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This volume contains full-length papers of contributions presented at Second International Conference on Concrete Sustainability (ICCS 16), held at Escuela de Ingenieros Caminos, Canales y Puertos of Universidad Politécnica de Madrid (Civil Engineering School of the Technical University of Madrid). Madrid, Spain, 13-15 June 2016.

The conference program includes four plenary lectures and 168 contributions articulated in 34 sessions. A total of 140 full papers have been submitted by authors and are presented in the volume.

The First International Conference on Concrete Sustainability, ICCS13, took place in Tokyo, May 2013, where more than 200 people from 36 countries registered. The ICCS Conferences provides a forum in which people from around the world come together to share their ideas and to discuss sustainability issues related to the concrete and construction industries.

Sustainability is already a key aspect for construction and, in particular, in construction with concrete. In the future, it will become an even more significant aspect in construction, such as planning, design, execution, maintenance and dismantling. Many research and normative aspects have still to be developed. We hope that the ICCS16 will contribute to improving the construction with concrete in this way.

Our sincere appreciation goes to plenary lecturers and all authors who have contributed to the outstanding scientific quality of the conference as reflected in the proceedings. We also show our gratitude for the financial support provided by SIKA SAU (Platinum Sponsor) and MAPEI (Gold Sponsor). The work of the International Scientific Committee is sincerely appreciated. In addition, we would like to express our gratitude to all the participants for their interest in the Conference.

Madrid, 20th May 2016

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ENVIRONMENTAL IMPACT, PERFORMANCE AND SERVICE LIFETIME – PILLARS OF SUSTAINABLE CONCRETE CONSTRUCTION

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Key words: Sustainability, green concrete, service life design, reliability

Abstract. Green concretes with reduced cement content may provide an alternative for improving concrete sustainability independently of supplementary cementitious materials. However, concrete sustainability is not merely a function of the absolute technical performance, durability and ecological impact, but also dependent on the degree to which these co-dependent properties are optimized and exploited within the design of concrete structures. The resulting uncertainties make an objective evaluation of concrete sustainability during mix design difficult. To aid in this process the Building Material Sustainability Potential is introduced, allowing a first estimate of the potential of a concrete mix to comply with the principles of sustainable engineering. Considering the low cement content of cement-reduced concrete, a proper prediction of the service life of structures made of this material is essential for the evaluation of the sustainability potential. The paper at hand outlines the service life prediction of cement-reduced concrete by probabilistic methods and discusses the subsequent evaluation of the sustainability potential of cement-reduced concrete.

1 INTRODUCTION

The building industry is affected by the ongoing sustainability debate more than any other industry, due primarily to the pronounced environmental impact resulting from the production of building materials, the erection of buildings and structures and the subsequent use thereof [1]. This holds especially true for concrete structures, as the production of this material – and here especially the production of the raw material cement – is highly energy intensive and the source of substantial emissions of CO$_2$ [2]. Reducing the environmental impact of concrete production independently of resulting consequences for the performance and durability of the material, however, is inadequate. Since the required service life of concrete structures normally ranges between 50 to 100 years, their environmental impact is spread over a long time period. Therefore, increasing the sustainability of building structures requires a reduction of the environmental impact associated with the erection, maintenance and operation processes and a concurrent increase of the durability of the structures at their maximum technical performance. This relation is described in Eq. 1 (see also [3]).
Even though the definition given above differs from standard definitions of the term sustainability, it is well in line with the latter, addressing the three basic pillars of sustainability – i.e. environmental aspects (by introducing the environmental impact) as well as social and economic aspects (hidden in the service life and performance parameters). As social and economic aspects, however, are extremely difficult or even impossible to evaluate during the concrete development process (i.e. the mix design), the definition given in Eq. 1 provides engineers with a simple tool to quantify the advantages and disadvantages of a specific concrete type with regard to its potential as a sustainable material. The exploitation of this potential during the design and construction process depends on the designer and user of the building or structure.

According to Eq. 1, three basic approaches to a sustainable use of concrete exist: 1st is the optimization of the composition of the concrete regarding its environmental impact while maintaining an equal or better performance and service life; 2nd is the improvement of the concrete’s performance at equal environmental impact and service life; 3rd is the optimization of the service life of the building material and the building structure at equal environmental impact and performance. Finally, a combination of the above named approaches seems reasonable.

In this paper the sustainability of so-called eco-concretes, i.e. concretes with a strongly reduced cement content, will be discussed. The development principles applied during the mix design procedures of the presented concretes are explained in the contribution of Moffatt et al. [4] to this conference. The focus of the paper at hand rather is placed on outlining the calculations related to the Building Material Sustainability Potential (BMSP) of such materials.

2 INVESTIGATED RAW MATERIALS

Following the approach of minimizing the environmental impact of concrete during the design phase, materials were selected with low environmental impact as judged by environmental impact indicators. Table 1 presents an overview of environmental impact indicator data representative of the materials used. The data in Table 1 demonstrate that the constituent material cement is critical for the environmental impact of concrete due to its high global warming potential (GWP). While the (GWP) of superplasticizers is similar to that of cement, it is of minor importance on account of the small dosages of this substance in concrete.
Table 1: Typical life cycle inventory data for cements and inert granular concrete constituent materials

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Non-renew.</td>
<td>Renew.</td>
<td>[MJ/kg]</td>
<td>[MJ/kg]</td>
<td>[kg CO₂/kg]</td>
<td>[kg R11/kg]</td>
<td>[kg SO₄/kg]</td>
</tr>
<tr>
<td>Cements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM I 32.5</td>
<td>5.650</td>
<td>8.74·10⁻²</td>
<td>0.951</td>
<td>1.64·10⁸</td>
<td>5.31·10⁴</td>
<td>3.30·10⁻⁵</td>
<td>2.20·10⁻⁶</td>
</tr>
<tr>
<td>CEM I 52.5</td>
<td>5.800</td>
<td>9.71·10⁻²</td>
<td>0.476</td>
<td>1.79·10⁸</td>
<td>5.74·10⁴</td>
<td>3.50·10⁻⁵</td>
<td>2.36·10⁻⁵</td>
</tr>
<tr>
<td>Cement industry (EPD)</td>
<td>2.451</td>
<td>6.58·10⁻²</td>
<td>0.691</td>
<td>1.50·10⁸</td>
<td>8.30·10⁴</td>
<td>1.2·10⁻⁴</td>
<td>1.0·10⁻⁴</td>
</tr>
<tr>
<td>Stone powders and aggregates</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Quartz powder 0-0.22 mm</td>
<td>0.820</td>
<td>3.16·10⁻²</td>
<td>2.34·10⁻²</td>
<td>4.98·10⁹</td>
<td>1.58·10⁴</td>
<td>6.75·10⁶</td>
<td>5.57·10⁶</td>
</tr>
<tr>
<td>Quartz sand</td>
<td>0.539</td>
<td>1.29·10⁻²</td>
<td>1.02·10⁻²</td>
<td>2.10·10⁹</td>
<td>7.54·10⁵</td>
<td>3.00·10⁻⁶</td>
<td>2.58·10⁻⁶</td>
</tr>
<tr>
<td>Sand</td>
<td>0.022</td>
<td>1.49·10⁻³</td>
<td>1.06·10⁻³</td>
<td>2.30·10⁻¹⁰</td>
<td>6.57·10⁻⁶</td>
<td>2.99·10⁻⁷</td>
<td>2.39·10⁻⁷</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.022</td>
<td>1.49·10⁻³</td>
<td>1.06·10⁻³</td>
<td>2.30·10⁻¹⁰</td>
<td>6.57·10⁻⁶</td>
<td>2.99·10⁻⁷</td>
<td>2.39·10⁻⁷</td>
</tr>
<tr>
<td>Superplasticizer (PCE based)</td>
<td>27.95</td>
<td>1.20</td>
<td>0.944</td>
<td>3.29·10⁻⁸</td>
<td>1.19·10⁻²</td>
<td>5.97·10⁻⁵</td>
<td>5.85·10⁻⁴</td>
</tr>
</tbody>
</table>

As binders, two cements, the first being a CEM I 52.5 R according to [8] and the second being a micro-cement with strongly reduced particle size, were selected for the investigations. No product specific life cycle inventory data were available for the micro-cement, but as it is produced by separating the fine particles from a CEM I 52.5 R, it is expected that the data will be very similar with a slight increase in renewable primary energy consumption, assuming the separation process is powered by a renewable energy source. As the availability of secondary cementitious binder materials may decline relative to future concrete demand, no secondary cementitious materials were included in this research.

Coarse and fine aggregate fractions consisting of inert quartz gravel and sand fractions, inert quartz powders and a silica fume were selected to make up the majority of the solid material in the granular matrix of the concretes. Selected properties of the cements and inert materials used are presented in Table 2.
Table 2: Properties of cements and inert aggregates investigated

<table>
<thead>
<tr>
<th>Property</th>
<th>Dimension</th>
<th>CEM I 52.5 R</th>
<th>Micro-cement</th>
<th>Silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density [9]</td>
<td>[g/cm³]</td>
<td>3.117</td>
<td>3.110</td>
<td>2,225</td>
</tr>
<tr>
<td>Blaine value [10]</td>
<td>[cm²/g]</td>
<td>5800</td>
<td>6900</td>
<td>-</td>
</tr>
<tr>
<td>Time of initial set [11]</td>
<td>[min]</td>
<td>170¹</td>
<td>77</td>
<td>-</td>
</tr>
<tr>
<td>Compressive strength $f_{c,28d}$ [12]</td>
<td>[MPa]</td>
<td>68.0¹</td>
<td>106.3</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>Dimension</th>
<th>Quartz powder 1</th>
<th>Quartz powder 2</th>
<th>Sand 0.1/1 mm</th>
<th>Sand 1/2</th>
<th>Gravel 2/8 mm</th>
<th>Gravel 8/16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density [9, 13]</td>
<td>[kg/dm³]</td>
<td>2.648</td>
<td>2.650</td>
<td>2.650</td>
<td>2.61</td>
<td>2.51</td>
<td>2.54</td>
</tr>
<tr>
<td>Water absorption [13]</td>
<td>[m.-%]</td>
<td>-</td>
<td>-</td>
<td>0.2</td>
<td>0.3</td>
<td>1.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Blaine value [10]</td>
<td>[cm²/g]</td>
<td>18.000¹</td>
<td>1448</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

¹) Data supplied by producer

The particle size distribution of all granular constituents was optimized using the CIPM Model by Fennis [14] and adjusted to yield mixes with maximum packing density and minimum voids content. A detailed description of this procedure can be found in [3, 4]. The particle size distribution of the solid materials used is shown in Fig. 1. The silica fume is not included in herein, as agglomeration causes the measurement of an unrealistically coarse particle size distribution in densified product. Additionally, a superplasticizer was also included in the mixtures and dosed according to the recommendations made in [14].

![Figure 1: Particle size distribution curves of the cements and inert granular constituent materials used](image-url)
3 COMPOSITION AND PROPERTIES OF INVESTIGATED MIXES

Based on the raw materials detailed in Sec. 2, a total of 6 different concrete mixes with cement contents ranging between 4 vol.-% and 10 vol.-% of all solid particles were developed. The composition and selected properties of the mixes are detailed in Table 3.

Table 3: Mixture composition of the developed concretes

<table>
<thead>
<tr>
<th>Raw material / characteristic value</th>
<th>Dimension</th>
<th>Concrete mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement content in dry mix [vol.-%]</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Grain size distribution (fit parameter n) [-]</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>Cement type [-]</td>
<td>CEM I</td>
<td>µCEM</td>
</tr>
<tr>
<td>Mixture composition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement [kg/m³]</td>
<td>113</td>
<td>111</td>
</tr>
<tr>
<td>Quartz powder 1</td>
<td>96</td>
<td>96</td>
</tr>
<tr>
<td>Quartz powder 2</td>
<td>120</td>
<td>121</td>
</tr>
<tr>
<td>Sand 0.1/1 (mm) [kg/m³]</td>
<td>519</td>
<td>520</td>
</tr>
<tr>
<td>Sand 1/2 (mm)</td>
<td>434</td>
<td>435</td>
</tr>
<tr>
<td>River gravel 2/8 (mm)</td>
<td>482</td>
<td>483</td>
</tr>
<tr>
<td>River gravel 8/16 (mm)</td>
<td>506</td>
<td>507</td>
</tr>
<tr>
<td>Water</td>
<td>87</td>
<td>85</td>
</tr>
<tr>
<td>Superplasticizer (PCE based) [-]</td>
<td>6.5</td>
<td>6.4</td>
</tr>
<tr>
<td>w/c-ratio [-]</td>
<td>0.64</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Mixture properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Dimension</th>
<th>Concrete mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength $f_{cm,28d}$ [MPa]</td>
<td>76.9</td>
<td>79.0</td>
</tr>
<tr>
<td>Degree of compactability c [-]</td>
<td>1.25</td>
<td>1.21</td>
</tr>
<tr>
<td>Flow value a [mm]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Inverse carbon resistance $R_{ACC}^{-1}$ $[(10^{-11} \text{m}^2/\text{s})/ (\text{kg/m}^3)]$</td>
<td>18.91</td>
<td>0.39</td>
</tr>
<tr>
<td>Mean value / standard deviation</td>
<td>6.83</td>
<td>0.33</td>
</tr>
<tr>
<td>Global warming potential (GWP) [kg CO₂/m³]</td>
<td>75</td>
<td>74</td>
</tr>
</tbody>
</table>

The mix design process consisted of the following steps: Firstly, the raw materials of the concrete were selected with the objective of minimizing the content of materials with pronounced environmental impact within the concrete mixture.

Secondly, the cement content within the concrete was defined to be decreasing from 10 vol.-% to 6, 5 and 4 vol.-% of the total solids volume contained in each mixture. Each mixture contained only one cement, either the CEM I 52,5 R or the micro-cement described in Sec. 2.

Thirdly, the volume content of each inert granular material was adjusted to maximize the inert material content in the concrete while taking into consideration the influence of cement particles on the packing density. The particle packing model CIPM by Fennis [14] was used to judge the particle packing density while adjusting the granular mixture composition.
Finally, the fresh concrete properties of the mixtures were optimized by adjusting the water content in each mixture. Each mixture was provided with a PCE-based superplasticizer according to the recommendations made in [14].

The composition of the mixes detailed in Table 3 is characterized by cement contents between 113 kg/m³ to 268 kg/m³ in the fresh concrete of either the CEM I 52.5 R or the micro-cement. Additionally, in one mixture the cement CEM I 52.5 R was combined with micro-silica fume by replacing 5 % by mass of the cement by the corresponding mass of micro-silica fume (referred to as SF-CEM I). Hereby the effect of an improved interfacial transition zone was studied. The reference concrete was adjusted to have a w/c-ratio of 0.43 with a cement content corresponding to the minimum requirements of EN 206-1 [15].

The fresh concrete was tested for its compactability according to [16] or its flow value according to [17] depending on the flow characteristics of the mixture. Specimens were casted, demolded at the age of 2 days, cured in water until the age of 7 days and stored at 20 °C and 65 % r. h. until the age of 28 days, then tested for their compressive strength according to [18]. The corresponding results are detailed in Table 3 and show that the investigated concretes provide high compressive strengths combined with significantly reduced environmental impact compared to standard concretes. The environmental impact of each concrete is represented here by its global warming potential (GWP) and has been calculated based on the environmental impact and content of each raw material as specified in Sec. 2.

Besides the properties in the fresh state and the mechanical properties, the concretes were also tested for their durability under common environmental exposures such as freeze-thaw attack with de-icing salt, carbonation and chloride ingress. These experimental results served in the calculation of the service-lifetime expected of these concretes.

Fig. 2 shows the results of freeze-thaw tests conducted according to the CDF-method as described in [19] and [20].

As can be seen from the results detailed in Fig. 2, neither the tested reference concrete with...
a cement content of 10 vol.-% corresponding to 268 kg/m³, nor the concretes with reduced cement content fulfilled the requirements for a concrete corresponding to exposure class XF4 (high water content with chloride attack) according to [15] with a maximum allowable spalling of 1500 g/m². This result was expected. However, the experimental data also shows that the capillary suction and the freeze-thaw resistance of mixes with 4 vol.-% of cement show lower water absorption and a lower spalling than mixes with cement contents of 5 and 6 vol.-%, respectively. Despite its significantly higher w/c-ratio of approximately 0.63, the mix containing 4 vol.-% of micro-cement exhibited a similar, though slightly inferior freeze-thaw resistance than the reference concrete with a cement content of 10 vol.-% and a w/c-ratio of 0.43.

This result in combination with the declining performance of mixes with increasing cement content can be explained by the reduced surface area of hardened cement paste per unit area of concrete under attack as the cement content is reduced. Since only the hardened cement paste is susceptible to a freeze-thaw attack, this effect obviously offsets in part the detrimental effect of an increased w/c-ratio. Unfortunately, the amount of data available is still too small to derive a general law which quantifies both effects.

In order to investigate the influence of the interfacial transition zone (ITZ) on the durability of concretes with low cement content, in the mix designated “4 % SF-CEM I”, 5 % by mass of the Portland cement were replaced by a micro-silica fume. It was dosed to the coarse aggregates in order to enhance a localization of these particles on the coarse aggregate surfaces. The comparison of this mix with the corresponding reference mixture, i.e. the mix containing 4 vol.-% of Portland cement, does not show any difference in the freeze-thaw behaviour. Here it appears the w/c-ratio of the cement matrix is generally too high for the ITZ to have any significant effect on the freeze-thaw resistance. Small differences, however, become apparent when comparing the results of the water absorption test. Here the mix containing micro-silica fume exhibits higher water absorption than the mix without silica.

A very important aspect in the evaluation of the durability of the investigated concretes is their resistance against a CO₂-induced carbonation. Therefore, beam shaped samples with dimensions of 100 x 100 x 440 mm³ were casted, demoulded after 2 days and stored in water at 20 °C until the age of 7 days. Then the beams were removed from water storage and exposed to dry conditions at 20 °C and 65 % r. h. until the age of 28 days. At this age, half of the beams were removed from the climate chamber and exposed to an increased CO₂ concentration of 2 vol. % at 20 °C and approximately 70 % r. h. Both the samples carbonating under normal and under increased CO₂ concentration were investigated for their carbonation depth by splitting the samples at four points along the length of the beam and applying phenolphthalein to the surfaces of the split cross sections. The carbonation depth of each concrete was determined using one beam, measuring inward at 3 points along each of the 4 edges of the split surfaces. The mean value of the carbonation depth was formed for each mixture out of the 48 measurements taken from the corresponding beam.

As can be seen from Fig. 3 (left), the reference concrete (w/c = 0.43) subjected to normal carbonation (i.e. approximately 0.04 vol.-% of CO₂) does not show any carbonation at all, whereas the samples with reduced cement content exhibit a significantly increased carbonation. The worst performance in this comparison was also observed with the mix containing 6 vol.-% of cement, followed by the mixes with 5 and 4 vol.-% cement. While the differences between the 6 vol.-% mix compared to the 4 and 5 vol.-% mixes are of statistical
significance, the differences between the latter two are not. The same is true regarding the differences between the composite cement containing micro-silica fume and the corresponding mix without silica. Similar results with regard to the ranking of the performance of the investigated concretes can be found for the samples exposed to an accelerated carbonation at 2 vol.-% of CO₂ in Fig. 3 (right). In this test setup the reference concrete also did not exhibit any carbonation. The best performance of all cement-reduced concretes was found for the mix with 4 vol.-% of micro cement. Independently of the test setup, the carbonation depth was lower than 1 mm, showing a good carbonation resistance, albeit a diminished carbonation resistance when compared to the reference mixture.

4 SERVICE LIFE DESIGN AS A KEY TO SUSTAINABLE BUILDINGS AND STRUCTURES

As illustrated by Eq. 1, maximizing the lifetime of a building or a structure is a very efficient way to improve the sustainability of our built environment. Methods to predict the service life of a concrete structure and to design the structure accordingly are essential tools in the sustainability assessment process for sustainable buildings and structures. However, this aspect is often neglected in the current life-cycle assessment debate, leading to a single sided focus on a pure reduction of environmental impact while neglecting the durability and thus the sustainability of the designed structures.

The service life design process is dominated by assessing the alteration – i.e. ageing and
often deterioration – of the material on one hand and the varying environmental exposures on the other. This requires in-depth knowledge of the deterioration mechanisms of concrete and on the variance of the influencing factors over time. The procedure of service life prediction will, in the following, be illustrated by means of the carbonation process applied to green concretes as presented in Sec. 3.

The time dependent carbonation of concrete can be described using Eq. 2, in which \( x_c(t) \) describes the carbonation depth in (mm) at the time \( t \). The dimensionless parameters \( k_e, k_c \) and \( k_t \) take into account environmental conditions, curing and testing effects. \( R_{ACC}^{-1} \) is the inverse effective carbonation resistance of concrete and \( \varepsilon_t \) is the corresponding error term in \( \text{(mm}^2/\text{years})/(\text{kg/m}^3) \). \( C_S \) describes the surrounding \( \text{CO}_2 \)-concentration in (kg/m\(^3\)) and \( W(t) \) is the dimensionless weather function, see [21]. With the experimental data depicted in Fig. 3, \( R_{ACC}^{-1} \) can be calculated for the green concretes (see Table 3).

\[
x_c(t) = \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot C_S \cdot \sqrt{t} \cdot W(t)}
\]  

(2)

As a limit state criterion \( x_c(t) = c \), with \( c \) being the concrete cover, is introduced. The failure probability \( p_f \) is defined as the probability for exceeding this limit state within a defined reference time period.

The loss of durability, i.e. the increase of the deterioration with time, reduces the reliability of a structure. In order to be able to evaluate this reliability at any age of the structure, a reference period for the service life has to be specified [22]. Based on Eq. 2, the time at which depassivation of the reinforcement occurs can be determined and an appropriate maintenance management established, which can significantly increase the intended service life. By introduction of the reliability index \( \beta \), a direct correlation between \( \beta \) and the failure probability \( p_f \) is obtained. In case of a normally distributed limit state function \( Z = R - S \) (R: Resistance, S: Action), the failure probability \( p_f \) can be directly determined using Eq. 3.

\[
p_f = p(Z < 0) = \Phi(-\beta)
\]  

(3)

The variable \( \Phi(-\beta) \) denotes the distribution function of the standardized normal distribution (see [23]). The correlation between various values for the failure probability \( p_f \) and the reliability index \( \beta \) is shown in Table 4. Note e.g. that the often used 5 % quantile in civil engineering is equal to a failure probability of \( 5 \cdot 10^{-2} \), which corresponds to a reliability index \( \beta = 1.645 \).

<table>
<thead>
<tr>
<th>( p_f )</th>
<th>( 10^{-1} )</th>
<th>( 10^{-2} )</th>
<th>( 10^{-3} )</th>
<th>( 10^{-4} )</th>
<th>( 10^{-5} )</th>
<th>( 10^{-6} )</th>
<th>( 10^{-7} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>1.28</td>
<td>2.32</td>
<td>3.09</td>
<td>3.72</td>
<td>4.27</td>
<td>4.75</td>
<td>5.20</td>
</tr>
</tbody>
</table>

Table 4: Values for the failure probability \( p_f \) and the related reliability index \( \beta \) [23]

The target values of the reliability index, \( \beta_{\text{target}} \), depend on the consequences of failure (loss of serviceability) and the relative cost of safety measures. Table 5 contains target values of the reliability index \( \beta \) for building components in the serviceability limit state (SLS), see [24, 25]. Considering the case of depassivation of the reinforcement due to carbonation or chloride ingress, the target reliability index is recommended as \( \beta = 1.3 \) according to [21].
Table 5: Target values of the reliability index $\beta$ depending on the relative cost of safety measures

<table>
<thead>
<tr>
<th>Relative cost of safety measures</th>
<th>Reliability index $\beta$ [24]</th>
<th>Reliability index $\beta$ [25]</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>$1.3 \ (p_f \approx 10%)$</td>
<td>$1.0 \ (p_f \approx 16%)$</td>
</tr>
<tr>
<td>Moderate</td>
<td>$1.7 \ (p_f \approx 5%)$</td>
<td>$1.5 \ (p_f \approx 7%)$</td>
</tr>
<tr>
<td>Low</td>
<td>$2.3 \ (p_f \approx 1%)$</td>
<td>$2.0 \ (p_f \approx 2%)$</td>
</tr>
</tbody>
</table>

Fig. 4 shows a comparison of the resulting service life prediction for concrete structures subjected to carbonation with 40 mm mean concrete cover thickness (8 mm standard deviation) using a green concrete containing 113 kg/m³ (see Fig. 3, 4% CEM I, w/c-ratio = 0.64) and a reference concrete containing 320 kg/m³ of a CEM I 42.5 R with a w/c = 0.60 as described in [26, 27]. Further parameters in Eq. 2 were set according to the example in [27], representing environmental exposure conditions in the city of Munich, Germany. The reference concrete reaches the chosen target reliability index of $\beta_{\text{target}} = 1.5$ after 100 years, the selected green concrete after 72 years.

Combining the measured performance of the green concretes with the durability parameters determined by experiment and the probabilistic service life prediction, it is now possible to evaluate the sustainability potential as described in Eq. 1. Table 6 contains the results for the BMSP of a green concrete as compared to a normal concrete evaluated for a moderate reliability index of 1.5 (see Table 5) in the case of CO₂-induced carbonation described above. Although the predicted service life of the green concrete is thirty years shorter than that predicted for the normal concrete, its high performance and reduced environmental impact compensate for this deficit within the sustainability potential index.
Table 6: Evaluation of the sustainability potential of a green concrete in comparison to a standard concrete

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Dimension</th>
<th>Normal concrete</th>
<th>Green concrete 4% CEM I 52.5 R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>-</td>
<td>CEM I 42.5 R</td>
<td>CEM I 52.5 R</td>
</tr>
<tr>
<td>Cement content</td>
<td>kg/m³</td>
<td>320</td>
<td>113</td>
</tr>
<tr>
<td>Water to binder ratio</td>
<td>-</td>
<td>0.60</td>
<td>0.64</td>
</tr>
<tr>
<td>Inverse carbonation resistance $R_{acc}^{-1}$ (mean value / standard deviation)</td>
<td>$(10^{-11} \text{m}^2/\text{s})/(\text{kg/m}^3)$</td>
<td>13.4 / 5.2</td>
<td>18.9 / 5.6</td>
</tr>
<tr>
<td>Calculated service life</td>
<td>years</td>
<td>100</td>
<td>72</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>MPa</td>
<td>38.4</td>
<td>76.8</td>
</tr>
<tr>
<td>Environmental impact</td>
<td>kg CO₂/m³</td>
<td>214</td>
<td>76</td>
</tr>
<tr>
<td>BMSP (See Eq. 1)</td>
<td>MPa-years/(kg CO₂/m³)</td>
<td>17.9</td>
<td>72.8</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

The sustainability of concrete is difficult to quantify during the concrete mix design process, as the three interdependent parameters of performance, durability and environmental impact must be evaluated and concurrently optimized. The Building Material Sustainability Potential (BMSP) is thus introduced as a simple indicator for sustainability during mix design.

It has been demonstrated that cement-reduced concrete can be produced while maintaining or even improving performance in compressive strength, raising potential for discussion of minimum cement contents within concrete standards. To evaluate the sustainability potential of the resulting concretes, however, their durability characteristics must also be considered.

Probabilistic service life design methods, relying on experiments and improved deterioration mechanism models, can be used to predict effectively the service life of concrete structures under defined environmental exposures. While experimental results indicate a deficit in the durability characteristics of cement-reduced concretes, this deficit may be insignificant depending on the intended exposure conditions. Due to significant increases in performance and strongly reduced environmental impact, the evaluation of the BMSP for one such concrete compared to a standard concrete indicates potential for a significant sustainability benefit when choosing the green concrete. Whether this benefit outweighs any potential drawbacks will also depend on the proper management of necessary maintenance measures when the service life of the structures indeed expires.

The cement-reduced concrete mixtures presented are a first step toward producing sustainable concrete and abstaining from supplementary cementitious materials. While the BMSP of the examined green concrete greatly exceeds that of the reference concrete presented, more research regarding the durability of these mixtures must be performed.

ACKNOWLEDGEMENTS

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EXPANDING KNOWLEDGE AND RESOURCES FOR MODERN
CONCRETE PROFESSIONALS: INNOVATION, SUSTAINABILITY,
AND RESILIENCE

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Key words: Cement, Climate Change, Concrete, Construction, Contractor, Design, Disaster-Protection, Engineer, Engineering, Excellence, Innovation, Jobsite Safety, Materials, Professor, Resilience, Resilient, Safety, Student, Sustainability

Abstract. Now more than ever, concrete design and construction projects must integrate innovative techniques and technologies to keep pace with ever-evolving economic, environmental, and aesthetic demands. Structures must be designed to protect against disasters and be resilient against climate change. Jobsite safety remains paramount. This presentation will highlight the many demands facing professionals in the concrete design, construction, and materials industry, and include examples of sustainability, innovation, and excellence. Also included will be American Concrete Institute initiatives to expand knowledge and resources to support modern engineers, contractors, researchers, and students.
1 INTRODUCTION

About the American Concrete Institute – Founded in 1904 and headquartered in Farmington Hills, Michigan, USA, the American Concrete Institute is a leading authority and resource worldwide for the development and distribution of consensus-based standards, technical resources, educational & training programs, certification programs, and proven expertise for individuals and organizations involved in concrete design, construction, and materials, who share a commitment to pursuing the best use of concrete. ACI has over 101 chapters, 93 student chapters, and nearly 20,000 members spanning over 120 countries. In recent years, the Institute has dedicated its resources to understanding and advancing the use of concrete in sustainable and resilient development through committees, articles, research, publications, partnerships, and events.

About Baker Concrete Construction, Inc. – Founded in 1968, Baker Concrete Construction, Inc., is one of the USA’s leading concrete construction firms and specializes in all types of cast-in-place concrete construction. In addition to an industry-leading commitment to safety, both on and off the job, Baker Concrete Construction has earned a reputation for excellence through finely-honed skills, fierce determination, and a commitment to high performance. Baker believes that success is driven by the commitment of its workforce to live “Incident and Injury Free” (IIF) – a culture not exclusively about rules and procedures, but about each employee examining his or her own relationship to safety. Baker Concrete Construction is ranked first in the 2015 Top 20 Concrete Specialty Contractors category in Engineering News-Record, and has been ranked in the top three annually since 1989.

2 SUSTAINABILITY

Sustainable development is defined by the Brundtland Commission as “development that meets the need of the present without compromising the ability of future generations to meet their own needs.” Resources have been developed both by the American Concrete Institute and Baker Concrete Construction on the topic of sustainability and sustainable development.

2.1 Sustainability at the American Concrete Institute

The American Concrete Institute’s first position statement on sustainability, approved in 2005, stated that the Institute shall: 1) encourage the development of sustainable structures through the application of environmentally friendly and sustainable concrete design, materials, and construction; 2) position concrete to compete effectively; and 3) raise the level of awareness and seek support for increased sustainability inside and outside the concrete industry.

In 2008, the American Concrete Institute began hosting its annual Concrete Sustainability Forum. This annual event includes global experts sharing insight on: 1) the need to work more diligently to clarify our industry’s environmental responsibility to society; 2) disseminating the significance of sustainability among members of the concrete sector, to
collect scientific data related to the environmental impacts of concrete, and to develop indicators and simple tools for the evaluation of those impacts; and 3) to host a forum on sustainability in concrete featuring representatives from the world’s diverse concrete organizations and innovators.

In 2010, ACI’s Board of Direction updated the position statement. It states the American Concrete Institute’s intent to: 1) support the consideration of economic, social, and environmental balance for sustainability criteria in concrete design, materials, and construction; 2) identify and, as appropriate, remove barriers to the adoption of more sustainable concrete practices, technologies, standards, specifications, etc.; and 3) raise the level of awareness and seek support for increased sustainability practices and designs within the concrete industry and beyond.

Also in 2010, ACI formed a new technical committee to develop and report information on the sustainability of concrete (ACI Committee 130: Sustainability of Concrete). This technical committee subsequently formed seven subcommittees to strengthen, broaden, and institutionalize its efforts. In tandem, several ACI committees are working on specific areas of sustainability and resilience, which they have been doing since the mid-1900s, and have published state-of-the-art reports guiding industry practice.

(1) ACI Technical Committee Documents

The term "technical committee document" refers to all ACI technical committee works, whether published or under development, such as codes, specifications, reports, and guides. All technical committee documents are developed by ACI technical committees in accordance with the ACI Technical Committee Manual, and approved by the ACI Technical Activities Committee. ACI has several documents currently available on topics relating to sustainability, including:

- Removal and Reuse of Hardened Concrete (ACI 555R-01);
- Guide to Thermal Properties of Concrete and Masonry Systems (ACI 122R-14);
- Use of Fly Ash in Concrete (ACI 232.2R-03);
- Report on High-Volume Fly Ash Concrete for Structural Applications (ACI 232.3R-14);
- Slag Cement in Concrete and Mortar (ACI 233R-03);
- Guide to the Use of Silica Fume in Concrete (ACI 234R-06);
- Concrete Repair Guide (ACI 546R-14);
- Guide to Durable Concrete (ACI 201.2R-08); and
• Specification for Pervious Concrete Pavements (ACI 522.1-13), among others.

Additional ACI technical committee documents are under development on the topics of insulating concrete forms, concrete wind turbine towers, alternative cementitious materials, and more.

(2) Educational programs

The term "educational programs" refers to all ACI educational committee works, whether published or under development, and include seminars, webinars, and courses through the ACI University. Educational programs are practice-oriented and are intended to raise the competence level of concrete producers, contractors, technicians, engineers, and others within the concrete industry. ACI has several educational programs currently available on topics relating to sustainability, including:

• Concrete Sustainability Basics;
• Concrete Sustainability – Incorporating Environmental, Social, and Economic Aspects;
• Green Cements;
• High-Volume Fly Ash Concrete for Structural Applications
• Sustainability of Concrete Pavement; and
• The Art of Thermal Mass Modeling for Energy Conservation in Buildings; among others.

(3) Symposium publications

ACI “symposium publications” include a collection of manuscripts authored by an ACI technical committee or by individuals. Manuscripts are selected and reviewed by the sponsoring ACI committee in accordance with the policies established in the ACI Technical Committee Manual. ACI has several symposium publications currently available on topics relating to sustainability, including:

• Recycling Concrete and Other Materials for Sustainable Development (SP 219);
• Thirteenth International Conference on Advances in Concrete Technology and Sustainability Issues (SP 303);
• Sustainable Performance of Concrete Bridges and Elements Subjected to Aggressive Environments: Monitoring, Evaluation & Rehabilitation (SP 304); and
• Concrete: The Sustainable Materials Choice (SP 269); among others.
(4) U.S. Green Concrete Council publications

To meet the need for credible technical resources on concrete sustainability, the American Concrete Institute formed the U.S. Green Concrete Council, a subsidiary of the Institute, to develop two publications: “The Sustainable Concrete Guide – Strategies and Examples” (English and Spanish) and “The Sustainable Concrete Guide – Applications” (English). Unveiled in 2010, these publications set a new standard within ACI for timely publication of industry-critical resources, and over 10,000 copies have been distributed.

(5) Concrete Construction Sustainability and Resilience Assessor

The ACI Certification Programs Committee, with the guidance of the ACI technical committee on concrete sustainability, is researching the development of a program to certify individuals as Concrete Construction Sustainability and Resilience Assessors. The ACI-certified assessors would be responsible for identifying opportunities during concrete design and construction to support environmental objectives. A program launch date has not yet been established.

2.2 Sustainability at Baker Concrete Construction, Inc.

As stewards of resources and environment, Baker Concrete Construction strives to ensure a solid future for current and future generations of people, communities, the nation, and the world through a balanced pursuit of the following:

- Social and people-related sustainability – working with integrity to enhance the quality of life for all;
- Environmental stewardship – working in a responsible and ethical manner to improve the built environment, seeking to rectify the practices of the past, and setting new industry standards through innovation and continuous improvement;
- Economic sustainability – ensuring long-term financial stability through social and environmental initiatives.

Baker Concrete Construction contributes to sustainability by focusing on three complementary strategies:

1. Offering a selection of sustainable concrete construction products and services to customers;
2. Pursing its own every-day sustainability program on jobsites, in construction trailers, and in its offices; and
3. Working closely with suppliers and subcontractors to align their processes with Baker’s own.
In addition, Baker Concrete Construction encourages special “Acts of Green” initiatives by each project, each function, and by individual co-workers. These Acts often originate from the Baker Safety, Sustainability, Quality and Productivity (SSQP) continuous improvement program.

Baker’s Safety, Sustainability, Quality and Productivity continuous improvement program generates many ideas that are implemented. Biggest successes have been with co-worker safety, but Baker has also improved quality, productivity, and social sustainability impact. Select impacts that have come from the program include:

**Jobsite offerings that promote sustainability include:**

- Support local charities: Toys for Tots, Make-a-Wish, food banks, building community playgrounds, etc. Giving is part of culture; becoming part of local communities when traveling to remote jobsites;
- Improvement in products that we buy;
- Promote buying local: labor, services, products;
- By example, filled a need for 800 workers for steel mill project and hired 80% local co-workers while installing approximately 400,000 yd$^3$ of concrete in less than one year;
- Promote cleaner project sites;
- Encourage a range of strengths (e.g.: 56, 91 days);
- Encourage zero discharge sites; separate dumpsters for steel and recycle concrete on modernization projects. When recycling is not practical, reuse concrete as fill or rip rap;
- Encourage paperless projects and processes; several recent nuclear projects included over 3,000,000 pages of records. Baker invested in enterprise content management software and processed most pages electronically and with electronic work flows. Shredded and recycled paper that was used;
- Increase efficiency of labor and materials;
- Optimize aggregates; review mix designs to assure that they are not gap graded and workable – enhancing the quality of work;
- Optimize mix designs: reduce CO$_2$ in concrete; routinely use 30% fly ash mixes and have used 50% fly ash mixes for mass concrete:
- Implement storm water pollution protection plans on projects;
• Avoid use of toxic materials;
• Recycle water on jobsites/batch plant;
• Recycle: waste on jobsites, use recycled products;
• Reduce dust (collection shrouds, socks, fans, etc.);
• Reduce wood use though proper formwork selection, make-up processes, and proper use of form oils;
• Energy-saving lighting with motion detectors in permanent facilities
• Reduce fuel-consumption (study, alternative fuels, reduce engine running time); use bio-diesel to fuel compressors and generators on a several projects; no idling equipment and vehicles on jobsites;
• Reduce noise;
• Reduce waste; and encourage use of bio-degradable materials for products that must be landfilled; and
• Reduce water use.

General sustainability initiatives that promote sustainability include:

• Health insurance, flu shots, wellness programs part of “Incident and Injury Free” (IIF) culture;
• The Safety, Sustainability, Quality and Productivity continuous improvement program includes sustainability discussions. This contributes to better awareness of the impact on communities;
• All co-workers receive “Incident and Injury Free” indoctrination that encourages behavior based approach to safety;
• Wellness includes coaching regarding co-workers well-being and daily stretch & flex and encouragement; and
• Baker University offers a wide range of classes that enhance the lives of our co-workers. These classes go beyond technical skills needed to perform work and support co-worker growth with languages, math skills, etc.

Business development offerings that promote sustainability include:

• Polishing damaged slabs as opposed to removing and replacing;
• Offer restoration as an alternative to new construction; Baker works on many modernization projects;

• Participate on brownfield re-development site and practice site reclamation;

• Participate on many LEED-certified projects; and

• Provide design assist services and Building Information Modeling support to make projects more constructible and consuming fewer resources.

3 JOBSITE SAFETY

Paramount to all aspects of sustainable construction is the strict accordance to published safety guidelines. Hard hat, eye protection, safety vest, long-sleeved shirt, long pants, boots, and ear protection are minimum safety equipment for concrete work, and dust masks should be worn when necessary. Since fresh portland cement concrete is highly alkaline and can cause skin irritation and burns, it is important for all to comply with these additional precautions to avoid injury:

1. Keep cement products off skin — experienced concrete craftsmen protect their skin with boots, gloves, clothing, and knee pads. Skin injury may result from clothing that is wet from cement mixtures;

2. Don’t let skin rub against cement products — Many cement products are abrasive. Rubbing increases the chances of serious injury. Keep concrete out of cuffs and boots;

3. Wash skin promptly after contact with cement products;

4. Keep cement and cement products out of eyes — Concrete workers should wear safety glasses or goggles. If any cement or cement mixtures get into the eye, flush immediately and repeatedly with water and consult a physician promptly; and

5. Keep products out of the reach of children.

3.1 Safety Resources from the American Concrete Institute

The American Concrete Institute works to disseminate consensus-based information through publications and educational products. Several Institute publications include content on jobsite safety, including:

• Concrete Fundamentals;

• The Contractor's Guide to Quality Concrete Construction (published jointly with the American Society of Concrete Contractors); and

3.2 Jobsite Safety at Baker Concrete Construction, Inc.

To be the preeminent concrete contractor through a commitment to delivering unparalleled safety, quality, and value and caring for employees, clients, and the environment, Baker has developed several guiding principles that prioritize jobsite safety. These principles are broader than jobsite safety, and include:

- Promoting an “Incident and Injury-Free” lifestyle;
- Promoting and demonstrate a commitment to sustainable construction;
- Striving for performance excellence in all areas;
- Promoting honesty and integrity;
- Leading by example and working toward continuous improvement through innovation;
- Being positive contributors to the concrete industry;
- Encouraging frequent and open communication;
- Being professional at all times;
- Delivering on promises; and
- Respecting individuals while operating as a team.

4 RESILIENCE

Resilience is defined by the United Nations International Strategy for Disaster Reduction as the ability of a system, community, or society exposed to hazards to resist, absorb, accommodate to and recover from the effects of a hazard in a timely and efficient manner, including through the preservation and restoration of its essential basic structures and functions. As further stated by the UNISDR, economic loss risk continues to increase across all regions – and seriously threatens the economies of low-income countries. Development with a renewed focus on resilience can offer opportunities for mitigating the impact of these events.

Resources are in development and have been developed both by the American Concrete Institute and Baker Concrete Construction to aid in creating resilient buildings, infrastructure, and communities.
4.1 Resilience and Disaster Protection at the American Concrete Institute

ACI Committee 133: Disaster Reconnaissance – This committee is working to report on the effects of major disasters on concrete construction worldwide. It aims to build institutional and individual member capabilities in disaster reconnaissance, and to leverage reconnaissance programs by other organizations, with special emphasis on concrete construction. Following the magnitude 7.8 earthquake in Nepal in April 2015 that caused more than 8,500 casualties and left over 22,000 people injured, ACI Committee 133 activated a team to research the performance of low- and high-rise reinforced concrete buildings in the region. Thousands of buildings were destroyed and many more rendered unsafe. The team catalogued their findings for use by ACI and the government of Nepal.

ACI Committee 216: Fire Resistance and Fire Protection of Structures (joint ACI-TMS committee) – This committee is working to develop and report information on the fire resistance and protection of concrete and masonry structures, while ensuring that ACI’s “Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies (ACI 216.1)” remains up to date.

ACI Committee 341: Earthquake-Resistant Concrete Bridges – This committee is working to develop and report information on bridges to resist earthquakes. It aims to publish a new document on recommendations for design of earthquake-resistant reinforced concrete bridge pier walls; complete bridge seismic isolation guide suitable for publication; develop a new document on performance-based seismic design of bridges; update ACI’s seismic retrofit document; and continue to develop content for seismic design of bridge columns based on drift.

ACI Committee 377: Performance-Based Structural Integrity & Resilience of Concrete Structures – This committee is working to develop and report information on performance-based structural integrity and resilience of concrete structures. It is working to examine the current ACI 318 integrity requirements; identify collapse resisting mechanisms in load redistribution in case of initial damage; determine how detailing can enhance structural integrity and resilience; and propose approaches and methods for functional and disaster-resistant design of structural components and systems.

5 THE ACI EXCELLENCE IN CONCRETE CONSTRUCTION AWARDS

New for 2015 and occurring annually hereafter, the ACI Excellence in Concrete Construction Awards provide a platform to recognize concrete projects at the forefront of innovation and technology, and showcase projects that inspire excellence in concrete design, construction, and sustainability.

5.1 Excellence Award

In 2015, the highest award was presented to the Museum of European and Mediterranean Civilizations located in Marseille, France. As an icon of urban rejuvenation located on the
banks of the 2,600-year-old harbor at Marseille, France, the 3-exhibition story building houses the first French National museum outside of Paris. The technical performances of ultra-high performance fiber-reinforced concrete (UHPFRC) were uniquely applied for the first time at such a scale for a building structure in order to widen exhibition spaces, minimize material consumption, and provide shelter while allowing natural light to filter through.

### 5.2 First Place, Second Place, and Honorable Mentions

Additional global projects recognized in 2015 include:

**Low-Rise Buildings** – 1st Place: MuCEM: Museum of European and Mediterranean Civilizations in Marseille, France; 2nd Place: Louisiana Sports Hall of Fame and Northwest Louisiana History Museum in Natchitoches, LA, USA.

**High-Rise Buildings** – 1st Place: Al Hamra Business Tower in Kuwait City, Kuwait; Honorable Mention: Tower One of World Trade Center New York City, NY, USA; Honorable Mention: Adobe Corporate Campus Phase 1 in Lehi, UT, USA.

**Infrastructure** – 1st Place: Cable-stayed Foot Bridge in Celakovice, Czech Republic; 2nd Place: Interstate 5 Willamette River Bridge Project in Eugene, OR, USA.

**Decorative Concrete** – 1st Place: Jean Bouin Stadium in Paris, France; 2nd Place: Downtown Doral Park Pavilion in Doral, FL, USA.

**Repair & Restoration** – 1st Place: Mission Bridge Seismic Retrofit in Abbotsford, BC, Canada.

### 6 CONCLUSIONS

As concrete design and construction projects continue to integrate innovative techniques and technologies in keeping pace with ever-evolving economic, environmental, and aesthetic demands, the American Concrete Institute and Baker Concrete Construction are advancing the industry. Providing guidance to protect against disasters, be resilient against climate change, ensure safe jobsites, practice sustainable design and construction, and showcase truly excellent concrete construction, ACI and Baker are continually expanding knowledge and resources to support modern engineers, contractors, researchers, and students. And, as each of us act locally, collaborate, and maintain a global perspective, we will accelerate the concrete industry’s contribution to ensuring a safe, sustainable, and resilient future.

On behalf of my 2,500 Baker Concrete Construction co-workers and 40,000 fellow ACI committee leaders, committee members, members, chapter members, and students who are championing sustainability, resilience, and technical excellence across the globe, I look forward to continuing this sustainability journey together.
6 REFERENCES


RECYCLING OF CONSTRUCTION AND DEMOLITION WASTE
AN OVERVIEW OF RILEM ACHIEVEMENTS AND STATE OF THE ART IN THE EU

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Key words: Sustainable Construction, Construction & Demolition Waste, Recycling, Circular Economy, Standards, Quality Control

Abstract. Technical research work on recycling started in RILEM back in 1981 in the TC 37-DRC chaired by Torben Hansen. The initial theoretical work on Demolition and Recycling of Concrete was quickly followed by industry driven actions as the interest in the topic was growing in practice. TC 121-DRG undertook under the leadership of Erik Lauritzen to come up with Guidelines for Demolition and Reuse of concrete in 1989.

Environmental legislation in Europe has been putting the topic of Construction and Demolition waste high on the agenda in recent year especially under impetus of northern Europe where now up to 90% of all the C&D waste is being recycled as part of a focus on sustainable construction and circular economy. Most recently in its communication on ‘Resource Efficiency Opportunities in the Building Sector’ COM (2014)445 the European commission focusses on the establishment of a common European approach to assess the environmental performance of buildings throughout their lifecycle, taking into account the use of resources such as energy, materials and water. The Commission announces in this context to support projects in H2020, COSME and Life+ related to research, innovation and demonstration showcasing how collaboration between public authorities and the private sector can create viable markets with recycled materials.

In the endeavor to fully implement a circular economy approach in the construction sector quality control and traceability of the origin of demolition waste is of utmost importance.

1 INTRODUCTION

The construction industry has to support a world of continuing population growth and social and economic development by providing homes, schools, offices, shops, hospitals,
airports, bridges, roads and infrastructures of all kind whilst improving the quality of life both in towns and on the countryside. At the same time, construction must respect the increasing and widespread social interest in environmental protection, preservation and restoration. So the construction sector has an important role to play with respect to economic, social and environmental issues and as such it has a central and crucial role to play in the development of our societies, our welfare and our environment.

The broad ‘Construction’ sector is generally interpreted to embrace all the activities that contribute to the creation, operation and maintenance of the built environment. It thus includes not only the design and construction of housing, other buildings and infrastructure works, but indeed also their operation, maintenance, repair, refurbishment and ultimate demolition and recycling. All associated activities related to the extraction and supply of construction materials, products, systems and services are of course also included. Construction thus defined is a major economic sector worldwide. As an example in Europe it represents a turnover of over 1000 billion Euros and it accounts for well over 10% of European GDP while employing directly and indirectly some 12 million people. From the above it is evident that the importance of the construction industry for the three elements of sustainable development, namely economic growth, social progress and effective protection of the environment, cannot be disregarded. Sustainable means lasting and enduring, therefore, sustainable development is economic development that lasts. Sustainable construction is obviously of great importance in Europe considering that according to different sources at present broadly speaking:

- 30% of all traffic is construction related,
- 40% of the energy consumption in Europe is construction related,
- 50% of material resources taken from nature are construction related,
- over 30% and up to 50% of national waste production comes from the construction sector.

This paper reviews in the perspective of sustainable construction some past and present technical evolutions with respect to recycling of construction & demolition waste in Europe this with a specific focus on the situation in Belgium especially in the densely populated Region of Flanders. Specific problems are indeed to be addressed especially in densely populated and congested areas. While striving to a circular economy, it can indeed hardly be accepted to dump valuable materials that have a potential for recycling or reuse on one side while extracting and transporting (over long distances) virgin materials on the other side. This is especially true for concrete and masonry waste materials.

2 SOME HISTORY ON RILEM’S INVOLVEMENT

Considered to be a “classic” in the sustainability movement, “The Limits to Growth” issued in 1972 by the Club of Rome http://www.clubofrome.org was the first study to question the viability of continued growth in the human ecological footprint. The report broke new ground as the first global study commissioned by an independent body rather than a government or the UN and inspired many research teams to explore in their field of work the contributions they could deliver to implement changes in the existing linear consumption
model in use at that point in time. Obviously this study of the Club of Rome got also the attention of researchers active in the context of RILEM. RILEM is the Union of Laboratories and Experts in Construction Materials, Systems and Structures. It endeavors especially to advance scientific knowledge related to the use of concrete, masonry, wood, asphalt, and other construction materials, and to encourage transfer and application of this knowledge world-wide. The essential work of RILEM is done in Technical Comities (TC’s) which are set up in a Bottom-Up approach, i.e. the corporate members and individual members bring forward the proposals for action. Those are then evaluated on their merits by a technical advisory committee who also provides in monitoring of the progress of the TC’s. Work within RILEM on recycling of construction and demolition waste started back in 1981 in the TC 37-DRC chaired by Torben Hansen. The initial theoretical work on Demolition and Recycling of Concrete was quickly followed by industry driven actions as the interest in the topic was growing in practice. TC 121-DRG under the leadership of Erik Lauritzen came up with Guidelines for Demolition and Reuse of concrete in 1989. A conference was held in Denmark in 1993 and a State of the art Report, and later also a RILEM recommendation were issued as final work of this TC. Further work in RILEM was taken forward amongst others in the TC’s 165-SRM “Sustainable Raw Materials - construction and demolition waste” chaired by Charles Hendriks and 198-URM “Use of Recycled Materials” chaired by Charles Hendriks and Enric Vázquez. Today RILEM work on recycling concrete and masonry is still continuing and will be addressed later in this paper.

3 DEVELOPMENTS IN BELGIUM AS A CASE STUDY

Although there has been extensive research and several demonstration projects in Belgium and abroad, the high end reuse of recycled aggregates in concrete is at present still quite limited. Several barriers were identified: a lack of practical knowledge and experience, a standardization framework that does not allow for much recycling and the need for a specific quality assurance scheme. However, an evolution is ongoing in this domain. In this section different recent new developments are described, with focus on two main axes: practical experiences and support of the professionals on the one hand, and development of standards and new quality assurance approaches on the other.

3.1 Early developments

At the end of the 1970s, the first Belgian research on recycling of construction and demolition waste (C&D Waste) was executed, in order to develop the whole recycling process for the so-called stony fraction: from demolition techniques over crushing, sieving and sorting processes to the identification of applications for recycled materials. Besides the limited „ad hoc“ experiences of recycling of rubble of the Second World War in Belgium and Germany, large scale application of C&D Waste as construction material did not really exist. In the following years, pioneering work of the BBRI was transferred from research to real practical applications. When a larger alternative to the Zandvliet lock in the Antwerp harbor was needed in 1982, it was decided to use recycled aggregates for the concrete for the construction of the new lock [1]. The recycled aggregates were in fact produced from the demolition waste of the old lock’s embankment walls demolition. This approach allowed for the first real
industrial scale recycling plant to be set up in Belgium in 1986 and for the recycling of 200,000 tons of concrete aggregate in the concrete of the new Berendrecht lock. This knowledge was later on also applied in other exemplary projects like BBRI's Recycled House and the Centre for Sustainable Construction CeDuBo.

3.2 Role of the government

One of the main barriers for expansion of the sector in the 1990s was the lack of technical prescriptions. Thanks to a collective initiative, i.e. the set-up of a working group of the Flemish ministry for infrastructure LIN - “Hergebruik van Afvalstoffen” (Reuse of waste materials), several specific instructions for government example projects were drafted, allowing the use of secondary aggregates in road construction applications. This resulted in the uptake of technical prescriptions for recycled aggregates in the Standard Specifications for Road Constructions (SB250) in the Flemish region by 1996.

Also in the Walloon region, initiatives were supported by the government in order to start up the recycling industry. With the support of the Walloon government, the organisation TraDeCoWall was founded in 1991 in order to start recycling C&D Waste, to identify practical and reliable solutions for the management of waste originating from building and demolition sites and to develop valorisation options. This initiative was later on also supported by the publication of Walloon technical specifications for public road works in which recycling was permitted in some areas.

Besides the technical part, also the aspects of environmental protection were covered by authority initiatives. Following the Waste Decree of 1994, the OVAM was responsible for the definition of the Strategic Waste Management Plan in 1995, in which the ambition of a 75% recycling rate of C&D Waste in Flanders by the year 2000 was defined. In 1997, the first version of VLAREA regulation was published. The VLAREA regulates the administrative and chemical requirements (leaching, ...) for C&D Waste in order to be allowed for reuse in or as –amongst others- building materials. The VLAREA also implied a quality control and certification scheme for recycled aggregates, and was in this field the first legislation in Europe. In those days, the “COPRO” certification scheme emerged and allowed for the marketing of certified recycled aggregates.

Back in 1993, high grade applications such as recycled concrete with recycled aggregates were heavily debated. The already mentioned LIN working group even prepared technical specifications for “recycled concrete”, but notwithstanding the earlier positive demonstrations by BBRI, the market and government bodies were still reluctant to allow such implementations in general practice. At the time this specifications which were based on the RILEM recommendations were regretfully not approved.

3.3 Ongoing actions

By now, anno 2016, the recycling of construction & demolition waste in Belgium can be considered as a ‘grown up’, well-developed economic activity in an established market. However despite many efforts, only an estimated amount of 1% of the available recycled
aggregate (which equals about 200,000 tons on 18 million tons each year) is currently not being down-cycled but used as aggregate for structural concrete. Several barriers to a wider application of recycled aggregate concrete were identified. There is still a lack of widespread experience in the use of recycled aggregates in concrete. Therefore, on standardisation level, there is a reluctance to open up the Belgian concrete standards for wide application of recycled aggregates. And without an adapted Belgian standard, also quality assurance and certification, which is based on the Belgian concrete standard, is a difficult issue to tackle in everyday practical projects. However, an evolution is ongoing: on the one hand, more and more construction projects are realized using recycled aggregate concrete. This leads to more knowledge on the performance of concrete with recycled aggregates in real circumstances, but also gives more insight in the practical attention points during the execution process, which are not always described in standards or quality assurance manuals, but are a large surplus for more confident application. On the other hand, the new European standard EN 206 on concrete, published in 2013, contains an informative annex E on the application of recycled aggregates in concrete. This has led to the discussion on Belgian level to adapt the normative framework in order to allow for more use of recycled aggregates in concrete. Since 2012, Belgium has a concrete standard that allows the use of recycled concrete aggregates and specific types of slags in structural concrete: NBN B15-001:2012 [2]. Replacement up to 20% (in volume) of the coarse fraction is allowed in concrete meant for interior applications (X0 and XC1 according to EN 206-1). Other application domains are allowed, on condition that ‘specific suitability is proven’. In order to expand this application domain, additional data and research results are needed. Three initiatives on Belgian level in order to anticipate on these evolutions and to stimulate the use of recycled aggregates in concrete and the uptake of recycled concrete aggregate in the standards and the Belgian quality assurance system have therefore been undertaken:

- The compilation of existing knowledge and experience in a Technical & Practical guidance document
- Research and developments in the field of standardisation
- Elaboration of a specific quality assurance scheme for recycled aggregate concrete

3.4 Practical guidance report

An important barrier for the use of recycled aggregate concrete remains the lack of practical experience and well documented knowledge. Therefore in 2013, a new pilot project campaign was started, in order to have a new set of demonstration projects that are well documented and can be monitored for several years, in order to assess the real life durability performance. On the other hand as mentioned above, several research projects took place in the 1990s and 2000s, that have led to a considerable body of scientific results on the use of recycled aggregates in concrete [3,4,5]. Both the practical experiences and the research results are now bundled in a BBRI Technical Report [6,7]. This report is meant as a practical guidance document, complimentary to the standards and existing certification schemes, allowing parties in practice to make arrangements on the use of recycled aggregates in concrete for buildings or road works. It describes the whole production chain, from demolition to the production of qualitative aggregates and concrete, as well as the actual placement on site. In each of the steps, a technical description is given on relevant criteria &
information, and practical recommendations are given to achieve the desired requirements &
criteria. An important perquisite for qualitative concrete, is the use of high quality aggregates. Two
important aspects to obtain qualitative recycled aggregates are the sourcing on the one
hand (demolition of road structures, prefab concrete, … with high compression strength and
avoiding contaminations like plaster, wood, plastics, …) and the production process on the
other hand (applying the correct crushing technique, crushing twice in order to retain only the
best concrete aggregate, …). The most suitable indicators for the quality of the aggregate are
density and water absorption. The report summarizes the influence the use of recycled
aggregate can have on the concrete properties, such as workability, compression strength,
creep and shrinkage, durability. Distinction is made between lower replacement ratios (<
30%) and higher replacement ratios (>30%). This allows the designers to make well-thought
choices on the applications where they want to use recycled aggregates in concrete, and gives
concrete producers insights in the technological aspects of concrete production. This last
element is further elaborated in more practical guidelines on how to take into account the
water absorption behavior of the recycled aggregates in the mixing of the concrete (pre-
saturation, mixing protocol, storage of aggregates, calculation of effective water/cement-ratio,
…). The third chapter gives more information on the execution process. This includes the
attention points like workability and slump loss, time of setting & finishing, … but also gives
guidance on the quality assurance process and the extra testing required when using recycled
aggregates in specific applications that fall outside of the current standard framework. The
final part of the technical report gives recommendations and aids decision makers to align the
different phases in the process: quality and production of aggregates, choice for the correct
concrete applications and replacement levels, quality control scheme and execution aspects.

3.5 Standardization and research

As mentioned before, the Belgian standard NBN B15-001 allows for the use or recycled
aggregates in concrete since 2012. 20% (volume) of the coarse fraction can be replaced for
concrete in interior applications. Some specific elements have to be taken into account:
- The required quality of the concrete aggregate is well defined and the requirements
  are more severe than those for ‘regular concrete aggregate’ used in road base
  constructions
  o d ≥ 4 mm, D ≥ 10 mm and the fines content should be below 1.5%. Thus,
    only coarse and well-graded aggregate can be used
  o ρrd ≥ 2200 kg/m³: only pure, uncontaminated concrete aggregate is allowed
  o The variation on the declared water absorption on 24h is limited to ± 2%, in
    order to avoid fluctuations
  o Other requirements are LA 35, FI20, A40, SS0.2 but should not be difficult to
    obtain if the requirements stated above are fulfilled.
- One is allowed to use recycled aggregates up to 20% in other application domains
  (higher compression strength classes, other environments) on condition that the
  specificity of use is demonstrated for the intended use and the intended concrete
  composition. This means the concrete producer has to demonstrate that the concrete
  composition with recycled aggregates has the required compression strength,
  durability and fresh concrete properties. Unfortunately, the standard does not
specify the way this aptitude for use should be demonstrated.

As the actual moisture content and the effective water absorption behavior of the recycled aggregates in the concrete are hard to determine, a safety factor is applied in the calculation of the effective W/C-ratio. The $W_{A24}$ of the recycled aggregates should be lowered with 4%, assuming this to be a safe maximum. BBRI has recently started research to further investigate and refine these requirements, together with BRRC and CRIC. The first step in the research consisted of the characterization of the recycled concrete aggregates available on the market today, in order to assess the quality improvement needed to be in line with the normative requirements. 9 concrete aggregate types fulfilling the “$d \geq 4 \text{ mm}, D \geq 10 \text{ mm}$”-requirement were collected and their properties were determined. The first results show that:

- The required composition (large amount of concrete & unbound aggregates) is obtained for the selected sources
- Most of the aggregates contain too much fines ($< 63 \mu m$). Either the quality of the fines should be checked, either more attention should go to the production process and the circumstances (wet or dry weather has an influence on the fines content)
- The water absorption (24h) varies between 2.9% and 6.6% from source to source. The lowest water absorption is obtained from aggregate produced from specifically selected road pavement concrete, the higher values are linked to crushers that accept a broader range of C&D waste
- Chemical agents like chlorides, sulphates, binding altering substances, … are present in very low doses for recycled concrete aggregates
- There is no link between the frost resistance of the aggregates and their water absorption. However, aggregates with a water absorption rate (24h) below 5%, appear to have a better frost resistance.

In the next phase of the research, several aggregates will be used to make concrete, in order to investigate the concrete quality, with a specific focus on the influence on the concrete characteristics due to the variance in the different aggregates: e.g. will the higher water absorption of some aggregate types have influence on the frost resistance, the carbonation resistance, … or will the concrete using aggregates with more fines be more difficult to obtain a certain workability.

### 3.6 Quality assurance & certification

Quality assurance plays an important role in the Belgian construction sector. Most public clients require the “BENO R label” as proof that the concrete delivered on site is conform to the Belgian concrete standard. A specific technical prescription document exists for this third party certification. However, this scheme is not developed for concrete using recycled aggregates. A working group is now developing these specific requirements, with special attention for the following aspects. First of all, the quality of the recycled aggregates should be guaranteed & certified as well. In addition to the requirements of NBN B15-001, the maximum diameter of the aggregates is limited to 20mm. This is done to avoid
carbonation tests on the concrete level. Also specific requirements are imposed on the storage and processing of the recycled aggregates, in order to avoid contamination with other fractions, and in order to guarantee a controlled humidity environment (to take into account in the mixing process). A distinction is made between concrete types and families. There are the ‘basic’ or standard concrete types on the one hand, requiring the same quality control aspects as conventional concrete without recycled aggregates, expanded with specific Initial Type Testing tests for each concrete composition in terms of compression strength and workability. On the other hand, the application domain where durability is an issue, more tests are required in the initial phase, specifically on carbonation resistance and frost-thaw resistance of the concrete with recycled aggregate.

4 FURTHER WORK

4.1 RILEM / fib collaboration

Recently a new RILEM TC on the “Structural behavior and innovation of recycled aggregate concrete” was approved. The terms of reference of this TC point to collecting case studies regarding environmental design of concrete and concrete structures. Background research will be studied and carried out as well. The proposed TC members are from academics, universities, national labs, engineers, research students and collaboration with fib is targeted. Indeed a Memorandum of Cooperation (MoC) was signed recently at the RILEM general council in Melbourne between RILEM and fib, the main aim is to develop further synergy in the field of concrete and especially to set up joint committees. It is nice to see that the TC on “structural behaviour and innovation of Recycled Aggregate Concrete” RAC chaired by Jianzhuang XIAO will be one of the first committees putting this RILEM-fib MoC in practise.

4.2 EC Initiatives

Environmental legislation in Europe has also been putting the topic of Construction and Demolition waste again high on the agenda especially under impetus of northern Europe where now up to 90% of all the C&D waste is being recycled as part of a focus on sustainable construction. Most recently in its communication on ‘Resource Efficiency Opportunities in the Building Sector’ COM (2014)445 the commission focusses on the establishment of a common European approach to assess the environmental performance of buildings throughout their lifecycle, taking into account the use of resources such as energy, materials and water. The Commission announces in this context to support projects in H2020, COSME and Life+ related to research, innovation and demonstration showcasing how collaboration between public authorities and the private sector can create viable markets with recycled materials.

5 TOWARDS A CIRCULAR ECONOMY

A circular economy approach is based on maintaining the value of the materials and the energy used in products in the value chain for as long as possible, and on minimizing production of waste and consumption of resources. It therefore promotes competitiveness, innovation, a high level of protection for human beings and the environment, and brings major economic benefits, thus contributing to growth and job creation. A circular economy
The model foresees the application of systemic approaches and interventions at several levels, e.g.

- new methods for the production and use of materials in order to reduce the quantity of materials produced and the amount of energy consumed, as well as
- new methods for the lengthening of the use phase of products the, redesign of products for easier maintenance, repair, upgrading, remanufacturing or dismantling, and recycling (i.e. eco-design); reducing the use of materials which are hazardous or difficult to recycle; the development of markets for secondary raw materials;
- incentivising and supporting waste reduction and high-quality separation by consumers; incentivising separation, collection systems that minimise the costs of recycling;
- improving cross-sectoral cooperation and facilitating clustering of activities to prevent by-products from becoming wastes (industrial symbiosis); and
- the development of new business models (e.g. renting or sharing versus buying).

The building and construction sector can contribute to this process as an important player. The construction industry is an ideal field for the use of recycled materials. This is because in this industry large quantities of materials are consumed, and even in cases where such materials are not environmentally inert it might be possible, by means of various substances or processes, to permanently immobilize hazardous components, however due attention is then also to be given to the reuse of such immobilize materials. The characteristics of new products can be equal to or even better than those which are conventionally produced. Cooperation with waste owners and the planning of processes in which by-products are formed is of great importance in order to produce good-quality construction products. The present legislation permits such an approach: materials are not discriminated against, regardless of their origin or denomination; what matters are simply their characteristics, their potential uses, and their environmental footprint. In 2013, a new and additional essential requirement (No. 7) was introduced in the revised version of the Construction Products Regulation, which promotes the efficient and sustainable use of resources. Also, the Waste Framework Directive 2008/98/ES introduced a new waste management hierarchy, where reuse and recycling are the priorities. The big challenge for the building and construction sector is therefore how to find the most realistic and balanced option for the utilization of recycled waste into new innovative products, in terms of both the nature and quantity of the waste, taking into account the location of potential end users, the current market demand for such products, their environmental impacts, the costs of recycling processes, and, in particular, the feasibility of applications for standard civil engineering technology, so that new materials can be produced, transported, installed and recycled again using existing civil engineering machinery and technology. The higher added value of a new product or environmental technology is always welcome, but it is not an absolute priority. It is also important that the final product is not harmful to the environment, and that the used solutions are robust and can be reused also in a next life cycle.

Other potential areas where the construction sector could become a role model in the circular economy approach are, for example: energy and material efficient innovative buildings which are easy to build, use, maintain, dismantle and recycle, which produce rather than use energy and water, new business models based on the renting and sharing of systems without
jeopardizing consumers' interests in terms of costs, protection, information etc., improvement and implementation of green public procurement principles, improving information about the sustainability of construction products based on LCA assessment (Environmental Product Declaration), life cycle thinking in building design (e.g. design for de-construction), awareness raising about de-construction and redesign among users and decision-makers to ensure that construction products are recycled at their end of life, improved techniques for selective demolishing, improvement of health and safety in the construction sector…. A lot of these issues will require as well an appropriate processing techniques as a good traceability of the life cycles of the materials used.

6 TRACEABILITY

Crucial for a circular economy approach to demolition waste will clearly be the level of quality control which can economically be installed. Tracking of construction and demolition waste and consequently the quality of the whole recycling process has long been a bottleneck in the industry. Together with relevant stakeholders such as demolition contractors, transportation firms, recycling companies, environmental agencies and researchers the Flemish Confederation of Construction Contractors has taken the initiative to put a demolition management organization into practice: TRACIMAT. Tracimat aims to promote a selective demolition process and to trace the origin of the demolition waste delivered to a recycling plant. This can guarantee the quality of the demolition waste delivered for further processing and recycling. The whole tracing starts with the preparation of a strict demolition planning prepared by an expert. This expert should always be a neutral party independent of the demolition contractor. Such an approach clearly has great potential not only with respect to guaranteeing the environmental quality of the waste delivered on site but with respect to its technical qualities. It is expected that this approach will give a further boost to the recycling of C&D waste and implementing circular economy aspects in the construction sector.

7 CONCLUSIONS

The paper gives an overview of general consideration with respect to recycling as well as current initiatives and projects in Belgium, leading to a well-developed framework for the application of recycled aggregates in concrete. The documentation and investigation in practical experiences and large scale projects is complemented by work on procedures and standards that keep evolving towards a more broadened application domain for recycled aggregates in concrete. Both technical aspects together (standards & practical experience) combined with a well-controlled management system are believed to be essential to lead to a higher confidence level and a wider use of recycled aggregates in the concrete sector.

REFERENCES


SUSTAINABILITY EVALUATION OF THE CONCRETE STRUCTURES

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Abstract. Sustainability is a new and settled concept in advanced societies, even though the term is sometimes inappropriately used, which may represents a threat. For this reason it is suitable to supply methods of measuring sustainability. Given that the sustainability concept involves distinct requirements and criteria, it seems reasonable to use multi-criteria methods in the decision making process. This work shows a brief review of such methods applied to concrete structures. The MIVES method is applied in this work for assessing the sustainability of the concrete structures. Three examples are proposed to show the capacity of the method. One of them is based on the general application of the Spanish Structural Code for Concrete Structures (EHE-08), and the other two assess the sustainability of reinforced concrete columns by adopting distinct type of concrete and building procedures.

1 INTRODUCTION

Sustainability approach, if implicit in the history of human development, has had a remarkable boom in the last three decades after the definition of the term within the framework of the United Nations (Bruntland Report, 1987).

The scope of sustainability, with its three pillars (economy, environment and social), covers the activities of the humanity and in its various forms (goods, services, etc.). Obviously, it is included the construction sector. In this sector, concrete as a structural material is one of the most used (Sakai, (2009), Ahmad and Saker (2014)) and one of the cheapest.

However, from the environmental point of view, the concrete, mainly by the contribution of cement, it is one of largest producers of CO₂ emissions into the atmosphere. The global production is approximately 5-6% of the total CO₂. Distinct actions have been developed for reducing this contribution of CO₂ (Metha, 2009): a) optimization of structural design, by incorporating an innovative engineering that reduce dimensions of elements, b) selection of more advanced ages as contractual age (56 or 91 days) to minimize the amount of cement, as it is already done in dams, c) advances in the binders that will reduce the content of clinker.

From the social point of view, the perception of the sustainability will be different, depending on the situation of the observer, both from a standpoint of social position, and depending on the general circumstances of the country. Thus, in a developing country, a new cement factory creates new jobs, but under the view of a developed country the same fact can be received as negative for the environment.
As a summary, from the point of view of sustainability, there are advantages and disadvantages in the use of concrete as the building material, both in construction in-situ and precast elements. Given that, the assessment of the sustainability of each alternative proposed by the construction sector seems a promising way for the decision making process.

In concrete structures, once fulfilled the functional requirements and safety, progress in the pillars of sustainability measures is uneven. From the economic point of view, historically there are numerous methods and the level of development is high. Latest environmental methods are incorporated, whether of general type: referring to several as more specific type (ACV, and others). The lowest level of progress, at the level of measures, occurs in the social pillar, although there are jobs in that direction.

This is usually done in a disaggregated way without integrating the set in an indicator for measuring sustainability. To address the issue jointly, the multi-criteria methods can be a reasonably approach to measure the indicators, variables or attributes.

The main objective of the present work is to highlight the importance of measuring the sustainability, as a tool for comparing alternatives. In addition, some examples are presented for showing the possibilities of the multi-criteria methods in the assessment of the sustainability.

2 CURRENT SITUATION AND TRENDS

Nobody disputes that the world changes quickly, exponential somewhat, and that the social, environmental and economic changes of the past 25 years, following the report of UN (1987), have led to a change of paradigm in decision-making at all levels and, of course, also in the sector of the construction, in relation to the project construction and the operation of our infrastructure. The concept of sustainability has become an aspect to consider that it can even influence the construction of the infrastructure.

The new paradigm in decision making process includes the incorporation of the point of view of the actors involved and affected by new construction or infrastructure. In this sense not only the promoter (public or private) and the technician decide, but also rather the opinion of society and its benefit come a growing weight.

When designing a structure, apart from the classical requirements, usually used: safety, durability, functionality, etc., it should be considered the requirements of sustainability: economic, environmental and social aspects. This process represents a disaggregated approach for measuring the sides of a polyhedron, which each face is every one of these requirements. However, it is possible to move forward and consider a joint assessment that represents the volume of the polyhedron (each of those requirements is a side).

Some improvements have been done in distinct directions, as evidenced Jato et al. (2014), showing by the increasing number of published papers in international journals focused on this topic. The direction of action may be different; on the one hand, the measure of the classical parameters, such as safety, durability, economic aspects, and even environmental and social parameters. On the other hand, the attempt for integrating measures of each plane into some set value, demonstrating, in any case, the need to measure to place the structure in its context and move forward.

The way to do these advances, in a multi-criteria approach, includes the incorporation of new requirements, since the methodology used in one of them. For example, from the
economic point of view is often to work on cost/benefit issues and intends to assess the economic value of the social and environmental aspects. This way faces great difficulties and disagreements among experts.

An additional way, in areas with less experience, is to start the evaluation with the use of checklists, regarding, for example, to social or environmental aspects. In such approach the models type BREAM, GBC, LEED and others may be a reference. It is also possible to perform more complex models, from the point of view of environmental such as Life Cycle Assessment (LCA) or other associated calculations, but in practice, they are difficult to apply to the standard type projects. There are steps of integration of similar methods at various levels, for example integrate Life Cycle Cost (LCC) and ACL in civil structures by means of Analytic Hierarchy Process (AHP), Kim et al. (2013).

There are distinct integration methods in the sector of the construction such as the MCDM multi-criteria methods: ANP, DEA/ELECTRE, TOPSIS, AHP, PROMÉTHÈE and other (Jato et al., 2014), although each of them is only used in one geographical framework or habit of work determined. The approach is based on the treatment of problems with homogeneous alternatives, i.e. of the same type.

These advances are also shown by the regulations. An example is series of European standards that are developed by the CEN/TC 350 (see Figure 1), used in Europe for assessing the sustainability building construction. This Committee discusses the environmental efficiency of the buildings, the various aspects on the analysis of the life cycle of the buildings and the problems at the level of the products. The AEN CTN 198 "Sustainability in construction" Committee (Tenorio and Vega, 2011) follows the CEN/TC 350 regulations and proposals in Spain. In general, while the concept is clear, in practice there is a certain imbalance that shows a bias towards environmental issues, confusing a part to the whole, when the analysis must be global, integrated manner with other sustainability requirements.

With considered bias, not surprising that the concrete is now penalized by the high value of CO₂ emissions and its moderated recyclability. In addition to this bias, different studies do not defined properly the boundaries of the system by not including relevant aspects of transportation.

3 MIVES METHOD

The above exposed shows the convenience to find multi-criteria methods (MCDM) that allow incorporate heterogeneous and, to the same time, flexible alternatives for measure the sustainability. These should be versatile for adapting to the different situations of projects (Basic projects or projects construction very elaborate) and to the different complexities of the structure (simple or very complex). The integrated model of value for sustainable assessments (MIVES) satisfactorily meets this challenge.

MIVES is a multi-criteria method that began with a research project¹ in 2002, led by Prof. Antonio Aguado, in coordination with UPC, UPV and TECNALIA. Then, other universities (UaC, UIC and UPM) were incorporated to the project. As a result, since 2005, the year in which the first PhD thesis of this line was defended, a total of 16 additional PhD

theses have been presented, in four Spanish universities. In addition, a significant number of papers and communications to congresses have been made.

The MIVES method responds to a classical structure of tree structured, usually in three levels (requirements, criteria and indicators), as shown in Figure 2. The sustainability requirements are economy, environmental and social aspects. The involved requirements and criteria, allow structuring the decision and easy the communication to third parties. The indicators allow measuring both variables and attributes.

In the evaluation is conversely as described in Figure 2. For the evaluation of each indicator a function of value previously established and agreed is used. It is a sigmoid of four parameters shape function; details may be consulted in Alarcón et al. (2011). The result of the measured indicator \( V_{lk} \) (using variables or attributes), enters the abscissa (see Figure 2) and, by using the value function the value of the indicator is obtained (values from 0 to 1).

The value of a generic criterion \( V_{Cj} \) is supplied by Equation 1. The Equation 1 shows the addition from \( K = 1 \) to \( n \) (where \( n \) is the number of indicators that there are in the criterion) of the product of the value of each indicator group \( V_{lk} \), by the weight associated with the same \( w_{lk} \).

\[
V_{Cj} = \sum_{k=1}^{n} w_{lk} \cdot V_{lk} \quad [\text{Eq.1}]
\]

Same approach is used in the assessment of the requirements, obtaining the value of a requested generic \( V_{Ri} \) in the Equation 2, which expresses the sum from \( j = 1 \) to \( i \) (being \( i \) the number of criteria that are in the requirement) of the products of the value of each criterion of

![Figure 1: Approach to the sustainability proposed by the CEN/TC 350 Committee.](image-url)
the Group \((J_{cv})\), by the weight associated with the same \((w_{Cj})\).

\[
V_{Ri} = \sum_{j=1}^{i} w_{Cj} * V_{Cj}
\]  
[Eq.2]

Finally, the rate of sustainability of a building \((SI)\) is obtained by the sum of the dimensionless values of each of the requirements \((V_{Ri})\) multiplied by the weight corresponding to each of them \((w_{Ri})\) as shown in equation [3]. The subscript \(i\) represents the number of established requirements, which is 3 (economic, social and environmental) for sustainability studies.

\[
SI = \sum_{i=1}^{I} w_{Ri} * V_{Ri}
\]  
[Eq.3]

The assessment of the indicators can be obtained deterministically or with probability approaches. The method allows a separation of components, for example, in the case of a building: structure, façade, foundations, facilities, etc., performing the assessment of the sustainability based on the contribution of each component of the building. For the assignation of weights, it can be used distinct methods. MIVES usually adopts directly assignation or AHP, in both cases, previously agreed, before studying alternatives.
The versatility shown by the MIVES method allows facing problems of decisions from different points of view, even with high plurality. The MIVES Method may evaluate highly technical aspects of construction, the social perception of a company, the selection of sites, the ranking of the staff of University departments, or prioritization of investments. The result of the sustainability index (SI) is a dimensionless value.

The above previously shown can directly apply when homogeneous alternatives are being evaluated. Nevertheless, in complex problems with heterogeneous alternatives, a phase called homogenization that fit the perception of the decision-maker between these heterogeneous alternatives is required (Pardo y Aguado, 2016).

4 CONSIDERATIONS ON A PRACTICAL USE OF THE METHOD

The method leads to good results if the decision tree is correctly built. It is of the highest importance that the decision taking party takes part in the definition of requirements and weights. Every theoretical development must be made without any alternative solutions to be independent when choosing indicators.

With respect to indicators it is advisable that only relevant ones should be considered. We propose that only indicators that have a relevance of more than 5% should be considered. Also only indicators that take into account differences for the alternatives that are under study should be considered.

The method does not need to consider many indicators but just the principal indicators that are relevant for the comparison. With this procedure we can save time in the calculation and higher accuracy because otherwise principal indicators may be shadowed by a large quantity of less important indicators.

Value functions of each indicator must be defined before the alternatives are discussed. Better results are obtained if are defined by consensus of experts with the help of seminars.

Of course, when we compare alternative solutions, limits of the system have to be homogeneous and consequently have to introduce associated transportations to resources movements like aggregates, cement and other products. It is also a key factor to consider decision taking party for each aspect like for example the selection of a typology for a construction decision may be different if we only consider promoting agency, contractor, end user or just a citizen. The decision is not a generic one but has to take into account the point of view of the final decision party and also economic, social and environmental aspects when the decision is taken.

Decision tree has to incorporate all aspects to be considered. If we try to measure resilience of a solution or the survival against unknown changes (like climate change) this concept has to be included explicitly in the tree.

On the other hand, it is important to state at the early stages which are boundary conditions that the alternatives that will be studied have to comply with. If some of the boundary conditions are not fulfilled, then the alternative cannot be tested. Also if all boundary conditions are fulfilled then what it has to be evaluated are the increments over the base value. For example, it protection time against fire as a starting point is 120 minutes and one of the alternatives guarantees 150 minutes, does these supplementary 30 minutes have to be considered? In this same direction are other aspects related with service life of a structure that
A decision tree can be defined to evaluate which alternative solution is better to make better a process or production from the sustainability point of view. For example, the ones already said in point 1 by Metha (2009) or Sakai (2009). Also other aspects related with them in a decision tree or in a general point of view. We see then that when we include a decision branch for the use of resources in environment part of the tree we are also considering design aspects that reduce the use of resources (aggregates, etc.). Other example can be when considering different types of binding components, that can reduce the amount of clinker (Josa et al. 2005 and Josa et al. 2007) it is also taken into account in emissions branch (i.e. CO₂) within the environmental branch.

5 EXAMPLES

To make evident the use of the method we describe some examples of it application that cover several aspects in decision taking:

- Sustainability Contribution Index for Structures (ICES) used in Spanish Normative EHE08
- Sustainability evaluation of precast products made of different materials like concrete or steel
- Selection of building process taking into account sustainability

Sustainability Contribution Index for Structures (ICES)

Current Spanish Normative on structural concrete (EHE-08) (M.F., 2008) includes Annex 13 (not mandatory) with the title Indice de contribución de la estructura a la sostenibilidad (Contribution index of the structure to sustainability). It has been in international normalization for structures that at this level a sustainability index is included. ICES evaluation is done after structural design is developed and after the comply to structural and functional requirements. In that 1st version it was done under the environmental aspect only. It also included other aspects from the social point of view and indirectly from the economic point of view.

With this development as are explained by Aguado et al., (2012) one could obtain the ISMA or Environmental Sustainability index for the later incorporation of social aspects, as for equation 4.

\[
\text{ICES} = a + b \times \text{ISMA} \quad [\text{Eq.4}]
\]

Where: \(a\) is a social contribution coefficient, that considers several factors in this aspect and \(b\) is a coefficient for the extension of service life of the structure.

The decision tree for this example is described in Figure 3. We can see that the different branches considered in the ISMA decision tree includes measurements as previously described (Metha, (2009), Sakai (2009)). In the same figure the different weights considered for each branch can be seen and also that the decision tree is not large to make every aspect.
decisive. The tree is designed so that homogeneous solutions, that are built with different processes, can be evaluated (in situ or with precast elements). In both solutions also different transport distances can be considered for materials to take into account emissions and costs.

After the publication of the structural concrete normative the same idea has been implemented in the structural steel normative (EAE-10) and in the composite structures normative. Also in the revision of the not published structural code also is considered the same model with a slightly more precise different approach.

**Figure 3:** Decision Tree for EHE-08 (Sources: M.F. (2008) y Aguado et al. (2011))

**Sustainability evaluation of precast products made of different materials**

Other problem that can be studied with this method is the sustainability assessment of precast elements it is different solutions with constructive differences or different materials (concrete, steel, plastics, etc.). For these solutions an example of the evaluation of the sustainability of sewerage pipes with different alternative solutions (Viñolas (2011), de la Fuente et al., (2016)). There are also other examples for the evaluation of sustainability of wind turbines (de la Fuente et al., 2016).

In Table 1 are presented different solutions of pipes with the following names: HM: Unreinforced concrete R class, HA: Reinforced concrete, class IV, PP: Structural Polypropilene, class SN8, PVC: Compact Vynil Polycloride, class SN, PVC: Compact Vynil Polycloride, Class 10.
In Table 2 it can be reviewed the decision tree with the considered assigned weights. In the table it has been considered the three columns of sustainability, incorporated for this example. In this case the additional functionality has been incorporated in the decision tree.

<table>
<thead>
<tr>
<th>Alternativa</th>
<th>Diámetro ext.</th>
<th>Diámetro int.</th>
<th>Espesor</th>
<th>Peso (kg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HM 400 mm</td>
<td>520 mm</td>
<td>400 mm</td>
<td>60 mm</td>
<td>240,00</td>
</tr>
<tr>
<td>HA 800 mm</td>
<td>1000 mm</td>
<td>800 mm</td>
<td>100 mm</td>
<td>705,00</td>
</tr>
<tr>
<td>HA 1200 mm</td>
<td>1480 mm</td>
<td>1200 mm</td>
<td>140 mm</td>
<td>1395,00</td>
</tr>
<tr>
<td>HA 2000 mm</td>
<td>2430 mm</td>
<td>2000 mm</td>
<td>215 mm</td>
<td>3650,00</td>
</tr>
<tr>
<td>PP 450 mm</td>
<td>450 mm</td>
<td>400 mm</td>
<td>50 mm</td>
<td>8,32</td>
</tr>
<tr>
<td>PVC 800 mm</td>
<td>800 mm</td>
<td>748 mm</td>
<td>26 mm</td>
<td>87,87</td>
</tr>
<tr>
<td>PE 1200 mm</td>
<td>1200 mm</td>
<td>1030 mm</td>
<td>85 mm</td>
<td>67,50</td>
</tr>
<tr>
<td>PRFV 2000 mm</td>
<td>2047 mm</td>
<td>1958 mm</td>
<td>44,5 mm</td>
<td>383,66</td>
</tr>
</tbody>
</table>

Table 1: Geometrical data of the alternatives.

Table 2.- Decision tree for sewerage pipes made of different materials.

Results arising from this study are presented in Figure 4, in which results from 3 different scenarios are presented (A: favourable conditions, B: Intermediate conditions, C: Unfavourable conditions). They are associated to some of the environmental indicators (% of recycled water, sensibility to environment in production plant). We can observe that, for small diameters (400mm) the solution with structured polypropylene, class SN 8 (PP) is the solution that obtains the best result while for larger diameters reinforced concrete solutions obtain the best results and are clearly the best for large diameters (2000mm).
Analizing the results, it can be observed that when diameters are small (400 mm or less), plastic alternatives (e.g., polypropylene) have similar evaluations to concrete alternatives. Whether an alternative is more sustainable than another depends mainly on the production plant, that is, on the scenario considered. In this same graphic, it can be visualized that although in terms of average, the concrete alternative of 400 mm diameter is slightly worse, this difference is very small and depending on the type of concrete or polypropylene plant producing the alternative, the sustainability index of one alternative against another can be greater, similar or less depending on the case.

All these results were presented in the seminar held in Zaragoza, which is explained in the previous chapter. Experts’ perception in sanitation matters was consistent with the obtained results. As with the results when the diameters are small (400 mm or less) depending on the case, plastic or concrete pipe can be placed. However, when the diameters are 800 mm or greater, the alternatives chosen, in most cases, are concrete pipes due to their better overall performance.

The ability to perform this analysis through a seminar with experts in water matters reinforces even more the results obtained. Since, if non-compliant results with perception always these data that could appear more strange can be reviewed.

Other example is for the evaluation of sustainability in different process of construction solutions. To show this solution an example is now described. The example is based on the comparison of different solutions of building columns with the following parameters: characteristic strength of concrete (hormigón (HA-25, HA-50 y HA-75), compacting of concrete (Self compacting, vibrated), geometry of the cross section (rectangular, circular). More information can be found in (Duran (2011) y Pons y de la Fuente (2014)).

For the sake of comparison the forces resisted are the same in all cases and also steel is B500SD (characteristic yield stress f_yk of 500 N/mm² and Young Modulus E_s of 200.000 N/mm²). In Table 3 it is shown different combinations from the study and the dimensions of the columns.

To make the different alternative solutions comparable regarding concrete specific and not generic corresponding to the different components used in each type of concrete depending on the compaction system and strength as it is shown in Table 4. In this table it can be checked how aggregates differ and also the fines content. Other data with relation to formwork, costs and other variables can be found in Duran (2011) y Pons y de la Fuente (2014).

The defined decision tree in this evaluation and the weights adopted are presented in Table 5. Both decision tree and weights were decided in a seminar with an expert committee. They considered which indicators were fundamental to decide the alternatives.
<table>
<thead>
<tr>
<th>Type $f_{ck}$</th>
<th>Compacting system</th>
<th>Section</th>
<th>Dimensions (cm)</th>
<th>$A_c$ (mm$^2$)</th>
<th>$A_s$ (mm$^2$)</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>C25</td>
<td>Vibration</td>
<td>Rectangular</td>
<td>40x40</td>
<td>158743</td>
<td>1257</td>
<td>C25/V/S</td>
</tr>
<tr>
<td></td>
<td>Self Compacting</td>
<td>Circular</td>
<td>50</td>
<td>195331</td>
<td>1018</td>
<td>C25/V/C</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rectangular</td>
<td>40x40</td>
<td>158743</td>
<td>1257</td>
<td>C25/SC/S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Circular</td>
<td>50</td>
<td>195331</td>
<td>1018</td>
<td>C25/SC/C</td>
</tr>
<tr>
<td>C50</td>
<td>Vibration</td>
<td>Rectangular</td>
<td>30x30</td>
<td>88743</td>
<td>1257</td>
<td>C50/V/S</td>
</tr>
<tr>
<td></td>
<td>Self Compacting</td>
<td>Circular</td>
<td>35</td>
<td>96211</td>
<td>792</td>
<td>C50/V/C</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rectangular</td>
<td>30x30</td>
<td>88743</td>
<td>1257</td>
<td>C50/SC/S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Circular</td>
<td>35</td>
<td>96211</td>
<td>792</td>
<td>C50/SC/C</td>
</tr>
<tr>
<td>C75</td>
<td>Vibration</td>
<td>Rectangular</td>
<td>25x25</td>
<td>61243</td>
<td>1257</td>
<td>C75/V/S</td>
</tr>
<tr>
<td></td>
<td>Self Compacting</td>
<td>Circular</td>
<td>30</td>
<td>70685</td>
<td>679</td>
<td>C75/V/C</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rectangular</td>
<td>25x25</td>
<td>61243</td>
<td>1257</td>
<td>C75/SC/S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Circular</td>
<td>30</td>
<td>70685</td>
<td>679</td>
<td>C75/SC/C</td>
</tr>
</tbody>
</table>

Table 3.- Alternatives considered in the example.

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>HA-25</th>
<th>HA-50</th>
<th>HA-75</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>AC</td>
<td>B</td>
<td>AC</td>
</tr>
<tr>
<td>Cement</td>
<td>CEM</td>
<td>I</td>
<td>(kg)</td>
</tr>
<tr>
<td>Additions</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filler (kg)</td>
<td>-</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Nano silica (kg)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Aggregates (kg)</td>
<td>1851</td>
<td>1725</td>
<td>1803</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>145</td>
<td>175</td>
<td>180</td>
</tr>
<tr>
<td>a/c</td>
<td>0.55</td>
<td>0.58</td>
<td>0.4</td>
</tr>
<tr>
<td>Pozzolith (ligno)</td>
<td>% spc</td>
<td>0.7</td>
<td>0.9</td>
</tr>
<tr>
<td>(kg)</td>
<td>1.8</td>
<td>2.7</td>
<td>1.4</td>
</tr>
<tr>
<td>Glenium (PCE)</td>
<td>% spc</td>
<td>0.3</td>
<td>1.5</td>
</tr>
<tr>
<td>(kg)</td>
<td>0.8</td>
<td>4.5</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Table 4.- Components for each type of concrete

<table>
<thead>
<tr>
<th>Requirements</th>
<th>Criteria</th>
<th>Indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1. Economic (50%)</td>
<td>C1. Construction costs (67%)</td>
<td>I1. Building costs (85%)</td>
</tr>
<tr>
<td></td>
<td>C2. Efficiency (33%)</td>
<td>I2. Non acceptance costs (15%)</td>
</tr>
<tr>
<td>R2. Environmental (33%)</td>
<td>C3. Emissions (67%)</td>
<td>I3. Maintenance (60%)</td>
</tr>
<tr>
<td></td>
<td>C4. Resources consumption (33%)</td>
<td>I4. Habitability (40%)</td>
</tr>
<tr>
<td>R3. Social (17%)</td>
<td>C5. Negative effects on the producer industry (80%)</td>
<td>I5. CO$_2$ Emissions (100%)</td>
</tr>
<tr>
<td></td>
<td>C6. Effects to third party (20%)</td>
<td>I6. Concrete consumption (90%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I7. Steel consumption (10%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I8. Workers’ inconveniences (20%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I9. Workers’ safety (80%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I10. Environment nuisances (100%)</td>
</tr>
</tbody>
</table>

Table 5.- Decision Tree and considered weights
The results of the Sustainability Index (SI) of each one of the alternatives are presented in Table 6. It can be observed that are all with a minimum value of 0.558 (corresponding to C25, C25, vibrated and square cross-section) and a maximum value of 0.852 (corresponding to C75, self-compacting and circular) because in a way to the reduced use of raw materials and to the increment of usable space in the building. This corresponds to the higher strengths used in high-rise buildings.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>SI</th>
<th>Ref.</th>
<th>SI</th>
<th>Ref.</th>
<th>SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>C25/V/S</td>
<td>0.608</td>
<td>C50/V/S</td>
<td>0.662</td>
<td>C75/V/S</td>
<td>0.707</td>
</tr>
<tr>
<td>C25/V/C</td>
<td>0.558</td>
<td>C50/V/C</td>
<td>0.716</td>
<td>C75/V/C</td>
<td>0.794</td>
</tr>
<tr>
<td>C25/SC/S</td>
<td>0.623</td>
<td>C50/SC/S</td>
<td>0.717</td>
<td>C75/SC/S</td>
<td>0.771</td>
</tr>
<tr>
<td>C25/SC/C</td>
<td>0.564</td>
<td>C50/SC/C</td>
<td>0.768</td>
<td>C75/SC/C</td>
<td>0.852</td>
</tr>
</tbody>
</table>

Table 6. Result of Sustainability Index (SI)

On the other hand it can be checked that in general the alternatives that use self compacting concrete result in higher SI values that the ones that use vibrated concrete. This corresponds in a way to aspects that have to do with non-quality costs, for voids in the base of the columns due to its compaction difficulties.

Lastly square columns result in higher values when concrete strengths are low (C25). When concrete strength is higher (C50 and C75), circular alternatives allow better results for sustainability due mainly to construction costs.

6 CONCLUSIONS

We can extract the following conclusions from the examples explained above:

- Sustainability as a concept is a well-established idea in our society and it is slowly being considered in the construction sector where it is difficult to produce changes. Evaluation methods are a necessary tool that will allow producing advances in the process.
- MIVES method is a very flexible tool to evaluate sustainability in structures from different points of view and different levels. It allows studying and evaluating in situ or precast solutions if they are efficient to solve a certain problem.
- Each decision requires a specific decision tree because if only a general is adopted it will not evaluate detailed aspects. This is shown in the examples described, each one on a different aspect.

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SUSTAINABILITY OF BRIDGE STRUCTURES. INDICATOR SYSTEM

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Key words: Sustainability, Bridge Structures, Railway structures, multy-criteria method, MIVES

Abstract. The impacts on ecosystem services due to human activity are now a fact: the loss of biodiversity, the climate change, desertification, health and illness, the broad social and economic differences, … coupled with an increasingly committed society required to give an immediate response to the new challenges today in short and long terms.

The amount of waste generated by the construction industry and the resources consumed (energy, raw materials) in the life cycle (construction, operating, maintaining, deconstruction) contribute to the enhancement of human ecological footprint.

There are new sustainability goals in civil engineering projects. The techniques to reach these goals in construction sector have been analyzed and a methodologic frame of sustainable management have been proposed.

In the recent years, there have been different attempts to respond to the need of more sustainable infrastructure projects from different perspectives or through the creation of certain instruments. One of these instruments is the Sustainability Assessment Methods, which have been already implemented on buildings, but are still taking their first steps in its implementation to the infrastructure project scale.

An indicator system is proposed in this project for assessing sustainability in railway infrastructure projects from early stages (previous analysis of alternatives) to allow classification of the different alternatives according to the impact on the environment, society and economy, and thus to select the most sustainable alternative. After developing an extensive study of the state of knowledge on sustainable construction worldwide and, according to exiting initiatives, it the sustainability assessment of railway infrastructure projects is proposed using a set of criteria and indicators.
In order to elaborate this proposal, a through comparative analysis has been undertaken about the tools and methods more widely used at the present, which are LEED for Neighbourhood Development, BREEAM for Communities and MIVES. Also other tools have been revised, with lower or more local uptake, which pose interesting or complementary approaches.

The indicator system is also applied to a case study, Spanish bridges of railway infrastructure, through a multi-criteria analysis, to identify existing constraints to the implementation of sustainability criteria from early stages considering the project life cycle. The indicator valuation in later stages and their possible application in the comparison between projects will be taking into account the interest of controlling and monitoring.

1 INTRODUCTION

To satisfy present needs without compromising future generation needs (Bruntland report, UN 1987) defines Sustainably concept. During the last decades this idea has been more popular until it has today a relevant position in decision taking within developed countries. Basic criteria are now being applied to building planning, design and construction. These criteria are also slowly being applied to developing countries.

In building there is still a need for indicators and enough confirmed data with respect to sustainability evaluation. Anyhow there are some indicators that give some information that allows evaluation of economic and environmental criteria towards sustainability.

There are important challenges in building construction regarding the evaluation of materials and its origin. Also aspects related to raw materials, energy consumption and energy savings, pollution, waste management, materials recycling and other are still to be evaluated more in detail and with a new perspective.

We are living in a constant change: globalization, huge innovation technological rate, quicker and more efficient communications, virtual environment, competition at every level, transformation in time and space make that sustainability has to be understood in a global dimension (Social, Environmental and Economic) as it is defined among other in “Sostenibilidad y Construccion” (Ache).

Nowadays there are several applications that have been developed to evaluate second energy certification or environmental certifications that promote energy efficiency in all stages of construction. These applications are manly dedicated to building construction but there are not any of these dedicated to infrastructure construction.

2 REFERENCES

Since 1987 the report “Our Common Future” also known as Bruntland report informed on how environment could affect the future of humankind and the need of a more sustainable future. After there was the Rio Conference in June 1992 in Rio de Janeiro organized by UN in which it was produced the “Rio Declaration on environment and development” the concept of
sustainable development was clarified. After in Kyoto in 1997 there was a convention of climate change on the need for reduction of emissions.

In recent Paris Conference (2015) on climate change (CMNUCC) there has been for the first time a historic agreement over the methods to reduce climate change that will be mandatory for at least 50 countries that represent at least 55% of global emissions of greenhouse gases. The agreement will be applied from 2020 because until that date Kyoto protocol will be active.

In the convention European Union has suggested a compromise to reduce emissions by 40% in 2030 compared to 1990. It has also has been developed a normative framework for sustainable development.

In Spain strategy is in the same direction as the European Union for next ten years. Spanish Government is willing to deepen its productive model with a Ley de Economia Sostenible (Sustainable Economy Law) that establishes the framework for the next decades.

UE2020 strategy states as key priorities: to create a competitive economy, connected and more respectful to environment, not only creating new industries but also accelerating and modernising existing industrial sectors. At the same time to stimulate more efficient use of resources, in particular of energy and also the application of new technologies which are more respectful with environment and to promote new employment and services in these activities. This perspective places the bases to transform production models to more efficient models in the use of natural resources and human activity.

This strategic framework, with the help of all the society, will procuce structural changes that will change the way we build and mantain present structures.

3 SUSTAINABILITY IN CIVIL ENGINEERING.

Regarding sustainability concept as described above it is necessary to consider bridges and other infrastructures taking into account the whole life cycle that includes different aspects like Economic, Environmental and Social. Civil structures are designed to a long working life, longer than most of structures and products and therefore it is specially important to consider social, economic and environmental aspects in their life cycle (Gu and Karoumi, 2012)

We must consider the different phases that constitute their life cycle:
  - Design, component manufacturing and materials
  - Construction
  - Use
  - Maintenance and repair
  - Deconstruction and waste management

Sustainability will consider to maintain in the long term the integration of the three principal aspects: Social, Economic and Environmental.
4 MULTICRITERIA METHOD TO EVALUATE SUSTAINABILITY. MIVES

MIVES method (6) consists of the following steps:
- Define the problem to be solved and the decisions to be made;
- Produce a basic diagram of the decision model, establishing all the aspects that will be part of the assessment in the form of a requirements tree that may include qualitative and quantitative variables;
- Establish mathematical functions that make it possible to convert the qualitative and quantitative variables, with their various units and scales, into a set of variables with the same units and scales;
- Define the importance or relative weight of each of the aspects that are to be taken into account in the assessment;
- Define the various design alternatives that could be used to solve the previously identified problem;
- Evaluate and assess those alternatives by using the previously created model; and
- Make the right decisions and choose the most appropriate alternative. In some cases it could be some kind of overlapping between these steps.

In this particular case, the problem to be solved (stage A of MIVES) is to assess the sustainability index of a precast concrete structure or product to be compared with other possible alternatives.

Regarding the requirements tree (stage B of MIVES), this consist in a hierarchical diagram (Figure 1) in which the various characteristics of the product or processes to be evaluated are defined in an organized manner, normally at three levels: indicators, sub criteria, and criteria (36). At the final level, the specific requirements under evaluation are defined and the previous levels (criteria and indicators) are included in order to desegregate the requirements, permitting, on the one hand, (1) having a global view of the problem, (2) organizing the ideas and (3) facilitating the comprehension of the model to any stakeholder involved in the decision process. On the other hand, the tree is useful to carry out the subsequent mathematical analysis.

Afterwards, mathematical elements from the general multicriteria decision theory are used (7) (8) (9) to formalize a methodology to convert the different criteria magnitudes and units into a common, non-dimensional, unit that will be called value (stage C of MIVES). In this sense, it should be noticed that this method accounts for both qualitative and quantitative variables related with the indicators.

In any multi-criteria decision problem, the decision maker have to choose between a group of alternatives (38), this being discrete or continuous. But, in any case, in the field of the precast concrete technology, it is possible to enumerate all the existing alternatives to give response to a specific problem.
MIVES uses Equation 2 as the basis for defining individual value functions $V_i$.

$$V_i = K_i \cdot \left[1 - e^{-\frac{p_{i,x} - p_{i,1,0}}{\mu_i}}\right]$$  

(2)

**5 SELECTION OF CRITERIA. BRIDGE STRUCTURES**

A model has been developed to evaluate sustainability of bridges. The model is based on Mives methodology in which requirements, criteria and indicators are defined along the three axis of sustainable development, Environmental, Social and Economic.

Methodology considers the requirements for the whole life cycle of the structure and allows to incorporate different needs of a project from the different views of sustainable development.

Once all components are defined in a requirements tree, then this tree is constant for different structures that can be applied to it. For each case importance or weight of each indicator can be modified if there is a need to it. The flexibility generated through this system makes it simple that it can be used by public or private members in the construction industry.

In following table 1 it can be seen the indicator system for the evaluation of railway bridges with requirements, criteria and indicators.

---

**Figure 1.** General requirements tree.
### Table 1. Indicator system for railway bridges

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Criteria</th>
<th>Indicator</th>
<th>Units</th>
<th>Value function</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>R₁ Economic</strong></td>
<td><strong>C₁ Total Costs construction time.</strong> (λ₁C₁ = 70%)</td>
<td>I₁ Total costs (Direct + Indirect) (λ₁I₁ = 85%)</td>
<td>Mil €/m²</td>
<td>Decreasing S-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₂ Non quality costs (ISO 9001) (λ₁I₂ = 15%)</td>
<td>%</td>
<td>Increasing S-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₃ Cost. Maintenance. Dismantling costs. (CSM) (λ₁I₃ = 90%)</td>
<td>€/m²</td>
<td>Decreasing Cc-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₄ Resilience. Risk of Disaster + lack of use (λ₁I₄ = 10%)</td>
<td>Uds.</td>
<td>Increasing Cc-shape</td>
</tr>
<tr>
<td><strong>C₂ Operating Cost.</strong> (λ₂C₂ = 30%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>C₃ Raw materials consumption</strong> (λ₂C₃ = 40%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>R₂ Environmental</strong> (λ₂R₂ = 45%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>C₄ Emissions</strong> (λ₂C₄ = 15%)</td>
<td></td>
<td>I₄ CO₂ emissions (ECO). Materials manufactured and transport (concrete ECH) + transport waste (ECV) + dismantling (ECD) (λ₄E₄ = 100%)</td>
<td>TnCO₂-eq/m²</td>
<td>Decreasing Cx-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₅ Total waste (RTA). Manufactured Materials (concrete RGH + steel RGA) + Construction and Maintenance (RCG) + dismantling (RGD). (λ₁₀ = 100%)</td>
<td>Tn/m²</td>
<td>Decreasing S-shape</td>
</tr>
<tr>
<td><strong>R₃ Social</strong> (λ₃R₃ = 15%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>C₅ Sensibility local community.</strong> (λ₃C₅ = 85%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₆ Public information. Public consultation. (λ₁₃I₆ = 20%)</td>
<td>Uds.</td>
<td>Increasing S-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₇ Recruitment and training of local staff. nº local staff (NPL) + nº training courses (NCI) (λ₁₇I₇ = 40%)</td>
<td>Uds.</td>
<td>Increasing S-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₈ Affectation others (MIT) = Construction time (APC) + Maintenance (APS) + dismantling (DPD). (λI₈ = 40%)</td>
<td>Uds.</td>
<td>Increasing S-shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I₉ Resilience. Risks of Disaster + lack of use. (λ₁₉I₉ = 15%)</td>
<td>Uds.</td>
<td>Increasing S-shape</td>
</tr>
</tbody>
</table>
6 MODEL APPLICATION TO REAL CASES

The model has been applied to two different real already built railway bridges that are built with different techniques. One of the bridges is a concrete bridge and the other is a mixed construction bridge, both near Madrid.

The application of the model shows the flexibility of the model that allows to evaluate different solutions with the same tool.

CASE 1º.- CONCRETE BRIDGE OVER M-40 EN MADRID.

Viaduct of 649 m in length, that crosses the Super Sur junction of M-40 and future service roads of M-40. It is a concrete box cross section. The first part of the bridge is an incremental launching bridge and the second is built with a complete scaffolding. (Figure 2)

The bridge is built of 13 spans of 36,85 + 53,60 + 67,00 + 3x60,30 + 67,00 + 53,75 + 4x40 + 30 m and a total length of 649,1 m, from p.k 104+498,383 to 105+147,483. The solution consists on a prestressed constant depth box girder of 4,5 m depth with variable width of the webs and flanges. Webs are inclined with a slope of 1/10.

The viaduct has a vertical distance with the M-40 roads underneath of 7,97 m at its most limiting place. Longitudinal profile has an ascending slope of 1,8% until span 6 and a parabolic connection from PK 104+817. In plan the viaduct is straight in its first part until Pier 5 and from that point a curve that leads to a circle with a radius of 2500 m.

The pier are designed octagonal with dimensions of 3,50 x 3,50 m. Abutments are designed as closed. Abutment E-1 is fixed with the deck and E-2 allow movements with the deck. Foundations are designed with deep pile foundations. Piles are 1,5 and 1,8 m in diameter. The structure is designed for 100 years.
CASE 2º.- COMPOSITE BRIDGE OVER C/ PEDRO BOSCH IN MADRID.

The structure lets the railway pass over the Calle Comercio near Atocha station in Madrid. Railway lines are high speed train and local trains.

The bridge is made of a steel truss at both sides of the tracks connected with steel beams each 3.50m. Over the beams a concrete deck is poured creating a mixed construction. Span is 96.49m within the km points 100+145.117 and 100+241.607. Supports are skewed with regard to the line of the tracks to an angle of 17.75º in abutment 1, E-1 and with a bigger skew angle in E-2 of 56.99º. (Figure 2)

Minimum height over Calle Comercio are over 10.25m in every case and over C5 local railway line es 6.31m. The width of the trusses is constant and equal to 1.50m. On the right bank of the platform there is a configuration of a noise reduction barrier to isolate the traffic noise from the trains to nearby houses. The length of the noise reduction barrier goes to 350m until Atocha Railway station. The design of this barrier is special to be integrated with the environment.

The deck is designed with Corten S355 J2G2W Steel, weather resistant to reduce to the maximum its maintenance. Internal steel elements are protected with a epoxy aluminium paint. Construction has been designed so that the affection to local roads and public are minimum so it interferes as little as possible to normal city life.

CASE COMPARISON

After the application of the model to both bridges a comparison has been made. The application of the model has been done without any modification of the indicators as both are comparable structures.

Comparison of indicators has been done considering m2 as the relevant factor so both structures can have comparable values even if both bridges have different lengths, 129m in the composite bridge and 649m in the concrete bridge.

In Figure 3 it can be seen the values obtained in comparison to the maximum possible value in each of the three different parts of sustainability, Economic, Environmental and Social.
In a second phase a comparison in each of the indicators have been made for both bridges as seen in Figure 4.

Finally a graph indicating each bridge has been done to be able to compare values in each part. This comparison can be seen in Figure 5.
From the analysis of Figure 5 we can observe some conclusions:

- Composite bridge has a lower weight on environmental part, which means that has a value in this part of 70% compared to 35% of the concrete bridge.
- Concrete bridge has a higher value on economic part with a very important result of 88% compared to 39% of the composite bridge.
- Social part is more even in both bridges but composite bridge gets a higher value of 61% compared to the value of 43% of concrete bridge.

We can therefore see that environmental values are higher in the composite bridge while economic value is higher in the concrete bridges. Social aspects are similar and both have no very good values in this respect.

Global values obtained with the same model in both bridges gives us very similar values. We get a global value of 0,59 for concrete bridge and 0,56 for the composite bridge. We can compare individual and global values in Figure 5.

Social values are well below environmental or economic values which indicates us that social aspects have to be more deeply investigated.

7 CONCLUSIONS

This paper reviews the concept of sustainability as it appeared in 1987 and its importance and influence that has constantly acquired for decision taking in a world wide perspective. The future, especially after Paris Summit of 2015 and its agreements will surely bring more importance to these aspects.

Although it has been identified a clear lack of a clear and precise scientific methodology for the identification, selection and focusing of indicators for sustainability in the construction sector and in particular of railway bridges. Therefore it is necessary to consider in construction projects and in particular en railway bridges from a life cycle perspective indicators regarding environment, social and economic aspects.
In this case Mives method has been used to prepare a multi-criteria analysis system that breaks down criteria into indicators that allows to evaluate sustainability of bridges considering economy, social and environmental aspects.

The model has been applied to two railway bridges built with different design and construction methods, one of them built in prestressed concrete and the other a composite steel and concrete bridge. It has been observed that the model has enough sensibility to evaluate differences in each aspect for both bridges and therefore it can be used to evaluate sustainability for railway bridges considering a full life cycle and social, economic and environmental aspects.

REFERENCES


CONTRIBUTING TO SUSTAINABILITY OF CONCRETE BY USING STEEL FIBRES FROM RECYCLED TYRES IN WATER RETAINING STRUCTURES

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Key words: Recycled Steel Fibre Reinforced Concrete (RSFRC), Water retaining structures, Sustainability, Cracking

Abstract. This paper shows how large the benefits of the use of Recycled Steel Fibres in water retaining structures can be, both from the point of view of economy and sustainability.

1 INTRODUCTION

Within the framework of European FP7 project, Anagennisi (Rebirth/Renaissance) [1], focused on the recycling of all components of end-of-life tyres into concrete (www.Anagennisi.org), FHECOR Consulting Engineers, one of the 16 partners of the project, has been analysing the potential for the use of recycled steel fibres (RSF) for cracking control in the design of water retaining structures (WRS).

In WRS made from concrete, the reinforcement is determined from considerations pertaining to crack width, whose allowable value ranges from 0.2 mm to 0.1 mm. In traditional RC design, this means that the amount of reinforcement needed for Ultimate Limit State needs to be doubled or tripled in order to effectively control crack opening to admissible values.

Due to the fact that steel ratios for this type of structures are low, steel fibres are very effective in controlling crack widths. If these fibres come from recycled end-of-life tyres, concrete structures become more sustainable as well as less expensive. There is already experimental evidence ([2],[3]) that (RSF) are effective in controlling crack widths for reinforced concrete structures with small reinforcement ratios. They will be even more effective if these ratios can be further reduced.

In this paper a typical example is considered, studying the influence of admissible crack...
width, and assuming a very common shape for the constitutive law of RSFRC in tension, which can be easily obtained with fibre contents of around 40 kg/m³. The analysis is made in terms of cost, production of CO₂ and consumption of energy.

2 DESCRIPTION OF THE BASIC PROBLEM

In order to focus the study, a specific problem will be considered. The problem, taken from a real case, is the building of a circular water reservoir for a capacity of 12 300 m³. The structural concept proposed for the deposit is that of a perimetral cylindrical wall with a footing and a slab on grade with a joint between the two elements. The wall height is 5.43 m and its internal diameter is 56.00 m. The maximum water level is 5.25 m. For durability reasons, the concrete for the wall is C45/55. The strength of steel is 500 MPa and the material partial factors considered are \( \gamma_c = 1.5 \) for concrete and \( \gamma_s = 1.15 \) for steel.

This wall is supported on a footing. The wall depth is 0.20 m. The wall has been analyzed using the 2-D model shown in Figure 1, representing 1 meter of horizontal wall width. The 3-dimensional effect of the curvature of the wall is modelled by supporting the wall on springs with a stiffness given by Eq. (1), where \( K_{wall} \) is the stiffness of the springs per meter of wall height, \( A \) is the area of the one meter of wall measured vertically and \( E_c \) is the modulus of elasticity of concrete. The actual stiffness of each individual spring is obtained by multiplying \( K_{wall} \) by the distance between springs, which, in this case is 0.25 m.

\[
K_{wall} = \frac{E_c A}{R^2} = \frac{36.28 \text{ GPa} \left( \frac{22}{10} \right)^{0.3} \times 0.2}{28.1^2} = 9189 \text{ kN} / \text{m}^2
\]

Figure 1: 2-D Wall model
The loads that are considered relevant for this problem are the self-weight of the wall (SW), the temperature gradient (T), which is 16.5°, differential shrinkage between foundation and wall due to different concreting times (SH), which has been evaluated as 150 mm/m and water pressure (W).

The hoop forces due to the individual load cases are represented in Figure 2. Hoop forces are obtained from the reactions of the model springs using the expression of Eq. (2), where $F_{hoop}$ is the force per vertical meter of wall, $R_{spring}$ is the reaction in the corresponding spring, $R$ is the reservoir radius (28.1 m) and $A_{spring}$ is the area corresponding to the spring equal to 1.00 m times the spacing between springs (0.25 m in this case).

$$F_{hoop} = \frac{R_{spring} \times R}{A_{spring}}$$

![Figure 2: Hoop forces due to individual actions](image)

Bending moments due to individual actions, obtained from the wall model are shown in Figure 3.
The determining situation for the dimensioning of reinforcement is SLS. The corresponding hoop forces and bending moments in the wall due to the different load SLS load combinations are provided in Figure 4 and Figure 5.
It is remarked that for ULS, the necessary reinforcement has to be able to resist factored forces due to loads. Imposed deformations can be disregarded in ULS. It will not be possible to reduce the amount of reinforcement below the following values:

- The vertical reinforcement has to be able to resist the factored moment due to water pressure. The minimum vertical reinforcement necessary for ULS, to comply with this condition, is given by Eq. (3). In this equations there is also a check for minimum reinforcement using Eq. 7.1 of EN 1992-1-1[4]. In these expressions $f_{cd}$ is the factored concrete strength, $b$ is the width of the section, $d$ is the effective depth, $f_{yd}$ the factored strength of steel and $A_s$ the necessary reinforcement. For compliance with minimum reinforcement conditions, the vertical reinforcement must be at least 7\(\phi12\) on the outer side of the wall.
3.1 Design of reinforcement for crack control

Table 1 summarizes the vertical reinforcement necessary to control cracking due to flexural bending to be placed on the inner face of the reservoir wall. This reinforcement has been determined considering two values of the admissible crack width ($w_{adm}$) 0.1 mm and 0.2 mm. It has also been determined for both a conventional reinforced concrete solution and an alternative solution using Recycled Steel Fibre Reinforced Concrete (RSFRC). In this last case it has been assumed that the mean residual tensile strength of steel at serviceability, $f_{Ftsm}$, is equal to 0.5×$f_{ctm}$, that is, 1.9 MPa. Even though its value has little influence in SLS calculations, the residual tensile strength at ULS, $f_{Ftu}$, has been taken as half of $f_{Ftsm}$. The constitutive law of concrete in tension is shown in Figure 6. Table 1 also shows some intermediate values such as the maximum crack spacing, $s_{r, max}$, the mean difference between steel strain and concrete strain, $\varepsilon_r$, the depth of the neutral axis, $x$, the stress in steel, $\sigma_s$, and the actual characteristic crack width, $w_k$. The crack width has been determined according to the model of EN 1992-1-1 [4].
Table 1: Vertical reinforcement needed for bending in the tension

<table>
<thead>
<tr>
<th>$w_{adm}$ [mm]</th>
<th>Concrete</th>
<th>Reinf.</th>
<th>$cm^2/m$</th>
<th>$s_{r,\text{max}}$ [mm]</th>
<th>$\varepsilon_r$ [mm/m]</th>
<th>$x$ [m]</th>
<th>$\sigma_s$ [MPa]</th>
<th>$w_h$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>RC</td>
<td>$7\phi20$</td>
<td>21.99</td>
<td>249</td>
<td>0.41</td>
<td>0.047</td>
<td>126</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>$7\phi12$</td>
<td>7.91</td>
<td>300</td>
<td>0.04</td>
<td>0.047</td>
<td>13.45</td>
<td>0.01</td>
</tr>
<tr>
<td>0.2</td>
<td>RC</td>
<td>$10\phi12$</td>
<td>11.30</td>
<td>268</td>
<td>0.75</td>
<td>0.037</td>
<td>231</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>$7\phi12$</td>
<td>7.91</td>
<td>300</td>
<td>0.04</td>
<td>0.047</td>
<td>13.45</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Figure 6: Constitutive law assumed for RSFRC

Table 2: Horizontal reinforcement needed for hoop forces

<table>
<thead>
<tr>
<th>$w_{adm}$ [mm]</th>
<th>Concrete</th>
<th>Reinf.</th>
<th>$cm^2/m$</th>
<th>$s_{r,\text{max}}$ [mm]</th>
<th>$\varepsilon_r$ [mm/m]</th>
<th>$\sigma_s$ [MPa]</th>
<th>$w_h$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>RC</td>
<td>$2\times8\phi32$</td>
<td>128.64</td>
<td>339</td>
<td>0.28</td>
<td>88</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>$2\times7\phi25$</td>
<td>68.74</td>
<td>294</td>
<td>0.31</td>
<td>104</td>
<td>0.09</td>
</tr>
<tr>
<td>0.2</td>
<td>RC</td>
<td>$2\times8\phi25$</td>
<td>78.56</td>
<td>386</td>
<td>0.48</td>
<td>144</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>$2\times9\phi16$</td>
<td>18.09</td>
<td>320</td>
<td>0.60</td>
<td>199</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Table 2 shows the same results pertaining to the horizontal reinforcement which is designed to control cracking from hoop forces.
3.2 Evaluation in terms of cost

Given the above design a cost analysis for one meter of wall perimeter has been carried out. For this, it has been assumed that the cost of one placed cubic meter of concrete is 120€, that the cost of one placed kilogram of reinforcing steel is 1.1€ and that the cost of one placed kilogram of recycled fibres is 0.7€.

The results of the cost analysis are summarized in Table 3. It can be seen that the cost of 1 meter of wall using RSFRC is 63% of that of a traditional RC solution for a crack width limitation of 0.1 mm and 68% for a crack with limitation of 0.2 mm. This makes the use of RSFRC already attractive from a purely economic point of view. However as it will be shown in the next paragraph its appeal increases when sustainability is considered.

Table 3: Cost Analysis

<table>
<thead>
<tr>
<th>wadm [mm]</th>
<th>Type</th>
<th>m³ concrete</th>
<th>m²/m Reinf Tension face</th>
<th>kg vert. reinf.</th>
<th>kg hor. reinf.</th>
<th>kg fibres</th>
<th>Cost [€/m]</th>
<th>Relative cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>RC</td>
<td>1.068</td>
<td>7ø20 22.0 5ø10 3.9 2×8ø32 128.6 111 539 0</td>
<td>843</td>
<td>100%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RSFRC</td>
<td>1.068</td>
<td>7ø12 7.9 5ø10 3.9 2×7ø25 68.7 51 288 43</td>
<td>531</td>
<td>63%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>RC</td>
<td>1.068</td>
<td>10ø12 11.3 5ø10 3.9 2×8ø25 78.6 65 329 0</td>
<td>562</td>
<td>100%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RSFRC</td>
<td>1.068</td>
<td>7ø12 7.9 5ø10 3.9 2×9ø16 36.2 51 152 43</td>
<td>380</td>
<td>68%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3 Evaluation in terms of reduced production of CO2

Table 4 compares the traditional solution with the RSFRC solution in terms of production of greenhouse effect gas CO2. The analysis is based on the assumption that the production of CO2 for every cubic meter of concrete, each ton of reinforcing steel and each ton of recycled fibres are 0.4, 0.9 and 0.018 (see Table 5). This last figure is a rough estimate and assumes that the production of new recycled fibres from recycled tyres involves the generation of roughly 2% the amount of CO2 when compared to manufacture of steel from iron mineral.

Table 4: Analysis in terms of reduced production of CO2

<table>
<thead>
<tr>
<th>wadm [mm]</th>
<th>m³ concrete</th>
<th>kg steel</th>
<th>kg fibres</th>
<th>kg of CO2</th>
<th>Relative value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>RC</td>
<td>1.068</td>
<td>650</td>
<td>0</td>
<td>585.35 100%</td>
</tr>
<tr>
<td>RSFRC</td>
<td>1.068</td>
<td>339</td>
<td>43</td>
<td>306.02</td>
<td>52%</td>
</tr>
<tr>
<td>0.2</td>
<td>RC</td>
<td>1.068</td>
<td>394</td>
<td>0</td>
<td>355.33 100%</td>
</tr>
<tr>
<td>RSFRC</td>
<td>1.068</td>
<td>202</td>
<td>43</td>
<td>183.18</td>
<td>52%</td>
</tr>
</tbody>
</table>
Table 5: Production of CO₂ for the manufacture of construction relevant materials

<table>
<thead>
<tr>
<th>Assumed CO₂ production per unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>ton CO₂/m³ concrete =</td>
</tr>
<tr>
<td>0.400</td>
</tr>
<tr>
<td>ton CO₂/t steel =</td>
</tr>
<tr>
<td>0.900</td>
</tr>
<tr>
<td>ton CO₂/t recycled fibre =</td>
</tr>
<tr>
<td>0.018</td>
</tr>
</tbody>
</table>

It can be seen that, with the given assumptions, the RSFRC solution produces half the amount of CO₂ compared to the traditional solution, irrespective of the crack with limit.

3.4 Evaluation in terms of energy savings

A similar analysis carried out for energy consumptions is summarized in Table 6 which has been drafted based upon the assumptions shown in 0. The analysis shows a reduction in energy input of 21% for admissible crack width of 0.1mm and 16% for an admissible crack width of 0.2mm.

Table 6: Analysis in terms of required energy

<table>
<thead>
<tr>
<th>w_adm [mm]</th>
<th>m³ concrete</th>
<th>kg steel</th>
<th>kg fibres</th>
<th>kWh</th>
<th>Relative value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>RC</td>
<td>1.068</td>
<td>650</td>
<td>0</td>
<td>881</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>1.068</td>
<td>339</td>
<td>43</td>
<td>695</td>
</tr>
<tr>
<td>0.2</td>
<td>RC</td>
<td>1.068</td>
<td>394</td>
<td>0</td>
<td>728</td>
</tr>
<tr>
<td></td>
<td>RSFRC</td>
<td>1.068</td>
<td>202</td>
<td>43</td>
<td>613</td>
</tr>
</tbody>
</table>

Table 7: Energy needed for production of material units

<table>
<thead>
<tr>
<th>Energy consumption</th>
</tr>
</thead>
<tbody>
<tr>
<td>kWh/m³ concrete =</td>
</tr>
<tr>
<td>460</td>
</tr>
<tr>
<td>kWh/t steel =</td>
</tr>
<tr>
<td>600</td>
</tr>
<tr>
<td>kWh/t recycled fibre =</td>
</tr>
<tr>
<td>12</td>
</tr>
</tbody>
</table>

12 CONCLUSIONS

From the above analysis the following conclusions can be drawn:

- RSFRC solutions for water retaining structures are very advantageous even from the point of view of construction costs. Fibres are very effective in controlling cracking in such structures because they are typically low reinforcement ratio structures for which tension stiffening effects play an important role. Using fibres is one way of magnifying these effects. In the example considered, material costs were shown to be cut by 37% to 32%. Given these figures the fact that no example of use of fibres in water retaining structures is known to the authors can only be the result of lack of
diffusion among designers of the benefits which have been shown in this paper.

- From the point of view of sustainability, it was shown that the use of RSFRC leads to a reduction of almost 50% in the production of greenhouse effect gas CO2 and to a 16% to 21% cut in energy consumption. These figures can only add to the already attractive results from a purely economic point of view.

- These features should make the use of RSFRC a standard for the design of water retaining structures in a future not too far off. The realization of this vision can only be hindered by recycled fibre production capacity.

REFERENCES


**FEASIBILITY STUDY ON THE UTILIZATION OF ALKALI-TREATED GROUND MUNICIPAL SOLID WASTE INCINERATION BOTTOM ASH AS CEMENT REPLACEMENT**

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**Key words:** incineration bottom ash (IBA); supplementary cementitious material; alkali treatment; compressive strength; heavy metal immobilization

**Abstract.** As municipal solid waste incineration bottom ash (IBA) contains metallic aluminum, crack and expansion issue could happen due to hydrogen gas generation when IBA is used in cement matrix. In this paper, ground IBA powder was first pre-treated with alkaline solution under certain conditions to remove metallic aluminum, and after that the treated IBA was used as cement replacement in cement mortar. The alkali treatment effects on metallic aluminum removal, chemical composition of IBA and compressive strength of cement-IBA mortar were evaluated. The results showed that metallic aluminum in IBA could be completely removed by treatment with sodium hydroxide solution, and the alkali treatment did not change the major components of IBA though some aluminum compounds changed. From the compressive strength of cement-IBA mortar, it showed that some pozzolanic property of IBA could sacrifice when IBA is treated with alkaline solution. However, the strength activity index value of cement-IBA mortar using IBA treated with sodium hydroxide solution under low temperature and low PH can still reach more than 80%, which is meaningful for practical application. In general, this paper revealed a viable way to treat IBA to be suitable for cement replacement.

1 INTRODUCTION

With economic development and population increase, municipal solid waste (MSW) generation has increased year by year in some big cities around the world. Compared with direct landfill of MSW, incineration has been developed to be a good way to deal with MSW, as through incineration not only the mass and volume of MSW can be reduced by 80% and 90%, respectively [1], but also electricity power can be produced. However, after incineration, the remaining by-product, i.e. incineration ash, still needs to be dealt with, normally by landfill. In some cities with scarcity of land, like Singapore, there is not enough landfill resource. Therefore, to utilize incineration ash, which can reduce the amount of waste and prolong the lifespan of landfill, is encouraged. There are two types of incineration ash, i.e.
incineration fly ash (IFA) and incineration bottom ash (IBA). Compared with IFA, IBA is less hazardous [2] and is classified as a non-hazardous waste according to the European Waste Catalogue. Furthermore, IBA occupies 75% to 80% of total incineration ash, while IFA is only around 10% to 15% [3]. Therefore once IBA can be utilized, it will be of great help to prolong the lifespan of landfill.

Supplementary cementitious material is a kind of material which contributes to the hardened property of concrete by hydraulic or pozzolanic reactions. The use of such material offers the viability of partial replacing cement in concrete production. Supplementary cementitious material greatly reduces the demand on cement and significantly lowers the greenhouse gas emission associated with cement clinker production.

Most supplementary cementitious materials originate from industry by-products such as coal fly ash and blast-furnace slag. It has been reported that IBA has pozzolanic properties and can be a potential supplementary cementitious material [4, 5]. The utilization of IBA as supplementary cementitious material will be of great significance. Not only waste IBA can be utilized and lifespan of landfill can be prolonged, but also cost of concrete production and greenhouse effect can be reduced. However, due to the aluminum products like beverage cans, aluminum container and aluminum packaging which we use in our daily life, there is some metallic aluminum remaining in IBA. Metallic aluminum can react with calcium hydroxide which is a by-product of cement hydration to generate hydrogen gas as showed in the following equation:

\[ 2\text{Al} + \text{Ca(OH)}_2 + 2\text{H}_2\text{O} \rightarrow \text{Ca(AlO}_2\text{)}_2 + 3\text{H}_2 \]  

The resulted hydrogen gas will lead to expansion and crack of concrete [6-8]. Therefore, removal of metallic aluminum from IBA to prevent hydrogen generation and aeration is crucial for the successful application of IBA as a supplementary cementitious material. Bertolini et al. used wet grinding as the pretreatment method to remove metallic aluminum in IBA before it can be used as supplementary cementitious material [9]. However, Bertolini’s results also showed that metallic aluminum in IBA cannot be completely removed by this way perhaps due to low pH value or temperature. Alkali or acid treatment is an alternative way to remove metallic aluminum in IBA. Compared with acid treatment which may not only removes metallic aluminum but also dissolves the main compositions of IBA such as CaO and SiO₂, alkali treatment seems a more possible way to deal with IBA to be used as supplementary cementitious material as it is also reported that alkali treatment can be used to treat other ingredients applied in cement paste, like fiber [10-12] and coal fly ash [13]. However, the effect of alkali treatment on raw material, i.e. IBA in the current study, as well as its impact to the resulted product, needs to be evaluated.

In this paper, alkali treatment was applied to remove metallic aluminum in IBA. Treated IBA was then used to partially replace cement for evaluating the strength activity index (SAI) in accordance with ASTM C311. The effects of different alkali treatment conditions, i.e. pH, temperature, treatment time and type of alkali media, on the properties of IBA and the resulted cement-IBA mortar were reported.
2 EXPERIMENTAL PROGRAMME

2.1 Ground IBA powder

The IBA was collected from Keppel Seghers Tuas Waste-to-Energy incineration plant, Singapore. The IBA was oven dried first, followed by ball milling. Fig. 1 shows the gradation of ground IBA powder. The average particle size is around 30 microns and maximum particle size is about 150 microns.

![Particle size gradation curve of grinded IBA powder](image)

2.2 Alkali treatment of IBA powder

Ground IBA powder was treated with either sodium hydroxide or calcium hydroxide at different temperatures, pH and duration. Table 1 summarizes the treatment conditions. A total of nine conditions were applied for IBA treatment. The alkaline solution used to treat IBA was prepared with either sodium hydroxide or calcium hydroxide and distilled water. Two treatment durations of 15 minutes and 120 minutes were used. Sodium hydroxide solution

<table>
<thead>
<tr>
<th>S/N</th>
<th>Notation</th>
<th>Temperature (°C)</th>
<th>PH</th>
<th>Alkaline medium</th>
<th>Treating duration (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>H$_2$OT70D120</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>NaOH</td>
</tr>
<tr>
<td>2</td>
<td>Na14T70D15</td>
<td>25</td>
<td>7</td>
<td>12</td>
<td>Ca(OH)$_2$</td>
</tr>
<tr>
<td>3</td>
<td>Na14T70D120</td>
<td>25</td>
<td>7</td>
<td>12</td>
<td>NaOH</td>
</tr>
<tr>
<td>4</td>
<td>Na14T25D15</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>NaOH</td>
</tr>
<tr>
<td>5</td>
<td>Na14T25D120</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>Ca(OH)$_2$</td>
</tr>
<tr>
<td>6</td>
<td>Na12T70D15</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>NaOH</td>
</tr>
<tr>
<td>7</td>
<td>Na12T70D120</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>Ca(OH)$_2$</td>
</tr>
<tr>
<td>8</td>
<td>Ca12T70D15</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>NaOH</td>
</tr>
<tr>
<td>9</td>
<td>Ca12T70D120</td>
<td>70</td>
<td>7</td>
<td>12</td>
<td>Ca(OH)$_2$</td>
</tr>
</tbody>
</table>
was used to treat samples 2 to 7, in which the effects of temperature (25 °C and 70 °C) and PH (12 and 14) were evaluated. Calcium hydroxide solution was applied to treat samples 8 and 9 to reveal the influence of type of alkaline solution for IBA treatment. To identify alkali treatment effect on heavy metal immobilization in IBA, a distilled water treated IBA sample (1) is used for comparison.

The treatment procedure follows, a) preparing alkaline solution to the pre-determined pH at room temperature, b) heating alkaline solution using a hot plate stirrer to the pre-determined temperature, c) adding IBA powder into the alkaline solution, d) stirring the solution until pre-determined treating duration. The hot plate setup used for the above-mentioned procedure is shown in Fig. 2 and the solid (IBA powder) to liquid (alkaline solution) ratio is around 1:10.

After the treatment, the alkali-IBA solution was centrifuged, followed by filtration to separate IBA powder and filtrate. The IBA powder was washed 2-3 times to remove alkalis. The treated IBA powder was then oven dried at 105 °C for 2 hours prior to further characterization and usage as cement supplement.

![Figure 2: Hot plate setup used for alkali treatment of IBA](image)

### 2.3 Mix design and processing of cement-IBA mortar

In accordance with ASTM C311/C311M, the determination of strength activity index (SAI) is based on the 20% cement replacement of control mortar sample by interested pozzolans. Table 2 summarizes the mix design of cement-IBA mortar. Mix 1 is the control mix without IBA, while the other mixes consist of 20% of either treated or original untreated IBA. Only IBA with the treatment duration of 120 minutes were used because shorter treating duration cannot prevent further hydrogen generation.

The mixing, casting, and curing procedure follows, a) Cement, IBA and sand were dry-mixed for 3 minutes in a mixer, b) water was slowly added into the mix and mixes for 5 minutes until a homogenous and consistent paste was achieved, c) the fresh paste was poured into 50 mm cubic moulds and vibrated for 30 seconds, d) the surface of moulds is covered by plastic sheet to prevent water evaporation, e) the specimens were cured at room temperature and demolded after 24 hours, and f) samples were cured in saturated lime water after demolding until the age of 7 days, 28 days and 90 days for testing.
Table 2: Mix design of cement-IBA mortar

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Notation</th>
<th>Cement Mass (g)</th>
<th>Original IBA Mass (g)</th>
<th>Treated IBA Mass (g)</th>
<th>Sand Mass (g)</th>
<th>Water Mass (g)</th>
<th>SP Mass (ml)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Control (w/o IBA)</td>
<td>500</td>
<td>0</td>
<td>0</td>
<td>1375</td>
<td>242</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>Original IBA</td>
<td>400</td>
<td>100</td>
<td>0</td>
<td>1375</td>
<td>242</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>Na14T70D120</td>
<td>400</td>
<td>0</td>
<td>100</td>
<td>1375</td>
<td>242</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Na14T25D120</td>
<td>400</td>
<td>0</td>
<td>100</td>
<td>1375</td>
<td>242</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Na12T70D120</td>
<td>400</td>
<td>0</td>
<td>100</td>
<td>1375</td>
<td>242</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>Ca12T70D120</td>
<td>400</td>
<td>0</td>
<td>100</td>
<td>1375</td>
<td>242</td>
<td>0</td>
</tr>
</tbody>
</table>

### 2.4 Tests

XRF and XRD were used to reveal chemical composition and structure of IBA powder before and after treatment. Inductively Coupled Plasma Mass Spectrometry (ICP-MS) was used to determine heavy metal content in the filtrate solution from treatment process.

Dry density was tested for the cement-IBA mortar samples of each mix. The sample is first dried in the oven at 105 °C for 24 hours. Dry density was then calculated by taking the ratio of mass to volume of the sample.

Compressive test was conducted to reveal the strength of cement-IBA mortars. A minimum of four samples were tested for each mix. Strength Activity Index (SAI) value is defined by the following equation:

$$SAI = \frac{B}{A} \times 100\%$$

where A is the compressive strength of control (w/o IBA) and B is the compressive strength of mortar with 20% cement replacement with IBA.

### 3 RESULTS AND DISCUSSION

#### 3.1 Alkali treatment for metallic aluminum removal

Surface foaming was observed when IBA was added to the alkaline solution due to gas generation from ground IBA powder as shown in Fig. 3(a). The reaction equation is:

$$2Al + 2OH^- + 2H_2O \rightarrow 2AlO_2^- + 3H_2$$

With prolonged treatment duration, the foam layer was reduced and a clear solution can be obtained as shown in Fig. 3(b), which suggests seizing of further gas generation from IBA. It is concluded that 15 minutes for alkali treatment is not sufficient to prevent further gas generation from IBA in all treatment conditions. A prolonged treatment of 120 minutes for all alkaline conditions is needed in order to obtain a clear solution which may suggest seizing of further gas generation from IBA. For this reason, only 120-minute treated IBAs were selected for cement-IBA mortar preparation.
Fig. 4 shows the dry density of cement-IBA mortar mixes. Density ratio defined as the density of sample to the density of control is also given. As can be seen, sample with 20% original IBA replacement has a much lower density (85.67%) than that of the control sample due to aeration from the untreated IBA powder. Samples with treated IBA, on the other hand, have a much higher density of above 95% of the control. This indicates alkali treatment is able to prevent further gas generation from IBA.

As for the comparison between the four treatment conditions, mortar samples with 20% IBA treated with NaOH at high molarity and high temperature (Na14T70D120) show the highest density which is just slightly lower than the control due to 20% IBA replacement and a lower density of IBA (2.65g/cm$^3$) than that of cement (3.15g/cm$^3$). It is also interesting to observe that a 45 oC reduction in treatment temperature (Na14T25D120) has more influence on gas generation and dry density than the two order reduction in alkaline molarity (Na12T70D120).

It is also observed that the density of sample with 20% Ca(OH)$_2$ treated IBA (Ca12T70D120) is further reduced as compared to the NaOH treatment at the same alkaline molarity and temperature (Na12T70D120). This may be due to the precipitation of insoluble calcium aluminate as shown in Eqn. (1) on the surface of IBA, which inhibits the reaction of IBA with alkaline solution and prevents further gas generation from IBA. The calcium aluminate layer may be broken by the mechanical force during paste mixing which enables further reaction of IBA with cement paste and gas generation, which lowers the density of
3.2 Compressive Strength and Strength Activity Index of cement-IBA mortar

Fig. 5 shows the XRD pattern of treated and untreated IBA. As can be seen, the major compositions of untreated IBA are crystalline material Quartz (SiO$_2$) and Calcite (CaCO$_3$). Minor components in untreated IBA are Magnetite (Fe$_2$O$_3$), Aluminum Oxide (Al$_2$O$_3$) and some compounds of calcium, silicon and aluminum, such as Gehlenite (Ca$_2$Al$_2$SiO$_7$), Calcium Silicate (CaSiO$_3$) and Calcium Aluminate Oxide (Ca$_3$Al$_2$O$_6$). After alkali treatment, the peaks of most compositions still remain, and Quartz and Calcite are still the major compositions.

Figure 5: XRD pattern of treated and untreated IBA

Compared with untreated IBA, there are two main differences of treated IBA XRD pattern:

1. The peaks between 31° to 34° of alkali treated IBA are less sharp after treatment. This is due to the dissolution of aluminum oxide. As is known, aluminum oxide can react with sodium hydroxide to generate soluble sodium aluminate as shown in the following equation:

$$\text{Al}_2\text{O}_3 + 2\text{NaOH} \rightarrow 2\text{NaAlO}_2 + \text{H}_2\text{O}$$  \hspace{1cm} (4)

2. A new peak which is at around 28° and marked as 8 in Fig. 5 appears in sodium hydroxide treated IBA samples. This is due to the new generation of aluminum silicate. Such kind of aluminum silicate generation is quite significant under the treatment of low PH (the treatment condition of sample Na12T70D120) or low temperature (the treatment condition of sample Na14T25D120) and not obvious for treatment of high temperature and high PH value. There is nearly no newly generated aluminate silicate in sample Ca12T70D120, which is treated with calcium hydroxide solution.

To further understand the chemical reaction during the alkali treatment process, aluminum content in the filtrate solutions is tested by ICP-MS method and the results is shown in Table 3. The aluminum content in the filtrate solution comes from two parts: reaction of metallic aluminum and alkali as shown in the equation (3) and reaction of aluminum compound like aluminum oxide and alkali as shown in equation (4). From Table 3, comparing sodium
hydroxide treated samples, Na12T70D120 which has the most significant aluminum silicate peak contains the least amount of aluminum content in filtrate solution, while Na14T70D120 which has the least significant aluminum silicate peak contains the most amount of aluminum content in filtrate solution. This phenomenon implies that under low PH value aluminum tends to react with silicate to generate aluminum silicate solid and under high PH value and high temperature aluminum tends to be dissolved into the solution, or we can say aluminum silicate is more stable under low PH value and not stable under high PH value and high temperature.

From Fig. 5 and Table 3, we can also see that calcium hydroxide treated sample Ca12T70D120 almost has no aluminate silicate peak and less aluminum content in filtrate solution. This may still be due to the calcium aluminate precipitate as mentioned in 3.1 which inhibits the dissolution of metallic aluminum or aluminum oxide in alkaline solution and the reaction of aluminum with silicate.

| Table 3: Aluminum content in filtrate solutions tested by ICP-MS |
|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Al content (ppb)            | Na14T70D120 | Na14T25D120 | Na12T70D120 | Ca12T70D120 |
| 191700                      | 144200      | 12800       | 64900        |

| Table 4: Oxide composition of treated and untreated IBA |
|-------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Formula     | Original IBA (%) | Treated IBA (%) | Treated IBA (%) | Treated IBA (%) |
|             | Na14T70D120 | Na14T25D120 | Na12T70D120 | Ca12T70D120 |
| CaO         | 32.47      | 34.93       | 34.5         | 34.9         | 33.47            |
| SiO2        | 25.89      | 29.02       | 28.99        | 29.17        | 30.16            |
| Fe2O3       | 9.923      | 12.52       | 12.62        | 11.49        | 11.89            |
| P2O5        | 7.75       | 8.07        | 8.01         | 7.64         | 7.64             |
| Al2O3       | 6.47       | 6.51        | 6.77         | 7.24         | 7.15             |
| Na2O        | 10.9       | 2.77        | 2.89         | 3.4          | 3.53             |
| MgO         | 2.38       | 2.23        | 2.33         | 2.34         | 2.44             |
| TiO2        | 1.18       | 1.42        | 1.39         | 1.39         | 1.33             |
| K2O         | 1.53       | 0.775       | 0.76         | 0.818        | 0.785            |
| CuO         | 0.453      | 0.532       | 0.533        | 0.449        | 0.455            |
| ZnO         | 0.449      | 0.396       | 0.396        | 0.469        | 0.476            |
| Cr2O3       | 0.222      | 0.278       | 0.272        | 0.269        | 0.258            |
| PbO         | 0.147      | 0.244       | 0.213        | 0.173        | 0.186            |
| MnO         | 0.113      | 0.148       | 0.147        | 0.194        | 0.176            |
| SrO         | 0.0637     | 0.1         | 0.0962       | 0.0633       | 0.0631           |
| NiO         | 0.053      | 0.072       | 0.0693       |               |                  |

Table 4 shows the oxide composition of treated and untreated IBA from XRF test. Compare with untreated IBA, the Na2O and K2O contents are greatly reduced in all four types of treated IBA. Sodium and potassium in IBA normally exist in the form of salts, such
as sodium chloride and potassium chloride, which are easily dissolved and washed away. The composition percentage of other components increases correspondingly in treated IBA due to the reduced content of N2O and K2O. It is also seen that aluminum content in the treated IBA does not lose a lot for all treated IBAs, which means the metallic aluminum content in IBA is low. Sample Na14T70D120 has the least content of aluminum in the four types of treated IBA and sample Na12T70D120 has the highest percentage of aluminum content. These results are coincident with the results shown in Table 3, i.e., less soluble aluminum content results in high percentage of aluminum content in the solid sample. The XRF result of sample Ca12T70D120 which shows high aluminum content is also due to the low dissolved aluminum content as shown in Table 3.

3.3 Effect of alkali treatment on the chemical composition and structure of IBA

Fig. 6 shows the compressive strength results of cement and cement-IBA mortars. To evaluate the pozzolanic property of IBA, the compressive strength improvement (%) from 28 days to 90 days is calculated as shown in Table 5 because the pozzolanic reaction mainly happen at later cement hydration stage, i.e. after 28 days. Sample Original IBA using untreated IBA as supplementary cementitious material has much lower compressive strength than control sample and other samples using treated IBA as shown in Fig. 6 due to the lower density from hydrogen generation. However, sample Original IBA has 124% compressive strength improvement from 28 days to 90 days and is much higher than sample Control (only 28% improvement), which means untreated IBA has good pozzolanic properties. The cement mortars using alkali treated IBA has higher compressive strength than sample Original IBA due to the removal of metallic aluminum resulting in the improvement of density, so alkali treatment is really helpful for IBA used as supplementary cementitious material. But comparing compressive strenght improvement from 28 days to 90 days, the alkali treated samples have much lower strength improvement than sample Original IBA, which means that though alkali treatment can remove metallic aluminum, the pozzolanic property of IBA sacrifice to a certain degree. From the comparison within alkali treated samples, sample Na14T25D120 and sample Na12T70D120 have relatively higher strength improvement percentage, and both of them are higher than the control sample, therefore IBA can still remain some pozzolanic property under low temperature or low PH sodium hydroxide treatment. However, IBA treated with sodium hydroxide under high temperature and high PH value lost its pozzolanic property and even affected cement hydration at later age so that sample Na14T70D120 using such kind of IBA has only 5% strength improvement from 28 days to 90 days. The high temperature and high PH value alkaline solution dissolve the most amount of aluminum as shown in Table 3, and this may also be applied to active silicate content though we have no silicate content data here. Such loss of aluminum and silicate may be the reason of pozzolanic property loss of Na14T70D120 sample. But the lower PH value or lower temperature treated IBA has less aluminum loss and the new generation of aluminum silicate phase also remains some aluminum and silicate content, which keeps the pozzolanic capacity of IBA to some extent, though they are also much smaller than original IBA.

Compared with sample Na12T70D120 and sample Ca12T70D120, both of them used IBA treated under the same temperature and similar PH value but with different alkaline media. Sample Ca12T70D120 using calcium hydroxide treatment IBA has higher early age (7 days)
strength than sample Na12T70D120, though its dry density is lower than sample Na12T70D120 from Fig.4. When IBA is treated in saturated calcium hydroxide solution, some calcium hydroxide precipitate may deposit on the surface or inside the pores of IBA. When such kind of IBA is used in cement, rather than waiting for cement hydration to generate calcium hydroxide, the existing calcium hydroxide can have the pozzolanic reaction of IBA to happen earlier and increase the early strength of cement-IBA mortar. However, with the consumption of pozzolanic constituent at early age, there is little pozzolanic reaction at later stage from 28 days to 90 days for sample Ca12T70D120, and like the IBA used in sample Na14T70D120, the IBA used in sample Ca12T70D120 also affected cement hydration at late stage so that sample Ca12T70D120 has only 3% strength improvement from 28 days to 90 days.

**Table 5:** Median compressive strength improvement (%) from 28 days to 90 days

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Median compressive strength improvement (%) from 28 days to 90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>28%</td>
</tr>
<tr>
<td>Original IBA</td>
<td>124%</td>
</tr>
<tr>
<td>Na14T70D120</td>
<td>5%</td>
</tr>
<tr>
<td>Na14T25D120</td>
<td>33%</td>
</tr>
<tr>
<td>Na12T70D120</td>
<td>31%</td>
</tr>
<tr>
<td>Ca12T70D120</td>
<td>3%</td>
</tr>
</tbody>
</table>

**Figure 6:** Compressive strength of cement and cement-IBA mortars
Table 6 shows the SAI value of cement-IBA mortar Na14T70D120, Na14T25D120, Na12T70D120 and Ca12T70D120. Though the above analysis shows that the pozzolanic property of IBA with alkali treatment can decrease a lot compared with untreated IBA, the SAI performances of Na14T25D120 and Na12T70D120 which use IBA with low temperature or low PH value treatment have meaningful values, i.e., larger than 80% for 28 days and 90 days, which meets 75% requirement set by ASTM standard C618. So these two kinds of treated IBA can be used as supplementary cementitious material in practical production.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Strength Activity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>Na14T70D120</td>
<td>68.59%</td>
</tr>
<tr>
<td>Na14T25D120</td>
<td>88.25%</td>
</tr>
<tr>
<td>Na12T70D120</td>
<td>69.16%</td>
</tr>
<tr>
<td>Ca12T70D120</td>
<td>84.12%</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

An alkali pre-treatment method to remove metallic aluminum in IBA was introduced, followed by evaluation of treated and untreated IBA to be used as supplementary cementitious material in cement mortar. The main conclusions are as following:

1. Metallic aluminum in IBA can be removed by alkali treatment for 2 hours. Sodium hydroxide is more efficient to completely remove metallic aluminum, while calcium hydroxide still remains some unreacted metallic aluminum in IBA, which results in a little bit lower density in the final cement-IBA mortar product.

2. Alkali treatment does not change the main chemical composition of IBA, and only wash away sodium and potassium salt. Quarts and calcite which are the two main components in IBA still remain unchanged after alkali treatment, but other compounds have some change. Especially for aluminum compounds, new aluminate silicate product was generated under sodium hydroxide treatment with low temperature or low PH value.

3. The compressive strength of cement-IBA mortars show that when IBA is treated with alkaline solution, some pozzolanic property, i.e. later age strength improvement, could sacrifice. However, the SAI value of cement-IBA mortar using IBA treated with sodium hydroxide solution under low temperature and low PH can still reach more than 80%, which is meaningful in practical application. The cement-IBA mortars using IBA treated with calcium hydroxide solution have high early strength.

6 ACKNOWLEDGEMENT

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3. NTU-IGS
REFERENCES


Keywords: Cement, Durability, Ladle Furnace Slag, Masonry Mortar, Pastes, Shrinkage

Abstract. A significant by-product of the steelmaking industry is ladle furnace (white or basic) slag, produced in the secondary or basic refining process of steel. The reactivity at outdoor temperatures of some of its key compounds, such as calcium aluminate, free calcium oxide, and free magnesium oxide are examined in this paper. Their different properties are studied in masonry mortar mixes that incorporate ladle furnace slag in amounts of 10% and 20% by weight of cement. The study concludes that ladle furnace basic slag can induce slight hydraulic reactivity and that its partial inclusion has no negative effect on the principal properties of cement masonry mortars.

1 INTRODUCTION

Over the past few decades, the reuse of many waste and industrial by-products in the construction sector has sought to respond to a key challenge: the exploitation of materials that would otherwise be of no economic value. Today, a key challenge facing our society is to develop processes that reuse these by-products, giving them material value and environmental, social and functional utility.\[1\].

White or basic Ladle Furnace Slag (LFS) produced in secondary or basic steel-refining processes is a significant by-product of the steelmaking industry. This manufacturing process yields two types of basic slag that are either low or high in silica or alumina, depending on the saturation method.

Secondary industrial materials can be used in various construction and civil engineering applications. Over the past few decades, several research groups have analyzed the mineralogy and hydraulicity of LFS in the context of its instabilities. Other key research projects have focused on the application of LFS in construction materials. Initial studies verified the use of LFS in cement matrices, followed by studies on soil stabilization for civil works. Those applications are based on the reactivity of certain LFS compounds, such as free magnesium oxide, free calcium oxide and calcium aluminates, under environmental
conditions. Current research is focusing on the characterization of (non-structural) pastes and masonry mortars containing these by-products in partial substitution of binders (cement) and fine aggregates.

This study specifically examines the reactivity at outdoor temperatures of certain compounds, particularly calcium aluminates, free calcium oxide and free magnesium oxide. Various analytical techniques are employed: DRX, TGA, chemical analyses, volumetric stability and a series of controlled hydration reactions. Hence, this research work focuses on the study of these slags and their performance in cement matrixes.

Different properties are studied: mechanical behaviour under compression and flexural loads and durability issues, exposed to both weathering and ageing, which mainly refer to efflorescence and wetting-drying ageing cycles.

Interesting results were achieved with incorporation of 10% and 20% LFS by weight of cement. The white slags with high alumina content showed better physical-chemical performance than mortar designs incorporating a content of high silica slag. Our study concluded that LFS can induce slight hydraulic reactivity and that partial incorporation of amounts of LFS below 20% by weight in cement masonry mortars has little or no negative effects on their mechanical performance and durability.

2 MATERIALS

This study of LFS in cement matrices was done by casting cement pastes and non-structural masonry mortars, for the analysis of their volumetric instabilities and shrinkage. High-alumina LFS and high-silica LFS were labelled EB1 and EB2, respectively. EB1 presented an irregular-shaped piece material of medium cohesion with a specific gravity of 2.75g/cm$^3$. Its main content by weight was 22% Al$_2$O$_3$ and 2.9% SiO$_2$. EB2 presented a powdery appearance with highly rounded particles and a specific gravity of 3.03 g/cm$^3$. Its main content by weight was 4.3% in Al$_2$O$_3$ and 22.4% in SiO$_2$.

Figure 1 shows the morphology of the irregular-shaped particles of medium cohesion found in EB1 and the powdery highly rounded particles of EB2. In addition, it shows the fine limestone sand aggregates, supplied by the construction company Bikain. Table 1 details the mineralogical analysis by XRD of both LFS types.

![Figure 1: LFS types (EB1 and EB2) and sand limestone](image)

Mortar fabrication was done with limestone aggregates of 0-4 mm in size: filler (0-1 mm) and sand (0-4 mm, with a fine fraction of 16%). Two Portland cement types were used as mortar binders (UNE 80103:1986): CEM I 52.5 R with a unimodal particle size distribution.
of 20µm, and CEM II/A-M (V-L) 42.5 R with a bimodal particle size distribution (19µm and 34µm).

Table 1: XRD analysis of Ladle Furnace Slags: EB1 and EB2

<table>
<thead>
<tr>
<th>Components</th>
<th>EB1</th>
<th>EB2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-lime/CaO (%)</td>
<td>16</td>
<td>-</td>
</tr>
<tr>
<td>Free periclase/MgO (%)</td>
<td>3.8</td>
<td>-</td>
</tr>
<tr>
<td>Brucite Mg(OH)₂</td>
<td>*</td>
<td></td>
</tr>
<tr>
<td>Tricalcium aluminate Ca₃Al₂O₆</td>
<td>***</td>
<td></td>
</tr>
<tr>
<td>Calcium-olivine Ca₂SiO₄</td>
<td>**</td>
<td>*****</td>
</tr>
<tr>
<td>Calcite CaCO₃</td>
<td>*****</td>
<td></td>
</tr>
<tr>
<td>Jasmundite Ca₂₀,₆₈Mg₁,₃₂(SiO₄)₃O₄S₂</td>
<td>***</td>
<td></td>
</tr>
<tr>
<td>Mayenite Ca₁₂Al₁₄O₃₃</td>
<td>*</td>
<td>***</td>
</tr>
<tr>
<td>Katoite Ca₃Al₂(OH)₁₂</td>
<td>***</td>
<td></td>
</tr>
<tr>
<td>Sjögrenite Mg₆Fe₂(OH)₁₆₅H₂O CO₃</td>
<td>**</td>
<td></td>
</tr>
<tr>
<td>Periclase MgO</td>
<td>**</td>
<td></td>
</tr>
<tr>
<td>Vaterite CaCO₃</td>
<td>**</td>
<td></td>
</tr>
<tr>
<td>Fluorite CaF</td>
<td>***</td>
<td></td>
</tr>
<tr>
<td>Hydrotalcite Mg₄Al₂(OH)₁₂CO₃</td>
<td>***</td>
<td></td>
</tr>
<tr>
<td>Oldhamite CaS</td>
<td>**</td>
<td></td>
</tr>
</tbody>
</table>

Both LFS types were studied under TGA analysis. The natural humidity of both EB1 and EB2 disappeared at temperatures of below 100°C. Hydrotalcite and sjögrenite dehydration took place at 280°C and calcium-aluminate dehydration at 370°C. Brucite decomposition into periclase occurred at 420°C and the decomposition of portlandite with water loss occurred at 460°C. Finally, calcite decarbonisation with carbon dioxide loss occurred from 760°C.

In EB2, brucite decomposition into periclase (molecular water loss) occurred at 420°C; decomposition of a low magnesite content (not evidenced by XRD) at 680°C; the oxidation of sulphurs (oldhamite) due to oxygen gains occurred at around 900°C and the oxidation of metallic iron (Fe⁺² to Fe⁺³) at around 950-1000°C. Finally, decomposition (not observed in XRD) of low calcite contents may be mentioned.

2.1 Volumetric instability of LFS

In the present study, we were unable to stabilize EB1, which revealed the presence of hydrotalcite, a compound associated with expansiveness[7]. In laboratory tests, two stabilization procedures (Type 1 and Type 2) were applied: firstly, to hydrate its expansive compounds and, secondly, in an attempt to stabilize them.

In the Type 1 procedure, EB1 remained submerged in water, while in Type 2, EB1 was sprayed with water (Figure 2). Neither procedure was effective at stabilizing the EB1 slag.

Because of the above-mentioned factors, it is recommended that LFS should not enter into contact with water. In Figure 2, the expansion of LFS was determined according to UNE 1744-1:2009 and two laboratory type tests.
With regard to the controlled hydration of LFS under lab conditions, SEM imaging (Figure 3) revealed equidimensional crystals of calcite and needle-like compounds, similar in structure to the hydrotalcite family.

3 DESIGN OF CEMENT MIXES

As presented in Figure 1, the EB2 slag type was of an acceptable grain morphology and size, however EB1 had to be gently crushed.

3.1 Cement pastes

Having replaced 10% and 20% of cement types I and II by weight of cement with LFS, several trial (10x10x60 mm) specimens were fabricated with a water/cement ratio (w/c) of 0.4 (see Table 2), respectively. Mixes 10 and 11 (partial replacement of limestone filler), were produced to analyze possible hydraulicity because of LFS, despite the foreseeable reduction in compressive strength. The addition of limestone fillers reduced compressive strength at early ages by a higher value than in the case of partial substitution of LFS. LFS therefore produced some hydraulic effect, although both replacements showed similar mechanical attributes at 28 days of curing.
Table 2: Cement paste mixes. Partial substitution of cement by LFS

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cem I (%</th>
<th>Cem II (%</th>
<th>Filler (%</th>
<th>EB1 (</th>
<th>EB2 (</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7days</td>
</tr>
<tr>
<td>0-ref.</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>65.5</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>0</td>
<td>55.3</td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>0</td>
<td>0</td>
<td>20</td>
<td>0</td>
<td>44.3</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>54.4</td>
</tr>
<tr>
<td>4</td>
<td>80</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>20</td>
<td>42.9</td>
</tr>
<tr>
<td>5-ref.</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>40.1</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>90</td>
<td>0</td>
<td>10</td>
<td>0</td>
<td>38.4</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>80</td>
<td>0</td>
<td>20</td>
<td>0</td>
<td>35.9</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>90</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>25.0</td>
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<tr>
<td>9</td>
<td>0</td>
<td>80</td>
<td>0</td>
<td>0</td>
<td>20</td>
<td>23.4</td>
</tr>
<tr>
<td>10</td>
<td>90</td>
<td>0</td>
<td>10</td>
<td>0</td>
<td>0</td>
<td>50.5</td>
</tr>
<tr>
<td>11</td>
<td>80</td>
<td>0</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td>44.4</td>
</tr>
</tbody>
</table>

The mechanical behaviour of the design cement pastes that incorporated the partial substitution of cement by 10% and 20% of LFS (in both cement types), led in both cases to a reduction in their compressive strength. However, partial substitution of 10% as a (low cost) admixture to clinker could be realistic for future applications for the purpose of obtaining an additional CEM II.

Finally, it may be mentioned that mineralogical analysis by XRD revealed tricalcium aluminate (AC3), katoite, periclase and other hydrated compounds, in similar proportions to LFS content. As is usual in the presence of LFS, other hydrated compounds were also present in similar proportions.

3.2 Masonry mortars: mechanical behaviour and dimensional instabilities

Several mixes were prepared with overall slumps of 175±10 mm (M7.5, UNE-EN 998-2), in order to achieve realistic applications in commercial mortars, as detailed in Table 3.

Table 3: LFS masonry mortars designs and mixing details

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Cem I</th>
<th>Sand</th>
<th>Water</th>
<th>10%Filler</th>
<th>20%Filler</th>
<th>10%EB1</th>
<th>20%EB1</th>
<th>10%EB2</th>
<th>20%EB2</th>
</tr>
</thead>
<tbody>
<tr>
<td>MM0-ref.</td>
<td>350</td>
<td>3150</td>
<td>472.5</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM10F</td>
<td>315</td>
<td>35</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM20F</td>
<td>280</td>
<td>---</td>
<td>70</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM10EB1</td>
<td>315</td>
<td>---</td>
<td>35</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM20EB1</td>
<td>280</td>
<td>---</td>
<td>---</td>
<td>70</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM10EB2</td>
<td>315</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>35</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>MM20EB2</td>
<td>280</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>70</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

The mixtures in this study were only based on cement type I. The reason, as demonstrated in previous studies[7], is that these combinations were better suited than CEM II (plus LFS by-
products) to obtain viable industrial applications in the field of masonry mortars (building sector). The mechanical properties were measured following the procedure detailed in UNE EN 196-1:2005, by testing prismatic pieces of 40x40x160mm, as detailed in Table 4.

Table 4: Masonry mortars with LFS: fresh (workability) and hardened (mechanical) behaviour, respectively.

<table>
<thead>
<tr>
<th>Masonry Mortar</th>
<th>Fresh gravity (g/cm³)</th>
<th>Slump (mm)</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>7 d</td>
<td>28 d</td>
<td>45 d</td>
</tr>
<tr>
<td>MM0-ref.</td>
<td>2.18</td>
<td>175</td>
<td>11.4</td>
<td>17.6</td>
</tr>
<tr>
<td>MM10F</td>
<td>2.19</td>
<td>179</td>
<td>9.7</td>
<td>15.1</td>
</tr>
<tr>
<td>MM20F</td>
<td>2.17</td>
<td>174</td>
<td>7.7</td>
<td>12.5</td>
</tr>
<tr>
<td>MM10EB1</td>
<td>2.17</td>
<td>178</td>
<td>9.3</td>
<td>15.0</td>
</tr>
<tr>
<td>MM20EB1</td>
<td>2.18</td>
<td>191</td>
<td>7.6</td>
<td>11.8</td>
</tr>
<tr>
<td>MM10EB2</td>
<td>2.14</td>
<td>178</td>
<td>7.4</td>
<td>12.1</td>
</tr>
<tr>
<td>MM20EB2</td>
<td>2.14</td>
<td>177</td>
<td>6.1</td>
<td>10.2</td>
</tr>
</tbody>
</table>

Under compression loads, a larger proportional reduction in compressive strength with higher LFS contents was observed. However, in view of the expected mechanical requirements for alumina-saturated LFS types (EB1), a maximum partial substitution of 20% LFS (by weight of cement) may be assumed. Quite unlike EB2, the alumina-saturated LFS types showed some hydraulic properties at an early age and after approximately six-months (mainly due to their high aluminate content), which gave them greater strength in comparison with the admixtures containing limestone fillers.

Importantly, a dimensional variation of the cement mortars with 10% EB1 and EB2 in substitution of CEM I was also observed in the mortar shrinkage analysis. Both the EB1 and EB2 slag mortars were expansive: 0.09 mm/m and 0.05 mm/m, respectively, at 28 days, and, 0.120 mm/m and 0.08 mm/m, respectively, at 75 days of ageing, at which point expansion stabilized.

The average expansion value was 0.1 mm/m (both mortar slags), the effects of which could partially compensate the expected shrinkage of dry mortars (around 0.5 mm/m). However, it should be remarked that uncontrolled expansion may be very damaging, even though it may compensate shrinkage contraction.

4 DURABILITY STUDIES

Two main aspects, strongly related to masonry mortar applications in the building sector, guided the experimental work. Firstly, the efflorescence evolutions were tested at lab scale and, secondly, the effect of wetting-drying cycles.

4.1 Mortar efflorescence at lab scale

Mortar efflorescence, a key factor of great importance in building applications, prompts aesthetic changes that are unsightly, for example, in façade applications. When dissolution enters the porous system in a mortar, by capillary action, it can precipitate mineral salt phase changes, the crystallization of which generates certain mechanical effects due to variations in pressure. Cyclic dissolution/precipitation of these salts, due to variations in temperature and humidity, is one of the most important deterioration processes in porous materials such as
stone, involving aesthetic modifications and their loss of cohesion.

In this research, the effect of an aggressive experimental method was applied and standardized using a total of 6 specimens (40x40x160mm) per mortar type (Table 3): MM0-ref, MM10EB1 and MM20EB1. The standard was based on UNE 83830 EX Spanish code, using the following test method: 7 days at room conditions (20±2°C plus 65±5%RH) and, additionally, the aggressive procedure maintained the specimens for 7 days in a climate chamber at 40±2°C and 50±5%RH. These conditions produced stronger water vaporization and, consequently, salt precipitation or efflorescence. Some of the testing details are presented in Figure 4.

![Efflorescence at lab scale: room conditions (left) and climate chamber (right)](image)

From the analysis of the results, it may be stated that the partial substitution of 10% and 20% by weight of cement type I for EB1 ladle furnace slag had no aesthetic effect on any of the analyzed mortars.

4.2 Wetting-drying cycles at lab scale

Wetting-drying cycles at lab scale analyzes the effect of natural weathering conditions related to water saturation and the drying effect on masonry mortars. The intention is to use accelerated procedures under laboratory conditions to simulate dry and rainy days (see Figure 5).

As previously stated, this test is of a great importance, because of the potential expansion of some of the LFS components. This property was tested according to ASTM D–559–03 in 30 wetting-drying cycles (24h per cycle) applied to the mortars described in section 4.1.

A total of 12 prismatic specimens (40x40x160mm) of each mortar type were tested. Following their accelerated weathering, the changes in compressive strength were evaluated at four different ages: 0 days, 28, 90 days, 180 days and 1 year (after casting), respectively.

Although there were minimal gains in mortar weight due to the presence of LFS, as may be seen from the results in Table 5, the effect of these by-products implies a degree of hydraulic
reactivity (see above discussions) over time. At the end of weathering cycles, none of the specimens appeared to be damaged.

Figure 5: Wetting-drying testing stages: climatic chamber, submerged in water and MIP technique

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Weight (%)</th>
<th>Average $\bar{\Omega}_pore$ ($\mu m$)</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0d  1y  28d  90d  180d  1y  7d  90d  180d  1y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MM0-ref.</td>
<td>+0.08</td>
<td>0.35  0.38  17.2  19.2  18.2  19.5  4.5  4.9  5.2  4.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MM10EB1</td>
<td>+0.09</td>
<td>0.48  0.44  14.1  15.0  14.6  14.4  3.8  4.2  4.3  3.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MM20EB1</td>
<td>+0.13</td>
<td>0.67  0.44  10.7  12.8  12.3  11.0  2.9  4.1  3.5  2.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Due to the effects of the ageing cycle, the average diameter of small pores was reduced. These changes in pore diameter were measured by Mercury Intrusion Porosimetry (MIP). Unlike the changes in the reference mortar, the LFS mortars had lower average pore sizes, probably due to new hydration products, or new phase precipitations, associated with the hydration of embedded ladle furnace slags.

As regards their expected mechanical strength, the compression load responses of LFS mortars, after exposure of over 1 year, had improved their overall value by 5% with regard to early ages (0 days), in a similar way to that observed in the reference mortar. However, after one year of exposure, all of the flexural strength values of the masonry mortars (with or without EB slags) remained constant.
5 CONCLUSIONS

- The present research has characterized the hydration and expansion processes of two types of ladle furnace slags (LFS): EB1 (high-alumina) and EB2 (high-silica). Additionally, both by-products were used as raw materials in the fabrication of corresponding pastes and masonry mortars, in partial substitution of cement.
- The physical-chemical performance of high-alumina LFS was better than the high-silica LFS, underlining the importance of not wetting LFS.
- With a view to their real application in the near future (building materials sector), this experimental study has investigated the performance of mixes based on LFS, Portland cements (type I and II), and natural aggregates (limestone sands) in relation to the following aspects: mechanical behaviour (under compression and flexural loads) and durability under weathering ageing (efflorescence and wetting-drying cycles).
- LFS can induce slight hydraulic reactivity. Partial substitution of less than 20% by weight of cement for LFS demonstrated no negative effects on either the mechanical performance or the durability of the masonry mortars analyzed in this study.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude for funding from: the Basque Regional Government (IT781-13 Research Group), the Spanish Ministry of Science and Innovation (BIA2014-55576-C2-2-R) and FEDER Founds. We also thank the Iñaki Goenaga Foundation and the University of the Basque Country (PIF 2013 grant). Additionally, this work would not have been possible without the construction materials generously supplied by Arcelor Mittal (Sestao) and Morteros and Revocos Bikain.

REFERENCES

LARGE INFRASTRUCTURE ECONOMIC, SOCIAL AND ENVIRONMENTAL SUSTAINABILITY ASSESSMENT.
AN EARLY APPROACH TO THE CANAL DE NAVARRA IRRIGATION AREA CASE

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Key words: Large infrastructure sustainability, Agricultural infrastructure impact,
Sustainability Assessment, Canal de Navarra.

Summary. The author is undertaking a PhD research project, which will attempt to develop
a generally accepted tool for assessing lineal infrastructures sustainability. As an initial
approach to the matter, the paper focuses on the Environmental, Social and Economic
aspects of infrastructure sustainability assessment, deepening into the specific impacts of
major agricultural projects. It deals with the specific case of making an early sustainability
assessment of a 22,500 ha irrigation infrastructure in northern Spain, five years after
construction completion, using the Berne Compass model.
1 INTRODUCTION

On December 12th, 2015 Laurent Fabius -Frenchman Minister of Foreign Affairs- approved the Climate Summit Paris Agreement, hitting the table with his little green deck. That move sealed the 196 countries pact to the legally binding text, for the first time committing themselves to "keep the temperature increase below 2 degrees compared to pre-industrial levels and pursue efforts to limit the increase to 1.5 degrees". 28 years before the United Nations had already established in the Bruntland Report [1] the generally accepted criteria of sustainability: to ensure satisfying the needs of the present without compromising the ability of future generations to meet their own needs.

A large infrastructure construction involves the spending of many public resources, it affects a significant part of the population, and generally implies large variations on environmental conditions before and after its existence. In order to pass the test of time, infallible judge for the success or failure of the Public Administration that decided to implement it, any infrastructure should be able to maintain the balance between the positive and negative long-term impacts it produces. It must be sustainable.

The 2008 global economic crisis has clearly shown, particularly in developed countries, that infrastructure economic, social and environmental sustainability has not been sufficiently weighted as decisive criteria for determining its construction. If public authorities had reliable and transparent tools for making an objective assessment -on initial planning phases- of economic, social and environmental sustainability of large infrastructures, society would be closer to ensure satisfying the present needs without compromising the future ones.

2 THE CANAL DE NAVARRA IRRIGATION AREA PROJECT

The Itoiz–Canal de Navarra hydraulic system is an ambitious Government of Navarra project whose main objective is taking advantage of excess water out of the northern part of the region -with average annual rainfall reaching 2,000 mm/year- to the middle and south of Navarre, much drier, with contributions from 400 to 800 mm/year.

A 418 hm³ reservoir and a 197 km long channel, with 45 m³/s head flow capacity, feed an irrigation scheme of 59,200 hectares, also supplying water to 650 hectares of industrial areas and complementing the drinking water supply of 70 % of the Navarra population. The development of the project is planned in three phases.

The first phase of the irrigation area, covering 22,500 hectares, was awarded in 2006 by a concession contract to Aguacanal. The agreement includes the design, construction, financing and operation and maintenance of the infrastructure for a 30-year period. Works began in 2007 and were completed in 2011, and meant an initial investment of 185 million euros.

The pipeline distribution network, 800 km long, transports water from the channel outlets to the more of 6,000 farmers’ pieces of land. It is real-time managed by the concessionaire, along with the 900 km road infrastructure, the power lines and the pumping stations.
3 IMPACT OF THE CANAL DE NAVARRA IRRIGATION AREA

3.1 A three legged stool

Margaret Robertson, member of the American Society of Landscape Architects (ASLA), defines sustainability [2] as humanity’s rapidly-evolving response to the urgent planetary challenges we all face, referred to systems and processes that are able to operate and persist on their own over long periods of time.

Planet earth can be seen, from a sustainability point of view, as a three-legged stool. Those permanently connected three legs -also called the triple bottom line, or the “three E’s”- are:

- environment: health of living systems
- economics: fair distribution of prosperity
- equity: equal opportunities for all, living today or yet to come

Jeffrey Sachs, director of the Earth Institute at Columbia University and known as one of the world's leading experts on sustainable development, adds a value-based approach to the concept [3], promoting the idea that including integrity, moderation and justice into the sustainability recipe should lead to common benefit.
3.2 Impact of large infrastructures

As indicated by the AT Kearney report [4] on the contribution of infrastructures to economic and social development, these are essential for social welfare and contribute to improving the quality of life of citizens, increasing their safety by facilitating their mobility and contributing to reduce pollution.

For example, improving the road network benefits connections between cities and promotes structuring of the territory. Investment in high-speed rail network reduces travel time by train, making it competitive with road and air transport which, in turn, provides environmental benefits. The development of urban infrastructure reduces the number of accidents, the amount of travel wasted time and air pollution.

However, these improvements are also accompanied by a high negative impact. The budget needed to make such investments affects the distribution of wealth in a city, region or entire country and, indirectly, the pockets of all citizens through taxes. The economic impact can also be seen from the point of view of the user of the infrastructure, in the case where they pay for the use of it as, for example, in a toll road.

 Needless to explain the possible negative environmental effects of a project of this kind, due to the nature of its own big dimension.

It shows especially important the analysis of the possible negative consequences of large infrastructures in the social field, as they are not generally studied in the same depth during the planning phase.

In particular, large linear infrastructures—such as highways, railroads, channels, etc.—produce a "barrier effect" that divides the territory in two, as cut by a giant knife, so that often the development of one of the banks is different from the other one.

3.3 Agricultural infrastructures

The case studied, the irrigation area of the Navarre Canal, belongs to the agricultural infrastructure sub-sector, whose impact on the planet and people living on it has specific characteristics.

The positive impact is self-evident; we all need to eat every day, either with products provided directly by the land, either with animals fed by those products. There are also beneficial effects such as population making a living in the countryside, the maintenance of local species, etc.

The negative impact of agriculture, pointed out by Sachs in his last book as the main responsible for climate change [3], is also undeniable:

✓ Water and energy consumption
✓ Emission of large amounts of CO2
✓ Production of methane and nitrous oxide from livestock and fertilizers
✓ Destruction of the natural habitat
✓ Introduction of invasive species
3.4 The Canal de Navarra Irrigation Area Case

The Canal de Navarra project, even though it is a large agricultural infrastructure, has some specific features that promote the positive effects, not suffering so much the downsides.

In terms of water consumption, it takes advantage of the annual winter water surplus on the Pyrenees, that otherwise would be lost in the river flow. Until now it was just not being used. In addition, there have only been installed highly efficient irrigation systems -pressure pipelines and sprinklers- instead of the traditional irrigation ditches, assuring that all the water running at the beginning of the channel gets to the farmers.

As for energy efficiency, more than 90% of the farmland is irrigated by natural water pressure -because the channel runs higher than the cultivated fields- so only near 2,000 ha need pumping stations. Even in case of needed extra energy, the maximum pumping elevation is less than 150m.

The land irrigated now by the Navarra Canal was, until 2006, being rain fed, so no deforestation or big landscape change has been done. More so, in dry summers the old farmers used to take from the river the needed water that did not fall in the form of rain, drying it and killing natural species.

The special characteristics of the concession agreement between the Navarra Government and the Concessionaire for the development of the irrigation area, a Public Private Partnership, determine the main economic and social impacts of the project:

- Design, Build, Finance and Operation scheme: the public part transfers to the private one all the steps for service providing. No initial capital investment is needed on the Government side. On the other hand, a significant part of the next years’ public budget is captive, not allowing for other investments.

- Long-term agreement: on a 30-year term contract the Concessionaire becomes a Government collaborator and trust between both parts is vital. The flipside is inflexibility. If new social demands arise, the agreement scheme may play as a boundary for what both parts are otherwise ready to implement.

- Shadow toll payment system: the user does not pay for the provided service –only around 5% of the total amount- putting into place some kind of “unfairness”. All Navarra taxpayers are sustaining the agricultural industry main infrastructure.

- Key Performance Indicators: the balancing system between project risks and monetary profit transferred to the private entity lays on the three contractual KPI. If in any given period of time indicators go below possible 1,000 maximum value, the Concessionaire receives less money. They are defined as follows:
  - K1 (efficiency indicator): percentage of water taken from the channel inlets delivered at the farmland outlets
  - K2 (effectiveness indicator): 24/7 water pressure and flow delivered at the hydrants, as compared to expected standards
  - K3 (client satisfaction indicator): time needed to solve any problem
4 A PRELIMINARY SUSTAINABILITY ASSESSMENT

4.1 The need for Sustainability Assessment

Regional and country Governments face nowadays -more than ever before- the need to answer their voters’ questions related to political action impacts. Therefore, feasible and practical implementation of sustainable development principles plays a very important role.

It is important that projects are evaluated *ex ante* from the point of view of all three sustainability dimensions. Political decision-makers can choose more targeted development projects if they are aware of whether, overall, they will improve or worsen the sustainable development of a region or a community. The problem is that quantitative data on the impacts of a project cannot be obtained easily before it is implemented.

4.2 The Sustainability Compass of Berne [5]

It isn’t easy to find a simple, balanced and generally accepted sustainability assessment tool for taking decisions in the context of local politics. For the purpose of this paper -a preliminary and simplified assessment of a young large infrastructure- I will make use of the *Sustainability Compass of Berne*, a pragmatic and qualitative evaluation tool aimed at filling this gap. It is based on a grid of targets and indicators for the ecologic, economic and social sustainability dimensions. It produces a simplified evaluation of the potential effects of a project on these dimensions.

The aim of the sustainability compass is to provide a simple qualitative evaluation of the impacts of important projects on the sustainable development of a given region or community. By using the tool, planned projects can be evaluated in a standard, comparative and comprehensive manner with respect to their ecologic, economic and social impacts.

The evaluation is made up dividing each dimension into target areas, and each of these into specific indicators. Each indicator gets a value from -2 to +2, depending on the impact effect, and the mean is bottom up calculated for all target areas and all dimensions. A green, yellow or red traffic light is finally awarded to the project, indicating the level of global acceptance.

![Hierarchical structure of the Sustainability Compass](image)

Figure 2: Hierarchical structure of the Sustainability Compass
### 4.3 A Canal de Navarra Irrigation Area Sustainability Evaluation

An initial approach to the project sustainability evaluation has been made, using the aforementioned Berne Compass, and the obtained result is a positive impact on each of the three sustainability pillars, with a global mean of 0.22/2, indicating that Project promotes sustainable development. These are the detailed results:

#### Sustainability compass: Evaluation

**Project:** Canal de Navarra Irrigation Area. Phase 1  
**Evaluating by:** J.E. Arizón  
**Date:** Jan. 5th 2016  
**Scope:** Construction: 2006-2011; O&M: 2011-2016  
**Reference:** ZRCN Sust Asse v6

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<th>Target area</th>
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<td>Social security</td>
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**Overall score:** 0.22

![Figure 3: Sustainability Compass evaluation results](image-url)
The assessing difficulty lies in the capability of the evaluator to conduct a fair and balanced appraisal of each indicator, according to the information available (e.g. annual evolution of water consumption, labor market statistics, etc.).

Due to the length limit of the present work, no explanation is provided for each criteria, having been used as reliable sources -besides the author's own knowledge because of his direct participation in the project-, documents from Gobierno de Navarra [6] and Cámara de Comptos de Navarra [7].

5 CONCLUSIONS

- Large infrastructure projects, when implemented, cause large impacts on territory, finance and people. Agricultural infrastructures produce specific sustainability impacts, due to the use of great amounts of water, energy and soil.
- Globalization and limited resources make now, more than ever before, necessary to have available \textit{ex ante} sustainability assessment of public initiatives.
- No generally accepted tool is available for large infrastructure sustainability assessment. The Berne Compass is close enough to be used for analysis purposes.
- The 22,500 ha Canal de Navarra Irrigation Area Phase 1 shows, five years after construction completion, signs of having positive sustainable development impact, on each of the sustainability pillars: economic, social and environmental.

REFERENCES

SELF-HEALING PERFORMANCE OF MAGNESIA BASED PELLETS IN CONCRETE
ICCS16

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Key words: Self-healing, concrete, pellet, magnesium oxide, film coating

Abstract. This study presents the potential of using magnesium oxide (MgO) based pellets to improve crack self-healing capability of concrete. Commercial hard burnt MgO pellets were used in this work. Pellets were enclosed in a film coating layer of polyvinyl alcohol (PVA) based material to sequester the core materials until the crack time. The coated pellets were replaced fine lightweight aggregates by 10% in mortar concrete mix. Two more mortar mixes were prepared; control and another mix including 10% of uncoated MgO pellets. The self-healing efficacy was verified through three-point flexural strength and monitoring cracks closure using optical microscope images. The results indicate that the coated MgO pellets presented 14.6% and 18.5% strength regain at 30 and 120 days respectively. In contrast, the control samples and the samples contained uncoated pellets did not exceed at best 9% strength recovery at both ages. Moreover, the microscopic investigation showed complete cracks closure for concrete samples contained the coated pellets compared to partial closure for the pristine samples and those contained uncoated pellets.

1 INTRODUCTION

Surface opening cracks are common defects in concrete structures. They allow penetration of water and/or other agents that result in loss of durability earlier than expected. Thus, their repairing is very essential and inevitable. Currently, maintenance and repair of concrete structures generally relies on regular inspection programs, which are expensive, and it also depends on a combination of non-destructive testing (NDT) and human perception [1]. In case of severe damage, the structural component is replaced entirely while repairs are attempted for less extensive damage. Vast amounts of money are spent each year on inspection and repair as direct and indirect costs, the latter often being much larger than the former. For instance, in the USA, the annual economic impact associated with maintaining, repairing, or replacing deteriorating structures is estimated at $18-21 billion [2]. The American Society of Civil Engineers estimated that $2.2 trillion is needed for five years starting from 2012 for repair and retrofit; a cost of $2 trillion has been predicted for Asia’s infrastructure for the same period [3]. Europe spends more than half of its annual construction budget on repair works [4], while in
the UK, repair and maintenance costs account for over 45% of the total expenditure on construction [5]. Moreover, repair works have a significant adverse environmental impact particularly in cases that partial or complete replacement of structures is required. It is known that the production of 1 tonne of Portland cement (PC), as often being the main constituent on concrete, releases about 0.85 to 1.1 tonnes of CO₂ [6]. Approximately 3.6x10⁹ tonnes of cement were produced worldwide in 2014 [7]. The CO₂ emissions associated with the production of cement are very significant, and are estimated at a 7% of the global anthropogenic CO₂ emissions [6].

Therefore, developing innovative technologies to overcome these challenges has become utmost necessity. Over the last two decades, the idea of concrete structures were able to heal their cracks without any external aids has emerged as a possible cure to enhance durability and serviceability, improve safety and reduce maintenance costs [8]. Different approaches and materials have been extensively developed. For instance, some approaches are aimed at improving the natural mechanism of autogenous crack healing, which is generally attributed to the hydration of un-hydrated cement grains and may be aided by the precipitation of calcium carbonate [9–11]. Others are designed to modify concrete by embedding microcapsules or hollow fibers with adhesive agents, to heal cracks autonomously [4,12]. Some research groups also suggested using specific bacteria spores with nutrients in concrete to promote precipitation of calcium carbonate in cracks [13,14].


Although the above minerals have shown some promising results as self-healing agents, it has been difficult to apply this approach as there has been a significant loss in workability of fresh concrete and possibly reduction in the self-healing efficiency of the hardened concrete, due to unavoidable further reactions between the incorporated powder, water and other products during the mixing and hardening processes of concrete [20,21].

Therefore, this study aims to introduce magnesium oxide (MgO) as a potential self-healing agent in form of pellets by enclosing each pellet in a film coating layer to avoid the foregoing disadvantages of the direct application of powder self-healing agents.

MgO is an expansive mineral produced mainly from calcination of magnesite and a smaller proportion of the world’s MgO production comes from seawater and brine sources. It is widely used as a cement or as a shrinkage reducing mineral additive in concrete [22]. The main product of the hydration process of MgO is brucite Mg(OH)₂ [23]. Brucite has a layered structure and its morphology varies depending on the magnesium source and forming conditions. An equilibrium pH of Mg(OH)₂ in water is 10.5. It is less soluble, less reactive, and less mobile than Portlandite, hence more durable. In MgO-PC blends, both brucite and the hydration products of PC are formed, which means that PC and MgO hydration occurs independently. There is a clear difference between the temperature development profile of PC and MgO. MgO has much higher temperature and earlier peak corresponds with its final set, which means that
MgO is more reactive than PC [22,24]. In the presence of moisture and CO₂, Brucite can yield one or more hydrated magnesium carbonates (HMC). What is important here that both the hydration and carbonation reactions are expansive with volume increase from MgO to Brucite of 2.2 times and from Brucite to HMC above of 1.8-3.1 times depending on the carbonate formed [27].

2 MAIN MATERIALS

CEM 1 (52.5N) Portland cement supplied by Hanson, UK was the principal material for all mortar mixes. The mortar mixtures were also prepared with fine lightweight sintered fly ash aggregates (4 mm maximum particle size), referred as LYTAG and obtained from LYTAG Ltd., UK. The oven dry bulk density of the fine LWA is 900±100 kg/m³.

Commercial hard burnt MgO pellets (CP) with (1-2) mm diameters were examined in this study as a self-healing agent. These pellets were obtained from Martin Marietta Magnesia Specialties, USA.

For coating purposes, polyvinyl alcohol (PVA) was used as the main coating material. Soluble starch was used as an additive to modify the PVA characteristics and formaldehyde solution 37-40 % (w/v) was used as a cross-linker. The PVA used has a degree of hydrolysis of 98-98.8 % and an average molecular weight of 31,000-50,000. All the coating reagents were obtained from Fisher Scientific, UK.

3 EXPERIMENTAL PROCEDURE

3.1 Enclosing pellets in polymer based coating

The coating solution was prepared by adding a predetermined amount of PVA to distilled water to achieve a solution of 10% solid content. The mixture was slowly heated to about 90 °C with stirring of 250 rpm until a homogeneous solution was formed. Water was added to compensate for any moisture loss that may have occurred during the heating process [25]. Desired amount of starch and glycerol was slowly added under continuous stirring. Butanol was used to avoid frothing. And then, formaldehyde of 10 wt% was added and stirring was maintained for 3 hours to completely gelatinise the starch. As explained in [25–27], the application of formaldehyde leads intermolecular and intramolecular linkage of -OH between starch and PVA molecules and therefore the hydrophilic groups were reduced and thus decreased the water absorbency of the blend.

The CP pellets which passed through sieve 2 mm and retained on sieve 1 mm were selected for coating. The coating solution was applied to the pellets by using spray coating technique associated with simultaneous drying by blowing a stream of hot air onto the surface of the pellets. DP-14 “Agglo-Miser” a disc pelletiser supplied by Mars Mineral was used in this work. During rotation of the disc pelletiser, the pellets were sprayed with the coating solution. The spray gun used in the coating process is Gravity Feed Mini-HVLP gun with 1 mm nozzle size. The coated pellets were dried at room temperature for 48 hours before stored in an airtight plastic container until tested. Ten pellets were taken randomly to examine their coating thickness by using scanning electron microscope (SEM) images. Hereinafter the coated pellets are referred to as CCP.
3.2 Mortar concrete mixes

Three mixes of mortar concrete specimens with water-to-cement ratio (w/c) of 0.4 and 1:1 cement to fine LYTAG proportions were prepared for this study. The pellets were partially replaced the fine LYTAG by 10%. The mixes were the reference, the mix containing CCP pellets, and the mix included uncoated pellets (CP). These mixes are referred to as M1, M2 and M3 respectively. For each mix, 12 cubes (40 mm X 40 mm X 40 mm) were prepared for compressive strength analysis and six prism samples (40 mm X 40 mm X 160 mm) were prepared for crack forming through a three-point flexural test, and crack healing analysis overtime. The samples were demoulded after 1 day of curing and then cured in a water tank at temperatures of 20°C ±2°C and a relative humidity 60 °C ±2 °C until the designed testing age.

3.3 Unconfined compressive strength (UCS)

The compressive strength testing was carried out using Controls Advantest 9 with a maximum capacity of 250 kN and a loading rate of 2400 N/s. Triplicate cubes were tested at ages of 7, 28, 56 and 90 days and the strength reported was the average of the three specimens.

3.4 Three-point flexural strength test

Cracks with a controlled width of 0.30 mm were induced in the prism specimens at age of 7 days by three-point bending test. This test was carried out using 30 KN INSTRON static testing frame and attached with a clip gauge to monitor the crack mouth opening. In a compliance with BS EN 12390-5:2009, the testing specimen was placed upon a base of two supports with a span of 120 mm. Then, the loading shaft was settled at mid span and gently conducted with the prism top surface. The ramp speed was adjusted into 0.1 mm/min and cyclic loading was disabled. Prior to cracking, a 1.5 mm deep notch was sawn on the underneath of all beams at the mid-point. The cracked specimens were vertically setup into the water tank in order to keep the crack surfaces in contact. The six specimens from each mix were re-cracked for the second round until failure at 30 days after the first crack. Three of them were returned back to the water tank for testing any further potential healing of the new cracks. Likewise, the three samples were cracked for the third round until failure after further 90 days. The strength recovery after the second and third rounds of cracking was calculated according to equation (1) [28]:

\[
\eta \% = \frac{\sigma_2}{\sigma_1}
\]

Where: \(\sigma_1\) is the maximum stress for the virgin specimen and \(\sigma_2\) is the maximum stress for the healed specimen.

3.5 Visual evaluation of the cracks sealing

The crack surfaces sealing of the prism specimens were monitored via digital microscope images depending on the elapsed time. GXCAM 1.3 type stereoscope supplied by GT Vision Ltd was used. Specimens were removed from water weekly for stereomicroscopic inspection and photographic imaging for quantification of crack-healing in time.

Additionally, the widths of crack faces were measured weekly at three locations for each specimen. The average value for the three readings was taken to analyse the crack surfaces sealing with time. It is noteworthy that despite the specimens were cracked for a controlled
width of 0.30 mm, upon load removal all specimens had remaining crack width less than 0.15 mm.

4 RESULTS AND DISCUSSION

4.1 Evaluation of the pellets coating

Figure 1 (a) shows SEM image of an individual pellet which completely coated by the PVA based coating layer. Thickness of the coating layer was examined for different pellets as one of them shown in Figure 1 (b). The thickness of the PVA shell varies between 10 µ to 50 µm.

![Figure 1: SEM images of coated MgO pellet: (a) individual pellet completely coated, and (b) thickness of the coating layer](image)

4.2 Influence on the unconfined compressive strength (UCS)

The compressive strengths of all mixes at ages 7, 28, 56, and 90 days are shown in Figure 2. The UCS of the samples containing CCP pellets were slightly lower than the control samples at all ages. This might be attributed to the interfacial bonding between the PVA coating and the hydrated cement products. On the other hand, the existence of some uncoated pellets probably led to unwanted early reaction of MgO. However, the addition of CP had a positive influence on UCS for all ages except at 90 days. For instance, the 28 and 56 days UCS values of M3 exceeded the reference value at the same ages by ~ 3.5 and 3 MPa respectively. After 56 days, the compressive strength of M3 samples have not changed. Generally, the addition of coated or uncoated MgO pellets showed a minimal influence on the compressive strength at all ages.

4.3 Flexural strength recovery

According to equation (1), Figure 3 shows the percentage of flexural strength regain at the second and third rounds of cracks. The control samples reached a maximum of only 2.3% strength regain at both rounds. While, the specimens contained CCP pellets demonstrated
14.6% and 18.5% strength regain at the second and third rounds of cracking respectively. It is strongly believed that this advantage of M2 samples is due to precipitation of brucite and HMC as hydration products of magnesium oxide released from inside pellets. On the other hand, the third round of cracking at 120 days exhibited better strength recovery compared with the second round. This indicates that some quantities of MgO pellets remained intact for longer times. This is due to the slow rate of MgO hydration in comparison with Portland cement ingredients [29].

Figure 2: UCS of all mixes at 7, 28, 56 and 90 days

M3 samples achieved much better strength regain than the control samples but still less than M2 samples. This can be attributed to some early reactions between the uncoated pellets and the existent water. This is an indication of the efficacy of coating pellets to avoid like these unwanted early reactions.

Figure 3: Strength regain of all groups for the second and third rounds of three point bending test
4.4 Evolution of crack surfaces sealing with time

The microscopic image analysis was conducted out to monitor the change in the width of cracks induced in the prism mortar specimens. The crack-sealing process is used as an indication for efficiency of the self-healing cementitious composites. Figure 4 presents typical stereomicroscope images for crack sealing just prior to the second and third rounds of cracking. It is obvious that all cracked samples had undergone a certain extent of sealing and showed some crystal depositions on the crack faces and walls at both ages. However, M2 samples distinctively showed complete surface sealing at 30 days after the first round of cracking and similarly at 90 days after the second round of inducing crack. This in turn backs up the strength regain results and shows the significant effect of addition coated self-healing pellets to the concrete compared to the pristine samples and those contained uncoated pellets.

Figure 4: Representative microscope images of the progress crack surfaces sealing
5 CONCLUSIONS

Preliminary results indicate that the coated MgO pellets showed promising results in both sealing cracks regain the flexural strength of mortar samples compared with the control samples. However, as the hard burnt MgO is a low reactive material; the coated pellets demonstrated a relatively low strength regain percentage compared with other approaches. Enclosing MgO as an expansive mineral in an adequate coating layer helped to control the timing of autonomic healing. PVA based coating could withstand the friction between pellets and the mixer blade in fresh concrete mixing due to its stiffness.

Further investigations are needed to figure out the exact mechanism of the healing process and the characteristics of the healing products.

REFERENCES


STUDY OF CONCRETE MODIFICATION EFFECT WITH RECYCLED AGGREGATE TREATED BY CARBONATION

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Key words: Recycled aggregate, Carbonation, Density, Strength, Drying shrinkage

Abstract. Ready-mixed concrete in Japan, if that were not used in the field, concrete that would remain back to the ready-mixed concrete plants is increasing year by year. In Tokyo, in order to reduce the return concrete, it has been taken also measures such as providing a penalty in return concrete, however it can’t be easily reduced. Therefore, it is desired the technique for reusing of these concretes. By separating all materials from return concrete, the aggregate is realized a method of utilizing the recovered aggregate. However, in case of that time has elapsed from the mixing on initial ready-mixed concrete, it is difficult to separate them.

In this study, these return concretes are once hardened, it was investigated how to reuse as an aggregate by crushing. However, since the mortar is attached to the surrounding aggregate, the quality of the produced aggregates are not good. Therefore, even lower performance as recycled aggregate, it is difficult to use as aggregate for concrete. However, if it added a high degree of crushing and grinding, in the fact, that energy and cost is also high, the environmental impact is significant. So, in this study, after the rough grinding, a mortar of aggregate was modified by carbonation.

As a result, it was possible to increase the density and water absorption as compared to the original rough milled aggregate. Therefore, to produce a concrete with modified aggregate was significantly improved effect on the concrete strength and the drying shrinkage. By using this technique, it is possible to improve the processing problems of the return concrete. In addition, it is possible to contribute to the reduction of global warming gases by adsorption of carbon dioxide gas. Incidentally, carbon dioxide is considered to be generated from a number of industries and the techniques that lead to environmental reduction technology in society as a whole world.

1 INTRODUCTION

Toward the Construction of environmental impact reduction and recycling-oriented society,
it is desired that the promotion of concrete for using the recycled aggregate and also using the recycled aggregate concrete around some construction site [1]. In the future, when considering the maintenance of the existing structures, it is assumed that many structures are updated, and a large amount of concrete mass is expected to occur from many structures. In addition, it has been reported that the large amount of remaining concrete and return concrete were generated in the transportation of ready-mixed concrete in the structure construction site. These demolition and remaining concrete are industrial wastes. Therefore it is necessary to appropriately process. One of the methods, it is conceivable to utilize a recycled aggregate for concrete. This leads to the zero emissions of the entire construction industry also the concrete industry. However, in order to produce recycled aggregate, it spends a high energy and cost, it leads directly to high environment impacts, and it is a need for caution.

Furthermore, in the global problem of global warming, it is very important that reducing the emissions of carbon dioxide as a greenhouse gas. In the cement production on the concrete field, carbon dioxide emissions from energy and non-energy sources is a problem. It is the second of emissions in the manufacturing industry in Japan. Therefore, even in the entire concrete industry, there is a need for new efforts to reduce carbon dioxide emissions.

For this purpose, it is necessary to produce a recycled aggregate at low energy and low-cost as possible. On the other hand, there is important that it is possible to reduce the carbon dioxide emissions during cement production. In this research, focusing on the recycled aggregate production, the recycled aggregate produced at low quality is applying the carbonation technology. As a result we have to understand the property of reforming the recycled aggregate. It was further aimed to research the characteristics of the concrete using these aggregate. In addition, it was discussed reforming the mechanism of recycled aggregate.

2 CARBONATION TECHNOLOGY AND TECHNIQUE

2.1 Carbonation Technology

Calcium hydroxide is known to produce calcium carbonate by adsorbing carbon dioxide. In the concrete, when the carbon dioxide in the atmosphere penetrates into the concrete, the calcium hydroxide of the hydration product in concrete is form the calcium carbonate by adsorbing carbon dioxide. This means that it owns the ability of the concrete to absorb carbon dioxide. This phenomenon is called carbonation, the carbonation in the field of concrete, may not be considered rebar corrosion on reinforced concrete. Rebar in the concrete, normally in an alkaline exist thin oxide film, is in the corrosion difficult state. If carbonated concrete by carbon dioxide, concrete was alkaline to gradually shift to neutral. This is called a neutralization, the thin oxide film by the neutralization progresses is destroyed around the rebar in the concrete and also corroded by the supply of water and oxygen. After then leading to strength decrease of the structure. However, focusing only on concrete, in general, the calcium carbonate produced by the carbonation has a large volume as compared with the calcium hydroxide. Therefore, the resulting calcium carbonate fills the large porosity is also confirmed to be densified. Thus, a large pore is reduced, it is known that improving the strength and the permeability resistance of concrete. So, if we can densify the mortar unit that is attached to the recycled aggregate, it was thought that it might be able overcome the disadvantages in that reduction in strength and low permeability resistance of recycled aggregate concrete.
2.2 Improving Recycled Aggregate

In Japan, recycled aggregate which is depending on their quality, H and M and L classification of its property and concrete provisions have been made on JIS A 5021, JIS A in 5308 Annex and JIS A 5022, 5023. [2-5] The index is made by absolute dry density and water absorption. High-quality recycled aggregate H can be used as concrete aggregate. However, it needs the high cost and a lot of energy to manufacture. Meanwhile, the reproduction in the aggregate M and L, in general it is difficult to use as a concrete aggregate, there is only available in the precursor concrete of floor leveling concrete or underground. However the cost and energy is small in manufacturing for these aggregate. In the future, in order to spread the recycled aggregate is considered that it is necessary to manufacture by low cost and energy of high quality recycled aggregates. Therefore a low-quality recycled aggregates prepared at low cost and low energy tried to be improved using carbonation technology as introduced above [6].

<table>
<thead>
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<tbody>
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<td>Crash</td>
</tr>
<tr>
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<td>Crash</td>
<td></td>
</tr>
<tr>
<td>MB</td>
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<td>Dismantling</td>
</tr>
<tr>
<td>MC</td>
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<td>Dismantling*</td>
</tr>
<tr>
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</tr>
<tr>
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<tr>
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<td>Dismantling</td>
</tr>
<tr>
<td>OS</td>
<td>Out</td>
<td>Returning</td>
</tr>
</tbody>
</table>

* including a part of returning concrete

In this research, the recycled aggregate was carbonated for 7 days on the accelerated carbonation device (temperature 20 °C, relative humidity of 60%, concentration of carbon dioxide at 5%). The samples are stirred once every two days, carbon dioxide is to be able to fill absorbed throughout the aggregate. It is shown in Table 1 on the recycled aggregate that used in this study. In addition, it shows a list of physical properties of the improved before and after of recycled aggregate in Table 2. Aggregate was used coarse aggregate M and L produced from dismantled concrete and under L as non-standard (O) produced from the return concrete of construction site. It showed the physical properties of recycled aggregates on the
improved before and after for carbonation technology in Figure 1. In all of the aggregate have improved density and water absorption, it is understood that some aggregates are classes up. Thus was possible improvement of physical properties by carbonation technology. Figure 2 shows the relationship between the amount of mortar ratio and water absorption of recycled aggregate. Here amount of mortar ratio, including directly measured and calculated from

<table>
<thead>
<tr>
<th></th>
<th>Before improving</th>
<th>After improving (absorbed CO₂)</th>
</tr>
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<tbody>
<tr>
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<td>Water absorption</td>
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<td>-</td>
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<td>M</td>
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<td>LS</td>
<td>L</td>
<td>1.94 12.92 12.8 - 25.1</td>
</tr>
<tr>
<td>OS</td>
<td>Out</td>
<td>1.92 14.57 16.1 - 28.0</td>
</tr>
</tbody>
</table>

Figure 1 changing physical properties

Figure 2 Mortar content and water absorption

crushing value as a reference [7]. The amount of mortar ratio is not changed by accelerated carbonation, it can be seen that only the water absorption rate is improved.
3 EFFECT OF IMPROVING RECYCLED AGGREGATE APPLIED CONCRETE

3.1 Using material, mix proportions and testing methods

The concrete was produced using improving recycled aggregate. It was performed compression strength, splitting strength test and the length change test in the manufactured concrete. Concrete was using a blast furnace slag cement type B (replacement BFS ratio is 50%). Mix proportion of concrete is water cement ratio 50%, and s/a 50%, unit of water 170 kg/m³ was constant. For the compressive strength, it produced the concrete with all recycled aggregate. It should be noted that fine aggregate was using a crushed sand. On the other hand, we were using L and the aggregate of non-standard for the length change as drying shrinkage test. The reason is because the improving effect was observed in the compressive strength test. The mix proportion of concrete prepared are shown in Table 3. In addition it was also shown the measurement results of the slump, air content and compressive strength of fresh concrete in this table. Fresh concrete in all concrete was almost same properties.

<table>
<thead>
<tr>
<th>absorbed CO₂</th>
<th>corse</th>
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<th>Slump (cm)</th>
<th>Air (%)</th>
<th>Compressive strength (N/mm²)</th>
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<td>COS</td>
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<td>6.5</td>
<td>20.1</td>
</tr>
</tbody>
</table>

The test method was carried out to determine the physical properties due to improving recycled aggregate. Compressive strength [8] and splitting tensile strength test [9] is conforming to JIS (Japanese Industrial Standard). Test specimen is cylinder specimen of ø100 × 200mm. Curing is carried out on the 28 day to 20 °C water curing, the strength test was carried out in three specimens. It was calculated strength than its average value.

Drying shrinkage test of the concrete was performed with JIS A 1129-3. It was de-frame in 1 day after the casting of the concrete, curing was performed 7 days in water at 20 °C. Then, it was placed at temperature 20 °C and humidity 60% RH environment. The length change was measured in 1,2,4,8 and 13 weeks from the start of the drying.
3.2 Results of strength test

Figure 3 and 4 show the results of compressive strength and splitting tensile strength tests. Strength properties of concrete using improving recycled aggregate, it is not shown greatly improved. However it can be seen that strength of the improving recycled aggregate concrete is same or a little bit high compared with originated recycled aggregate concrete. Especially improving effect of strength is seen remarkably by using the aggregates of L and non-standard (O). In addition, it seems relatively high improving effect in splitting tensile strength.

![Figure 3 Results of compressive strength](image1)
![Figure 4 Results of splitting tensile strength](image2)

3.3 Results of drying shrinkage test

Figure 5 is the result of length change test (Drying shrinkage) in the drying age at 8 weeks (56 days). Drying shrinkage rate is small at all the mix proportions by improving recycled aggregate concrete compared with originated recycled aggregate concrete. From this result, recycled aggregate by adsorption of carbon dioxide is improving, it can be said that the drying shrinkage rate also decreases. Further, the drying shrinkage ratio of LCL concrete using the fine recycled aggregate is large as compared to the LCN concrete using normal fine aggregate. Drying shrinkage rate of concrete using recycled fine and coarse aggregate are tended to be larger than that of concrete using only recycled coarse aggregate. This is similar to what has been reported in previous literature. Also it was observed that the improving effect of the carbon dioxide adsorption in concrete using recycled fine aggregate. The improving effect was observed on COAN and COBCO concrete that was the improving recycled aggregate of non-standard (O). These recycled aggregate is used as a return concrete. Therefore, it said recycled aggregate which can be expected improving effect by the carbon dioxide adsorbed is an aggregate produced from the concrete without over time from the production, compared with the recycled aggregate from the long shared concrete.
4 MECHANISM OF DRYING SHRINKAGE REDUCTION ON IMPROVING RECYCLED AGGREGATE

Here, particularly it consider a mechanism of improving effect of aggregate properties and reduction of the drying shrinkage using the recycled aggregate as a return concrete. Therefore,
the results of the measurement of the amount of adsorption carbon dioxide on recycled aggregate was shown in Figure 6. The amount of carbon dioxide adsorption is much as a low-quality aggregate and concrete as short time from production.

Next it is shown in Figure 7 the results of the pore volume of the recycled aggregate is measured by the Archimedes method. Obviously porosity of improving recycled aggregate has decreased in all of the original recycle aggregate, it is densified. Therefore, the observation of the products of the pore using a SEM, production of calcium carbonate was confirmed as shown in Figure 8. This densification of the attached mortar on recycled aggregate due to carbonation is the effect to improving physical properties of aggregate and drying shrinkage reduction of concrete.
5 FUTURE WORKS OF IMPROVING RECYCLED AGGREGATE

In this way it is improving low-quality recycled aggregate by carbonation, it could suggest the possibility that can be applied to concrete. The measures to realize this system is considered as follows.

1) Return concrete (which returning concrete from construction site produced by the ready-mixed concrete plants) and the remaining concrete is expanded in the yard, and to be solidify.

2) It is crushed easily to adjust to the aggregate particle size.

3) Production of low-quality recycled aggregate will be transported to the plant that are burning fuel in close proximity to the ready-mixed concrete plants (for example, cement plants, biomass energy plant).

4) Using carbon dioxide from the plant of burning fuel, recycled aggregate is adsorbing the carbon dioxide.

5) The improving recycled aggregate is transported to ready-mixed concrete plant again, and it will be used to produce concrete replaced with normal aggregate.

By stepping on this process, it is considered that it also can contribute to the reduction of carbon dioxide emission in a plant of burning fuel. As a result we think that also lead to contributing to a recycling society. In the future, it is considered to determine the replacing ratio of recycled aggregate with ordinary aggregate for having required performances.

6 CONCLUSIONS

The summarized results obtained in this research as follows.

- The dry density increases and water absorption was small on recycled coarse aggregate and recycled fine aggregate using carbonation technology such as adsorption of carbon dioxide
- Strength properties of concrete using the improving recycled aggregate was equivalent or improved the original recycled aggregate concrete.
- The drying shrinkage rate is improvement due to the adsorption of carbon dioxide on recycled aggregate concrete.
- Adsorption ratio of carbon dioxide became larger as having a high attached mortar ratio. In addition, the aggregate for adsorption ratio of carbon dioxide is large, attached mortar part had become dense.
- By measuring the adsorption ratio of carbon dioxide to be screened is recycled aggregate of drying shrinkage reducing effect. To adjust the required performance of the concrete, it is considered that there is a possibility for contributing to the the low-quality recycled aggregates concrete using improvement by adsorption of carbon dioxide.
- If forced carbonation using exhaust gas such as a cement plant, leading to a reduction in emissions of carbon dioxide from the factory. And producing the low-quality recycled aggregates from the return concrete is low cost and low energy thereby reducing the environmental impact. Furthermore, for improving recycled aggregate easily by using the proposed carbonated techniques in this research, it was suggested that lead to concrete production to reduce the environmental impact.
REFERENCES

SUSTAINABILITY DIMENSION OF AN ELEVATED CORRIDOR OVER A GREENFIELD

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Key words: Sustainability, Green field, Landscaping

Abstract. Govt. of Delhi has planned express corridors, which are fully elevated for a long
distance, say 3 to 5 KMs within the city and intermittently connected by Ramps with Traffic
at Grade for better usage. One such corridor, namely Barapullah Elevated Corridor project
was planned before Common wealth Games 2010, but construction has been taken up in three
phases. 1st Phase which included straight elevated corridor from Sarai Kale Khan (in East
Delhi) to Nehru Stadium (in South Delhi) was completed before the Common wealth Games,
but some of its pre-conceived ramps could not be constructed due to scarcity of time and are
being built up now in 2nd phase. One such ramp that has been recently completed and opened
to traffic is crossing Silver Oak Park in Southern part of New Delhi is a part of 2nd Phase. The
alignment of this ramp was earlier decided way back in 2007 along with the planning of Phase
1 and II [1]. Incidentally, there was a time gap of 6 long years between planning and its
execution. When in 2013, PWD started the construction works of 2nd Phase, it was observed
that at the same location, Delhi Metro Rail Corporation (DMRC) has already built up an
underground station (Jangpura station) and underground tube rail. It was not possible to lay
the foundation of the ramp now and essentially the alignment had to be shifted to new
location. Spanning over the entire station building would have made a very long span,
requiring a special structure at a high cost and additional time in planning and
implementation. Hence due to economic considerations and also due to various other site
restrictions, only option was to take the new alignment through the silver Oak Park that is a
green filed with dense trees all-round. The sanctity of this aged old park was getting lost with
new alignment. Moreover the horticulture department was not inclined to allow the construction activities through the green belt. It was a challenging task and the sustainable solution to this problem is discussed in this paper.

1 INTRODUCTION

Silver Oak Park is a green field area, triangular in shape and developed in an area of about 45000 sqm with more than 500 numbers of Oak trees, Babool, Sirish, Tunte, Papri and Neem. Joggers have been using the Park in morning for jogging as well as morning exercise. Conducting of Yoga classes in the Park was also a common feature. An experience of cool breeze in evening by the residents was another attraction of the park. Seasonal flowers along with tall and thick trees was an attraction to everyone all round. Under such circumstances, taking a ramp in the midst of the Park was a hit to the ecology and disturbance to the overall environment of the park.

2 TWIN ISSUE IN SHIFTING THE ALIGNMENT

Shifting of the alignment resulted in twin issues that were environmentally related and making the project unsustainable. So taking the ramp along any alignment would not only cause the removal of some of the precious trees, but also dividing the Park into two parts disturbing the overall environment. Overall, it was a big challenge to redesign the alignment. The sustainable solution to the issue is discussed in the subsequent paras.

3 NEW ALIGNMENT THE RAMP

While new alignment of the ramp was chosen, the main consideration was to minimise the number of trees to be removed and also design in such a manner that the sanctity of the Park is maintained.
Whatever alignment is chosen, it will amount to depletion of substantial area of green cover. Trees had to be removed and even the transplantation for all species of trees was not feasible. Three different options were considered for new shifted alignment. Trees to be removed were counted for each of these three options.

The details of these 3 options are given below in Table 1, 2 and 3.

**Table 1 : Alignment 1**

<table>
<thead>
<tr>
<th>Numbers</th>
<th>Tree Species</th>
<th>Girth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Babool</td>
<td>0.9 to 1.35</td>
</tr>
<tr>
<td>11</td>
<td>Silver Oak</td>
<td>0.3 to 1.0</td>
</tr>
<tr>
<td>1</td>
<td>Sirish</td>
<td>0.70</td>
</tr>
<tr>
<td>2</td>
<td>Tunte</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>Papri</td>
<td>0.65 to 0.80</td>
</tr>
</tbody>
</table>

**Table 2: Alignment 2**

<table>
<thead>
<tr>
<th>Numbers</th>
<th>Tree Species</th>
<th>Girth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>Silver Oak</td>
<td>0.3 to 1.3</td>
</tr>
<tr>
<td>4</td>
<td>Sirish</td>
<td>0.5 to 0.7</td>
</tr>
<tr>
<td>2</td>
<td>Tunte</td>
<td>0.5 to 0.7</td>
</tr>
<tr>
<td>4</td>
<td>Papri</td>
<td>0.45 to 1.1</td>
</tr>
</tbody>
</table>
Table 3: Alignment 3

<table>
<thead>
<tr>
<th>Numbers</th>
<th>Tree Species</th>
<th>Girth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Tunte</td>
<td>0.3 to 0.5</td>
</tr>
<tr>
<td>12</td>
<td>Papri</td>
<td>0.4 to 1.5</td>
</tr>
<tr>
<td>9</td>
<td>Silver Oak</td>
<td>0.4 to 1.5</td>
</tr>
<tr>
<td>1</td>
<td>Neem</td>
<td>1.2</td>
</tr>
<tr>
<td>3</td>
<td>Babul</td>
<td>0.2 to 1.3</td>
</tr>
<tr>
<td>3</td>
<td>Sirish</td>
<td>0.5 to 0.85</td>
</tr>
</tbody>
</table>

It was observed that alignment 1 required removal of 18 trees, alignment 2 required 26 trees to be removed and alignment 3 required 32 trees to be removed. So alignment 1 having minimum trees interference was adopted and thus with the best planning, only 18 trees (11 silver Oak and 7 others) were removed with girth varying from 0.3 m to 1.35m.

Further 180 trees in lieu of these 18 trees have also been planted so as to compensate the ecological disturbance to the extent possible.

4 DESIGNING THE ALIGNMENT

4.1 Change from Solid structure to solid slab over Piers

As per the initial planning, the corridor was to be constructed as filled up portion with retaining walls (or Reinforced Earth wall) on either side. But since the corridor was passing over the midst of the Park, it was decided to modify the structural arrangement from solid structure to open structure so that the movement of people within the boundaries if the existing park is not restricted and the park is not divided into 2 parts. This helped in maintaining the oneness character of the park.

Main reason for designing earlier as filled up (solid) ramp instead of designing on piers was primarily on cost consideration. The deck area of the ramp was about 3000 sqm. But shifting of alignment resulted in longer ramp and this 3-lane carriageway has now become about 5500 sqm. Out of this, the area over the silver Oak Park was about 1400 sqm.

4.2 Changing the levels of the Park by lowering the surface

Since the ramp was descending, it was almost touching to grass level at the lowest point. Thus the opening so created was getting a triangular one and that would have been very unpleasant cross-corridor with limited headroom for the passer-byes. In order to increase the headroom, the park has ben redesigned by depressing the ground level. For facilitating it, the pile caps below the piers were kept at lower level so that surcharge over the pile caps was reduced and the clear height between ground and soffit of the solid slab structure was sufficient enough for a pleasant movement and saving from the feeling of claustrophobia.
4.3 Landscaping the entire area around the Piers

As a part of future planning the area underneath the structure including the piers will be given green touch and hard landscaping so that overall area including the ramp and trees around forms an environment friendly belt without loosing the ecology of the area and also becomes a tourist attraction spot.

Figure 4: Completed loop

The columns will be given artistic touch by making murals so that it becomes part of overall landscape and the oneness character of silver Oak Park is also retained.

Figure 4: Movement across the park beneath the loop
5 CONCLUSIONS

- Once the construction of corridor is complete and opened to traffic, it will be a part of overall scheme of Silver Oak Park with attractive landscaping and murals.
- Overall Environment, greenery and social and economical factors are taken care of while modifying the scheme and implementing at site.
- If some issues are observed in the implementation of a planned corridor due to delay for any reason, then all efforts are required to be put in to find the most sustainable solution to the problem without losing the basic character and functionality of the Project [2,3].

REFERENCES


SUSTAINABILITY EVALUATION OF A NEW TYPE OF CONCRETE BRIDGE STRUCTURE
- Butterfly Web Bridge -

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Key words: Prefabricated Panels, Fiber Reinforced Concrete, Light Weight, CO2 Emissions

Abstract. In order to reduce the superstructure weight and keep the durability of the structure, a new type of bridge called “Butterfly Web Bridge” was developed by Sumitomo Mitsui Construction Co., Ltd. in Japan. In a butterfly web bridge, the butterfly-shaped prefabricated panel is used for the web. The 80 MPa concrete is used for the panels and it has a thickness of 150 mm. This new type of structure enables the reduction of the amount of materials such as concrete, steel reinforcements, and prestressing steels. Therefore, it reduces the dead load of the superstructure by 10% compared with an ordinary box girder, as a result, the size of the substructure is also reduced. Three butterfly web bridges have already been constructed in an express way. From the calculation of CO2 emissions, it was found that this type of bridge could reduce CO2 emissions by approximately 10%.

1 INTRODUCTION

It is important to reduce the superstructure weight in an earthquake prone country like Japan. Therefore, corrugated steel web bridges have been applied in many projects. However, the maintenance cost is relatively high to keep the durability of the structure during their design life time.

In order to solve such issues, a new type of bridge called “Butterfly Web Bridge” was developed. The butterfly web bridge uses prefabricated butterfly-shaped panels in the web. Those web panels are not continuous in the longitudinal direction, therefore, with respect to shear force acting on the web it behaves similarly to a Double Warren truss (Figure 1). Consequently, shear force is broken down into compressive force and tensile force in the web. The butterfly web panel has a thickness of 150 mm. The material for the components of the panel is 80 MPa concrete and tensile stress area is reinforced with prestressing steels. The amount of prestressing steels is determined such that there is no tensile stress intensity under the dead load and no cracking occurs under the service load. It has no steel reinforcements in the web area and dowels are arranged in the upper and lower ends to connect the panel and slabs.
Compared with an ordinary box girder, this structure has thin web components and enables the reduction of the dead load of the superstructure by approximately 10% (Figure 2). This also enables the reduction of the size of the substructure. Therefore, the amount of materials such as concrete, steel reinforcements, and prestressing steels can be reduced. As a result, this indicates that this structure has a smaller environmental impact.

2 FEATURE OF BUTTERFLY WEB

2.1 Material

The butterfly web comprises of precast panels fabricated off-site at a plant using high strength fiber reinforced concrete with specified design strength of 80 MPa. Steel fibers with tensile strength of 200 MPa, 0.2 mm in diameter, 22 mm in length, and 0.5% in total volume are used to enhance the panel’s shear capacity (Figure 3). Inside the panels, prestressing steels are placed in alignment with the orientation of tension acting on the panels. Pretensioning system is used for prestressing. Prestressing steel components are 15.2 mm
diameter strands with debossed surfaces to enhance their adhesion with concrete (Figure 4). Steel pipe filled with mortar are used as dowels arranged at top and bottom of the panels. No reinforcing steels are used, which makes the panels easy to work with and maintain.

2.2 Prefabrication

The butterfly web panels are fabricated in a plant using industrial processes away from the construction site, and transported to the site by truck. Although external shape and thickness are standardized, panels used at different points in the design required different amount of prestressing steels and dowels. Therefore, several types of panels have to be fabricated. However, it is not possible to fabricate different types of panels at the same time. In order to enhance the fabrication process, four panels of the same type are produced at one time. Since prestressing force is applied early at the fabrication stage, steam curing is used to accelerate strength gain. The fabrication stages is shown in Figure 5, 6 and 7.
2.3 Erection

The butterfly web bridge can be erected by different methods such as staging method, cantilever method, and precast segment method. Cantilever method (Figure 8) has the least environmental impact because the ground conditions under the bridge need not to be modified. Three bridges were erected by this method. Specifications of a butterfly web bridge applying cantilever method are described as follows: Each butterfly web panel are designed 2.9 m long and installed at every 3.0 m. Panel weighs approximately 3.25 t and enables the main girder lighter than an ordinary concrete web. The usual segment length in a cantilever method is from 2.5 m to 4.0 m which depends on the form traveler capacity. On the other hand, erection segment length of 6.0 m can be used for the butterfly web bridge, equivalent to the length of two web panels (Figure 9). Thus, in a butterfly web bridge, segments can be lengthened which leads in reduced segments number. As a result, the construction period can be substantially shortened. Besides, the butterfly web panels are not continuous in the longitudinal direction, thus there is no need to join adjacent web elements which enhances the efficiency of the construction process.

3 SUSTAINABILITY EVALUATION OF BUTTERFLY WEB BRIDGE

There are three butterfly web bridges erected by cantilever method in Japan. Sustainability evaluation has been performed and one bridge is discussed in the following sections.

3.1 Akutagawa Bridge

Akutagawa Bridge is located in Osaka Pref. and forms part of the Shin-Meishin Expressway that runs between the major cities of Osaka and Hyogo. It is a 6-span continuous rigid frame bridge and has 348.0 m in total length including 75.0 m maximum span length (Figure 10, 11, 12). 50 MPa strength concrete is used for the main girder. Girder height varies from 4.5 m at pier head to 4.0m at mid span. Transverse rib having 250 mm thickness are arranged at every 3.0 m in order to gain transversal stiffness. This bridge was planned as ordinary concrete box girder originally, but it was redesigned as a butterfly web bridge in...
order to make the superstructure lighter and reduce the pier and foundation size. Comparison between an ordinary concrete box girder bridge and a butterfly web bridge is shown in the next section.

![Figure 10: Side View](image)

![Figure 11: Cross-sectional View](image)

![Figure 12: Akutagawa Bridge](image)

### 3.2 Social Aspects

The structural comparison between an ordinary concrete web and butterfly web is shown in Table 1. Referring to Table 1, the weight of the superstructure can be reduced by approximately 11% and the number of cantilever erection segment was also reduced from 9 to 5 using butterfly web. The reduction of the superstructure weight led to the scaling down of the substructure size. The width of the pier could be reduced from 7.0 m to 6.0 m, and the diameter of the foundation was narrowed from 9.0 m to 8.0 m.

In the case of an ordinary box girder bridge, the construction period using cantilever method is 10 days for 1 segment. With 9 segments on each pier side, the total construction period is 90 days. On the other hand, the butterfly web bridge requires 11 days for 1 cantilever segment. However, the segment is quite longer than an ordinary box girder. And with only 5 segments per pier side, the construction period sums up to 55 days. Therefore, the butterfly web bridge could shorten the construction period by 35 days considering one span of the pier.

Japan is a mountainous country, and difficulty often arises in construction of temporary access roads which leads to an increase of construction costs. Therefore, existing bridges or roads are sometimes used as access roads for the adjacent construction site. As for the Akutagawa Bridge, it was also planned to be used as an access road for the adjacent tunnel construction. Consequently, the shortened construction period enabled the early start of
construction of the adjacent site and was eventually merited for its early opening.

Table 1: Structural comparison

<table>
<thead>
<tr>
<th>Ordinary Box Girders (concrete web)</th>
<th>Butterfly Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure Weight</td>
<td>46700 kN (1.00)</td>
</tr>
<tr>
<td>Segment Length</td>
<td>2.5m ~ 4.0m</td>
</tr>
<tr>
<td>Segment Number</td>
<td>9 segment</td>
</tr>
<tr>
<td>Construction Cycle days</td>
<td>10 days / segment</td>
</tr>
<tr>
<td>Construction Period</td>
<td>90 days / pier</td>
</tr>
<tr>
<td>Segment Length</td>
<td>6.0m</td>
</tr>
<tr>
<td>Segment Number</td>
<td>5 segment</td>
</tr>
<tr>
<td>Construction Cycle days</td>
<td>11 days / segment</td>
</tr>
<tr>
<td>Construction Period</td>
<td>55 days / pier</td>
</tr>
</tbody>
</table>

Moreover, as Table 2 shows, girder and pier stress are almost the same between an ordinary box girder and the butterfly web bridge. This means that the structure’s safety was not sacrificed in reducing the member sizes which was initiated from the reduction of the superstructure’s weight.

Table 2: Stress comparison

<table>
<thead>
<tr>
<th>Girder stress (Lower edge)</th>
<th>Ordinary Box Girders</th>
<th>Butterfly Web</th>
<th>Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under Dead load</td>
<td>0.87 N/mm²</td>
<td>1.08 N/mm²</td>
<td>&gt; 0.0 N/mm²</td>
</tr>
<tr>
<td>Under Service load</td>
<td>-0.89 N/mm²</td>
<td>-0.88 N/mm²</td>
<td>&lt; -1.27 N/mm²</td>
</tr>
<tr>
<td>Pier Stress (Reinforcement)</td>
<td>Seismic Condition</td>
<td>214.0 N/mm²</td>
<td>212.6 N/mm²</td>
</tr>
</tbody>
</table>

3.3 Economical Aspects

Regarding the initial construction cost of the superstructure, butterfly web bridge requires higher cost by approximately 5% compared with an ordinary box girder bridge because of the use of high quality materials. On the contrary, the substructure construction cost can be reduced by as much as 10 to 15% due to its reduced size. In the case of Akutagawa Bridge, the butterfly web bridge reduced the total construction cost by 4%.

3.4 Environmental Aspects

Table 3 shows the comparison of CO₂ emissions calculated using Reference 5 between an ordinary box girder and butterfly web. 40 MPa concrete is used for the superstructure of an ordinary box girder bridge and 50 MPa concrete is used for the girder of a butterfly web bridge as described before. Since, the compressive strength of concrete referred in Reference 5 is from 30 MPa to 40 MPa, the data for high strength concrete is estimated according to water-cement ratio. And as the butterfly web panel is prefabricated at a plant, the CO₂ emission data is estimated by using the data of a prefabricated product from the same reference that includes the influence of curing. Moreover, the emission due to transportation from the plant to the construction site is also considered in the case of butterfly web bridge.
Table 3 indicates that CO₂ emissions of the superstructure between an ordinary concrete girder and butterfly web is almost the same. This is because, though butterfly web bridge reduces the material quantity used, it requires high-strength concrete for the web and girder. On the other hand, thanks to the reduction of the substructure size, the emission can be reduced by 12% in this case. In total, the butterfly web bridge enables the reduction in CO₂ emission by 7% compared to an ordinary box girder bridge.

Table 3: CO₂ emissions comparison

<table>
<thead>
<tr>
<th>Akutagawa Bridge</th>
<th>Con(Cast-in-situ)</th>
<th>Precast Panel</th>
<th>Re-bar</th>
<th>Prestressing steel</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CO₂ volume</td>
<td>CO₂ quantity</td>
<td>CO₂</td>
<td>CO₂ ratio</td>
<td></td>
</tr>
<tr>
<td>Ordinary Box Girder</td>
<td>3869m³</td>
<td>1273.5t</td>
<td>0.0t</td>
<td>769.9t</td>
<td>121.8t</td>
</tr>
<tr>
<td>Butterfly Web</td>
<td>3074m³</td>
<td>1226.4t</td>
<td>566.7t</td>
<td>179.3t</td>
<td>638.4t</td>
</tr>
<tr>
<td>Ordinary Box Girder</td>
<td>7573m³</td>
<td>1613.3t</td>
<td>0.0t</td>
<td>762.1t</td>
<td>1023.0t</td>
</tr>
<tr>
<td>Butterfly Web</td>
<td>6342m³</td>
<td>1367.8t</td>
<td>0.0t</td>
<td>1224.8t</td>
<td>939.1t</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In addition to the comparison based on material quantity, the use of butterfly web bridge can shorten the construction period as was described previously. This strongly indicates, that the CO₂ emissions coming from the operations of construction machineries can also be reduced. Considering these effects, it can be said that the total reduction of CO₂ emissions becomes much larger.

Additionally, as can be seen in Figure 12, openings on the webs or ribs of the butterfly web bridge provides some aesthetic accent in terms of appearance and makes a harmonious scenery with the neighboring landscape.

4 CONCLUSIONS

The use of butterfly web enables the reduction of the superstructure weight resulting to a reduced size of the substructure. This makes a remarkable effect not only on the reduction of structure’s environmental impact, but also in terms of economic and social aspects. In addition to a lighter superstructure, the butterfly web bridge requires fewer number of erection segment increments which leads to a shorter period of construction. Furthermore, maintenance is easier as the web panels do not use steel reinforcements and are of high quality which was ensured by production using industrial fabrication processes. Overall in comparison with an ordinary box girder, this new structure excels in terms of social, economic, and environmental aspects, and therefore is regarded as a sustainable structure.

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THE OPTIMIZATION OF RAILWAY CONCRETE SLEEPERS FOR INCREASING OF DURABILITY AND SUSTAINABILITY

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Key words: Ballast layer, Support conditions, Optimization, Rail-Seat, Mid-Span

Abstract. The focus of this paper is on presenting optimization possibilities of railway concrete sleepers’ geometry using conventional calculation ways which are based on the method of Zimmerman. Using the above mentioned calculation models, the authors analysed the appearing bending moments in the rail beams and the bearing capacity of sleepers depending on the variation of geometrical characteristics of sleepers, and the support conditions of these structural elements in ballast layer. As one of the major problems in the real tracks, it is the difficulty to determine, whether the support condition would be partial or full hence both support conditions were taken into consideration during the studies. The variation of bearing capacity of the rails and sleepers are compared in the case of 24 proposed sleepers with different dimensions, aiming to find the best solutions for achieving a structural optimized tie.

1 INTRODUCTION

In the last decades due to the continuously increasing speed and axle loads of trains the engineers has been forced to develop structures of railway tracks to keep and consolidate the railway system role in the transportation system. Railway transport represents a cost effective, energy efficient and sustainable communication technology compared to other transportation systems. In order to keep this competitive position over the other transportation systems, the railway needs to update its operating technical system and to catch up its whole system on the current requirements of passenger and freight transport.

Even today the main type of railway tracks are the ballasted track, therefore in the modernization process for providing the sustainability of the railway transportation system an important role is played by the railway track structures mainly the connection between rails, sleepers and ballasted layer.

Sleepers are laid transverse to the supported rails fixed on them, transferring the loads from rails to the ballast and subgrade below [1, 2].

For good performance of sleepers to fulfil the functions and objectives an ideal sleeper should possess the following [2]:

- The sleepers to be used should be economical, they should have minimum possible initial and maintenance costs.
- The fittings of the sleepers should be such that they can be easily adjusted during maintenance operations such as easy lifting, packing, removal and replacement.
- The weight of sleepers should not be too heavy or excessively light, they should have moderate weight, for ease of handling.
- The design of sleepers should be such that the gauge, alignment of track and levels of the rails can be easily adjusted and maintained.
- The bearing area of sleepers below the rail seat and over the ballast should be enough to resist the crushing due to rail seat and crushing of the ballast underneath the sleeper.
- The sleeper design should be such as to facilitate easy removal and replacement of ballast.
- The sleepers should be capable of resisting shocks and vibrations due to passage of heavy loads of high-speed limits.
- The design of the sleepers should be such that they are not damaged during packing processes.
- The insulation of rails should be possible for track circuiting, if required, through sleepers.

![Figure 1: Principle of classic, ballasted track structures, cross section](image)

Therefore the optimization of concrete railway sleepers is necessary since it enables the reduction of energy and use of material on the short term, but also decreases the costs of production of these elements; in long term it increases the sleepers’ durability against the variable load cases and environmental influences.

2 DESIGN AND CALCULATION METHODS OF RAILWAY SLEEPERS

Dimensioning and study of elements of railways’ superstructures is already a very old process, it had begun simultaneously with the appearance of the railway. Initially the engineers designed the railways structures by completely relying on previous experiences and practices but over the time many other methods had been developed for the calculation of railways superstructures [6].

Current practices in the world concerning the calculation of railway structure are mainly based on the method of Zimmermann, “beam on elastic foundation” [1, 3].

The basic idea in Zimmermann method is to transform the transversely sleepered track represented theoretical by a discrete supported beam to an equivalent longitudinally sleepered
track represented by a fictive, continuously supported beam on an elastic foundation (Figure 2) [3, 9].

During the studies the following assumptions were taken into account:
- The maximum design speed of trains, $V=200$ km/h.
- The axial load is considered to be $2P_L = 25$ Tons.
- Sleeper spacing, $k=600$ mm.
- The composition of the subsoil is considered as clayey and muddy (low quality) $C=0.05N/mm^3$.
- Maintenance condition of superstructure is good, $\delta=0.2$.

**Table 1:** Distribution of sleeper bearing pressure

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Partial ballast support condition when the sleeper is only supported in the areas beneath the rails (in the areas of blocks)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Full ballast support condition when the pressure distributes along the whole length of the sleeper but beneath the mid span, the pressure is half of the pressure beneath the blocks.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Full support along the whole length of sleeper with the same pressure value is the most unfavorable pressure distribution case.</td>
<td></td>
</tr>
</tbody>
</table>

Because in the structural design of the railway sleepers is very difficult to determine the exact contact pressure distribution between the sleeper and the ballast and its variation with time, this represents an important element in this process [4].
In this article for the analysis of the elements, both support conditions of the sleepers resting on ballast layer were considered, trying to take into account the most unfavorable conditions too. The ballast layer properties were selected according to local conditions, but unchanged during the studies.

3 PROPERTIES OF CURRENT T49 CONCRETE SLEEPER PROPOSED FOR OPTIMIZATION

As benchmark concrete tie was considered, the type T49 mono-block concrete sleeper, designed and manufactured in Romania and used by the Romanian Railways Company (CFR).

The design of this mono-block concrete sleeper is based on the type B70 German concrete sleeper, using the Vossloh rail fastening system. The concrete used for the manufacturing of these elements should be C50/60, the strength of the 28th day of cubic samples of the sleeper's concrete is considered to be more than 60 N/mm². The table 2 presents the dimensions and geometrical characteristics of this element.

Figure 3: Current T49 concrete sleeper dimensions
Section 1-1

Figure 4: Sections of the T49 concrete sleeper

Table 2: Dimensions of T49 concrete sleeper

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
<th>Dimensions [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Total length of sleeper in the bottom</td>
<td>2600</td>
</tr>
<tr>
<td>( l_b )</td>
<td>Length of the blocks in the bottom</td>
<td>1100</td>
</tr>
<tr>
<td>a</td>
<td>Length of bottom side of sleeper from end to rail seat axle</td>
<td>550</td>
</tr>
<tr>
<td>b</td>
<td>Length of bottom side of sleeper from rail seat axle to the end of block</td>
<td>550</td>
</tr>
<tr>
<td>c</td>
<td>Length of mid-span zone</td>
<td>400</td>
</tr>
<tr>
<td>( h_{l1} )</td>
<td>Height of rail-seat center</td>
<td>213</td>
</tr>
<tr>
<td>( h_{l2} )</td>
<td>Height of sleeper center</td>
<td>175</td>
</tr>
<tr>
<td>( b_{l1} )</td>
<td>Maximum width of end sleeper in the end side</td>
<td>300</td>
</tr>
<tr>
<td>( b_{l0} )</td>
<td>Width of sleeper in rail seat section</td>
<td>276</td>
</tr>
<tr>
<td>( b_{l0} )</td>
<td>Width of sleeper in the middle side</td>
<td>220</td>
</tr>
</tbody>
</table>

4 SUGGESTED SLEEPERS

In order to achieve the optimum geometrical properties of the T49 concrete sleeper, 24 variations of the dimensions of T49 are evaluated in this study. Using the conventional calculation models only the section beneath the rail axis (section 1-1) and the blocks length were changed compared to the original T49 tie. The middle section’s length of the sleepers was considered unchanged, the length of 400mm required by the European standards was preserved. Table 3 presents these dimensions.

Table 3: Geometrical properties of proposed concrete sleepers

<table>
<thead>
<tr>
<th>Sleeper no.</th>
<th>( L_s ) [mm]</th>
<th>( h_{l1} ) [mm]</th>
<th>( b_{l1} ) [mm]</th>
<th>( l_b ) [mm]</th>
<th>( c ) [mm]</th>
<th>( l_t ) [mm²]</th>
<th>Optimization steps</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conv.</td>
<td>2600</td>
<td>213</td>
<td>276</td>
<td>1100</td>
<td>400</td>
<td>180585094.57</td>
<td>Conventional T49 sleeper</td>
</tr>
<tr>
<td>1</td>
<td>2600</td>
<td>211</td>
<td>276</td>
<td>1100</td>
<td>400</td>
<td>175459575.08</td>
<td>Decreasing height, constant width and block length</td>
</tr>
<tr>
<td>2</td>
<td>2600</td>
<td>209</td>
<td>276</td>
<td>1100</td>
<td>400</td>
<td>170432415.57</td>
<td>Increasing width constant height and block length</td>
</tr>
<tr>
<td>3</td>
<td>2600</td>
<td>213</td>
<td>285</td>
<td>1100</td>
<td>400</td>
<td>187937256.23</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2600</td>
<td>213</td>
<td>295</td>
<td>1100</td>
<td>400</td>
<td>196097506.08</td>
<td></td>
</tr>
</tbody>
</table>
5 ANALYSIS OF RAILS AND SUGGESTED SLEEPERS

The appearing bending moments in the UIC 60 rail beams were evaluated depending on the variation of rail-seat sections’ geometrical properties and the length of the blocks, considering all support conditions of the sleepers.

For this calculation we used the following equation, based on the method of Zimmermann:

\[
M_r = \frac{E_r I_r}{4(E_r I_r + E_i I_b)} \cdot \sqrt[4]{\frac{4(E_r I_r + E_i I_b)}{b_0 C} P_s \cdot (1 + t \delta \varphi)}
\]  

The exact magnitude of the load transferred from the rails to the sleepers was determined by the equation 2:

\[
Q = \frac{P_s k}{2L}
\]

\[
L = \sqrt[4]{\frac{4(E_r I_r + E_i I_b)}{b_0 C}}
\]
5.1 Variation of the bending moment in rails

Figure 5: Variation of the bending moment in the rail beams depending on the geometrical optimization of sleepers and the variation of the bearing ballast pressure beneath the sleepers.

Figure 6: Percentage variation of the rails bearing capacity in function of the sleepers’ geometrical characteristics (stiffness of the sleepers).
5.2 Analysis of the bearing capacity of the optimized concrete sleepers

Figure 7: Variation of the positive bending moment in the rail-seat section of the sleepers

Figure 8: Variation of the negative bending moment in the mid-span section of the sleepers
5.3 Comparison of rails and sleepers bearing capacity

**Figure 9:** Comparison of the variation of the bending moments in rails and in sleepers’ rail-seat section with full support conditions under the ties

**Figure 10:** Comparison of the variation of the bending moments in rails and in sleepers’ mid-span section with full support conditions under the ties
6 DISCUSSIONS

In Figure 5, the variation of bearing capacity of rail beams is presented depending on the stiffness and the support condition of the sleepers. It seems evident that the bending positive moments appearing in the rail beams are significantly less, (with about 50%), when the distribution of the bearing ballast layer varies over time beneath the sleepers and the sleeper support condition transforms from partial support condition to full support condition.

Unfortunately the modification in the ballast layer’s structure in not favorable to the sleepers, mainly in the middle-span section of the ties. Since the pressure is distributed along the whole length of the sleepers, its values decrease below the rail-seat portions of the ties and thus the positive bending moment also decreases in the sleepers’ rail-seat sections (Figure 9).

On the other hand, simultaneous negative bending moments in the mid-span sections cause tensile stresses at the top fibers of the sleepers, which may induce the cracking of the concrete structure in the ties’ central zone.

It can be seen on the Figure 10 that while the positive bending moment in the rails decreased about 50% (20kNm) compared to the moments in rails when the sleepers had partial support condition (Figure 5), the negative bending moments in the sleepers’ mid-span section increased with up to 7-10kNm. Therefore, one of the major challenges in both the structural design of the sleepers and in the optimization process is to reduce the cracking sensitivity of the sleepers in the mid-span zones.

It can be observed on Figure 6 that the rails’ bearing capacity linearly depends on the stiffness and geometrical properties of the sleepers. In the case of sleepers 11, 12, 13 and 17, 18, 19, 20, whose rail-seat sections’ height was increased, the bending moment in the rails decreased by 10% and 20%, respectively models 7, 8, 9, 10 with shorter length of the blocks (1050mm) and with lower height in the section below the rails, were unable to ensure the same support conditions as the previous mentioned models, therefore the positive moment in the rails tracks increased.

The analysis of the bearing capacity of railway concrete sleepers reveals that, the variation of the bending moment in the rail-seat section and mid-span section of the sleepers are often opposite. By reducing the blocks’ length, the height of rail-seat section and widening of the section (model 7, 8, 9, 10), the positive bending moment in the sleeper’s section decreased by 4%.

When the height of the rail-seat section was increased (models 18 – 24), the decrement of the bending moment has doubled up to 8% (Figure 7).

On Figure 8 can be seen a completely opposite progress where the negative bending moment in the middle-span section of the sleepers increased by up to 25%.

It has to be mentioned that the differences and variations of the bending moment in the sleepers’ sections are acceptable in the case of models 11-17, where reduction of positive and negative bending moments was obtained.

7 CONCLUSIONS

This paper investigates the possible optimization steps in railway sleeper structural design in order to find the most suitable solutions for the optimum model with high mechanical strength, while reducing the manufacturing and sustainability costs in the same time. Nowadays, in the developed countries, railway companies undertake even higher construction
costs, if the maintenance costs will substantially diminish over the serviceability life span. But, in many developing countries in Eastern Europe, such as Hungary, Romania, Bulgaria, etc. the railway network is underdeveloped. Mainly the lack of maintenance has deteriorated the track conditions, demanding complete modernization to catch up with the level of modern railways. Therefore, in this modernization process the optimization of concrete sleepers plays a significant role. They must conform to the local conditions.

According to the above mentioned results, even if they are based on conventional calculation methods, it can be established that in terms of optimization of railway concrete sleepers, there still are unexploited possibilities.

It is evident that the negative and positive bending moments at the rail-seat and mid-span sections are significantly influenced by the ballast support condition. As consequence, without proper maintenance of the railway track it is impossible to ensure sustainable railway superstructure. In this case, the optimization of sleepers does not have significant positive effects in the service life of railway superstructures.

Considering the results, it can be stated that the height of the analyzed sleepers in the rail-seat section is optimal in interval 205 – 215 mm with and 275-300mm breadth.

The selection of the ideal height and breadth of sections mainly depends on the definition of the length of sleepers and blocks. These dimensions have direct effect on the negative moments appearing in the mid-span section of ties. The positive bending moments have decreased in the rail-seat section of the models with shorter blocks and lower height compared to the conventional T49 sleeper (sleepers 7, 8, 9, 10). It is true that, at the same time, decrease of the negative moment in the mid-span section of the sleeper appears, but it was investigated with full support conditions and the maximum negative moment is not higher than 10kNm, being considered acceptable. In this case the differences of the bending moments seem a little exaggerated.

The results are emphasizing the optimization possibilities in the design of sleepers with potential to be extended for a larger set of elements, considering sleepers with shorter length. The main objective was to find the optimal ratio between the length of the sleepers and blocks which can be followed by the sections’ geometrical optimization with optimal breadth.

REFERENCES


THERMAL MASS IMPROVEMENT OF LIGHTWEIGHT CONCRETE WITH MODIFIED AGGREGATES

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Key words: Lightweight concrete, thermal mass, lightweight aggregate, PCM

Abstract
The object of this research was to obtain an extra supply of thermal mass in building materials by the addition of phase change materials (PCM) by means of modified aggregates. Vacuum impregnated technique was used to incorporate the PCM in the porosity of lightweight aggregates and different commercial aggregates like expanded clay, perlite, sepiolite, pumice and vermiculite were used.

After the aggregates characterization, concrete specimens were produced and mechanical and thermal tests were carried out to analyse the influence of the PCM in the thermal inertia of the concrete structure.

Specific thermal test were carried out to concrete sample of 20x20x5 cm with different dosages. Comparison with traditional concrete (δ = 2300 kg/m³) showed that the increase of the thermal mass of the lightweight concrete with PCM had three times more energy storage capacity than traditional concrete specimens. Other parameters as thermal conductivity, thermal delay and thermal decrement were studied in the tested cases.

1 INTRODUCTION

Nowadays, the improvement of the energy efficiency in building is one of the main purposes for the European Union to achieve the objectives regarding the decrease of the greenhouse gases emission. Building sector consumes more than 40% of the total energy of the EU member states and more than 50 % of it, is used to space heating and cooling. One of the reasons of this result is the trend to use lightweight construction materials along the last 30 years with low values of thermal resistivity (Re-value) and thermal mass. These materials allow high temperature oscillations inside the building and a decrease in the thermal comfort of the occupants. Normally, the way to improve the energy performance is by adding insulating materials on the building structure or by increasing the thickness of the concrete or brick layers to increase the thermal mass of the building envelope and thus, its energy storage capacity. These incorporations cause decrease of the room spaces, greater skilled workers and higher number of hours and material spent, resulting in a higher cost of work.
Along the last 20 years, some new technologies have emerged to decrease the lightweight materials effect in construction and one is the use of phase change materials (PCM) [1]. PCM are materials which have high latent heats of fusion, allowing them to absorb or transfer large amounts of heat during the phase change from solid to liquid and vice versa. The process is reversible and proceeds at constant temperature. These materials have the capacity to improve the thermal mass of the materials without increasing mass neither space by means of the introduction of higher thermal storage capacity based on latent heat [2-4].

Numerous studies can be found regarding the use of PCM in building [5-7] but all of them can be classified by: use of macroencapsulated PCM, use of microencapsulated PCM [8-10] or materials with PCM embedded [11-12].

These studies show the increase of the thermal mass in the lightweight concrete using modified lightweight aggregate with PCM to improve the behaviour of buildings in terms of energy efficiency without the need to vary the structure of the walls, floors or ceilings. This incorporation of the PCM consists in a previous step of microencapsulation of the PCM inside the porosity of the aggregates. After that, the PCM integration in the concrete does not require any extra step in the mixture process and the use of this material becomes simple.

This encapsulation technique used is a vacuum impregnation that has been used by other authors to produce different types of PCM composites and enriched aggregates[13-19]. But most of these studies does not have information about the thermal behaviour after the incorporation of the impregnated material in the matrix (mainly concrete and plaster), only about the thermal characterization of the composite created.

The thermal behaviour of a construction material with PCM is a characteristic that must be studied to evaluate the thermal differences between conventional materials and PCM-modified materials. The capacity of energy storage and the thermal delays and decrement are parameters useful for the comparison of the behaviours that should be analysed and therefore specific tests should be carried out.

All this research is enclosed in a FP7 UE Project called Adaptiwall (Multifunctional lightweight wall panel based on adaptive insulation and nanomaterials for energy efficient buildings) and the specific objective of this task is to achieve a lightweight concrete buffer with the same thermal properties than a traditional concrete. After the laboratory development, 80 x 80 x 16 cm prototypes of concrete were produced for the installation of Adaptiwall multifunctional panels in the demonstration park that Acciona has in Algete (Madrid).

2 EXPERIMENTAL

2.1 Materials

Aggregates

Different lightweight aggregates were impregnated to analyse the storage capacity. The aggregates selected were those with a usual use in the building industry. Two different natures of aggregates were studied:

a. Siliceous aggregates: Perlite, Vermiculite, Pumice, Sepiolite
b. Clay aggregates: Expanded clay size 3-8 mm, Expanded clay size 0-4 mm

![Figure 1: Lightweight Aggregates: (a) Expanded clay; (b) Perlite; (c) Vermiculite; (d) Sepiolite](image)

After the impregnation process they were subjected to characterization techniques: differential scanning calorimeter and thermogravimetric analysis. With these technics, the weight percentage of PCM in the aggregates and the storage capacity were analysed.

**PCM**

There are different materials that can be used as phase change materials. Most usual are paraffins, as organic PCM, and hydrated salts, as inorganic PCM.

PCM of both natures were selected with a melting point between 20 and 30 ºC. Both were used in the impregnation of the aggregates and the results were very different between the use of salt hydrates and paraffins. Salt hydrates are very hygroscopic and the modification of the number of water molecules changes the physical properties of the salt, so these salts are very sensitive to the temperature changes. After the impregnation, the analysis made to the aggregates shown that the salt changed its properties and lose the thermal energy storage capacity in the interesting temperature range because of the temperature of the process. The changes in the nature of the salt make incompatible this PCM with the impregnation process and the use in concrete production. For this reason organic PCM was selected.

### 2.2 Experimental process

**Vacuum Impregnation**

The use of vacuum impregnation technics aims to introduce the PCM inside the small size pore of a porous material, lower the pore able to be impregnated and lower the vacuum pressure. This technics extract the air occluded in the pores and introduces the melted PCM. The steps carried out to the impregnation of the aggregates are shown in [REF] and
Figure 2 shows the laboratory set up and a bigger scale of the process using a vacuum oven:

![Figure 2](image)

Figure 2: Impregnation process (A [13] lab scale; B, impregnation by a vacuum oven)

To avoid leakage of the PCM outside the aggregates, a last step was added that consists on dry the aggregates in an oven and over an adsorbing felt along 24h.

At the beginning, the impregnation process was made in a laboratory flask with 200-300 g of aggregates. After the optimization of the process, a vacuum oven and 30 L container were used for the impregnation of 8-10 kg of aggregates.

Concrete production

Real scale concrete mixer was used with a production of 20-30 L of concrete in each batch. Lightweight concrete with PCM and traditional concrete was produced. In both cases the aggregates (no the lightweight with PCM) were from quartzite because it has better thermal conductivity than other mineral nature. Cement type I and superplasticizer additives were used to improve the rheology and cohesion.

2.3 Characterization techniques

The materials developed were characterized mechanically and thermally. Normalized equipment and laboratory set up were used to carry out this characterization.

Mechanical characterization

The concrete is subjected to a compressive strength test following the standard UNE-EN 12390-3. The mechanical requirements for the concrete are established by the project as C20-C25 and the density needs to be as lower as possible. Because of that the concrete dosage had to be deeply studied to achieve an optimum.

Thermal characterization

Standardized techniques for the PCM characterization as differential scanning calorimetry and thermogravimetry were applied to the impregnated aggregates to know the latent heat, the melting/freezing point and the gravimetric quantity of PCM in the aggregate [REF]. The same operation conditions were used to extract comparative data. Both equipments used were TA Instrument thermo-balances models DSC Q200 and SDT Q600.

Thermal conductivity is measured with a Netzsch HFM 436 equipment based on heat flux meters.

The thermal behaviour of the concrete samples was studied with a laboratory set up that consist on a hot plate in contact with the sample and after the insulation of the system, the temperature and heat fluxes through both faces of the sample are monitored.

Two different tests were carried out with this system: a heating step with a final constant
temperature and temperature cycles to simulate the day-night fluctuation. Both tests are used to compare the behavior of the lightweight concrete with PCM and traditional concrete.

3 RESULTS

3.1 Impregnation results

All the aggregates cited were studied in the impregnation process. The use of all of them has different pros and cons but the first characteristic we studied was the capacity of adsorb the PCM: weight % of PCM and latent heat after the impregnation process. Table 1 shows all these data.

<table>
<thead>
<tr>
<th>Aggregates</th>
<th>Morphology</th>
<th>Density (original)</th>
<th>Strength</th>
<th>Impregnation capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>% w/w (kg PCM/kg aggregate)</td>
</tr>
<tr>
<td>Expanded clay (3-8 mm)</td>
<td>Regular granules</td>
<td>350-450 kg/m³</td>
<td>High</td>
<td>39.5 %</td>
</tr>
<tr>
<td>Expanded clay (&lt;4 mm)</td>
<td>Regular granules</td>
<td>480-560 kg/m³</td>
<td>High</td>
<td>22.6 %</td>
</tr>
<tr>
<td>Perlite (1-6 mm)</td>
<td>Amorphous granules</td>
<td>250 kg/m³</td>
<td>Low</td>
<td>65 %</td>
</tr>
<tr>
<td>Sepiolite</td>
<td>Powder</td>
<td>&gt;1000 kg/m³</td>
<td>High</td>
<td>56.1 %</td>
</tr>
<tr>
<td>Pumice</td>
<td>Crushed Stone</td>
<td>760-850 kg/m³</td>
<td>Medium</td>
<td>36.7 %</td>
</tr>
<tr>
<td>Expanded Vermiculite (&lt;4 mm)</td>
<td>Small worm shape</td>
<td>90-140 kg/m³</td>
<td>Low</td>
<td>71.5 %</td>
</tr>
</tbody>
</table>

Perlite and vermiculite have very low mechanical strength so the use of these aggregates will decrease the mechanical properties of the concrete. On the other size, sepiolite and pumice have higher densities and in the case of sepiolite, the small size complicates the impregnation process. Finally, expanded clay has high strength, low density and regular shape but the clay with a smaller size has lower porosity than the higher size, so the concrete samples were produced with expanded clay (3-8 mm).

3.2 Concrete samples

The production of concrete studied was preceded for an intensive research in the optimization of the dosage to obtain good workability concrete and improvement of the thermal conductivity. After that, the optimized dosages are shown in Table 2.

<table>
<thead>
<tr>
<th></th>
<th>Traditional concrete</th>
<th>PCM Lightweight concrete (LWC+PCM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A /C</td>
<td>0.35</td>
<td>0.4</td>
</tr>
<tr>
<td>CEM I 52.5 R (kg)</td>
<td>11.55</td>
<td>8.784</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>4.7</td>
<td>3.9</td>
</tr>
</tbody>
</table>
3.3 Thermal behaviour

Samples of concrete of 20x20x5 cm were subjected to changes in its temperature by means of a controlled hot plate. The monitored parameters were the temperatures and heat flux in both sides: the hot side in contact with the plate and the cold side on the opposite. All the tests have been carried out with one sample of 5 cm of thickness and with two stacked samples to increase the thickness to 10 cm.

Two temperature programs were studied:

a) Step from 20 to 60°C: the hot plate is heated with a constant rate and the concrete suffers this change with an increment of its temperature. The information extracted with this test is the maximum gradients between faces, the time spent to achieve the final temperature (heating delay) and the quantity of energy stored for that increment of temperature.

b) Temperature cycle between 20-40 °C: 13 hours cycles from 20 to 40 °C with a simulated curve of a day-night oscillation. Real charge and discharge of the PCM is studied with this test, temperature decrements and heating delays are calculated.

**Temperature step tests**

After an initial temperature stabilization of the sample at 20° C, the test begins with the increment of the temperature with a rate of 0.2 °C/min in the fluid that heats the hot plate.

The behaviour of the temperatures in the cold side of the samples for traditional concrete and LWC+PCM is shown in the Figure 3. The maximum temperature gradients and the time necessary for achieve the constant temperature are calculated (time lag). These values are around 10°C of difference in the gradient between both sides of the panels and around 130 minutes of delay to achieve the final constant temperature.
The melting of the PCM is the reason for the increases in the time to heat and in the gradient between both sides of the panel.

**Thermal storage capacity**

The heat flux sensors give enough information to know the quantity of energy is absorbed by the sample along the test and that is needed to change its temperature. With the differences between the heat flux that goes in and out the sample and the temperature gradient is achieved along this time, can be calculated the thermal storage capacity of the sample. Strictly the system where the sample is measured must be an adiabatic box to avoid any leakage of thermal energy. In the case of the laboratory set up used, there is a thermal resistivity (Re-value) of 2 m²K/W around the sample. Because of that, the values obtained are only for comparative use between all the tests carried out.

**Figure 4** shows the heat flux of both samples and the accumulated flux along the test time. In the first part of the test there is an increment in the accumulated heat owing to the sample heating period and when the temperature stabilizes, the flux remains low and constant.

The TES capacity has been calculated in function of the temperature gradient obtained in each case (that is around 35 °C). In **Table 3** the results of TES calculated in the case of 5 cm of thickness and 10 cm of thickness is calculated. The values obtained in both thickness is very similar because this parameter should be independent of it, but that small differences are caused by the differences in the concrete composition because the concrete is an heterogeneous material and the final content of PCM depends of the lightweight aggregates size distribution (bigger is the size, bigger is the porosity and the absorption capacity). The results shown that finally it was designed a concrete with 3 times more thermal mass and 30 % lighter than a traditional concrete.
Table 3: Thermal parameters (* only comparative values)

<table>
<thead>
<tr>
<th></th>
<th>ΔT peak to peak (ºC)</th>
<th>ΔT max (ºC)</th>
<th>Δt HF (min)</th>
<th>ΔT peak to peak (min)</th>
<th>ΔT max: Heating Delay (min)</th>
<th>Thermal decrement (Cold side)</th>
<th>TES (kWh/m3/ºC) *</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traditional concrete (5 cm)</td>
<td>4,2</td>
<td>4,8</td>
<td>23,8</td>
<td>32,0</td>
<td>125,0</td>
<td>10,4</td>
<td>0,94</td>
</tr>
<tr>
<td>Traditional concrete (10 cm)</td>
<td>3,5</td>
<td>7,1</td>
<td>118,4</td>
<td>111,6</td>
<td>191,6</td>
<td>9,2</td>
<td>1,09</td>
</tr>
<tr>
<td>LWC+PCM (5 cm)</td>
<td>2,3</td>
<td>5,8</td>
<td>72,6</td>
<td>56,4</td>
<td>124,4</td>
<td>6,7</td>
<td>3,42</td>
</tr>
<tr>
<td>LWC+PCM (10 cm)</td>
<td>5,9</td>
<td>10,3</td>
<td>190,3</td>
<td>170,3</td>
<td>277,5</td>
<td>6,1</td>
<td>3,03</td>
</tr>
</tbody>
</table>

Figure 4: Heat Flux in both samples through the hot and cold sides.

Cyclic temperature
Following the procedure, an initial step of stabilization at 20 ºC was carried out and after that a program of cyclic temperature initiates.

The parameters measured in all the cases are:
- ΔT max: maximum difference of the temperatures between both sides along the test
- ΔT peak to peak: difference between the temperature of both side maximum temperatures
- Δt peak to peak: thermal delay between the maximum temperatures of two sides.
- Δt HF: delay between the maximum values of heat fluxes that go out the sample.
- Δt max: The heating delay when the maximum temperature differences occur.

Figure 5 and Figure 6 show the temperature oscillations in traditional concrete and LWC+PCM, and the parameters measured are shown graphically.

The results are shown in Table 3 and the more relevant results are regarding the thermal delays. The increment of the thermal mass of the lightweight concrete with PCM makes important delays in the heating of the cold surface because the energy, which the hot plate
gives to the sample, is used to melt the PCM before increase its temperature. The contrary process, freezing, makes the minimum temperatures are higher than traditional concrete keeping the thermal decrement narrower in the cold side than in the hot side.

4 REAL SCALE PROTOTYPES

Acciona has a demonstration park with specific facilities able to use in demonstration of construction technologies. Adaptiwall project installed there different prototypes with different concrete dosages to study the real thermal behaviour of the whole intelligent panel.

The dosage of concrete with PCM was one of the used and a concrete panel of 80x80x16 cm was produced for the demonstration activity. Different thermocouples and heat flux sensors were installed during the production to monitor the thermal performance of the
concrete. Results of these data are no shown because the project is in the data treatment stage. Figure 7 shows a picture of 4 concrete prototypes installed and the final panels after the installation of the other functional layers.

![Concrete prototypes](image)

**Figure 7:** Concrete prototypes (a) and the whole project panel installed (b)

5 CONCLUSIONS

After all the research developed and the test carried out the conclusion of this work are:

- Some lightweight aggregates used conventionally in construction sector are able to be impregnated with organic PCM.
- In the case of formulation of lightweight structural concrete, expanded clay is the most adequate aggregate. It has a capacity of increase more than 40% its weight by means of porous impregnation.
- The performance of the encapsulated PCM in aggregates must be studied inside the matrix that is going to be used. In the case of this study the matrix is concrete and requirements as mechanical properties and workability limited the use of the PCM (or the aggregates).
- The use of an amount of 0.18 kg of PCM/Lconcrete in a lightweight concrete gets a concrete with three times higher thermal storage capacity than a concrete 30% heavier.
- Thermal tests shown important increment in the thermal delays up to 60% higher in the case of the use of PCM. This behaviour means a decrease in the energy consumption in HVAC systems and an increase in the thermal comfort.

The final step of the research was the validation of the impregnation technics for the prototype production and the performance of the concrete in real conditions will be tested with the ongoing monitoring.

AKNOWLEDGMENTS

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WOOD-CONCRETE COMPOSITE FLOOR SYSTEM IN REHABILITATION

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Key words: Wood-Concrete Floor System, Connector, Building Heritage

Abstract. The present communication focuses on the rehabilitation of ancient floors made of wood by means of the demolition of its not structural elements and restructuring to a timber-concrete composite (TCC) system (concrete of new contribution and wood respecting the preexisting one) connected by a new connection system, which turns out to be much more efficient than previous systems, reaching higher stiffness (the quotient shear/slip at interface) and a failure mode by plastification and tensile strength, without plastification of the surrounding concrete nor pull up from wood.

1 INTRODUCTION

The rehabilitation of constructed heritage is one of the important aspects contemplated nowadays in sustainability criteria of construction area. It is evident that energetic cost measured in kg of CO₂ emitted to the environment in materials manufacture and constructive process included in a new building site is higher than the mentioned cost in rehabilitation processes of already constructed work. This aspect, added to the current Spanish (and some other countries) construction crisis, gives value to the use of procedures and technologies centred on the recovery of before existing materials of construction.

Accordingly, all 40s previous floors in Spain are made of wood with a filling (usually domed) of diverse materials. From the 40s and with great profusion from the 60s, the appearance of the unidirectional floor structure re-produced mimetically the structural behavior of the previous wood one with more resistant capabilities, nevertheless there are great quantity of made of wood floors in Spain yet, some of which are part of the best exponents of our built-up heritage.

The present communication, assuming the mentioned criteria of sustainability, centres on the rehabilitation of ancient wooden floor structure by means of the demolition of his not structural elements and restructuring to a timber-concrete composite structure TCCs (concrete of new contribution and remaining the ancient wood) properly connected for its joint work, which turns out to be much more efficient on having replaced material of landfill with
structural material. It manages to preserve the old wood joists, which supposes an energetic saving, besides an improvement of the aesthetic and acoustic quality against any other solutions.

2 TIMBER-CONCRETE COMPOSITE STRUCTURE. BACKGROUND

In the structures formed by elements with overlapping and not connected materials with different Young’s modulus, the behavior of each material is produced separately. In case of a pure bending (or compound bending if shear deformations are neglected), each part has different curvature tending to detach, and the whole tensional diagrams are the result of each separate bending, because the materials are far from their full mechanical ability. With no connection, no shear can be transmitted between the two materials.

In case of composite structures, building each part properly sized and connected, detachment is avoided and the curvature is unified. Stresses take advantage reaching better tensional distribution that improves the separate materials behaviour, with the consequent economic savings.

In the joint between the two materials a shear is mobilized, which must be resisted by the connection elements. If the connection is infinitely rigid (situation may be called “full interaction” or “total connection”), the deflection at midspan, the shear and the stress distribution are independent of the rigidity of the connection.

But this is an ideal and theoretical case, since there is no connector with infinite stiffness. In reality, the overall behavior of the whole, depends on the rigidity of the connector. This is evaluated by the coefficient $Kq/sq$ being $Kq$ the stiffness of a connector according to standard tests and $sq$ the separation between adjacent connectors.

This technique is consolidated in case of mixed pieces of concrete and steel, but it is relatively new when it comes to TCCS. Since the 80s more or less satisfactorily different specific connection have been developed from research, focused on analysing the slip module (or stiffness) of the connection [1], or on the use of lightweight concrete in the process [2].

Bruce L. Deam, et al. [1] compared strength, stiffness, and mode of failure of different connectors. Push-out tests were conducted with many different connectors. Namely: Round and rectangular concrete plugs with and without screw and steel pipe reinforcement, proprietary screws, lag screws with different diameters, sheet brace anchors and framing brackets.

Concrete plug specimens manifested no measurable movements until higher loads causing rapid slip increase and concrete plugs fracture. The reinforced steel pipe ones produced a similar initial response until higher loads, tilted the pipe behind the concrete, crushed both materials and forced the wood to lift off the concrete.

The addition of a vertical coach screw to the plug increased stiffness, strength and ductility, and at the end of the tests the screw had developed a double curvature and the concrete was crushed near his head.

The rectangular concrete plug reinforced with a coach screw was found to provide greater stiffness and strength, as well as favorable post-peak behaviour, but his failure mode was similar.

Specimens with steel brace anchors and fasteners embedded in concrete (with plates...
orientation at 45° and 90° to the beam axis, respectively) have displayed that the orientation have no significant effect on the performance.

Framing brackets (Pryda brand) shown similar responses. Failure mode was by spiral nails completely extracted from the wood and very large deformations at the end of the test, with all the slip concentrated in the nailed connection between the steel bracket and the wood.

Specimens with vertical different diameter coach screws exhibited little movements until low loads but then the timber began to fail in bearing and the screw began to bend crashing the surrounding concrete and wood.

The patented SFS screws, specifically developed for connections between existing timber floors and concrete slabs, were driven at 45 and 135° to the loading direction. The screw in tension fractured during the test whereas the screw in compression broke into three parts, but with brittle fracture, crashing the around concrete and wood.

On the other hand, Steinberg E. et al. [2] researched the use of lightweight concrete (LC) with connectors from the European market, fig. 1. They made push-out tests over Timber-Concrete Composite specimens (TCCS) connected with 4 different connectors, some of them being the most used patents of the market.

![Figure 1: Overview of the connectors A,B,C and D push-out tested by Eric Steinberg et al. [1]](image)

Push-out tests results are shown as load-slip curves for each connector in fig. 2.

Failure mode for all investigated connectors was exhaustion and crushing of the LC. The connectors themselves remained almost undamaged, but with visible bending deformations, that were severe in connector C. Furthermore, the screws in type A connectors, which were not subjected to tension, and the steel sheet of the type D connectors were affected by bending.

3 NEW CONNECTION SYSTEM

3.1 Previous connection systems disadvantages.

In order to determine the effectiveness of any of these connectors, it is important to analyse the push-out tests carried out in any investigation into two different aspects:
1st-Failure mode of the mixed structure set mounted with the connection when it is subjected to extreme loads. Analysing consulted literature, the failure modes of the investigated connections are shear connection failures, failure by crushing the surrounding concrete, pull-out the connection from the wood, separation of the surfaces; or in some cases, failure by crushing wood (when there are notches in the wood).

2nd- The other indicator of connection effectiveness is its slip modulus throughout the loading process until breakage. Normally, it is considered as initial modulus the ratio of the load corresponding to 40% of the ultimate load versus the corresponding slip. Usually the corresponding stiffness to 60% and 80% are also calculated, giving an idea of the load-slip behavior throughout the full load range.

If we classify the studied connectors by its resisting mechanism, we could speak of shear connectors, glued joints and connectors resisting mainly axial force.

Shear connectors results in more or less large slides involving the described failure modes. Glued joints are very rigid, but provide no ductility at all. Axial connectors provide the highest slip modulus with desirable ductility.

This axial behavior is only possible placing connectors inclined to the contact surface. For inclined lag screws connections, the viewed reports shown failure modes that are either failure by removing the screw, crushing of concrete subjected to compression around the lag head or surfaces separation. It never reaches a breaking strength of the lag without damaging the head or the concrete surrounding it, unless the lag screw was made by low resistant quality, in
which case the lag is not appropriate.

Based on these considerations, the main hypothesis of the research is that axial connections can be developed overcoming all connectors’ failure modes above exposed.

The objectives to achieve by new connectors design are:

-To get working mainly in tension. This configuration avoids problems arising from excessive slip (in shear working connections) or buckling (in compression working ones). To achieve it, inclined placed screws are designed, and so, the shear sense resulting from the subjected loads, makes them work primarily in tension. The elements usually change the tilt direction according to the position or nature of the loads, as shown in fig. 3.

- To avoid crushing in the concrete area next to the element, and in particular close to the connector head. Screws are used to achieve it, as the one shown in Fig. 4 with a widening washer-shaped head wider than the tightening bolt dimensions.

Figure 3: Inclined position of the connectors in a mixed wood-concrete beam to ensure that connectors are always in tensile strain.
To avoid failure modes pulling-out the connection from timber. Consequently it will be studied lag screws with adequate thread bolts in terms of depth, angle of the thread, pitch length and embedded depth in the wood.

If the element is properly designed, it should enable the shear stresses transmission process from the wood to the connector and it must be able to mobilize the corresponding tractions without pulling-out. It should channel these efforts, predominantly tension, to the head and it transmits them to a large mass of concrete that will not suffer plastifications. If connections are placed inclined and in tension, transverse load to screw shaft is much minor than when it is placed perpendicular to the contact plane, and no crushing will occur in concrete. The ultimate goal is to break the steel lag before crushing the surrounding concrete and to reach a failure mode by ductile tensile exhaustion of the connecting element, using the most efficient possible way to transfer shearing stresses from one material to another.

The new concrete slab can be lightweight. This involves less resistance to local compression and cracking tendency respect to a heavier concrete, but it is no yet a problem for the new wider head.

The formwork system concrete slab may be a provisional boarding supported on the sides of existing wooden beams. Fig.5
3.2 Tests program

To design the new connection some tests series have been made.

- **Previous tests with the necessary material:**

Regarding the lag, tests are performed to know (fig. 6):

- *The type of lag screw used to form the connection,* in terms of type of material, its tensile strength and diameter. Simple tensile tests with different lags screws types and diameters are made to find the appropriate diameter and resistance.
- The embedding depth on wood, neck and thread, in terms of depth, angle of the thread, pitch length and length of the thread. Pull-out tests inspired in UNE 1382 [3] helped by a specific load device were performed.
- The connector washer, in terms of diameter and thickness required. It is calculated with a simple load balancing.

![Figure 6: Tensile tests of lags, specific device and pull-out test.](image)

Concerning timber (sawn or laminated), tests are conducted to know (fig. 7):

- The tensile elastic modulus of wood, and its tensile and compression strength. Therefore load-unload flexural tests are performed, tensile strength from bending tests and compression tests with strain gauges were performed.

![Figure 7: Wood bending tests in elastic range and at breakage.](image)
Regarding the concrete (normal or LC), compressive strength tests are performed.

- **Push out tests with connection specimens:**

  They consist of a series of tests intended to reveal the sliding load curve until breakage and the failure mode of the TCCS.

  So, connections from 3 different types of screws were manufactured, and push-out tests are performed. (fig 8). The shape and dimensions of tests pieces are inspired by the standard specimens of Annex B of Eurocode 4 [4].

  In these tests an unexpected contribution of friction in resistance sliding of the connection is discovered, so it was decided to investigate the friction between wood and concrete, due to the compression component normal to the contact, and a new series of tests has been programmed.

- **Real scale prototype tests:**

  They consist in manufacturing a stretch of TCCS to real scale, with the designed connection, taking measurements of the deflection at midspan not only for elastic loading conditions but also in anelastic phase. It aims to analyse their behaviour upto break.
Two trials with two kinds of manufacture of the same dimensions are performed:
- Timber concrete composite floor with regular aggregates.
- Timber lightweight concrete (LC) composite floor.

### 3.3 Initial results.

The new connection system has been successfully designed. The new dimensions of washer-shaped head, thread, and neck diameter ensure failure mode by tensile yielding, which does not exhaust the surrounding concrete and connector is not extracted from the wood. The breaking load in push-out specimens with 6 screws and separation of 10 cm was 120 kN while the tensile strength of a $8 \text{mm}$ lag screw was 14 kN (that corresponds to 10 kN in the contact plane direction if the angle is $45^\circ$). The only plausible explanation for the additional 60 kN can be attributed to friction.

The stiffness of the new connection is higher than previous connections. This difference is greater especially in the initial stiffness (elastic phase), fig 10.

![Figure 10: Load-slip curves obtained superimposing the designed connection (Cone 3) and the obtained from push-out tests conducted by Eric Steinberg et al. [2]](image)

By placing the screws with inclined arrangement, if the load increases, the component that compresses the wood against the concrete also increases growing the friction too. Initial results suggest that the friction coefficient can reach values close to 1, so that the breaking
loads can double the load that the lag screw resists alone.

All dimensions of new connector have been obtained from mechanical characteristics of used materials by mechanical considerations. The design process can be used for other material characteristics leading to different geometrical values for a specific similar connection.

We are currently working on a mathematical model able to predict in terms of load and static and mechanical material parameters, the floor internal tensions and movements; not only in linear phase behavior, but also in non-linear one, although nonlinear phase model resolution must be numerical.

4 EXPERIMENTAL REHABILITATION OF A TERRACE FLOOR.

This system has been implemented in the rehabilitation of a floor terrace of a manor house, whose original building dates from the 20s of the last century, performing successfully. (fig. 11),

![Figure 11: Rehabilitation works with the proposed system and connection.](image)

Ancient 4 m length timbers with a 14x7 cm² cross section were furnished with connectors every 15 cm. Lightweight concrete was used for the 8 cm thick slab. Old look was preserved but strength and stiffness were substantially increased, saving ancient timber.

5 CONCLUSION

A new connection system for timber-concrete structures has been developed, able to avoid some inadvisable features of previous systems. It can be used either for rehabilitation of ancient timber floors and for completely new ones.

Connector is a special steel screw with a large diameter head able to avoid the local concrete crushing, even using lightweight concrete. It is screwed in the wood with an inclination to work mainly in tension and yields plastically prior to pull out.

Tests have shown a higher stiffness for this system retaining ductility for ultimate loads and leading to a more effective connection when compared with previous ones.

System has been successfully used in the rehabilitation of an 80 years old floor, preserving ancient timber and look.
REFERENCES


AUTOMATIC DESIGN OF BUILDING CONSTRUCTION PROCESSES BY SIMULATED ANNEALING. A MEASURE TO IMPROVE SUSTAINABILITY, TIME, FINANCIAL AND COMPUTATIONAL COSTS

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Key words: shoring; building structures; optimization; heuristic methods; reinforced concrete; simulated annealing.

Abstract. Reducing costs in building construction processes by using successive levels of shoring, involves two strategies: 1) reducing the time spent in each of the construction operations, and 2) attempting to recover most of the components used as soon as possible. Through these two strategies, more efficient building construction processes can be achieved. Since safety must be taken into account in the construction process, it is necessary to have a calculation method. Accordingly, a computer tool which calculates and verifies building construction processes was developed. The main novelty of this work lies in its automatic optimum design of building construction processes, which was created by using Simulated Annealing (SA) heuristic algorithm. This tool can improve four of the most important points involved in construction: sustainability, safety, time and financial cost. After applying this strategy to a case study, it was found that SA performed an intelligent search of the optimal solution. The optimal solution was achieved while reducing up to 42% of the cost of the shoring system compared to the real construction process, in shorter construction time, meeting all the usual requirements for the construction of buildings, and using fewer materials which means sustainable construction of concrete structures.

1 INTRODUCTION

Reducing building times, saving on costs, using fewer materials and improving safety are four important aspects of efficient construction processes. At the present time savings in time, costs and materials are achieved mainly by recovering all or part of the building construction components as soon as possible. To get this early recovery is necessary to consider the different types of construction processes currently available [1]. Safety is also extremely important because the general improvement is always subject to structural safety considerations.

Numerous authors have proposed a large amount of theoretical models to estimate
shore/slab load transmission, including complex models and simple calculation methods. Among these last methods Grundy and Kabaila [2], Duan and Chen [3], Fang et al [4] and Calderón et al [5] developed simplified procedures to estimate load transmission between slabs and shores in multistorey buildings.

The first objective of this study was to develop and validate a software tool that would provide users a quick and simple calculation method. The computer tool can be used to check the construction process defined and entered by the user, searching optimal solutions. Since the design of the optimal construction process is intimately related to the structural designer’s experience, he has to follow a strategy of trial and error and continually redefine the process until he finds a better solution than the original. This strategy is not automatic, and as it usually leads to construction processes in which safety is given the highest weight, construction times are longer, more materials are used and costs are higher than those of the optimal solution. It is therefore advisable to apply optimization techniques to obtain the best construction processes.

Accordingly, the second objective of this study was to achieve an automatic design of optimum building construction processes by optimization techniques. A review of non-heuristic optimization studies can be found in Sarma and Adeli [6]. Heuristic search methods can also be used intelligently to obtain optimal solutions in a reasonable computation time [7]. Its first application to reinforced concrete was by Coello et al [8] in a simply supported beam, and by studies on prestressed concrete beams by Leite and Topping [9]. Nowadays, many heuristic search methods are used [10-11], being more efficient when they are combined as hybrid optimization [12-14]. In this study, single optimization algorithm such as Simulated Annealing was used as a first approach to building construction processes.

The study is divided as follows. Section 2 describes the building construction processes used at present. Section 3 defines the automatic design of building construction processes done in this work. Section 4 analyzes a case study looking for the optimal solution and giving construction time reductions, savings on cost and using fewer materials. Finally, in Section 5 we give the main conclusions.

2 USUAL BUILDING CONSTRUCTION PROCESSES

Building construction processes based on successive floors of shoring is the usual construction system. This construction system involves two or more stages for each floor. Two necessary stages are: Shoring stage, when concrete is poured over the floor; and Striking stage, when shores are removed if concrete has enough resistance to bear the loads applied. On each floor one additional stage might be applied. Reshoring or Clearing (partial striking) is usually used as an intermediate stage. Reshoring consists of removing all the shores from a floor, within a few days, when concrete has enough resistance to only bear its selfweight. After that, shores are replaced. At this moment, shores only work for future load increments. In this way, reshoring allows a retrieval of formwork to be reused on the upper floors. On the other hand, clearing or partial striking consists on removing more than 50% of the shoring components (shores and straining pieces) and the entire formwork. These kinds of stages involve a higher and faster reuse of shoring system for its use within a few days on upper floors.

Nowadays, the most important construction processes applied internationally are: 1)
Shoring/Striking (SS); 2) Shoring/Reshoring/Striking (SRS); and 3) Shoring/Clearing/Striking (SCS). Figs. 1, 2 and 3 gives a graphical description of each construction process for two consecutive levels of shoring.

Figure 1: SS construction process

Figure 2: SRS construction process

Figure 3: SCS construction process
3 AUTOMATIC DESIGN OF BUILDING CONSTRUCTION PROCESSES

3.1 Computer tool

A computer tool programmed in FORTRAN language, which calculates and verifies building construction processes by a simplified procedure, was developed. Calderón et al's simplified procedure [5] was chosen because it is the latest method and it has better goodness of fit than the previous simplified methods [5,15-17]. This simplified procedure is able to consider the usual construction system types: Shoring/Striking (SS), Shoring/Reshoring/Striking (SRS) and Shoring/Clearing/Striking (SCS) for slabs, waffle slabs and girderless hollow floor slabs.

To apply this method requires knowing the deformability of slabs. Scanlon and Murray’s method [18] was considered for this purpose. The software was also programmed with the stiffness matrix method to calculate the required bending moments on slabs. This method also takes into account the different possible boundary conditions: end, corner and internal spans. In addition, in this work different values of the number of successive levels of shoring were considered, from a floor to four successive levels.

Thus, following Calderón et al’s simplified procedure [5], the computer tool calculates the loads on both slabs and shores. Once loads are known, the computer tool verifies that the conditions, described in depth in section 3.6, have been satisfied for each slab and building operation. Summarizing, these conditions consist of checking whether constructive aspects and loads on slabs are feasible. In this way, users can check if a particular construction process is possible or not, and know accurately the loads on slabs and shores.

3.2 Definition of the optimization problem

The problem of obtaining optimal construction processes consists of minimizing an objective function $F$. In this case, the cost of the shoring system was defined according to Eq. (1) which must comply with the different constraints $g_k$ considered in the Eq. (2).

$$F(x_1,x_2,...,x_n) = \text{cost of the shoring system} \quad (1)$$

$$g_k(x_1,x_2,...,x_n) \leq 0 \quad (2)$$

where $x_i$ are the values of the different variables that make up the complete construction process.

3.3 Variables

A total of $(5+4*n)$ variables define a complete construction process as a function of the number of floors ($n$) in the building. The first five variables adopt constant values in the whole building, defined as follows:

1. Type of process: shoring/striking (SS), shoring/clearing/striking (SCS) or shoring/reshoring/striking (SRS).
2. When an intermediate operation is involved (SCS or SRS) a decision must be made on the percentage of shores to be removed. For SCS the different alternatives consisted of removing 33, 50 or 66%. In SRS, reshoring involves removing 100% of shores and then replacing a certain percentage. We considered that 100% were
replaced, as this is the usual practice.

3. The number of consecutively shored floors: 1, 2, 3 or 4.

4. Straining piece separation according to slab type: for waffle or girderless hollow floor slabs 1, 2 or 3 times the distance between ribs; for flat slabs, 1, 2 or 3 m.

5. Type of shore. Only one type was considered in this study.

The four remaining variables can adopt different values on different floors:

1. Shores may be separated by 0.5, 1 or 2 m.

2. Number of days for casting a new slab; with possible values of between 5 and 14 days.

3. Number of days for clearing a slab, choosing from possible values between 2 and 7 days.

4. Number of days for reshoring a slab, with possible values of between 2 and 9 days.

All the variables considered were discrete and not continuous. So the dimension of the space solution is defined by the Eq. (3):

$$Dimension = 4 \times 3 \times 3^n \times 10^{(n-1)} \times (1 + 3 \cdot 6^n + 1 \cdot 8^n)$$

(3)

3.4 Parameters

The parameters of construction processes consist of all the values taken as data. These values are constant for all possible solutions. In the case of obtaining optimal construction processes for buildings, the parameters are the values that are normally set in the building design phase. Table 1 lists all the parameters considered in this work.

<table>
<thead>
<tr>
<th>Data on each floor</th>
<th>Building data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of slab</td>
<td>Type of span</td>
</tr>
<tr>
<td>Thickness</td>
<td>Continuity of span</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>Number of floors</td>
</tr>
<tr>
<td>Fast, normal or slow hardening concrete*</td>
<td>Ambient temperature</td>
</tr>
<tr>
<td>Lengths of spans</td>
<td>Concrete density</td>
</tr>
<tr>
<td>Cantilever lengths</td>
<td>Elasticity modulus of steel</td>
</tr>
<tr>
<td>Height between floors</td>
<td>Construction and project safety coefficients</td>
</tr>
<tr>
<td>Construction loads</td>
<td></td>
</tr>
<tr>
<td>Service loads</td>
<td></td>
</tr>
</tbody>
</table>

* $s = 0.20$ (fast); $s = 0.25$ (normal); $s = 0.38$ (slow) according to MC-2010 [19]

3.5 Objective function

The objective function considered consisted of minimizing the cost of the shoring system of a construction process. It was decided to consider the rental costs of the required material according to the following values:

- Rental of shores: €0.03/shore/day
- Rental of formwork panels: €0.10/m²/day
• Rental of straining pieces: €0.04/m²/day
• Rental of pivoting mechanism for clearing: €0.03/m²/day

The prices cited in the above costs were provided by specialist shoring companies and are those currently charged in Spain.

The result of the objective function is expressed in €/m² in order to make it applicable to the entire building, as only one span was actually analyzed. Also, by considering this objective function as the cost of renting the shoring equipment, building time and sustainability are also considered indirectly. In fact, the shorter the time and the fewer the resources required, the lower the cost assessed by the objective function.

### 3.6 Structural constraints

The constraints in Eq. (2) express the limiting conditions of the different solutions considered, both in verifying loads and building conditions. Firstly, for building and safety reasons, minimum and maximum variable values were established, within which high-cost solutions, and unusual or unsafe construction processes could be found (See Section 3.3). Secondly, the separation between straining pieces was considered the same on all floors. In this way the transmission of loads between the shores on the different floors is exclusively vertical, without introducing additional shear forces. Finally, the loads were verified by the condition of Calavera [20] and Fernández [21], which verifies the loads on the slabs during the different building phases and determines whether or not the different solutions considered are feasible. This last condition is based on the critical evolution of the concrete tensile strength in relation to its other mechanical characteristics:

$$\beta = \alpha \cdot \frac{\gamma'}{\gamma} \leq \frac{f_{ckt,i}}{f_{ckt,28}}$$  (4)

Where $\alpha$ is the proportion of loads received by the slabs under construction compared to the design loads, $\gamma'$ is the construction safety factor, $\gamma$ is the service safety factor, and therefore $\beta$ is the proportion of the load measured on the slab weighted by the safety coefficients, $f_{ckt,i}$ is the tensile strength of the concrete at the age of the concrete under study, and $f_{ckt,28}$ is the concrete’s tensile strength in service.

Thus, following Calderón et al’s simplified procedure [5], the computer tool calculates the loads on both slabs and shores and verifies that the above conditions have been satisfied for each slab and building operation, i.e. that $\beta$ is equal to or lower than the proportion of the acquired tensile strength.

### 3.7 Simulated Annealing heuristic algorithm

The heuristic search method used was Simulated Annealing, originally proposed in 1983 by Kirkpatrick et al [22], based on an analogy with the formation of crystals melted at high temperatures and allowed to cool slowly so that they remain in a state of minimum energy. This process is governed by the Boltzmann factor, $[\exp(-\Delta E/T)]$, when $\Delta E$ is the increase in energy of the new configuration and $T$ is the temperature. The optimizing process begins with a randomly generated solution at the initial high temperature. This is altered by small movements, i.e. little changes on the solution currently analyzed. The cost of the new solution is then assessed. Feasible low cost solutions are immediately accepted, while those with
higher costs are accepted when a random number between 0 and 1 is less than the expression \( \exp(-\Delta E/T) \), when \( \Delta E \) is the absolute value of the cost increase and \( T \) the present temperature. A specific number of iterations, known as Markov chain, are carried out at this temperature, which is then reduced by means of the expression \( T = k\cdot T \) where \( k \) is the coefficient of cooling, so that the likelihood of accepting higher cost solutions is reduced. The process usually ends when the temperature is reduced to a small proportion of the original, or after a certain number of Markov Chains with no improvements to the solution [7]. With this strategy, which allows worse solutions to be accepted, locally optimal solutions can be avoided in which other strategies would become stuck. In this method the different parameters, such as initial temperature, length of the Markov chain and coefficient of cooling, must be calibrated.

4 ANALYSIS OF A CASE STUDY

In this Section, case study is analyzed. Firstly, a description of building is given. Secondly, a comparison of usual construction processes, maintaining the construction of a new slab every 7 days, was analyzed. Finally, automatic design has been considered to show the several advantages of taking into account the optimization techniques in building construction processes.

4.1 Description of building

It was considered the construction of the Fine Arts Faculty of the Universitat Politècnica de València (Spain), which was studied in depth by Gasch [15] and Buitrago et al [23]. This building has six storeys and a basement with waffle slabs 0.40 m thick, 0.15 m rib and 0.80x0.80 m waffle. The spans were 5.50x8.00 m and 5.50x8.80 m. Height between floors ranged from 2.90 m to 4.00 m. The construction process studied by Buitrago et al [23] involved a construction process of shoring, clearing and striking on two consecutively shored floors, with seven and three days between each casting and clearing respectively. This construction process was used as a reference for section 4.2 and 4.3.

4.2 Comparing usual construction processes

Before using automatic design of building construction processes to achieve optimal solutions, it was compared the most usual construction processes described in depth in Section 2. All of them had similar characteristics: two consecutive shored floors, casting of a new slab every seven days and, in appropriate cases, clearing or reshoring three days after casting operation. The striking process of a floor was performed three days before casting two upper floors. The SCS process was studied in depth by Buitrago et al [23] and it was taken as a reference to compare with other types of processes. Fig. 4 gives financial costs of different types of construction processes with the same construction time (49 days). For this building it can be seen that SCS process was 30.65% cheaper than SS process. It also shows that SCS process was 13.76% cheaper than SRS process. In fact, SCS process was chosen as the construction system in the case study.
Fig. 5 shows the number of floors where the material was required at the same time. This number of floors gives an idea of the minimum amount of material required to carry out the entire construction process. As we can see in Fig. 5, SS process required all components of the shoring system for 2.0 floors. However, SRS and SCS processes needed less formwork (only for 1.0 floor), so the financial cost was lower. In addition, SCS process needed shores and straining pieces only for 1.5 floors. For this reason and maintaining construction time, SCS process was the cheapest and used fewer materials which meant a sustainable construction process of this concrete structure.
As shown, a correct choice of the construction process can greatly improve the possible solution adopted. But nevertheless, once it is chosen the best type of construction process, reaching the optimal solution is extremely difficult. For this reason, the authors of this study believe that automatic design is a fundamental tool in order to improve and optimize construction processes.

4.3 Automatic design. A solution to improve construction processes

Designing construction processes usually requires the experience of the structural designer. However, automatic design using simulated annealing heuristic algorithm avoid this necessary experience. As mentioned above, this method requires a randomly-generated initial solution, initial temperature, Markov chain length, cooling coefficient and a stopping criterion. Medina’s method [24] was chosen to set the initial temperature; this consists of choosing an initial temperature value and determining the number of solutions accepted. Obtaining between 20 and 40 % of accepted solutions is considered an appropriate threshold for this value. If the percentage is higher than the upper limit, the initial temperature is halved and if below the lower limit it is doubled until it converges. A double condition was adopted as the stopping criterion: reaching a Markov chain without improving the solution and a temperature below 0.01% of the initial. To calibrate the length of the Markov chain and cooling coefficient, a study was made for the values shown in Table 2.

| Table 2. Study for the calibration of the length of the Markov chain and cooling coefficient. |
|-----------------------------------------------|------------------|
| **Length of Markov Chains** | **Values** |
| 500 - 1,000 - 2,000 - 5,000 |
| **Cooling Coefficient (k)** | **Values** |
| 0.85 - 0.90 - 0.95 |

After this study, Markov chain lengths of 1,000 and cooling coefficients of 0.95 were adopted for the Fine Arts building.

Even though in this strategy the dependence of the initial solution is minimal, the building was calculated five times and each time the same optimal solution was obtained. Fig. 6 shows the results obtained in one calculation. This figure also gives the financial cost in relation to the computation time of the solutions accepted by this method. It is also shown the temperature variations adopted by the method; it can be seen that as the calculations advance the temperature drops, which reduces the probability of accepting solutions worse than the latest being accepted. In fact, it can be seen that the variation of the financial cost gets smaller as the calculations advance and converges on the optimal result obtained.

The best solution obtained had a cost of €6.96/m². Computation times were 95.90 minutes, showing cost reductions of 41.85 % respect to the SCS process and 59.68% respect to the SS process analyzed in Section 4.2. The main features of the optimal solution are summed up in Table 3.
Figure 6: Search process for optimal solution by Simulated Annealing for Fine Arts Building.

Table 3. Best optimal solution obtained by Simulated Annealing.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Fine Arts Building</th>
<th>Variable</th>
<th>Fine Arts Building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Process</td>
<td>SCS</td>
<td>Casting Level 6 [days]</td>
<td>5</td>
</tr>
<tr>
<td>Clearing or Reshoring Percentage [%]</td>
<td>66</td>
<td>Casting Level 7 [days]</td>
<td>5</td>
</tr>
<tr>
<td>Consecutive Shored Floors</td>
<td>2</td>
<td>Clearing Level 1 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Separation of Straining Pieces [m]</td>
<td>1.66</td>
<td>Clearing Level 2 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Shores Area [m²]</td>
<td>2.89E-04</td>
<td>Clearing Level 3 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Separation of Shores Level 1 [m]</td>
<td>2</td>
<td>Clearing Level 4 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Separation of Shores Level 2 [m]</td>
<td>1</td>
<td>Clearing Level 5 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Separation of Shores Level 3 [m]</td>
<td>2</td>
<td>Clearing Level 6 [days]</td>
<td>2</td>
</tr>
<tr>
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<td>Clearing Level 7 [days]</td>
<td>2</td>
</tr>
<tr>
<td>Separation of Shores Level 5 [m]</td>
<td>2</td>
<td>Reshoring Level 1 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Separation of Shores Level 6 [m]</td>
<td>1</td>
<td>Reshoring Level 2 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Separation of Shores Level 7 [m]</td>
<td>2</td>
<td>Reshoring Level 3 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Casting Level 1 [days]</td>
<td>0</td>
<td>Reshoring Level 4 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Casting Level 2 [days]</td>
<td>5</td>
<td>Reshoring Level 5 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Casting Level 3 [days]</td>
<td>5</td>
<td>Reshoring Level 6 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Casting Level 4 [days]</td>
<td>5</td>
<td>Reshoring Level 7 [days]</td>
<td>-</td>
</tr>
<tr>
<td>Casting Level 5 [days]</td>
<td>5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As can be seen from the optimal solutions obtained (Table 3), they are based on finding a
feasible solution with the maximum possible separation between straining pieces and shores and shortest possible operating times to further reduce costs. In this way, the minimum amount of equipment is used and all or part of it is soon recovered. The SCS process was found to be the optimal process for achieving this objective. It can therefore be concluded that any user of this type of tool will be able to find an optimal solution to the shoring problem that will reduce the most important aspects involved in construction: costs, construction times and sustainability, without the need to previously consider a specific construction process or to have any experience in designing these processes.

5 CONCLUSIONS

A tool was developed to calculate and verify loads on slabs and shores with the aim of obtaining the optimal construction processes for multistorey buildings with in situ casting. It has been shown that different types of construction processes are usually used to improve costs and construction times. However, important improvements on costs, times or sustainability are provided by the experience of the structural designer. Simulated Annealing heuristic method was used to search for the optimal solutions and avoid the need for the personal criteria of an expert structural designer. Optimal solution was obtained that involved savings of up to 42% of the cost of the shoring system (up to 60% when taking into account the choice of the construction process and automatic design), reductions in construction time up to 29% and using fewer materials which means sustainable construction of concrete structures.

REFERENCES


FABRICATION, PERFORMANCE AND ENVIRONMENTAL SAFETY OF FIRED BRICKS FROM LAKE SILT AND SEWAGE SLUDGE

ICCS16


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Keywords: Lake silt, Sewage sludge, Fired brick, Heavy metal, Dioxins

Abstract. Lake silt, cinder and sewage sludge were used as raw materials to prepare fired brick samples using the vacuum plastic extruder in laboratory. Physical and chemical properties, including plasticity index, oxide composition and mineral composition of raw materials and the linear drying shrinkage, water absorption, bulk density, compressive strength, thermal conductivity and freeze-thaw resistance of the fired brick samples were investigated. Results showed that the linear drying shrinkage, water absorption and compressive strength of fired brick samples made from 85% lake silt, 10% cinder and 5% sewage sludge were 5.35%, 16.5% and 20.5 MPa, respectively. Gas produced during sintering process was collected and analyzed. The results demonstrated that the toxic equivalent quantity of PCDDs/PCDFs was much less than the value specified in standard for pollution control of the municipal solid waste incineration. Leaching test showed that heavy metals in bricks were immobilized effectively during the sintering process. This study reveals that it is an effective way to comprehensively use lake silt, cinder and sewage sludge as raw material to produce bricks.

1 INTRODUCTION

In China, the lake covers a vast area of more than 91019 km², accounting for 0.95% of the entire area of the country [1]. Accompanying the population expansion in China, rapid development of the economy brings tremendous pressure on the environment. Many lakes have pollution problems, such as eutrophication [2, 3], heavy metals pollution, etc [4-6]. A survey on the water quality of sixty-seven lakes throughout China by Li [7] showed that about 80% of the lakes have high pollution levels [8]. One of the important measures to manage the lake problems is to dredge sediment, which not only improves the environmental condition of water body, but maintain the good function of the shipping channel. Consequently, large scale of dredged silts is produced, which would be another pesky environmental problem if appropriate ways to utilize the silts lacks. At present, the silts are mainly employed to
landscape, produce building materials, fertilize forms [9]. In brick making, it is a high value-added way to employ the dredged silts as one of the raw materials. On the other hand, a mass of sewage sludge is brought out in the process of disposal of municipal sewage. Some sewage sludge contained certain organic compounds and trace elements necessary for plant growth, and was used as fertilizer or soil conditioner to increase soil fertility, incineration and landfill, the preparation of lightweight aggregate and bricks.

In this work, using lake silt as main raw material and cinder as lean material, sewage sludge as pore-forming agent to produce bricks was investigated on bench scale. Properties, including linear drying shrinkage, bulk density, compressive strength, as well as frost resistance of the fired bricks, were studied. Gas produced during sintering process was collected and analyzed, and the heavy leaching test was conducted as well to evaluate the environmental safety during the manufacturing and practical use stages.

2 MATERIALS AND METHODS

2.1 Raw material

Lake silt was obtained from the Changdang Lake, Jiangsu, China; cinder was obtained from a local thermal power plant; sewage sludge was produced by sewage treatment plant. All the three raw materials were dried, and then crushed into powders, which then were screened by a 2mm-sieve.

2.1.1 Plastic index

The plastic index was evaluated by the Atterberg limits test according to the procedure as described in the national standardization (GB/T 50123-1999). Normally, the clay applied to brick making has a plasticity index (PI) value ranging from 7 to 15. Larger PI generally causes larger linear drying shrinkage, increasing the possibility of cracking. The PI value of lake silt in the experiment was 14.2.

2.1.2 Oxide composition

The chemical composition measured by x-ray fluorescence (XRF) of the raw materials is given in Table 1.

<table>
<thead>
<tr>
<th>Oxide</th>
<th>Lake Silt (wt.%)</th>
<th>Cinder (wt.%)</th>
<th>Sewage Sludge (wt.%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>69.74</td>
<td>39.04</td>
<td>22.97</td>
</tr>
<tr>
<td>Al2O3</td>
<td>16.20</td>
<td>9.39</td>
<td>9.68</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>5.78</td>
<td>19.50</td>
<td>25.67</td>
</tr>
<tr>
<td>CaO</td>
<td>1.29</td>
<td>15.44</td>
<td>22.22</td>
</tr>
<tr>
<td>MgO</td>
<td>2.11</td>
<td>0.91</td>
<td>1.40</td>
</tr>
<tr>
<td>Na2O</td>
<td>1.24</td>
<td>0.00</td>
<td>1.70</td>
</tr>
<tr>
<td>K2O</td>
<td>2.13</td>
<td>0.48</td>
<td>0.77</td>
</tr>
<tr>
<td>LOI</td>
<td>5.11</td>
<td>1.04</td>
<td>47.50</td>
</tr>
</tbody>
</table>
Silicon, aluminum and iron were the major compositions in lake silt, while the mass ratio of silicon and aluminum in cinder and sewage sludge were lower than that of silt’s. The loss on ignition (LOI) of sewage sludge was far more than silt, which indicated that the sludge contains a lot of organic matters.

### 2.1.3 Thermal analysis

Thermal behaviours of silt, sewage sludge were tested by thermogravimetry (TG) and differential scanning calorimetry (DSC) to determine sintering temperature, shown in Figure 1(a) and Figure 1(b), respectively. In Figure 1(a), there is an exothermic peak in the DSC curve at 310.6°C caused by the combustion of organic matter, and an endothermic peak at 577.0°C was attributed to the transformation of quartz α into β. There was an obvious weight loss between room temperature and 600°C in the TG curve in Figure 1(a), caused by the removal of adsorbed water and crystal water. The weight loss between 600°C and 900°C was caused by combustion of coke; there was almost no weight loss or increase beyond 900°C. The total loss on ignition was measured at 5.1%.

The combustion of sludge generally is divided into three stages: removal of adsorbed and crystal water, decomposition of organic matter, decomposition and carbonization of residual organic matter. In Figure 1(b), exothermic peak in the DSC curve at around 300°C was caused by decomposition of organic matter. In addition, there is a high exothermic peak at 406°C caused by combustion of organic matter and coke. Endothermic peak at around 655.3°C was caused by decomposition of carbonate. There was a continuous weight loss distributed between 25°C and 700°C in the TG plot of sludge, shown in Figure 1(b). Weight loss of 11.6% between 25°C and 200°C was observed in the TG curve, caused by the removal of adsorbed water. Weight loss of 30.6% between 200°C and 500°C was mainly due to volatilization of organic matter. While above 600°C, there was no weight loss or increase. The total loss on ignition was measured at 47.5%, higher than lake silt, mainly caused by the combustion of organic matter. Meanwhile, the process of combustion of organic matter produced a large amount of heat, which was helpful to sintering process of sludge.

![Figure 1: TG-DSC curves of lake silt (a) and solid sewage sludge (b)](image)

### 2.2 Methods
2.2.1 Preparation of fired brick

The sieved raw materials were blended to produce homogenous mixtures which contained 20 wt% water and gained adequate plasticity. The mixtures were aged for 3 days before being used. The mixture ratio used in the experiment was shown in Table 2.

Table 2: Raw mix compositions for laboratory brick production (wt.%)

<table>
<thead>
<tr>
<th>Mix</th>
<th>Lake Silt</th>
<th>Cinder</th>
<th>Sewage Sludge</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>90</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>85</td>
<td>10</td>
<td>5</td>
</tr>
</tbody>
</table>

Forming
Adobe samples were formed using the extrusion process. The following processing parameters were found optimal to produce adobes from the mixtures containing 20 wt% of moisture: 1 MPa of forming pressure, 0.075 MPa of pressure in vacuum zone. The extruded bars were cut into two sizes of smaller blocks in dimensions of 100mm×28mm×17mm and 50mm×28mm×17mm, respectively.

Drying
Adobe samples were firstly dried at room temperature in the laboratory for 24 h, then in an oven from 25°C to 105°C at a heating rate of 10°C/h, until the mass of the adobes did not change any more.

Sintering
Dried specimens were fired in an electric laboratory furnace with a heating rate of 100°C/h, and dwelled for 2h at the maximum temperature 950°C. After turning off, the samples were cooled down naturally with the furnace.

2.2.2 Testing method
Some important performance indexes (bulk density, water absorption, compressive strength) characterizing the quality of the resultant bricks were evaluated in accordance with the Chinese standard GB/T (2542-2012). Thermal Conductivity Analyzer (TPS 2500S) was used to measure the thermal conductivity. The microstructure of the final products was examined using a scanning electron microscope (SEM). Porosity and pore size distribution of the fired bricks was measured by mercury intrusion porosimetry (MIP).

3 RESULTS AND DISCUSSION

3.1 Properties of the bricks
Photographs of the bricks were shown in Figure 2. The surfaces of Brick 1 (Mix 1) and Brick 2 (Mix 2) were smooth and delicate. However, the addition of sewage sludge roughened the surface of Brick 3 (Mix 3), which was caused by the high LOI of sewage sludge. The properties of three types of bricks, including the linear drying shrinkage, bulk density, water absorption, coefficient of thermal conductivity were shown in Table 3.
Table 3: Properties of adobe and final bricks

<table>
<thead>
<tr>
<th>Mix</th>
<th>Linear Drying Shrinkage (%)</th>
<th>Bulk Density (Kg/m³)</th>
<th>Water Absorption (%)</th>
<th>Thermal Conductivity (W/(m·k))</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.45</td>
<td>1710</td>
<td>15.4</td>
<td>0.623</td>
<td>33.1</td>
</tr>
<tr>
<td>2</td>
<td>5.49</td>
<td>1686</td>
<td>17.2</td>
<td>0.610</td>
<td>25.4</td>
</tr>
<tr>
<td>3</td>
<td>5.35</td>
<td>1633</td>
<td>16.5</td>
<td>0.533</td>
<td>20.5</td>
</tr>
</tbody>
</table>

3.1.1 Linear drying shrinkage

The linear drying shrinkage in drying process of adobe samples is an important factor for controlling cracking of the product. As seen from Table 3, the linear drying shrinkage was measured for bricks of each mix design. The linear drying shrinkage of Brick 1 was 6.45%, which was higher than the other two mixes. This was due to the fact that the silt particle was smaller than that of cinder and sludge, and has high plasticity index. With the addition of 10% of cinder, the linear drying shrinkage of Brick 2 reduced to 5.49% due to the restraint effect of coarse cinder particles. The linear drying shrinkage of Brick 3 was 5.35%, a little smaller than Brick 2. This was because that dry sludge had large and hard particles which could act as skeleton to some extent.

3.1.2 Bulk density

The bulk density, which is strongly related to the loss on ignition, is a key parameter for the brick. As seen from Table 3, the bulk density of Brick 1 was 1710 kg/m³. With the addition of 10% of cinder, the density of Brick 2 reduced to 1686 kg/m³. The addition of sewage sludge further reduced the bulk density of Brick 3, which was 1633 kg/m³. The main reason for such a trend is the combustion of the organic matter during the sintering period. Compared to lake silt, sewage sludge had a very high LOI, as seen from Table 3. The proper reduction of the bulk density of brick is beneficial to the decrease of the weight of building and the increase of the heat insulation efficiency.

3.1.3 Water absorption and thermal conductivity

Water absorption and thermal conductivity are the key factors affecting durability and heat insulation performance of the bricks. Water absorption and thermal conductivity of fired brick are related to the internal pore structure, which came from evaporation of water in
drying stage and combustion of organic matter, decomposition of mineral.

Water absorption of the brick samples was tested, and the results are presented in Table 3. The 24h water absorption based on the amount of open pores, and 5h boiling water absorption based on the amount of total pores of Brick 1 was 15.4% and 17.4%, respectively. The addition of cinder increased the 24h water absorption and 5h boiling water absorption of Brick 2, which was caused by the combustion of the organic residues during the sintering period. Compared to Brick 2, the 24h water absorption and 5h boiling water absorption slightly dropped in Brick 3. The reason was that the combustion of significant organic matter from sludge produced a large amount of heat, which promoted the sintering of the particles around sewage sludge.

As seen from Table 3, the largest thermal conductivity were obtained in Brick 1. The thermal conductivity values of reference bricks (Brick 1) was 0.623 W/(m·k). When cinder was added to lake silt, the thermal conductivity values of Brick 2 decreased. The thermal conductivity value of Brick 3 was 0.533 W/(m·k), decreased by 14% after the addition of sewage sludge. This result demonstrates that the brick samples produced with sewage sludge addition could have good heat insulation performance.

3.1.4 Compressive strength

There is no doubt that the compressive strength is the most important property of construction materials. As shown in Table 3, the compressive strength of Brick 1 was 33.1MPa. With up to 10% cinder added to the brick, the strength of Brick 2 was 25.4MPa, which was 23% lower compared to Brick 1. The compressive strength of Brick 3 was 20.5MPa, which was 38% lower compared to Brick 1. The main reason is that sludge has great weight loss during the sintering period compared to lake silt and cinder. So, the addition of sludge to bricks should be strictly controlled. It should be noted that the compressive strength of all bricks still meets the requirement of Chinese standard GB/T (2542-2012).

3.1.5 Freeze-thaw resistance

![Figure 3: Freeze-thaw resistance of fired brick samples](image-url)

Figure 3: Freeze-thaw resistance of fired brick samples
In order to investigate the durability of the bricks, frost resistance of bricks was evaluated. The test was conducted on 5 specimens of each mix. The brick samples were subjected to cycles of freezing and thawing according to the procedure as described in the Test methods for wall bricks (GB/T 2542-2012).

The results were shown in Figure 3. It can be seen that the weight loss of all the bricks after 100 cycles was very low. According to GB/T 2542-2012, the weight loss of individual building brick should not be greater than 2%. Brick 1 performed the best under frost attack, with Brick 3 followed, while Brick 2 showed the highest weight loss.

3.2 Toxicity analysis

Gases released during sintering process were analyzed by the combined technique of high resolution gas chromatography and high resolution mass spectrometry (HRGC/HRMS) according to the specification (HJ77.2-2008) for testing the PCDDs/PCDFs in environment and waste gas via HRGC/HRMS. Results were given in Table 4. The results showed that the toxic equivalent of PCDDs/PCDFs was 0.176 ngTEQ/m$^3$, which was much less than the 1 ngTEQ/m$^3$, specified in GB18485-2014 (Chinese National Standards).

**Table 4: PCDDs/PCDFs produced during the sintering process of bricks**

<table>
<thead>
<tr>
<th>Test items</th>
<th>Concentration (ng/m$^3$)</th>
<th>Toxicity factor I-TEF</th>
<th>Equivalent concentration of toxicity (ng/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCDDs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2,3,7,8-T4CDD</td>
<td>0.00434</td>
<td>1</td>
<td>0.00434</td>
</tr>
<tr>
<td>1,2,3,7,8-P5CDD</td>
<td>0.0396</td>
<td>0.5</td>
<td>0.0198</td>
</tr>
<tr>
<td>1,2,3,4,7,8-H6CDD</td>
<td>0.0378</td>
<td>0.1</td>
<td>0.00378</td>
</tr>
<tr>
<td>1,2,3,6,7,8-H6CDD</td>
<td>0.0557</td>
<td>0.1</td>
<td>0.00557</td>
</tr>
<tr>
<td>1,2,3,7,8,9-H6CDD</td>
<td>0.0492</td>
<td>0.1</td>
<td>0.00492</td>
</tr>
<tr>
<td>1,2,3,4,6,7,8-H7CDD</td>
<td>0.256</td>
<td>0.01</td>
<td>0.00256</td>
</tr>
<tr>
<td>O8CDD</td>
<td>0.295</td>
<td>0.001</td>
<td>0.000295</td>
</tr>
<tr>
<td>PCDFs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2,3,7,8-T4CDF</td>
<td>0.0242</td>
<td>0.1</td>
<td>0.00242</td>
</tr>
<tr>
<td>1,2,3,7,8-P5CDF</td>
<td>0.0609</td>
<td>0.05</td>
<td>0.00305</td>
</tr>
<tr>
<td>2,3,4,7,8-P5CDF</td>
<td>0.138</td>
<td>0.5</td>
<td>0.0691</td>
</tr>
<tr>
<td>1,2,3,4,7,8-H6CDF</td>
<td>0.156</td>
<td>0.1</td>
<td>0.0156</td>
</tr>
<tr>
<td>1,2,3,6,7,8-H6CDF</td>
<td>0.160</td>
<td>0.1</td>
<td>0.0160</td>
</tr>
<tr>
<td>2,3,4,6,7,8-H6CDF</td>
<td>0.187</td>
<td>0.1</td>
<td>0.0187</td>
</tr>
<tr>
<td>1,2,3,7,8,9-H6CDF</td>
<td>0.0421</td>
<td>0.1</td>
<td>0.00421</td>
</tr>
<tr>
<td>1,2,3,4,6,7,8-H7CDF</td>
<td>0.481</td>
<td>0.01</td>
<td>0.00481</td>
</tr>
<tr>
<td>1,2,3,4,7,8,9-H7CDF</td>
<td>0.0604</td>
<td>0.01</td>
<td>0.000604</td>
</tr>
<tr>
<td>O8CDF</td>
<td>0.130</td>
<td>0.001</td>
<td>0.000130</td>
</tr>
<tr>
<td>TEQ(PCDDs+PCDFs)</td>
<td></td>
<td></td>
<td>0.176 ngTEQ/m$^3$</td>
</tr>
</tbody>
</table>
Following the Chinese specification HJ 557-2010 on leaching toxicity test method for solid wastes, heavy metal concentrations was conducted using ICP-AES. The results are shown in Table 5. It showed that heavy metals in bricks were about 5 magnitudes lower than that specified in specification for leaching toxicity evaluation of dangerous wastes (GB5085.3-2007), which demonstrated that heavy metals were effectively immobilized, and thus the environment would not be polluted by the use of the bricks.

<table>
<thead>
<tr>
<th></th>
<th>Cd</th>
<th>Pb</th>
<th>Cr</th>
<th>As</th>
<th>Cu</th>
<th>Zn</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.878</td>
<td>2.358</td>
<td>2.689</td>
<td>1.457</td>
<td>3.869</td>
<td>1.326</td>
</tr>
<tr>
<td>3</td>
<td>1.853</td>
<td>2.041</td>
<td>4.197</td>
<td>1.564</td>
<td>4.138</td>
<td>1.417</td>
</tr>
<tr>
<td>4</td>
<td>1.880</td>
<td>2.489</td>
<td>6.169</td>
<td>1.891</td>
<td>4.354</td>
<td>1.806</td>
</tr>
<tr>
<td>5</td>
<td>1.886</td>
<td>3.215</td>
<td>6.977</td>
<td>1.000</td>
<td>4.456</td>
<td>1.439</td>
</tr>
<tr>
<td>Limits</td>
<td>100000</td>
<td>500000</td>
<td>1500000</td>
<td>500000</td>
<td>10000000</td>
<td>10000000</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

- In this investigation, lake silt, cinder and sewage sludge were used as raw materials to prepare fired brick samples using the vacuum plastic extruder in laboratory. Following major conclusions can be made:
  - Sewage sludge can be used together with lake silt and cinder to produce brick which can meet the requirements of Chinese specifications. The incorporation of cinder and sewage sludge into lake silt reduced the linear drying shrinkage of adobe, but at the same time increased the water adsorption and lowered the compressive strength and the frost resistance of the fired brick.
  - The toxic equivalent quantity of PCDDs/PCDFs of gas collected during the sintering process was much less than the limit specified in standard for pollution control of the municipal solid waste incineration. Leaching test showed that heavy metals in bricks were immobilized effectively during the sintering process.
  - This study reveals that it is an effective way to comprehensively use lake silt, cinder and sewage sludge as raw material to produce bricks.

REFERENCES

[3] Q. Zhou, Y. Zhu, Potential pollution and recommended critical levels of phosphorus in


SHOTCRETE REINFORCED WITH RECYCLED FIBERS FROM SECONDARY WASTE OF END OF LIFE TIRES
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Key words: End-of-life tyres, recycled fiber, mechanical behaviour, shotcrete.

Abstract. The main objective of this research work is to study the possibility of obtaining a functional, sustainable, and less expensive concrete by using in its production metallic fibers produced from tyres that are no longer in use. In addition, the work includes the identification of any benefits originated by the use of these recycled fibers in pavements and shotcrete.

This research analyzes concretes produced with recycled fibers from different sources - cars and trucks - and obtained through different methods. The concretes’ mechanical characteristics and their spraying possibilities are studied in the laboratory prior to production for spraying in a construction site and analyzing the existing differences.

The spraying conditions are studied for dry and a wet process, and the technological difficulties in the process of introducing the fibers in the concrete mixer and during projection are discussed.

Regarding the flexural strength, the use and the quantity of different types of fibers do not introduce significant changes in the first crack strength. However, these parameters do introduce changes in the results of residual strength depending on type and amount of fibers. There have been differences observed between results obtained in specimens made by pouring and those made by spraying. Energy absorption results obtained are promising.

The difficulty of fiber-reinforced concrete spraying process is also considered adequate.

1 INTRODUCTION

Since 2003, the European Directive 1999/31/CE bans the deposit of entire used tyres in dumps, and since 2006 of shredded tyres. This Directive was transposed into Spanish law through Royal Decree 1481/2001. This regulatory framework favoured new uses of end-of-life tyres (ELT), as for example its retreaded, recycling for industrial use or use as a source of energy. [1][2]

About current applications of recycled materials of ELT, and in particular, of the rubber; the majority application still is the use of granule as grass infill. [2]

Secondary waste which contain steel fiber is often used as scrap metal at the steel sector. The Figure 1 shows the percentage distribution, in Europe, of the ELT destination [3].

ELTs are considered non-hazardous waste. In Spain is estimated that more than 300,000 tones will be produced before 2020. That is why it is essential the identification of new applications for these materials in alternative markets. [1]
Steel fiber have been used at certain intervals for concrete reinforcement [4], especially in tunnel and pavement applications, but also in structural elements.

The characteristics of the commercial fibers used for those applications are similar to the ones that appear as waste at the ELT; and the energetic cost to obtain fiber from ELT is lower than obtaining new fiber. Nonetheless there are little experience focused to the secondary product recycling [5].

The main objective of this paper is to analyze the viability of the valorization of those secondary waste, in a environmentally sustainable way, for the steel fibers production and its use for concrete reinforcement.

As specific objectives, are raised:

Improve the recycling process of ELT in order to obtain high quality steel fiber, adequate to its proposed use.

Corroborate the validity of the recycled steel fiber as alternative to commercial fiber, to be used in pavements or in tunnels by shotcreting.

This paper shows some results of the work made in the context of the project "Secondary Waste From ELT revaluation" Ref IPT-2012-0990-310000 founded by the Spanish government - Ministry of economy and competiveness into the subprogram "INNPACTO". leaded by GMN - COMSA emte group.

2 VALORIZATION PROCESS OF THE FIBERS FROM ELT

Recycled fibers were obtained as a secondary waste from the tyres recycling plant that the company GMN (COMSA emte group) has in Lleida. This waste is a product of the tyre treatment to obtain granule rubber. At present it is commercialized as scrap with small value. The waste contain steel fiber very contaminated with rubber and textile fiber. Two types of tyres were analyzed depending on its previous use (car and truck). Figure 2 shows the appearance of the used waste.

Steel fiber obtained from tyres can be grouped in two different types, depending on the part of the tyre that they are, and so, the diameter they have: fiber from the bead, with big diameter...
(around 1 mm in the case of car fiber and 1.5 mm in the truck case) and small diameter fiber from the tyre belt (around 0.25 mm in the case of car fiber and 0.30 mm in the truck case). (Figure 3)

**Figure 2**: Used waste tyres from a) car b) truck

![Figure 2: Used waste tyres from a) car b) truck](image)

**Figure 3**: Section of a tyre

![Figure 3: Section of a tyre](image)

To eliminate impurities from the waste of GMN fiber, steel fiber have been separated following a series of operations including different levels sieving, air flowing and magnetizing. Each kind of waste has a two level treatment involving the separation or non-separation of the bead fiber. Then 4 types of recycled fibers will be produced:

- CA-P1: car fiber phase 1 - or car with bead.
- CA-P2: car fiber phase 2 - or car without bead.
- TR-P1: truck fiber phase 1 - or truck with bead.
- TR-P2: truck fiber phase 2 - or truck without bead.

Table 1 shows the sieving types and the separation scheme used in every phase.

Table 2 shows the efficiency (as the percentage of final recycled fiber obtained with respect to the weight of the treated waste) of the proposed procedures to obtain fiber from the GMN waste. The fraction considered as non-useful as fiber in this process can still be used in the steel industry as previously planned.

The fiber type obtained with this procedure are shown in the figure 4 compared with the reference fiber: recycled fibers commercialized by ADRIA company (Adriatica Riciclaggio e
Ambiente) ABRUZZO SRL and commercial fibers DRAMIX RC 65/35 BN and RC 80/35 BN. All fiber have been geometrically characterized by determining its average diameter and length. Results are shown in the table 3.

Generally, obtained product shows a large variety of lengths and diameters, in particular before bead separation. In contrast, ADRIA fiber show more homogeneous dimensions and correspond to a more industrialized and selective treatment.

<table>
<thead>
<tr>
<th>Table 1: Scheme of fiber production</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product type</td>
</tr>
<tr>
<td>Treatment</td>
</tr>
<tr>
<td>two levels of mesh opening:</td>
</tr>
<tr>
<td>25x25 mm and 2x20 mm.</td>
</tr>
<tr>
<td>2.- Magnetizing:</td>
</tr>
<tr>
<td>to eliminate the finest particles of the rubber.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2: Efficiency of the fibers recycling procedures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product type</td>
</tr>
<tr>
<td>Useful % recycled of the initial weight.</td>
</tr>
</tbody>
</table>

Figure 4: Obtained fiber after treatment and reference fiber
Table 3: Geometrical characteristics of the fiber.

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Average length (mm)</th>
<th>Average diameter (mm)</th>
<th>Rubber</th>
<th>Textil</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADRIA</td>
<td>10</td>
<td>0.35</td>
<td>&lt;1%</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>CA-P1</td>
<td>36</td>
<td>0.5</td>
<td>&lt;1%</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>CA-P2</td>
<td>29</td>
<td>0.25</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>TR-P1</td>
<td>37</td>
<td>0.6</td>
<td>&lt;1%</td>
<td>0%</td>
</tr>
<tr>
<td>TR-P2</td>
<td>19</td>
<td>0.32</td>
<td>&lt;1%</td>
<td>0%</td>
</tr>
</tbody>
</table>

3 CONCRETE CHARACTERIZATION WITH FIBER IN LAB

To analyze the viability of the recycled fiber use, an experimental campaign was run in the lab to analyze the effect of the fiber on the concrete strength according to the criterion proposed in EN 14845-1 and EN 14845-2. The reference concrete mix proportioning is shown in table 4. The super-plasticizer dosing was adjusted in each case to obtain an enough workable concrete (slump close to 100 - 120 mm). This admixture demand increased as the fiber dosing increased.

Table 4: Concrete mix design.

<table>
<thead>
<tr>
<th>Component</th>
<th>Lab mix design (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement - CEM I 42,5 R</td>
<td>350</td>
</tr>
<tr>
<td>Water</td>
<td>192.5</td>
</tr>
<tr>
<td>Water / cement</td>
<td>0.55</td>
</tr>
<tr>
<td>Crushed limestone coarse aggregate</td>
<td></td>
</tr>
<tr>
<td>7/12</td>
<td>42</td>
</tr>
<tr>
<td>4/7</td>
<td>786</td>
</tr>
<tr>
<td>Limestone sand 0/4</td>
<td>990</td>
</tr>
<tr>
<td>Filler</td>
<td>10</td>
</tr>
<tr>
<td>Fiber</td>
<td>VARIABLE</td>
</tr>
<tr>
<td>Admixture sika viscocrete 5720</td>
<td>VARIABLE</td>
</tr>
</tbody>
</table>

The incorporation of recycled fiber to concrete presented a bigger difficulty than commercial fiber because of its tendency to create "clusters" while mixing. However the problem was solved by introducing progressively the fiber. Once in the mixer, concrete didn't show any separation problem.

For the concrete characterization, three test campaign were performed, testing mixes with different fiber dosage and types as shown in table 5. Compressive and flexural strength were determined according to EN 12390-3 y EN 14651 respectively at 28 days. Some determinations were repeated in different campaigns. To do that in every mixing three cylindrical specimens 150x300 mm and 3 or 4 prismatic150x150x600 mm specimens were casted and tested. Results in table 5 are the average value of all specimens from each mix. Figure 5 show the obtained flexural behavior for 35 Kg/m³ fiber dosing in the Dramix 80/35 fiber case, and 50 Kg/m³ for the recycles fibers.

All concretes show similar first cracking strength and, as expected, the tendency shows
that a bigger fiber amount means a higher residual flexural strength.

Table 5: Results of the characterization tests

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Fiber dosing kg/m³</th>
<th>( f_c ) (MPa)</th>
<th>( f_{r1} ) (MPa)</th>
<th>( f_{r3} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Campaign number</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Plain concrete</td>
<td>32</td>
<td>35,63</td>
<td>45,9</td>
<td>---</td>
</tr>
<tr>
<td>DRAMIX 65/35 10</td>
<td>44,1</td>
<td>---</td>
<td>---</td>
<td>2,53</td>
</tr>
<tr>
<td>DRAMIX 65/35 20</td>
<td>45,73</td>
<td>---</td>
<td>---</td>
<td>3,11</td>
</tr>
<tr>
<td>DRAMIX 80/35 35</td>
<td>41,6</td>
<td>37,73</td>
<td>---</td>
<td>2,6</td>
</tr>
<tr>
<td>ADRIA 30</td>
<td>44,88</td>
<td>---</td>
<td>---</td>
<td>1,36</td>
</tr>
<tr>
<td>ADRIA 40</td>
<td>45,54</td>
<td>---</td>
<td>---</td>
<td>2,12</td>
</tr>
<tr>
<td>ADRIA 50</td>
<td>---</td>
<td>42,98</td>
<td>35,04</td>
<td>---</td>
</tr>
<tr>
<td>CO-F1 30</td>
<td>47,57</td>
<td>---</td>
<td>---</td>
<td>3,31</td>
</tr>
<tr>
<td>CO-F1 40</td>
<td>48,84</td>
<td>---</td>
<td>---</td>
<td>3,62</td>
</tr>
<tr>
<td>CO-F1 50</td>
<td>---</td>
<td>44,23</td>
<td>42,22</td>
<td>---</td>
</tr>
<tr>
<td>CO-F1 60</td>
<td>37,4</td>
<td>---</td>
<td>---</td>
<td>5,49</td>
</tr>
<tr>
<td>CO-F2 30</td>
<td>39,55</td>
<td>---</td>
<td>---</td>
<td>3,61</td>
</tr>
<tr>
<td>CO-F2 50</td>
<td>---</td>
<td>45,46</td>
<td>---</td>
<td>4,92</td>
</tr>
<tr>
<td>CO-F2 60</td>
<td>34,73</td>
<td>---</td>
<td>---</td>
<td>3,73</td>
</tr>
<tr>
<td>CA-F1 30</td>
<td>38,74</td>
<td>---</td>
<td>---</td>
<td>2,61</td>
</tr>
<tr>
<td>CA-F1 40</td>
<td>42,97</td>
<td>---</td>
<td>---</td>
<td>4,14</td>
</tr>
<tr>
<td>CA-F1 50</td>
<td>---</td>
<td>40,84</td>
<td>---</td>
<td>3,03</td>
</tr>
<tr>
<td>CA-F1 60</td>
<td>35,12</td>
<td>---</td>
<td>---</td>
<td>4,44</td>
</tr>
<tr>
<td>CA-F2 30</td>
<td>32,54</td>
<td>---</td>
<td>---</td>
<td>2,16</td>
</tr>
<tr>
<td>CA-F2 50</td>
<td>---</td>
<td>40,79</td>
<td>---</td>
<td>3,49</td>
</tr>
<tr>
<td>CA-F2 60</td>
<td>34,75</td>
<td>---</td>
<td>---</td>
<td>4,19</td>
</tr>
</tbody>
</table>

All fiber type at all the tested dosage reached the minimum \( f_{r1} \) and \( f_{r3} \) residual strength flexural level to be considered as structural capacity according to MC 2010.

This means that with recycled fibers it is possible to reach enough capacity than with the commercial ones by only increasing the fiber dosage. For the rest of the testing program it was decided to fix the fiber dosing for DRAMIX 80/35 fiber at 35 Kg/m³, and for the recycled fibers (ADRIA, car and truck fibers) in 50 Kg/m³ in order to have a comparable results.

Both for pavements and sprayed concrete applications (the most extended fibers uses) usually demand to determined the concrete energy absorption capacity. This property is obtained by testing square slabs specimens according with 14488-5.

Moreover in tunneling the casting method by spraying concrete may also influence the fibers contribution. For this reason it was decided to analyze this property for the proposed recycled fibers. To do that a testing program was performed both casting concrete by pouring in laboratory and by spraying in a construction site. The mix design was adapted to the site possibilities and it is shown in table 6. The spraying method was dry way. (Figure 6)
It was not possible to introduce the Phase I recycled fibers into the sprayed procedure as thick fibers created clusters with risk to damaged the spraying hose. When casting was made in laboratory tree specimens were produced for each studied combination and when sprayed only one specimen was casted. Figure 7 shows the testing set-up and table 7 the average obtained results. Figure 8 shows examples of experimental complete results for some concretes.

Laboratory made specimens are signaled with L and worksite made specimens are signaled with WS.

Only some concrete panels made with ADRIA fiber has energy capability until 25 mm deflection under 500 J. The rest exceed this minimum value considered in EN 14487-1 to be characterized in a absorption energy class, and then may be adequate for tunneling.
applications. Even someone overpass the 1000 J value.

Again, recycled fiber obtained in Phase I have better results than those in Phase II, and again, samples made with CA-P2 show low values (although higher than 500 J) close to the limit.

Table 6: Concrete mix design for energy absorption test

<table>
<thead>
<tr>
<th>Component</th>
<th>Weight (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM I - 42.5</td>
<td>400</td>
</tr>
<tr>
<td>Nominal water</td>
<td>180</td>
</tr>
<tr>
<td>Coarse aggregate 4/12 mm</td>
<td>560</td>
</tr>
<tr>
<td>Sand 0/2</td>
<td>300</td>
</tr>
<tr>
<td>Sand 0/4</td>
<td>1040</td>
</tr>
<tr>
<td>Fiber VARIABLE (DRAMIX 80/35 :35; rest: 50)</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6: Spraying procedure

Figure 7: Energy absorption test set up
Table 7: Energy absorption test results

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Maximum load (kn)</th>
<th>Energy absorption at deflection 25 mm (J)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-PLAIN</td>
<td>46.23</td>
<td>---</td>
</tr>
<tr>
<td>L-DRAMIX 80/35</td>
<td>88.35</td>
<td>1735</td>
</tr>
<tr>
<td>WS-DRAMIX</td>
<td>59.33</td>
<td>1290</td>
</tr>
<tr>
<td>L-ADRIA</td>
<td>57.01</td>
<td>560</td>
</tr>
<tr>
<td>WS-ADRIA</td>
<td>38.84</td>
<td>622</td>
</tr>
<tr>
<td>WS-CA-P2</td>
<td>46.59</td>
<td>587</td>
</tr>
<tr>
<td>WS-TR-P2</td>
<td>55.06</td>
<td>972</td>
</tr>
<tr>
<td>L-CA-P1</td>
<td>108.68</td>
<td>1780</td>
</tr>
<tr>
<td>L-TR-P1</td>
<td>88.05</td>
<td>1209</td>
</tr>
</tbody>
</table>

Figure 8: Energy absorption test results (MPa/mm)
4 WET WAY SPRAYING TEST IN WORKING SITE

In order to verify the real possibilities of the application of recycled fibers a final test was performed in the construction of the ADUNA tunnel in Zuzurkil (Spain) by wet way as it is usual procedures for the high concrete volume in real conditions applications. A 2m³ mix experience casting in a concrete plant, transported (2 hours) in a mixer truck, and sprayed was performed. Mix design (by m³) was 425 kg of CEM I 52,5; sand 1350 kg; 4/8 mm aggregate 380 kg; w/c ratio 0,36 and 40 kg of recycled fibers in accordance with the second phase fibers selection.

Once the plain concrete was in the mixer truck, fibers were introduced at a very low rate (40 min) an then transported (2 hours) to the construction site, and then sprayed. (fig 9). Even than the fibers introduction time was too much, all the expected problems were easily overcome, and the experience may be considered promising.

Moreover, the mechanical results in the plate test to determine the energy absorption was 1540 J. That means even higher than needed.

Figure 9.- On site spraying of concrete

5 CONCLUSIONS

An experimental program to verify the possibilities of using recycled steel fibers obtained from ELT was performed. The main conclusions are:
- The method of production has been analyzed, reaching the conclusion that the best efficiency and performance is obtained with car and truck fiber with bead; while car and truck fiber without bead present difficulties and low efficiency.
- It is possible to obtain a good compaction and homogeneity if concrete is vibrated.
- In light of the results achieved at the flexural tests it is concluded that fibers doesn't change the first cracking strength but improves the residual one. Fibers obtained in the first phase selection (with thicker and longer fibers) shows higher residual strength.
- It can be observed the non-possibility of spraying of fiber with bead due to its length and thickness, showing the need of obtaining shorter fiber on the recycling pre-treatment. The
difficulty of obtaining fiber without bead doesn't justify a better resistance, but make possible the spraying process.
- Tests performed on site have demonstrate the viability of spraying and have shown the needs of improvements to be studied.
- With those ideas it is logical to propose a fibers selection in two phases in order to valorize the thick fiber to be used in pavements applications due to its better mechanical behavior, and to valorize the second phase fibers for sprayed concrete based on its better possibilities in the process.
- A promising way to valorize the secondary waste in the ELT recycling and to produce a more sustainable concrete was proposed and evaluated.

REFERENCES


[3] cambiar a inglés


A CASE OF STUDY FOR EMBEDDING RFID TAGS IN PRECAST CONCRETE

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Key words: Precast, labeling, RFID

Abstract. Passive Radio-Frequency IDentification (RFID) tagging can be a useful tool to leverage building information models (BIM) systems. Passive RFID tags can be used, for instance, to improve traceability in precast, transport of pieces and quality assurance for identifying unitary building pieces and its location following the BIM definition.

RFID tags, unlike other identification techniques in construction, such as labels and barcodes, which are usually located in the surface of BIM items, can be embedded into the concrete pieces themselves. In addition, as passive RFID tags do not require a power supply to operate, they provide a means for long-term labelling that dramatically improves both the traceability of items from the BIM system and their sustainability.

In this paper the authors present a preliminary study where different types of RFID tags have been embedded into concrete at different depths to assess the viability of adopting this technology for item labeling. Once the RFID tags were embedded in the concrete pieces, we tried to obtain a valid response from the tags using a RFID reader.

The result of this study suggests the viability of achieving long-term electronic labeling of concrete pieces by embedding RFID tags into the concrete. In addition, as the concrete pieces themselves protect the RFID tags during the transportation and building stages of BIM pieces, this improves the durability of the labeling. The data stored in the RFID tags can be also secured by resorting to standard data encryption methods and tools.

1 INTRODUCTION

When a concrete structure is built, the availability of all the information regarding its design and construction phases is crucial during the structure’s lifetime. All this information is often provided by the contractor and stored in different places. In addition, storing information about the quality of the concrete—such as concrete strengths for different batches, dates of reception of the materials and the quality control of the processes—are mandatory if the contractor meets the ISO 9000 and the ISO 14000 standards.

All these heterogeneous pieces of information are often stored in different systems and
remote locations, thus being a hard and difficult task to gather all this information. Therefore storing all this information into the pieces themselves, would enable the authorized personnel to obtain all this information in-situ.

In recent years several studies have suggested the possibility of embedding electronic tags in concrete [1][2]. Moreover, the advantages of combining radio-frequency identification (RFID) tags with building information models (BIM) systems [3][4][5][6] have been also highlighted, as well as its benefits in production chains [7].

In this paper we propose a labeling method which involves embedding electronic tags that can store any type of information including not only the identification of a piece itself, but also information from (i) building information models (BIM) systems, such as design plans, (ii) information about the contractor —e.g. traceability for the materials contained in the piece— (iii) transportation information, (iv) construction processes, etc. The possibility to retrieve all these information in-situ, directly from the concrete pieces, is a novel idea that could help simplifying structure and building management. Furthermore, it is also possible to update the information stored in the concrete pieces by resorting to a management application installed in a high-end cell phone or tablet PC used by the structure or building management team.

In this paper we also present a mobile application that we developed to enable the communication with the embedded tags. This application allows a bidirectional communication for both storing and obtaining information into concrete pieces.

2 METHODS

As mentioned in the previous section, this paper presents a pilot study for studying the viability of using RFID tags as a labeling method for concrete pieces. In the next subsections we briefly describe the RFID technology and the different types of existing RFID tags. In Subsection 2.3, we describe the design of the experiment.

2.1 RFID technology

RFID is an acronym that stands for “Radio Frequency IDentification”, and refers to a technology targeted at identifying and/or communicating with remote devices — commonly known as RFID tags— in a contactless manner. RFID tags and readers are electronic devices composed of (i) a low-power microcontroller unit (MCU) or a custom ASIC —an integrated circuit designed for a particular use—, (ii) a radio transponder and (iii) an antenna and matching network tuned to the operating frequency.

Common frequency bands employed in RFID technology include Low Frequency (LF): normally 125 KHz or 134 KHz and High Frequency (HF): most often 13.56 MHz. RFID devices can also operate in the Ultra-High Frequency (UHF) band: 868 MHz (Europe), 915 MHz (United States) or 960 MHz (Japan). However, UHF is often neglected since there is a plethora of radio devices already operating in these frequencies (e.g. wireless home appliances, wireless alarm systems, etc.) that could easily interfere with RFID transponders. In addition, the operating frequency and range (read distance) depends on whether the target RFID tag is active or passive.

Active RFID tags are equipped with a power supply that enables them to operate even at microwave frequencies (e.g. 433 MHz, 2.4 GHz or 5.8 GHz), and thus to be readable at
longer distances (about 100 meters) than those operating in the LF or HF bands. The power supply keeps the MCU powered at all times. This enables the designer to optimize the antenna for transmitting, thus improving the range. As the on-board battery has a limited lifespan (about 1-2 years) these tags cannot be used for long-term applications such as, for instance, embedding the tags in concrete.

Unlike active RFID devices, passive tags do not include an on-board battery to power the microcontroller unit. Instead, they grab the carrier wave emitted by the reader via the antenna. The carrier wave is then rectified and filtered and eventually used to power the MCU. Hence, the MCU is only powered when it enters the magnetic field emitted by the reader. The strength of the magnetic field drops dramatically with the distance from the emitter, and thus these tags are inherently short range. However, as these tags are powered solely by the reader, their lifespan is much longer and thus can be used in long-term applications.

2.2 Near Field Communication

Near field communication (NFC) is a subclass of the RFID technology operating in the HF band (13.56 MHz). NFC enables two-way contactless communication between endpoints over short distances (about 6-7 cm) by resorting to inductive coupling between two coils — i.e. the antennae belonging to the reader and the tag respectively.

NFC tags can be regarded as passive data repositories with storage sizes ranging between 96 and 8192 bytes (as of 2015) that can be read —and in some cases even written— using a NFC-enabled device such as smartphone or tablet PC.

NFC-enabled devices are currently being used in many different applications including contactless payment systems, public transportation, social networking (such as sharing contacts, photos, documents, videos, etc.) or for security and identification purposes (e.g. the Spanish ID card v3.0 employs this technology).

2.3 Embedding RFID tags into concrete pieces

A laboratory study was designed for assessing the performance of different types of RFID tags embedded into concrete batches.

Concrete used in the tests is designed as a normal concrete of 30Mpa characteristic compressive strength to analyze the most common concrete used in structures in Spain.

The design mix has a cement content of 367Kg/m3, sand 594 Kg/m3, gravel 1.221Kg/m3 and 185 liters of water. Concrete strengths were a mean of 28MPa at 7 days and 36MPa at 28 days.

For the experiment described in this paper passive tags have been selected since they do not use an internal power supply. As passive tags are powered by the reader, their lifespan is more suitable for long-term applications, thus ensuring the durability of concrete labeling. Furthermore, the absence of a power supply makes this kind of tags ideal for embedding into concrete preserving them from ruggedness of the harsh construction industry environment.

Into each concrete cylinder two 13.56 MHz passive RFID tags were introduced at different depths —2cm and 3cm — from the concrete surface. Lately the response of RFID tags was measured by communicating with tags at different stages of the concrete curation.
In order to test the communication of the tag reader (a mobile phone equipped with a NFC transponder) with embedded tags an Android application was developed. This application uses a NFC transponder and antenna to communicate with 13.56 MHz passive RFID tags. One of the main challenges when using RFID technology is data security and privacy. Additionally, currently there is no international standard for data storage and transmission. Moreover different RFID and NFC vendors offer mutually incompatible proprietary features such as communication protocols, encoding and cypher. For this reason, the developed application uses a custom read/writing protocol based in NFC including a security encryption step to be able to read and write the information securely from NFC terminals.

2.4 Information stored in RFID tags

For this work the information stored in each tag includes a registry with the following basic information about the piece: (i) concrete type and strength, (ii) batch number, (iii) batch date, and, (iv) element reference number. Since, information stored in the RFID component is limited by the tag capacity data linkage techniques can be used for pointing to detailed information. Thus, additional URLs can also be stored for expanding available information from the concrete piece with relevant structure design and maintenance facts.

When data read from a tag is decoded using the mobile phone, the stored information about concrete piece is shown in the application. Figure 2 depicts an example of RFID tag data reading, where the piece contains basic information of its composition, and, additionally it contains URLs linking to the BIM documentation of the piece.

As commented in previous section, the developed application uses a custom communication protocol. This communication protocol uses a secured AES algorithm based on a hashed password for both reading and writing information into the RFID tag. Thus, only mobile phones that possess the right password could read tag information and decode it correctly. Using this kind of encryption policy, the developed Android application could be easily generalized for supporting different groups of tags, basically by including a different encryption key for each group of tags.
3 RESULTS

In order to test the availability of communication using radio frequency through concrete, seven 13.56 MHz passive RFID tags were embedded into four different concrete batches shown in Figure 1(b). Four RFID tags were situated two centimetres from the batch surface, and the other three tags at three centimetres.

Using the previously commented Android application, different readings were performed at different epochs in concrete curation, concretely, measurements at 1, 5, 10, 20, 45, 60 and 90 days after concrete mixing where done. Since the first measurement the day after batches creation, we achieved to successfully communicate with six of the seven embedded tags. This communication includes reading and writing RFID tags using both RFID readers from computers, and also using NFC communication from mobile devices.

Figure 3: Mobile application showing encrypted data contained in a RFID tag. (a) raw data and (b) decrypted data
Figure 3 shows a data reading performed using a NFC capable mobile phone using the developed Android application. The application is able to read encrypted contents written in NFC tags and show decrypted information from concrete.

3 CONCLUSIONS

The study performed demonstrates the availability of achieving long term electronic labelling by using RFID technology embedded in the concrete pieces. Authors have reached a successful communication with all the RFID tags that where incrusted into essay concrete batches during different moments in the lifetime of the batches.

Using RFID tags within concrete pieces provides a durable, robust and reliable way of labelling during transportation and building stages of BIM pieces. Moreover, this labelling eases building management by providing information directly from concrete pieces. Additionally the developed solution for testing the study enhances the usage of RFID technology by securing the information stored in RFID Tags.

For this study an Android application has been developed offering a simple and affordable interface for reading information through NFC technology. A cloud based platform is planned to be developed offering services for managing, writing and storing information from different RFID Tag sets.

REFERENCES


AN EXPERIMENTAL STUDY ON PRECAST CONCRETE BEAM-TO-COLUMN CONNECTION USING INTERLOCKING BARS

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Key words: beam-to-column connection, precast concrete, interlocking bars, development length, static loading, quasi-static loading

Abstract. A new type of precast concrete exterior beam-to-column connection is introduced in this paper. The connection consists of a precast U-beam, precast column with corbel, and interlocking bars to connect the precast column and precast beam; these bars, act as the flexural reinforcement to withstand the hogging and sagging moment subjected during testing. The beam-column joint was designed according to the strong column-weak beam principle. The static-monotonic loading was applied to the first specimen (P1), while a quasi-static loading based on displacement control was applied on the second specimen (P2). Tip deflection and specimen cracking were monitored during the tests. Plastic hinges were formed in the beam of both specimens. P1 has no significant cracks in the joint, while P2 has ‘X’ cracks in the joint but it was still deemed acceptable. The precast columns of both P1 and P2 were free from cracks. The connection met the acceptance criteria presented in ACI 374.1-05. The development length of the interlocking bars, i.e. 800 mm from the column face, was sufficient to transfer the load from the beam to the joints and columns.
1 INTRODUCTION

Precast concrete systems have become more popular in the last few decades. These systems have some advantages in comparison with the conventional in-situ construction methods, i.e. quality, time and cost [1-3]. Nurjaman et al. [4] concluded that the reduction in the construction time is around 30%. In addition, the cost efficiency of using a precast concrete system is around 20% compared to that of conventional systems [5]. Precast concrete systems are also one of the construction systems which satisfy the 'green construction’ rules. Green construction is a part of the trend for sustainable construction that has been developing to face the issue of global warming [4].

On the other hand, precast concrete systems still face some technical problems, e.g. the behaviour of the connection and the ease of implementation of these joints. Sometimes, the connection is satisfactory, but is too complicated a detail or requires a high degree of precision, which causes difficulties in implementation. Furthermore, it can extend the construction time and increase construction costs.

Many types of connection have been developed, particularly beam to column connections in moment resisting frames, however, they are often partially bolted/welded/pre-stressed/cast-in-place (CIP) connection. Generally, though, they are categorized as either dry connection or wet connection, or strong connection and ductile connection [6].

Each type of connection has its own advantages and disadvantages. Welded connections, even though they satisfy strength and stiffness, can result in excessive heat, which can damage or cause cracks in adjacent precast concrete [7]. Furthermore, the implementation of this connection needs skilled labourers who can guarantee the quality of welding in the connection [8].

Bolted connections seem to be the easiest method to connect the precast elements on site. Unfortunately, these require a high degree of precision in placing the channels or steel plates before casting the precast elements. This is not easy to be done due to the sliding risk [8]. Another study anticipated this sliding risk by providing sufficient tolerance in the bolt holes. However, the hole tolerances caused a loss of initial stiffness in the connection [9].

The cast-in-place (CIP) concrete systems are more monolithic connections and are recommended for seismic resistant buildings [1]. It also provides more tolerance in precast connections. Unfortunately, it takes longer to construct as the concrete has to gain its strength; and it needs additional formwork and scaffoldings on site. All of these will lead to an increase in cost and time for construction.

The new connection developed in this paper was designed as a ductile connection. It combines a precast beam, corbel, interlocking bars and cast-in-place concrete. The precast beam is a combination of a solid beam and a U-beam. The interlocking bars were designed as flexural reinforcement which can withstand both hogging and sagging moments in the beam. The precast column is categorized as a one-piece multi-story column construction [10] which intends to accelerate the erection work for building the precast concrete column [11]. When constructing, the precast beams are put on the corbel, and then the interlocking bars are placed through the joint in the precast column and inside the U-beam. The concrete is then cast monolithically in the joint core and connection region.
The new connection offers some advantages. Particularly, the problem regarding the high precision which is needed for assembling the existing precast elements – this is minimized in this case. The use of the partial U-beam negates the use of formwork for casting the connection region. In addition, it lowers the volume of cast-in-place concrete. The number of scaffoldings can be minimized due to the existence of the corbel to support the precast beam. This new connection also avoids the use of welding, bolts and the pre-stressing process which therefore leads to a reduced need skilled labour and a reduction in the construction time. Overall, this new connection can be expected to offer a more economical and practical method.

The aim of this study is to develop a new ductile connection for a precast concrete moment-resisting frame. An experimental study of beam-column connections tested under static-monotonic and quasi-static loading was performed. The behaviour of the connection was evaluated based on the acceptance criteria in ACI 374.1-05 [12].

2 EXPERIMENTAL WORK

2.1 Detail of the Test Unit

The test specimen represents the exterior beam-to-column joint of a moment resisting frame. The length of the members was determined by the contra-flexure points resulting from a computer analysis. The column was designed to be stronger than the beam to meet the requirement of the strong column-weak beam principle, which is used imperative for seismic resistance structure. The reinforcement detail is presented in Figure 1.

The dimension of the precast column is 300 mm x 300 mm; the height is 2000 mm. The 12 reinforcing bars of 16 mm-diameter are used as longitudinal reinforcement; the reinforcement ratio is 2.67%. This meets ACI provisions [15] which states that the reinforcement ratio of the longitudinal steel bar in a column should be between 1% to 6%. The 8 mm-diameter steel shear link was spaced at 100 mm centres along the height of the column. There is a gap at mid-height of the precast column which is used for placing the interlocking bars. The height of the gap is 300 mm and is equal to the beam depth. The precast column has a corbel which is used for seating the precast U-beam.

The precast beam consists of two parts, i.e. the precast U-partial beam and the cast-in-place (CIP) reinforced concrete beam core. The outer dimension of the precast U-beam is 250 mm x 300 mm, and 1250 mm in length. The beam core is 150 mm x 250 mm in dimension, with a length of 800 mm. The beam core was cast monolithically with the joint core. The longitudinal bars of the beam core consist of 4 interlocking bars of 12 mm diameter, which act as both negative and positive moment reinforcement.

2.2 Test Setup and Instrumentation

The test setup used in this study is explained as following; both the column ends were restrained by steel plates which were bolted on to the test rig, while the beam end is free. The load was applied vertically to the tip of beam. No vertical axial load was applied to the top of the column since this tends to enhance the joint shear strength; hence, this is a worst loading
case scenario [13]. The static-monotonic loading was applied to the specimen P1 until failure. While a quasi-static loading was subjected to specimen P2 using the displacement control at 3, 8, 12, 18, 24, 36, 48, and 60 mm, as shown in Figure 2.

![Figure 1: Details of P1 and P2](image-url)
2.3 Material Properties

The precast elements were cast using normal concrete with a compressive strength of 30 MPa. The cast-in-place (CIP) were cast with higher compressive strength concrete. A maximum size of 10mm coarse aggregate was used to ensure that the fresh concrete filled the congested section. The concrete slump was between 75 and 125 mm.

**Table 1** shows the compressive strengths of the concrete. The compressive strength was obtained from 100 mm x 100 mm cubes, where the concrete cylinder strength $f_{c'}$ was taken as 80% of the cube strength $f_{cu}$ [14]. Yield strength of the steel reinforcing bars was 500 MPa; Standard Deviation was 30 MPa.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average cube strength $f_{cu}$ ($f_{c'}$)* N/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Precast concrete</td>
</tr>
<tr>
<td>P1</td>
<td>38.07 (30.46)</td>
</tr>
<tr>
<td>P2</td>
<td>40.95 (32.76)</td>
</tr>
</tbody>
</table>

*) Concrete cylinder strength was taken from 80% of the cube strength [14].

**Table 2**: Experiment results of P1

<table>
<thead>
<tr>
<th>Items</th>
<th>Load (kN)</th>
<th>Deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First-crack</td>
<td>15.00</td>
<td>2.0</td>
</tr>
<tr>
<td>Yield</td>
<td>51.88</td>
<td>16.0</td>
</tr>
<tr>
<td>Maximum</td>
<td>59.80</td>
<td>23.5</td>
</tr>
<tr>
<td>Ultimate</td>
<td></td>
<td>46.5</td>
</tr>
</tbody>
</table>

![Figure 2: Load history for the reversed cyclic load test used for specimen P2](image-url)
3 TEST RESULT OF P1

3.1 Failure Mode

Cracking was observed throughout the test. Figure 3 presents the crack-pattern of the precast concrete beam-column joint specimen P1 under negative bending moment (load applied to beam vertically downward).

The first crack appeared on the base of the beam adjacent to the column, in line with the tip of the corbel, where the maximum moment occurs. The first crack appeared on the surface on the beam core at the load level of 15 kN; the crack-width was 0.05 mm. This was followed by a delamination crack on the top surface of the beam core and the wall of the precast U-beam at the same load level. The next cracks again occurred on the beam core at a distance of 150 mm and 300 mm away from the column. In general, all cracks could be categorised as typical flexural cracks which were distributed along the connection region (cast-in-place) of the beam. The flexural cracks that occurred on the top surface of the beam were distributed at a distance of $2 \times$ the depth of beam (h) from the column face, which means $2 \times 300 \text{ mm} = 600 \text{ mm}$, which is the plastic hinge length stated in ACI Code.

The wall of the precast U-beam developed cracks at a load level of 20 kN. The cracks propagated vertically, perpendicular to the beam axis, and were an extension of the crack seen earlier in the beam core.

The longitudinal bars of the U-beam started to slip at a fairly high load level, i.e. 55 kN. This was indicated by a horizontal crack in the bottom of the U-beam which extended from the support until 200 mm from the corbel. That means that the beam core and U-beam acted as a composite beam and that there was therefore good bond between the walls of the U-beam and beam cores very well.

No significant cracks occurred in the joint core. A small delamination crack at the gap between the joint core and the upper precast concrete appeared at a load level of 25 kN. The crack stopped at the middle of the column width.

The final gap-width between the precast beam and the precast column at the end of the test was 8 mm. While, no crack occurred on the corbel.

3.2 Load-Displacement

Figure 5 shows the load-displacement relationship of the beam tip. From the curve, an attempt to estimate the first crack, yield, and ultimate load of the joint can be made. All values are presented in Table 2.

From the load-deflection graph, when slope of the curve changes slightly, the first-crack appears to be at 15 kN. This corresponds well with the load at which the first crack was visible during the test.

Although the yield condition is not clear either physically or from the graph several theories have been proposed by researchers to determine the yielding point from a load-deflection relationship [19-21]. By using the theory that the yield deflection is determined by taking the secant stiffness at 75% of the ultimate lateral load ($H_u$) of the real system [15], then the yield deflection can be determined and is presented in Table 2. The ultimate condition,
however, is very clear and the peak load, the deflection at the peak load and the maximum deflection can be determined easily.

4 TEST RESULT OF SPECIMEN P2

4.1 Failure Mode

Figure 4 shows the crack-pattern of specimen P2. The first crack occurred on the top surface of the beam core at a load of 15 kN, during the first cycle of negative loading (at the displacement level of 3 mm). The new cracks on the top surface beam core continued to develop at displacement levels of 8 mm, 18 mm, 24 mm, and 36 mm in negative loading along the connection region. The cracks extended to the wall of the U-beam at displacement level of 8 mm until 36 mm in both negative and positive loading directions.

Cracks in the joint-core initially appeared during the first cycle of displacement level of 8 mm. Then, the cracks extended forming an “X” crack in the joint core; the propagation of cracks stopped at the displacement level of 36 mm (DR = 3.5%). A thin delamination crack in the gap between the joint core (CIP-concrete) and the precast concrete (top and bottom part) occurred at the displacement level of 18 mm.

After the displacement level of 36 mm (DR = 3.5%), no more cracks developed in the joint core; the cracks were concentrated in the beam adjacent to the column. At the displacement level of 60 mm, the opening between the U-beam and the column was 15 mm and the part of the U-beam supported by the corbel was crushed.

Overall, P2 experienced a ductile failure with the plastic hinge forming in the beam.

4.2 Load – Displacement

Figure 6 illustrates the beam tip load-deflection for specimen P2 (the quasi-static load history is given in Figure 2). Two full cycles were applied at every level of displacement.

In order to understand the performance of this new connection when subjected to quasi-static loading, the results of P2 were evaluated based on ACI 374.1-05 (Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary), as presented in Table 3.

The hysteresis loop appeared stable from the beginning of test until the last cycle at the deflection level of 60 mm. There was no significant strength degradation after the peak load. All the maximum loads at each deflection increment were more than 75% of the peak load. The connection fulfilled the requirements stated in ACI 374.1-05, that the peak load of the last cycle of the drift ratio of 3.5% should be not less than 0.75 of the peak load.
Figure 3: Crack-pattern of specimen P1

Figure 4: Crack-pattern of specimen P2
Figure 5: Load – displacement relationship for specimen P1

Figure 6: Load – displacement relationship for specimen P2
The ACI 374.1-05 requires that the relative energy dissipation ratio shall not be less than 0.125 at the last cycle at the drift ratio of 3.5%. The definition of relative energy dissipation ratio ($\beta$) according this code is the ratio of the area of the hysteresis loop to the area of a circumscribed parallelogram defined by the initial stiffness during the first cycle and the peak resistance, as presented in Figure 7. As shown in Table 3 specimen P2 satisfied this requirement.

The secant stiffness is evaluated at the peak-to-peak stiffness of the load-displacement curves. The increasing the displacement level caused the decreasing of the stiffness of P2 due to the cracks formation and plastification of material. ACI 374.1-05 requires the secant stiffness at drift ratio of 3.5% (from positive to negative loading) should not be less than 0.05 times of the initial stiffness. As shown in Table 3, P2 satisfied this requirement. Even at the displacement level of 60 mm (DR = 5.83%) the stiffness was still over 0.05 times the initial stiffness.

Table 3: Comparison between test results of P2 and Acceptance Criteria ACI 374.1-05

<table>
<thead>
<tr>
<th>Items</th>
<th>Specimen P2</th>
<th>Acceptance Criteria*</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{\text{2nd}}/P_{\text{max}}$ Negative loading</td>
<td>0.84</td>
<td>$\geq 0.75$</td>
</tr>
<tr>
<td>Positive loading</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.23785</td>
<td>$\geq 0.125$</td>
</tr>
<tr>
<td>$K_{0.035}/K$</td>
<td>0.2863</td>
<td>$\geq 0.05$</td>
</tr>
<tr>
<td>$K_{0.035}/K'$</td>
<td>0.167</td>
<td>$\geq 0.05$</td>
</tr>
</tbody>
</table>

*) ACI 374.1-05  
$\beta$= relative energy dissipation ratio  
$K$ and $K'$ = initial stiffness for positive and negative loading for first cycle.  
$K_{0.035}$ = secant stiffness at drift ratio of 3.5% .

Figure 7: Relative energy dissipation ratio [12]
5 CONCLUSIONS

1) This type of beam-column connection performed well under both static-monotonic and quasi-static loading. Both specimens exhibited a flexural failure mode and behaved monolithically until failure.

2) With reference to the structural behaviour under static-monotonic loading (P1),
   - The first crack occurred on the top surface of the beam core at a load of 15 kN.
   - The cracks were typically flexural cracks, which occurred within the plastic hinge length of the beam.
   - There was no significant crack in the joint core.
   - The maximum deflection was 46.5 mm. With a yield-deflection of 16 mm, the displacement ductility was 46.5/16 = 2.9.

3) With reference to the structural behaviour under quasi-static loading (P2)
   - The first crack occurred on the top surface of the beam core at a load of 15 kN during the first cycle of loading.
   - The cracks were typically flexural cracks, which occurred within the connection region and in the wall of the U-precast beam.
   - An ‘X’crack formed in the joint core.
   - The plastic hinge was formed in the beam adjacent to the column face.
   - The connection met the acceptance criteria in ACI 374.1-05.

4) The interlocking bars connecting the joint core and the beam core can act as flexural reinforcement for the beam core, under both static and cyclic loading. The development length of the interlocking bars, i.e. 800 mm from the face of the column, is considered necessary to distribute the internal forces without bond slip occurring at the beginning of loading; this allows the interlocking bars to develop their true tensile strength through to yield.

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REFERENCES


CEMENT BASED FACADES FOR MID-RISE COMMERCIAL SUSTAINABLE AND RESILIENT BUILDINGS
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Key words: Cement based materials, Facade, Sustainability, Resilience

Abstract. The social, economic, and environmental impacts associated with natural hazards can threaten the sustainability of the building environment. Architects and engineers must consider the reconstruction impacts associated with unknown future extreme events that may occur during a building's lifetime jointly with initial construction impacts in order to design resilient and sustainable buildings.

Due to its external location on the building, the enclosure is the constructive system most exposed to natural hazards, as wind or flood, and has a main role on the global building performance regarding energy efficiency and vapor and air interchange. Hence, sustainability and resilience of buildings depend on the short and long term performance of the enclosure.

To evaluate the cement based facades (CBF) solutions for mid-rise office buildings, a study of the functional performance level, sustainability and resilience against coastal and seismic hazards was carried out with data and examples taken from a literature review. CBF were categorized considering the materials, the reinforcement and application (cast in place, prefabricated, multi-layered, etc.), self-weight, functional/control layers’ placement and interdependences with the structure. This work was part of a NSF Project RSB: Performance-based Decision Support System for Resilient and Sustainable Multi-Hazard Building Design (Award number 1455466).

CBF solutions must also be evaluated regarding their performance level (baseline, over-code and high performance) during normal operational conditions and amid climatic and natural hazards. Sustainability analysis takes into account the manufacturing and construction impacts, the operational costs in normal conditions, durability aspects and the end of life conditions. Resilience assessment considers the effect of multiple coastal and seismic hazards, expected damages and recovery measures. A vulnerability analysis of the CBF types’
expected damages, levels of damage regarding the baseline performance of functional layers and overall functionality loss is also necessary. The functional recovery considers the inspections and techniques required to evaluate damage, the repair and replacement measures required and the time until baseline operational performance is reached. The conclusions highlighted the advantages and the weaknesses of CBF that should be addressed to improve sustainability and resilience that could help early building design decisions.

1 INTRODUCTION

Sustainability and Resilience are two transversal strategies that nowadays have a deep impact on all the activities that take place in our society. Sustainability deals with the resources and environmental effects involved in the production and use of everyday goods while resilience is related to the ability of the community for adapting to unexpected and many times traumatic effects of extreme events. Accordingly, both concepts must be incorporated to any decision of the administration, the community and the production agents.

Mid-rise office buildings are the most common type for administrative and productive centers and, therefore, are in the core of resilient communities. Their performance is a consequence of the building systems and design and the Soil-Foundation-Structure-Enclosure (SFSE) interactions [1]. Among them, the enclosure (facades and roof) has a main role in sustainability issues related to energy efficiency due to its external location and its insulation function. Furthermore, the enclosure is the system of the building more exposed to climatic hazards and the level of damage on the multiple functions that the enclosure performs can seriously compromise the building’s functionality. The design selection must therefore balance multiple competing interests [2, 3].

Cement based facades (CBF) are considered to have a good performance and to be durable, sustainable and resilient due to its monolithic, continuity and mechanical capacity derived from the possibility of reinforcement. Many studies can be found in the literature highlighting the improved performance regarding to some low cost/low weight residential applications, as wood frames [4], that also show a very deficient performance under severe events and environmental hazards. However, those studies focus mainly on energy efficiency aspects and there is a lack of analyses that integrate sustainability, resilience and performance level of CBF facades.

The aim of this study is to define a framework for evaluating cement based façade solutions for mid-rise office buildings considering their functional performance level, sustainability and resilience against coastal and seismic hazards with data and examples taken from a literature review.

2 CEMENT BASED FACADES

Cement is a chemical binder and can be used to produce many different mixtures with diverse physical and mechanical properties. Conventional facades are composed by several layers that can constitute one or more leaves. The materials used and the layers arrangement defines the façade performance [5]. Then, the first approach considers the cement based materials and systems for façade. The layer arrangement is then analyzed, identifying the control layers that constitute a façade and the control functions developed. In a third step, the mechanical performance of the system and the structural design is considered.
2.1 Cement based materials and systems for facades

Cement based materials (CBM) are composed by cement and many other materials that can achieve very different performance levels depending on the composition and the production procedure. They can be shaped in the fresh state at room temperature, which makes them the most spread for almost any application in buildings. CBM can be used directly for on-site applications and can also be used for manufacturing preformed or precast elements. CBM can also be reinforced, overcoming their low tensile capacity and ductility. Figure 1 classifies the use of CBM in the facades [6]. The multiple combinations can produce high performance materials designed according to the specific application [7]. CBM have also been described to play a role on the interior air environment [8].

Regarding the specific application, CBF includes the use of wall systems (concrete masonry, insulating concrete forms, precast, removable forms, tilt-up, and autoclaved aerated concrete) and exterior finish products (stucco, concrete brick, fiber-cement siding, manufactured stone) [9].

![Figure 1: Cement based materials in the Facade, from [Thomas Herzog]]

2.2 Functional and Control Layers’ setting

From a functional point of view, the façade has to fulfill several requirements. Usually, each function is covered by a specific layer, as described in Figure 2. On one hand, it has to guarantee a basic mechanical capacity (Structural layer, S) to transfer the external loads to the structure. Besides, the façade acts as a filter between the interior and the exterior of the building. Accordingly, the façade has to control the heat (Thermal layer, T) and water transfer (Moisture, M, and Vapor, V), guaranteeing the air tightness (Air, A). According to this basic code, any CBF’s functional profile can be described. Figure 2 describes some functional configurations of conventional façade systems. When all the layers are incorporated in one leaf, the façade works monolithically. It’s the typical configuration of precast concrete sandwich panels. Other solutions are composed by different leaves which correspond to a layered configuration during the construction process.
2.3 Mechanical and structural requirements

From a mechanical point of view, the façade has to transfer the wind loads to the structure. Accordingly, at least one of the façade layers has to have flexural capacity and the ability to transfer those loads to the beams and floor slabs. In conventional CBF, a CBM wall is used as a structural façade layer.

Depending on the structural performance of the façade, another classification can be defined. If the façade does not participate on structural functions, it can be self-supporting or nonbearing. In this last case, the structural layer can be directly supported on a beam or slab and the façade-structure interaction depends on the friction-adherence. It can also be suspended and the interaction will hinge on the façade connections. If the façade takes part in the structure, it can be as a load bearing element or collaborating against horizontal loads, as advanced connections or collaborating joints [10].

3 PERFORMANCE BASED EVALUATION

The evaluation of the façade based on its performance is the key stone for the design of new solutions that can fulfill the actual and future requirements. The functional layers’ configuration, jointly with the materials selection and the façade-structure interaction would be assessed according to certain performance parameters. Those parameters can be analyzed based on performance functions that allows optimization process, energy efficiency and environmental analysis or damage/recovery functions.

Three aspects must be identified for each case: performance level expected, period of time considered and operation conditions.
3.1 Performance level

As a general approach, three levels of performance for facades are usually considered:

- Baseline performance: corresponds to the code requirements about safety (structural), economic and environmental (hygro-thermal, Solar-illumination) and habitability (Acoustic, vapor/air/water tightness).
- Improved/enhanced performance: fulfill over-code requirements defined by the owners, the designer, the insurance or the community [5].
- High performance: integrates and optimizes on a life-cycle basis all major high-performance attributes, including energy conservation, environment, safety, security, durability, accessibility, cost benefit, productivity, sustainability, functionality, and operational considerations [11].

Improved/enhanced and High performance are usually related to energy efficiency and environmental issues, evaluated by Life Cycle Analysis. The increase of thermal efficiency and the use of renewable energies have proved to be an effective way to reach a net-zero energy enclosure. However, the recent environmental concerns and the natural and man-made hazards [12] led to an upgraded concept based on sustainability and resilience. High performance CBF can be composed with fiber reinforced lightweight CBM, precast multilayered panels with advanced connections and collaborating joints.

3.2 Short vs. Long term performance

Performance during time is another issue that should be addressed. The long term effect on the enclosure materials’ performance, due to their aging, durability and maintenance has to be included in the equation. CBF are a good solution as CBM are durable and chemically stable. However, long term deformations and cracking can compromise CBF.

3.3 Normal Operation and Extreme events

Performance evaluation is usually carried out for normal operation conditions and do not consider the occurrence of extreme events that could compromise the actual performance level. However, the long term performance evaluation has to take also into account the effect of natural events expected according to realistic risk assessment. This unusual scenario must be considered when a high performance level is required. Accordingly, a performance based evaluation has to be carried out considering the damage or misfunction of some leaf or layer of the façade. The results of this evaluation can be used to determine the resilience level of a building depending on the remaining performance capacity.

4 SUSTAINABILITY

Sustainable design of building enclosures can be measured considering many different environmental and efficiency parameters [13]. CO₂ emissions [14], energy efficiency, footprint impact and durability are among those parameters. The evaluation takes into account the specific climatic conditions and the performance level expected under operation. Some evaluation tools based on life cycle analysis have been implemented for building enclosures, as LCA, LCEA or LCC [13]. It is generally accepted that the energy impact of materials manufacturing reaches 30 %, the transportation, construction and demolition/recycling can be
estimated as 20% and the operational and maintenance accounts for about 50%. Accordingly, the material selection affects the sustainability parameters in different ways. When CBF are considered, the passive energy effect of CBM can be summarized both on its thermal insulation capacity and the thermal inertia [15]. The use of advanced lightweight CBM combined with some engineered insulation materials can be a good option for high performance façade designs [16]. On the other hand, the low amount of cement required for the manufacturing of advanced CBM also produce good sustainable evaluations [17]. As a general rule, high performance CBF would achieve large passive energy saving values. The long term performance of CBF is considered to be another strength that can also be increased with an effective joint design [10].

5 RESILIENCE: COASTAL AND SEISMIC HAZARDS

The effect of the natural events on the façade operation thereafter will depend on the level of damage and the layer affected. In this paper, the most common coastal and seismic events are considered, as shown in Figure 3. Each event is characterized by the specific hazards that can produce on the enclosure and, specifically on the façade. The first aspect to point out is that any event could produce multiple hazards altogether. The hazards would produce diverse types of damages on the façade and the functional/control layers would also be different.

![Figure 3](image-url): Coastal and seismic hazards and expected damages.
5.1 Multiple Hazards

In the case of an extreme natural event, the façade is submitted to multiple hazards related to wind, water in the form of rain, flood or waves or shaking due to earthquakes. The effect that each hazard produces on the façade is different (Figure 3). Wind will produce pressure and suction on the external layer and the impact of debris. Rain will wet the external layer that would increase self-weight. Flood will produce pressure on the lower part of the façade, accumulations of water and the subsequent bio-contamination [18]. The effect of a wave would imply a massive impact on the lower stories and an earthquake would mobilize the interaction elements between the façade and the structure [19].

5.2 Damage functions

The type and level of damage on the enclosure under an extreme event depend on the specific hazard, the constitutive materials and the façade design. Figure 3 describes the expected damage associated to each coastal and seismic hazard. The last column of the graph enumerates the layers that could be affected in a conventional façade design. This initial evaluation allows the definition of damage functions and a sensitivity/vulnerability analysis can be performed. The result of such analyses would identify the expected damaged façade layers and the level of damage regarding a baseline performance (functional layers damaged and functionality loss). In order to minimize the loss of functionality two approaches can be implemented on the CBF designs. On one side, the use of high performance designs with redundant layers can save a baseline performance after an extreme event. On the other, some design solutions can be incorporated to conventional façades for damage mitigation. The use of advanced connections and collaborating joints can improve structural performance of the system, taking advantage of the mechanical capacity of reinforced CMB. The use of a thin external shield fiber reinforced CBM can protect the internal functional layers, absorbing external impacts and moisture penetration and, therefore, limiting the depth of the damage in the facade.

5.3 Recovery Functions

The damage evaluation must be completed by a recovery analysis, which would be directly related to the level of damage on the enclosure. However, in many cases the level of damage in the internal layers of an enclosure cannot be directly observed and functional evaluation must be performed to identify the extent of the damage. Level of damage Inspections and tests would be required, which would take time and have costs that must be considered. Once the damage is identified, the repair solution can be selected and effectively carried out. In some cases, the immediacy time of intervention required can be extremely important to avoid further damages -i.e. drying and cleaning after a flood [20]. Summarizing, apart from the repair costs, the inspection and diagnosis costs and the total time operational/baseline performance is reached have to be considered in a resilience evaluation. In the case of CBF systems, the more precast the system, the easier and faster would be the reparation and the lower the recovery costs.
6 CONCLUSIONS

- An effective design of a cement based façade (CBF) must consider the material, the building system, the façade-structure interaction, the performance level, the climate conditions and the exposure degree to extreme hazards. Accordingly, a specific analysis has to be managed for each combination.

- Advanced Cement based materials (ACBM) can adequate their physical and mechanical performance to the building system requirements.

- CBF systems must combine strength, thermal inertia and self-weight to satisfy sustainable and resilient standards. Redundant multilayered configurations jointly with advanced connections and collaborating joints are promising solutions to reduce damage due to coastal and seismic hazards.

- High performance facades are more sustainable and energy effective, although require a larger initial investment. Furthermore, high performance CBF have a larger capacity to remain operational after an extreme event.

- The use of high performance CBF for mid-rise office buildings, which are the most common building types for administrative and productive centers, would increase the resilience of the communities.

- The technical and economical evaluation has to consider the initial investment jointly with the maintenance costs and the direct and indirect impacts of damage under extreme events and recovery at least to a baseline level.

REFERENCES


DURABILITY OF CONCRETE EXPOSED TO SEA WATER AT EARLY AGE: FLOATING DOCK METHOD FOR CONSTRUCTION OF CAISSONS

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Key words: Caissons, concrete, durability, maritime environment, chloride

Abstract. ACCIONA Infrastructure, a Spanish contractor, uses the caisson method for the construction of breakwater structures. One of its floating docks, Kugira, is one of the largest of its kind in the world and it can produce concrete caissons measuring 70 m long by 36 m wide and 35 m high. Concrete caissons made in floating docks are gradually immersed in sea water as they are built; thus, the set concrete is exposed to sea water at a very early stage, within 18 to 48 hours after casting in sliding formwork. In order to study the durability of slag concrete using this construction method, the properties of concrete exposed to sea water at a very young age have been tested. Specimens were exposed to artificial sea water by the ponding method at different ages (16h, 1, 2, 3, 7 and 28 days) for a period of 6, 18 and 36 (analysis pending) months. The following properties were measured and compared with unexposed specimens: compressive strength, water and oxygen permeability, pore size distribution, migration coefficient DNT492 and chloride profiles. After 6 and 18 months ponding, lower chloride contents were measured for the specimens exposed to sea water after 28 days curing compared to those exposed at early ages. However, this difference significantly decreases within a small depth from the exposed surface. Also the migration coefficient DNT492 decreases significantly with the increase of the age of the concrete. With regards to water and oxygen permeability, no significant differences were found. In this paper, a summary of all these works is presented.

1 THE CONSTRUCTION METHOD

In the floating dock construction method, concrete is cured (mainly) under sea water; the caisson is progressively submerged as the caisson walls are casted and slide through the formwork. A typical caisson made with the Kugira floating dock (owned by ACCIONA Infrastructure) is 66.85 m in length, 34.60 m in width and up to 34 m in height. It is divided in 8 sections, in a total of 32 inner cells, weighing more than 12000 metric tons. More than 5000 m³ of concrete and 500 tons of steel are used for the construction of each caisson. The construction method can be described in the following main steps:
• With the pontoon fully out of the water, the deck is cleaned and the caisson structure is stacked.
• The caisson base moulds are assembled, the reinforcement of the base slab and wall fixings are placed and the bottom slab is casted.
• The sliding mould and the steel truss are lowered.
• As the sliding of the caisson shaft starts, the base slab mould is removed and concreting of the caisson shaft starts. The concrete is poured in successive 30 cm thick layers at an average rate of 2 m every 12 hours.
• The floating dock is stage ballasted by flooding the ballast tanks to ensure that both the caisson and the dock are sunk in a balanced and proportionated way.
• When sliding has reached 8 m, several 300 mm tubes are put in place to communicate between the different shaft cells; this way the caisson is divided in 8 sectors so that when ballast is introduced the caisson will remain horizontal.
• At 12 m, 8 more tubes are placed in the exterior screens (1 per sector) to be able to locate the valves needed for anchoring the caisson.
• At 17 m, the towing hooks are placed to allow the caisson to be towed out to sea.
• Once the sliding is finished, the mould is removed and the caisson is ballasted with sea water.
• The floating dock is ballasted until the caisson rises of the deck and floats.
• Finally, the caisson is towed to the quay wall and the valves are open to fill the cells with sea water until the hole block is settle in the sea bed.

Figure 1: Page layout

2 APPROACH

2.1 Objectives

The objectives of this study were:

• To investigate the influence of early exposure to artificial sea water on the concrete’s mechanical and transport properties.
• To determine the chloride transport properties in concrete by accelerated methods
• To analyze the influence of the use of nanosilica in the improvement of the strength and durability of concrete at real scale.

2.2 Test Program

All materials and the mix design are representative of those usually used in the construction process. Tables 1 and 2 show the composition and the sample matrix, respectively; the concrete follows the standard EN 206-1 (2000), fulfilling the minimum requirements for the exposure class XS3.

Table 1: Concrete Mix Proportions

<table>
<thead>
<tr>
<th>W/C</th>
<th>CEM III/A 42.5 N/SR (kg/m³)</th>
<th>Sand 0-4 (kg/m³)</th>
<th>Aggregate 4-16 (kg/m³)</th>
<th>Glenium CS03 SCC (% of CEM)</th>
<th>Pozzolith 488N (% of CEM)</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.45</td>
<td>350</td>
<td>918</td>
<td>1048</td>
<td>2.0</td>
<td>0.4</td>
<td>200</td>
</tr>
</tbody>
</table>

Specimens were tested at the ages of 16 h, 1, 2, 3, 7 and 28 d; these ages have been decided according to the details of the construction methods. The following curing conditions were employed:
• 16 h and 1 d: cured in the form;
• 2, 3 and 7 d: cured in the form for 1d and from then on under water;
• 28 d: cured in the form for 1 d, then 6 d under water and from then on stored at 20°C and 65 % RH.

At the age of exposure (16 h, 1, 2, 3, 7 and 28 d), the specimens were immersed in artificial sea water, the composition of which is given in Table 3.

Table 2: Sample Matrix

<table>
<thead>
<tr>
<th>Number/Type of samples</th>
<th>0.050mm</th>
<th>0.100mm</th>
<th>0.150mm</th>
<th>0.250mm</th>
<th>0.500mm</th>
<th>16h</th>
<th>1d</th>
<th>2d</th>
<th>3d</th>
<th>7d</th>
<th>28d</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (before exposure)</td>
<td>X</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Compressive strength (after 6 months immersion)</td>
<td>X</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1 profile (after 6 month ponding - cast surface)</td>
<td>X</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1 profile (after 18 month ponding - cast surface)</td>
<td></td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1 profile (after 36 month ponding - cast surface)</td>
<td></td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mercury Intrusion Porosimetry</td>
<td>X</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Microscopy (SEM)</td>
<td>X</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid chloride migration</td>
<td>X</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*2 extra specimens were tested at the age of 180 days
*2 extra cylinders (4 specimens) were tested at the age of 180 days
### Table 3: Composition of artificial sea-water

<table>
<thead>
<tr>
<th>Composition</th>
<th>Artificial sea-water</th>
</tr>
</thead>
<tbody>
<tr>
<td>(in %)</td>
<td>(in g/l) for 1001 (g)</td>
</tr>
<tr>
<td>Total salinity</td>
<td>35</td>
</tr>
<tr>
<td>Chloride</td>
<td>19.345</td>
</tr>
<tr>
<td>Sodium</td>
<td>10.752</td>
</tr>
<tr>
<td>Sulfate</td>
<td>2.701</td>
</tr>
<tr>
<td>Magnesium</td>
<td>1.205</td>
</tr>
<tr>
<td>Calcium</td>
<td>0.416</td>
</tr>
<tr>
<td>Potassium</td>
<td>0.390</td>
</tr>
</tbody>
</table>

2.2.1 Ponding tests and artificial sea water exposure

In these tests, the following properties were examined:

- Compressive strength before exposure to artificial sea water, at the ages of 16 h, 1, 2, 3, 7, 28 and 180 d, according to EN 12390-3 (2009);
- Compressive strength after exposure for 6 months in artificial sea water, for specimens exposed at the ages of 16 h, 1, 2, 3, 7 and 28 d, according to EN 12390-3 (2009);
- Pore size distribution on paste samples, at the ages of 16 h, 1, 2, 3, 7 and 28 d by mercury intrusion porosimetry between 0.004 and 100 μm. This pore size range is important for all transport phenomena including capillary water transport and chloride diffusion. At the selected ages, the samples were immersed in 2-propanol (alcohol exchange) for 3 days in order to stop the hydration, vacuum dried and stored in a desiccator containing nitrogen to avoid carbonation.
- Microstructural analysis on polished cross sections of concrete, at the ages of 16 h, 1, 2, 3, 7 and 28 d by scanning electron microscopy (SEM). At the selected ages, hydration was stopped by means of alcohol exchange. After dried, the samples were impregnated with epoxy resin and finely polished using petroleum as grinding liquid.
- Chloride profiles on concrete samples, tested according to ASTM C1543-10a (2010) (ponding method). At the ages of 16 h, 1, 2, 3, 7 and 28 d the samples were exposed to chloride solution (3% NaCl) during 6, 18 and 36 (analysis pending) months. At the end of the exposure period, the samples were ground on a lathe in the following steps: 0-2, 2-4, 4-6, 6-9, 9-12 and 12-15 mm for the specimens exposed during 6 months; 0-2, 2-5, 5-8, 8-12, 12-16, 16-20 and 20-25 mm for the specimens exposed during 18 and 36 (analysis pending) months. The acid soluble chloride content (total chloride content) was determined according to AASHTO T-260 (1997). Additionally, for the age of 28 d, the influence of the exposed surface was studied by comparing the chloride profiles for both cast and cut surfaces.
- XRD on powder samples of concrete, after 18 months ponding: for the samples exposed at the ages of 1 and 28 d, grinding steps 0-2, 2-5, 5-8, 8-12, 12-16, 16-20 mm; for the samples exposed at the ages of 2, 3 and 7 d, grinding step 0-2 mm.

2.2.2 Rapid chloride penetration test

The chloride migration coefficient, based on non-steady-state experiments was measured according to NT Build 492 (1999). In this test, an external electrical field (10-60 V DC) is applied axially across a test specimen (Ø 100 x 50 mm) to accelerate the process of chloride migration. At the end of the test period (typically 24 hours), the specimen is axially split into
two half and the chloride penetration depth is measured using a colorimetric technique (silver nitrate spray). The chloride migration coefficient is then calculated from the chloride penetration depth.

3 RESULTS

The first page must contain the Title, Author(s), Affiliation(s), Key words and the Summary. The Introduction must begin immediately below, following the format of this template.

3.1 Ponding tests and artificial sea water exposure

3.1.1 Compressive strength

Figures 2 shows the results from compressive strength measured before (left) and after (right) exposure to artificial sea water. Slow hydration continues throughout 6 months and the relative strength gain from 28 to 180 days is about 6 %.

Figure 2 shows that the age of the concrete at the time of exposure does not affect the strength development. The compressive strength after 6 months exposure is 5 to 10 % lower when compared to non-exposed specimens, which is within the variation of strength testing. From the results there seems to be no visible influence of the exposure to seawater on compressive strength.

![Figure 2: Compressive strength (left) before and (right) after 6 month exposure to sea water](image)

3.1.2 Pore size distribution

In practical terms, it is the capillary pores that are of major importance for the chloride ingress since the gel pores are too small to allow significant transport of species by diffusion or capillary forces. Although macro pores have no capillary action, they can also be of importance under submerged conditions.

The pore size distribution of cement paste samples at the ages of 16 h, 1, 2, 3, 7 and 28 d was characterized by mercury intrusion porosimetry (MIP).

The objective was to compare the pore size distribution of the cement paste at different ages of hydration. The results are shown in Figures 3. It can be seen that, as the hydration proceeds, there is a refinement of the pore structure together with a significant decrease (about 50 % from 16 h to 28 d) in the total pore volume. Most significant, is the increase in the gel pores (< 10 nm) and the decrease in both smaller (10-100 nm) and larger capillary pores (0.1-1 μm).
Regardless of the exposure environment, concrete exposed at an early age will have a higher chloride diffusion coefficient since both the moisture content and the volume of capillary pores is higher; however capillary suction is relatively small. As the hydration proceeds, the volume of capillary pores decreases and with it also the diffusion coefficient.

**Figure 3**: Absolute and total pore volume distributed according to the pore size at each age

### 3.1.3 Microstructural analysis

The microstructure of the concrete samples at the ages of 16 h, 1, 2, 3, 7 and 28 d was characterized by scanning electron microscopy. SEM images in Figure 4 show an overview of the cementitious matrix for samples 16 h, 1, 7 and 28 d. Detail pictures are shown in Figures 7 to 10.

![SEM images showing microstructure of concrete samples at different ages](image-url)
The microstructure of the slag cement containing concrete is, as expected, porous at young age ≤1 d, which is visible in the SEM backscatter images in form of black to dark grey areas (Fig. 4). Up to an age of 7 days the porosity is continuously decreasing (Fig. 4) and not so much different from the 28 day sample. This confirms the results from the development of the pore size distribution as shown in Fig. 3. The reaction of the slag is not visible at 16 h, where mostly the clinker phases are dominating the hydration. Some of the capillary porosity can be attributed to the clinker reaction: C3S and C2S are dissolving faster in the pore solution as hydration products are formed, which results in open space around the grains.

After 1 d, however, the slag started to react. In this stage only the partial dissolution of slag grains were visible. Hydration products in form of hydrotalcite like phases were not observable by SEM at this stage. After 7 d reaction of slag was clearly evident. In some of the grains the formation of a new Mg containing phase was observed. This phase is generally referred to as hydrotalcite Mg6Al2(CO3)(OH)16·4(H2O), which is a main hydration product of slag with higher magnesium content and which composition and crystal structure may vary within certain limits.

After 28 d the microstructure of the concrete is denser and the hydration of slag and the formation of hydrotalcite around the slag grains are more pronounced. The hydration process of slag will not stop under water saturated conditions but will go on for several months or even years.

3.1.4 Chloride profiles

The chloride profiles after 6 and 18 months exposure (ponding method) are summarised in Figure 5. After 6 months exposure, no significant difference was found between the ages of 1 and 7 days except for the surface chloride content (CS). At early ages (1 day), the higher concrete permeability leads to distinctively higher chloride ingress, which is indicated by higher Cl concentrations in the first 5 millimetres. However, after a certain time with the refinement of the pore structure (Fig. 3) and because chloride diffusion is, in general, a slow transport process, the diffusion coefficients decrease with time and therefore almost the same chloride concentrations were analysed at depths larger than 5 mm.

In general, the chloride ingress is more severe in specimens exposed at early age (1-7 days) when compared to those exposed at the age of 28 days. The main reason is associated with the strong decrease in porosity as shown in Figure 3, leading to lower permeability. While the curing age has, in general, a positive influence in reducing the total chloride content, it can be seen that this effect is more pronounced at the outermost layer. In fact, due to the combined transport mechanisms of capillary suction and diffusion, the relative difference in the chloride content between 28 days (circle) and 1-7 days (other curves) decreases with the penetration depth, since in the case of specimens exposed at 1-7 days the only relevant transport mechanism is diffusion.

The influence of the casting surface on the chloride ingress is also shown in Figure 5 for the case of specimens exposed at the age of 28 days. It can be seen that, up to a depth of 6 mm, the chloride content is higher for cast surfaces due to the higher cement content and therefore, chloride binding capacity; however, this does not seem to influence the chloride
Based on the mixture proportions the binder content can be calculated as 14% by mass of concrete. In addition, based on the experience from Tang et al. (2013), a difference of 4% binder per mass of concrete can be assumed between the outermost layer (paste rich) and the bulk concrete. Therefore, when expressed in terms of binder, the chloride contents shown in Figure 5 varied between 2.3-5% at a depth of 0-2 mm and between 0.4-0.9% at a depth of 10-15 mm.

After 18 months, however, there is a distinction between the chloride profiles of specimens exposed at different ages. Clearly, the influence of the exposure age becomes less pronounced. With exception of the surface chloride content, specimens exposed between 2 and 7 days show lower chloride contents when compared to those of specimens exposed at the age of 28 days.

Therefore, there is no reason to assume that the early age exposure has a negative influence on the chloride penetration. An interesting feature though is shown for the specimens exposed at the age of 7 days. The total chloride content is in general the lowest, showing also the lowest surface chloride content.

**Figure 5**: Profiles of chloride ingress in concrete exposed at different ages: (left) after 6 months exposure; (right) after 18 months exposure (36 months analysis pending)

### 3.2 Rapid chloride penetration test

Figure 6 shows the non-steady-state migration coefficients obtained at different ages. At the age of 1 day, the chloride migration coefficient is relatively high. However, only one day later (2 days old), this value drops drastically to about 50% and further on by about 90% at the age of 28 days. At the age of 28 days, the DNT492 obtained is within the range of those reported in ECO-Serve (2005), i.e. between 5 to 10 x10^-12 m2/s. A direct comparison between the two results, DNT492 and DF2, is not possible since these are obtained under different experimental conditions (i.e. migration and diffusion). In the field, chloride penetration through concrete exposed in marine environments occurs mainly by combined capillary suction/diffusion in atmospheric and tidal zones and by diffusion only in submerged zone.

Since the concrete is exposed at very early ages, capillary suction is rather limited and the diffusion coefficients are much lower than the migration coefficients measured by accelerated
methods. Nevertheless, accelerated methods give a good indication as to the resistance of concrete against chloride ingress.

![Graph: Evolution of the chloride migration coefficient DNT492 as a function of the specimen age](image)

**Figure 6**: Evolution of the chloride migration coefficient DNT492 as a function of the specimen age

## 4 CONCLUSIONS

- In this study, the influence of the early age exposure to sea water on the durability properties of slag concrete, generally used by ACCIONA Infrastructure for the construction of caissons by the floating dock method, was investigated.

- For the tested concrete composition, the **compressive strength** was approximately 70 MPa at the age of 28 days. Early age exposure to chloride environment did not influence the compressive strength when compared to concrete cured for the same period under normal conditions (20 °C and 65 % RH).

- **Mercury intrusion porosimetry** measurements have shown a significant decrease (about 50 %) of the total pore volume between 16 hours and 28 days mature samples. More importantly an increase in the gel pores and a decrease in the larger capillary pores was measured. Although the porosity (both total and capillary) is higher at early ages, its influence on the durability of the concrete, namely the resistance against chloride ingress, is limited. The main reason is that, at early ages, the degree of capillary saturation is expected to be higher than at the age of 28 days and thus diffusion is the only relevant mechanism for chloride transport.

- Microstructural studies using **scanning electron microscope (SEM)** confirmed the results obtained by mercury intrusion porosimetry. SEM images showed that at very early ages (≤1d), the porosity of the cement paste is high and mainly the clinker phases react. The porosity then significantly decreases until the age of 7 days. The reaction of the slag is visible already from 1 day but more pronounced after the age of 7 days where the formation of hydrotalcite around the grains is clearly visible.

- With regards to **chloride ingress**, after 6 months ponding no difference was found between specimens exposed at the ages of 1 to 7 days; lower chloride contents were measured for the specimens exposed after 28 days curing. However, the relative difference significantly decreases within a small depth, from 0.25% at the surface to 0.10% at a depth of 3 mm. The chloride diffusion coefficient appears to remain constant already after the age of 3 days. After 18 months exposure, only specimens exposed at the age 1 day show higher chloride contents. when compared to the remaining exposure ages. Again, no clear negative influence of the exposure age on chloride penetration was found: lower chloride values were measured for specimens
exposed at the ages of 2 to 7 days when compared to those measured for specimens exposed at the age of 28 days. Except for the specimens exposed at the age of 1 day, after 18 months exposure, the chloride diffusion coefficients are much lower than those measured after 6 months exposure. It is recommended to compare the effect of the exposure age on the resistance against chloride ingress for concretes made with slag cement and ordinary Portland cement; although slag concrete is known to have a high resistance against chloride ingress due to its refined pore structure and binding capacity, sometime is required for the secondary pozzolanic reactions to take place, which may not be achieved in such short exposure times.

- The **rapid chloride migration test** provides a good indication as to the resistance of the concrete against chloride ingress. The migration coefficient decreases significantly with the age of the concrete at the time of the test. However, at the age of 28 days, DNT492 was found to be in the range of values reported in the literature. In practice however, chloride transport through concrete exposed under submerged conditions occurs by means of diffusion mechanism meaning that the difference in chloride penetration between specimens exposed at the age of 1 day and 28 days is expected to be rather small (as indicated by the results from 6 months ponding experiments).

- In summary the results indicate an only weak influence of early age exposure on the investigated concrete properties. Chloride ingress was higher in the surface zone of specimens exposed to artificial seawater at early stage but after ca. 5 mm depth the chloride levels tend towards the ones which were exposed after 28 d. Higher chloride content in surface zones in slag containing concretes with subsequent strong reduction in concentration is well known from other studies (e.g. Hong and Hooton, 1999) independent of the curing age and time of exposure. And these earlier studies showed also the lower chloride concentration and higher chloride binding capacities of slag containing concretes.

REFERENCES


ECO-MECHANICAL ANALYSIS OF TWO LIGHTWEIGHT FIBER-REINFORCED CEMENT-BASED COMPOSITES

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Key words: Traditional lightweight concrete (TLC), Rubber lightweight concrete (RLC), Fiber-reinforced concrete, Compression tests, Bending tests, Eco-mechanical analysis

Summary. The mechanical and environmental performances of two different lightweight concretes are compared in the present paper. Since an optimal compromise between structural and environmental requirements needs to be reached in building materials, to tailor such cement-based composites the so-called eco-mechanical approach is adopted.

The first mixture is a traditional lightweight concrete, containing industrial aggregates made with expanded clay (to reduce the weight) and polymeric fibers (to reach a ductile failure) in place of the traditional stone aggregates and steel rebars, respectively. The second mixture is a non-conventional lightweight cement-based composite, in which recycled rubber from end-of-life tires partially replaces the aggregates. Also in this concrete, polymeric fibers have been used as reinforcement.

Concerning the mechanical properties, the results of an experimental campaign, performed on materials (i.e., uniaxial compression tests on cylinders) and on full-scale structures (i.e., plates in three point bending), are taken into consideration. In particular, the plates have been used to retrofit the sidewalks of a famous bridge in Italy. On the other hand, the carbon footprint and the embodied energy of the two types of concrete define the environmental impact.

If the analysis is performed at the material level, considering only the compressive strength, the traditional lightweight concrete behaves better than the non-conventional composite. Conversely, the plates made with fiber-reinforced rubber concrete show better eco-mechanical performances than those made with the traditional lightweight composite reinforced with fibers.

As a result, the above-mentioned differences suggest the necessity of a proper definition of the level (i.e., material or structural) where the eco-mechanical analysis needs to be performed.

1 INTRODUCTION

According to Model Code 2010 (MC2010)[1], the density of lightweight concrete varies from 800 to 2,000 kg/m³. To reduce the mass of normal weight structural concrete, stone aggregates are substituted by cellular structured particles. Such lightweight aggregates are generally produced by heating some raw materials (e.g., shale, clay, slates, fly ashes, etc.) to incipient fusion, and then cooling them in the so-called pyroprocessing method[2].
Lightweight concrete is mainly used to reduce the mass, and consequently the dead loads and the inertial seismic actions, of both new and existing structures, and to facilitate the transportation and the placement of precast elements. In general, to justify the use of lightweight concrete, which is more expensive than normal weight concrete, a lower cost of the project and/or an improved functionality must be attained [2].

A more sustainable lightweight concrete can be tailored by using a non-conventional aggregate made with rubber from end-of-life tires [3,4]. Indeed, the density of rubber granulates (with the dimensions of grains comprised between 0.8 and 20 mm [5]) is lower than that of the stone aggregates. The use of rubber in the concrete industry is also convenient from an environmental point of view. Specifically, hundreds of millions scrap-tires are generated each year worldwide, and their landfilling is becoming unacceptable due to the rapid depleting of the sites and to the associated risks [3].

Several studies on concrete containing rubber put into evidence not only the reduction of strength, with respect to normal weight concrete, but also some interesting mechanical performances [3,4]. More precisely, vibration damping, resistance to impacts, blasts, and to external constraints (temperature, settlements, etc.) are generally improved by using rubber aggregates. However, a direct comparison between the performances of Rubber Lightweight Concrete (RLC) and those of Traditional Lightweight Concrete (TLC) cannot be found in the current literature.

For this reason, the following sections report and compare the results of an experimental campaign conducted on both TLC [6] and RLC [7] specimens. Some of them have been reinforced with short polymeric fibers, already used to substitute traditional rebars [8]. The performances of the two lightweight concretes have been measured at material level, by means of uniaxial compression tests, and on the full-scale one-way plates of a real case [9], tested in three point bending.

In addition, to perform a more comprehensive comparative analysis of the two types of lightweight concrete, also the environmental impact has been considered. In fact, a compromise between mechanical and ecological performances needs to be reached in order to tailor an optimal building material [10].

2 EXPERIMENTAL INVESTIGATION

2.1 Concrete mixtures

Two TLC mixtures, made with expanded clay aggregates, are analysed herein. As it can be observed in Table 1, the first is a reference unreinforced mixture (i.e., TLC_R), whereas a cubic meter of the second mixture (i.e., TLC_F) is reinforced with 10 kg of short polymeric fibers (diameter = 0.48 mm, length = 54 mm, elastic modulus = 5.75 GPa, and tensile strength > 620 MPa). The fibers are commercially available and are made by a mix of polymers (mainly polypropylene). Due to the selected components reported in Table 1, TLC mixtures have a density of about 1,650 kg/m^3.

Similarly, a combination of rubber aggregates and stone aggregates has been used to tailor a RLC mixture containing 12 kg/m^3 of the polymeric fibers previously described (i.e., RLC_F in Table 1). Although the density of this concrete (i.e., 1,900 kg/m^3) is higher than that of TLC, it can be considered a lightweight mixture in accordance with the definition given by MC2010 [1].
Table 1: Material components referred to 1 m$^3$ of the three lightweight concretes.

<table>
<thead>
<tr>
<th>Components</th>
<th>TLC_R</th>
<th>TLC_F</th>
<th>RLC_F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water (kg)</td>
<td>140</td>
<td>140</td>
<td>168</td>
</tr>
<tr>
<td>Cement Type II A-LL 42.5R (kg)</td>
<td>500</td>
<td>500</td>
<td>352</td>
</tr>
<tr>
<td>Stone aggregate (kg)</td>
<td>700</td>
<td>700</td>
<td>1,110</td>
</tr>
<tr>
<td>Expanded clay aggregate 3-8 mm (kg)</td>
<td>300</td>
<td>300</td>
<td>0</td>
</tr>
<tr>
<td>Rubber granulates 3-20 mm (kg)</td>
<td>0</td>
<td>0</td>
<td>238</td>
</tr>
<tr>
<td>Superplasticizer (l)</td>
<td>3.6</td>
<td>3.6</td>
<td>6</td>
</tr>
<tr>
<td>Viscosity Modifying Agent (l)</td>
<td>1.2</td>
<td>1.2</td>
<td>0</td>
</tr>
<tr>
<td>Polypropylene fibers (kg)</td>
<td>0</td>
<td>10</td>
<td>12</td>
</tr>
</tbody>
</table>

2.2 Specimens and test setup

Three cylindrical specimens, with height of 300 mm and base diameter of 150 mm (Fig.1a), were cast with the three mixtures introduced in the previous section. Such samples were subjected to uniaxial compressive loads through a MTS testing machine (maximum load capacity = 1,000 kN), by imposing a constant velocity of the stroke (i.e., 0.037 mm per minute). In addition to the average strain measured on the whole height of the specimens, two LVDTs were placed on the central part of each cylinder to evaluate the local strain on a base of 100 mm (Fig.1a).

Three 1,000 × 1,000 × 100 mm plates (see Fig.1b) were also cast by using the lightweight concrete mixtures defined in Table 1. Such plates, which do not contain any steel rebar, correspond to the full-scale structure of a simply supported sidewalk\[9\], and, accordingly, they were tested in three point bending (Fig.1b). The load $P$, distributed along the midsection through a steel beam, was applied by means of a MTS machine. A load cell of 100 kN applied the load, and two LVDTs were used to determine the midspan deflection $\delta$ (depurated by the support displacements), as the stroke increases at a constant velocity of 0.360 mm per minute.

![Figure 1](image)
3 EXPERIMENTAL RESULTS

3.1 Uniaxial compression tests

Fig. 2a-c illustrates the stress-strain (\(\sigma - \varepsilon\)) curves obtained from the compression tests on the concrete cylinders depicted in Fig. 1a.

**Figure 2**: The results of the experimental investigations: (a)-(c) The stress-strain curves obtained from the uniaxial compression tests; (d)-(f) The load-deflection curves of the three point bending tests.
During the ascending branch, the strains are measured with the two LVDTs, whereas in the softening stage the shortening deformation of the whole specimen is assumed to be coincident with the stroke of the loading machine. The main parameters of the $\sigma - \varepsilon$ curves (see Fig.3a) are collected in Table 2. They include the compressive strength $f_{lc}$ (i.e., the peak of the $\sigma - \varepsilon$ curves), the tangent modulus of elasticity $E_{lc}$ (in the origin of the $\sigma - \varepsilon$ curves), the strain at the peak of stress $\varepsilon_{lc1}$, and the plasticity number $k_{lc} = E_{lc}/E_{lc1}$ (i.e., the ratio between $E_{lc}$ and the secant modulus from the origin to the peak of stress $E_{lc1} = f_{lc}/\varepsilon_{lc1}$).

As reported in Table 2, the experimental values of $f_{lc}$ measured in RLC_F specimens are about one half of the compressive strength of TLC. This is not only due to the different water/cement ratio of the two mixtures (see Table 1), but also to the different aggregate/cement paste interaction of the two mixtures [11]. Moreover, because the average compressive strength of RLC_F is lower than 16 MPa (i.e., the average strength of the minimum concrete grade admitted by MC2010 [1], LC8), it cannot be considered a structural concrete. However, it is worth noting the large ductility of RLC, which is remarkably higher than that of TLC. This is particularly evident in Fig.4, where the normalized stress vs. compressive strain curves of both TLC_F and RLC_F mixtures are compared. To be more precise, at the same level of strain, the normalized post-peak residual stresses of RLC always exceed those of TLC.

![Figure 3: The main parameters measured in the experimental campaign: (a) Uniaxial compression tests; (b) Three point bending tests.](image)

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$f_{lc}$ (MPa)</th>
<th>$E_{lc}$ (MPa)</th>
<th>$\varepsilon_{lc1}$ (%)</th>
<th>$k_{lc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLC_R_1</td>
<td>21.34</td>
<td>12,388</td>
<td>2.1</td>
<td>1.2</td>
</tr>
<tr>
<td>TLC_R_2</td>
<td>21.29</td>
<td>18,538</td>
<td>1.6</td>
<td>1.4</td>
</tr>
<tr>
<td>TLC_R_3</td>
<td>21.90</td>
<td>14,283</td>
<td>1.8</td>
<td>1.2</td>
</tr>
<tr>
<td>TLC_F_1</td>
<td>24.04</td>
<td>16,524</td>
<td>2.0</td>
<td>1.4</td>
</tr>
<tr>
<td>TLC_F_2</td>
<td>21.91</td>
<td>22,340</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>TLC_F_3</td>
<td>22.78</td>
<td>17,738</td>
<td>2.1</td>
<td>1.6</td>
</tr>
<tr>
<td>RLC_F_1</td>
<td>9.88</td>
<td>15,692</td>
<td>1.8</td>
<td>2.9</td>
</tr>
<tr>
<td>RLC_F_2</td>
<td>11.60</td>
<td>15,684</td>
<td>2.0</td>
<td>2.7</td>
</tr>
<tr>
<td>RLC_F_3</td>
<td>11.50</td>
<td>15,446</td>
<td>1.8</td>
<td>2.4</td>
</tr>
</tbody>
</table>
3.2 Three point bending tests

The results of the bending tests on one-way plates are reported in Fig. 2d-f, where the load-midspan deflection curves \( (P-\delta) \) are depicted. In all the tests, the effective cracking load \( P_{cr^*} \), at which the softening stage of the \( P-\delta \) curve begins, is clearly evident. In the case of unreinforced plates (i.e., TLC_R in Fig. 2d), \( P_{cr^*} \) is the absolute maximum. On the other hand, the \( P-\delta \) curves of fiber-reinforced plates show two relative maxima (Fig. 2e-f). The first coincides with \( P_{cr^*} \), and the second, at onset of strain localization, corresponds to the actual ultimate load \( P_{u^*} \). Fig. 3b illustrates an ideal \( P-\delta \) curve, whereas Table 3 reports the experimental values of \( P_{cr^*} \) and \( P_{u^*} \), and of the corresponding deflections \( \delta_{cr^*} \) and \( \delta_{u^*} \).

The brittle behaviour of unreinforced TLC plates can be observed in Fig. 2d, where the \( P-\delta \) curves show a dramatic drop after the peak. On the opposite, when fibers are present in the cementitious matrixes of both the lightweight concretes, a ductile response, with \( P_{u^*} \approx P_{cr^*} \), can be observed (Fig. 2e-f). Moreover, the flexural strength of RLC (i.e., \( P_{cr^*} \)) is comparable with that of TLC and, for each plate, \( P_{cr^*} \) is remarkably higher than the maximum design load \( P_d = 8.60 \text{ kN} \). Such a value is obtained by factorizing the dead loads (equal to 2.0 kN/m\(^2\)) and the variable load (4.0 kN/m\(^2\)) with the partial safety factors \( \gamma_G = 1.3 \) and \( \gamma_Q = 1.5 \), respectively.

Table 3: The structural parameters measured with the three point bending tests on one-way plates.

<table>
<thead>
<tr>
<th>Plates</th>
<th>( P_{cr^*} ) (kN)</th>
<th>( \delta_{cr^*} ) (mm)</th>
<th>( P_{u^*} ) (kN)</th>
<th>( \delta_{u^*} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLC_R_1</td>
<td>15.08</td>
<td>0.32</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>TLC_R_2</td>
<td>16.23</td>
<td>0.21</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>TLC_R_3</td>
<td>16.72</td>
<td>0.29</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>TLC_F_1</td>
<td>22.82</td>
<td>0.59</td>
<td>24.00</td>
<td>4.30</td>
</tr>
<tr>
<td>TLC_F_2</td>
<td>21.54</td>
<td>0.61</td>
<td>23.24</td>
<td>4.92</td>
</tr>
<tr>
<td>TLC_F_3</td>
<td>20.40</td>
<td>0.47</td>
<td>21.56</td>
<td>5.23</td>
</tr>
<tr>
<td>RLC_F_1</td>
<td>18.39</td>
<td>0.40</td>
<td>16.62</td>
<td>2.98</td>
</tr>
<tr>
<td>RLC_F_2</td>
<td>22.40</td>
<td>0.45</td>
<td>21.90</td>
<td>2.69</td>
</tr>
<tr>
<td>RLC_F_3</td>
<td>20.59</td>
<td>0.57</td>
<td>19.29</td>
<td>5.25</td>
</tr>
</tbody>
</table>
To quantify the ductility of the fiber-reinforced plates, the recommendations provided by MC2010\[^1\] can be applied. Specifically, the minimum conventional reinforcement in tension (made with steel rebar) can be avoided, if one of the following ductility conditions is satisfied (see the load-deflection curve reported in Fig.5\[^1\]):

\[
\delta_u / \delta_{sls} \geq 20 \quad (1.a) \\
\delta_{peak} / \delta_{sls} \geq 5 \quad (1.b)
\]

where \(\delta_u\) = ultimate deflection related to the required maximum rotation of the plate; \(\delta_{sls}\) = deflection at service load \(P_{sls}\); and \(\delta_{peak}\) = deflection at maximum load \(P_{max}\).

For the sake of the simplicity, only Eq.(1.a), generally applied to lightly reinforced structures, is taken into account. As illustrated in Fig.5, \(\delta_u\) is also related to the load \(P_u\) (not necessarily coincident with the load \(P_u^*\) reported in Fig.3b), which in turn can be obtained from the theoretical ultimate bending moment of the plates. Such moment is easily calculated under the hypothesis of linear strain profile, in which the maximum tensile strain is \(\varepsilon_{Fu} = 2\%\).

The parabola-rectangle is assumed as the constitutive law for concrete in compression, whereas a rigid-plastic constitutive relationship, defined by the ultimate residual strength \(f_{Ftu}\), describes concrete in tension. In the absence of direct material tests, \(f_{Ftu}\) is assumed to be one third of the flexural tensile strength \(f_{ct,fl}\) experimentally evaluated. This certainly brings to an over-estimation of \(P_u\), because MC2010\[^1\] states that \(f_{Ftu}\) must be one third of \(f_{R3}\), which is the significant residual strength for ultimate conditions.

For each fiber-reinforced mixture, the values of \(P_u\), and of the associated \(\delta_u\) read on the curves of Fig.2e-f, are reported in Table 4. The same Table also collects the values of \(\delta_{sls}\), measured on the curves of Fig.2e-f at \(P_{sls} = 4.40\) kN (from the quasi-permanent combination of actions with \(\psi_2 = 0.6\)), and the ductility ratios \(\delta_u / \delta_{sls}\). For both the types of lightweight concrete, the average ductility ratio (i.e., 486 for TLC_F and 21 for RLC_F) is higher than 20, then the plates can satisfy the MC2010 requirements also without conventional reinforcement\[^1\]. As \(P_u\) is over-estimated, \(\delta_u\) and the ductility ratio \(\delta_u / \delta_{sls}\) are underestimated, and therefore the mixtures TLC_F and RLC_F can be certainly used to cast the concrete plates of Fig.1b in the absence of rebars.

![Figure 5: The typical load-deflection curve used to define the ductility requirements of fiber-reinforced concrete structures\[^1\].](image-url)
Table 4: The parameters used to define the ductility requirements of fiber-reinforced concrete structures\textsuperscript{[1]}.

<table>
<thead>
<tr>
<th>Plates</th>
<th>$P_u$ (kN)</th>
<th>$\delta_u$ (mm)</th>
<th>$\delta_{sls}$ (mm)</th>
<th>$\delta_u / \delta_{sls}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLC_F 1</td>
<td>21.67</td>
<td>9.51</td>
<td>0.02</td>
<td>581</td>
</tr>
<tr>
<td>TLC_F 2</td>
<td>20.49</td>
<td>8.86</td>
<td>0.03</td>
<td>352</td>
</tr>
<tr>
<td>TLC_F 3</td>
<td>19.44</td>
<td>8.49</td>
<td>0.02</td>
<td>525</td>
</tr>
<tr>
<td>RLC_F 1</td>
<td>16.98</td>
<td>0.68</td>
<td>0.06</td>
<td>11</td>
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<tr>
<td>RLC_F 2</td>
<td>20.37</td>
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<td>0.04</td>
<td>34</td>
</tr>
<tr>
<td>RLC_F 3</td>
<td>18.86</td>
<td>1.49</td>
<td>0.08</td>
<td>19</td>
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</tbody>
</table>

4 ECO-MECHANICAL ANALYSIS

By introducing the so-called Ecological Index ($EI$) and Mechanical Index ($MI$), a more comprehensive comparative analysis, including the environmental impact of the two types of lightweight concrete, can also be performed\textsuperscript{[10]}. The following equation estimates $EI$ by combining the amount of the released carbon dioxide (i.e., $\alpha =$ carbon footprint) and the energy used in the product lifecycle (i.e., $\beta =$ embodied energy)\textsuperscript{[12]}.

$$EI = \alpha \cdot \beta$$ \hspace{1cm} (2)

Both $\alpha$ and $\beta$ are here measured for the most relevant concrete components, such as cement, lightweight aggregate (made with expanded clay or grained rubber from end-of-life tires) and polypropylene fibers. Table 5 collects all the data necessary to compute $EI$ in accordance with Eq.(2)\textsuperscript{[12,13,14,15]}

The mechanical performances can be defined by two indexes: $MI_1 = f_{lc}$ at the material level, and $MI_2 = \delta_u / \delta_{sls}$ at the structural level. The latter mechanical index represents the ductility of flexural members, as stated by MC2010\textsuperscript{[1]}. The average values of $EI$, $MI_1$, and $MI_2$ are reported in Table 6 for the three lightweight concretes under investigation. According to MC2010\textsuperscript{[1]}, both the fiber-reinforced lightweight concretes TLC_F and RLC_F can be used in the plates of Fig.1b without any rebar in the tensile zones. On the other hand, to avoid the brittle response of Fig.2d, TLC_R plates must be reinforced with the minimum amount of rebars suggested by MC2010\textsuperscript{[1]} (i.e., 4 bars having a diameter of 5 mm). By applying a moment-curvature algorithm\textsuperscript{[16]} to the TLC_R plate cross-section, $MI_2 = \delta_u / \delta_{sls} = 6$ can be obtained. The ecological and mechanical indexes of TLC_R (Table 6) are respectively used as the upper bound ($EI_{TLC_R}$) and lower bound ($MI_{TLC_R}$) values of the required performances. Referred to these bounds, an eco-mechanical analysis of the TLC_10 and RLC_12 mixtures can be carried out by means of the non-dimensional diagrams depicted in Fig.6 ($MI = MI_1$ in Fig.6a, and $MI = MI_2$ in Fig.6b).

<table>
<thead>
<tr>
<th>Components</th>
<th>Carbon footprint (kg)</th>
<th>Embodied energy (MJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>0.8</td>
<td>5</td>
</tr>
<tr>
<td>Expanded clay aggregate</td>
<td>0.3</td>
<td>4</td>
</tr>
<tr>
<td>Rubber granulates</td>
<td>0.2</td>
<td>4</td>
</tr>
<tr>
<td>Polypropylene fibers</td>
<td>2.7</td>
<td>100</td>
</tr>
<tr>
<td>Steel reinforcement</td>
<td>1.5</td>
<td>21</td>
</tr>
</tbody>
</table>
Table 6: The ecological and mechanical indexes of the three lightweight concretes.

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>EI (kg GJ/m³)</th>
<th>MI₁ (MPa)</th>
<th>MI₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLC_R</td>
<td>1,907</td>
<td>21.51</td>
<td>6</td>
</tr>
<tr>
<td>TLC_F</td>
<td>2,432</td>
<td>22.91</td>
<td>486</td>
</tr>
<tr>
<td>RLC_F</td>
<td>1,379</td>
<td>10.99</td>
<td>21</td>
</tr>
</tbody>
</table>

Figure 6: The results of the eco-mechanical analysis: (a) At the material level; (b) At the structural level.

At the material level (Fig.6a), neither TLC_F nor RLC_F contemporarily satisfy the ecological and mechanical requirements. More precisely, the ecological index of TLC_F is higher than $EI_{TLC,R}$ (i.e., TLC_F should not be used from an environmental point of view), whereas the compressive strength of RLC_F is lower than the minimum acceptable value (i.e., $MI_1 < 21.51$ MPa).

On the other hand, Fig.6b shows the potentialities of rubber lightweight concrete in a structural application, where RLC_F fulfils all the required eco-mechanical performances. Indeed, with respect to TLC_F, which only satisfies the mechanical requirements, the substitution of clay aggregates with rubber granulates, and a lower content of cement as well, can reduce the environmental impact.

5 CONCLUSIONS

The results of the experimental investigation, concerning the behaviour of two different lightweight concretes, can be summarized by the following points:

1. Compressive strength of RLC is remarkably lower than that of TLC. Nevertheless, uniaxial compression tests on cylinders reveal the higher ductility of RLC.
2. On the contrary, the flexural strengths of TLC and RLC are comparable, as measured in three point bending tests on one-way plates.
3. If the environmental and mechanical properties are contemporarily taken into account, fiber-reinforced RLC plates perform better than those made with fiber-reinforced TLC. This is not true at the material level, because of the lower strength of RLC.

Finally, the above-mentioned differences suggest the necessity of a proper definition of the scale (i.e., material or structural) at which the eco-mechanical analysis has to be performed.
REFERENCES

[5] European Committee for Standardization. Materials produced from end of life tyres - Specification of categories based on their dimension(s) and impurities and methods for determining their dimension(s) and impurities. CEN/TS 14243, (2010).
Innovative precast concrete structural floor as a part of a HVAC System. The real application experience in a building.

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INTRODUCTION

This paper summarizes the environmental behavior of a precast concrete floor that is located in a new construction building (Barcelona; Leitat Technology Centre. 2014 ). This concrete structural floor has been designed by Pich Architects and has as a main characteristic the connection with the HVAC system in order to minimize the energy consumption on the building. Moreover the paper deeply defines the state of the art of these technical solutions and introduces further necessary research in order to improve this system and to increase its use on the future low energy consumption buildings.

GENERAL SPECIFICATIONS

Continuously architectural and structural solutions are competing in order to provide the most suitable option at technical office to solve an architectural target. During lot of time the chosen factors were related with mechanical behavior, durability, fire resistance, price… However since last years energy parameters landed over the decision tables and their importance is increasing daily. Related with these construction parameters both concepts (energy during the use phase and embodied energy) has to be taken in consideration in order to suit for the most environmental solutions.

As it is known, on the sector of material production companies is mandatory to act over the embodied energy of its material reconsidering material quantities and material production process. However and related with the most importance period of energy consumption (during the use phase) any contribution on the energy savings is required. For this reason the research in this field has high potential.

Over this discussion building structure has a specific importance due its existence of any building and the weight and its cost is significant in the overall of the building.
The adaptability, shape capacities of concrete and the numerous structural capacities of this material push also these technologies to focuses on the energy behavior during the building uses.

At the market already exist two systems that use the concrete thermodynamically capacities on structural flooring to reduce the building’s energy consumption meantime that increase the indoor thermal comfort of the buildings.

These technologies are based on the air flow in the inner ceiling that is connected with the HVAC system of the building this flow allow a temperature exchange and activate the concrete ceiling it as and storage system with numerous energetic capacities:

- Concrete Cool Systems http://www.kieferklima.de/en/products/concrete-core-cooling-concretcool is concrete a core cooling with supply air utilizes the high storage capacity of the concrete ceiling. The supply air flows through the cooling tubes consisting of aluminum with high thermal conductivity which can be laid in the concrete ceiling the high energy exchange rate between the air in concrete ceiling allows the ceiling to serve as a decentral retro heater for the supply air while simultaneously cooling the ceiling. A heat transmission rate of up to 90% is achieved. Even this system it is being used across Europe is necessary remark that the main technology is based on the aluminum pipes. These pieces increase a lot the time on the construction phase and at the same time has a significant cost in comparison with conventional concrete floor systems.

Figure 1: Image of Concrete Cool Pipe and Duct system.

- Termodeck system is based on the concrete hollowcore flooring and not any specific tubes are introduced on the floor. http://www.termodeck.com/ However the design of this precast slabs and the characteristics of the concrete made it have no any specific difference in comparison with other commercial types. This reason not allows increasing the energy potentiality of the system and also requires specific and cost cleaning process in the interior of the hollowcore flooring. http://www.casavonen.com/SelladoPlacasAlveolares.htm

Figure 2: Picture of Termodeck system integrated in the precast concrete floor system.
RECENT PROJECT DEVELOPMENT

Picharchitects has experience on this system and detect high research future development opportunities opportunity related with it. However a specific research project in cement, concrete and hollowcore is necessary in order to ensure and to provide at the market the improved system: Precast Energy Slabs (PES)

Below is explained the recent development done by the studio on related with the energy uses of precast concrete

The building (Leitat Technology Centre with a total floor area 4.800sqm and 5.2M€ of construction cost), will be used for research spaces and laboratories in the areas Biotech, Nanotech and new technologies. The presence of the building tries to connect, with the existing building, creating, abstractly, continuity in the skin texture of the traditional city. Moreover the building was designed with independent components that can be built industrially as well as assembled on site in order to get a very flexible building for future uses, even its deconstruction.

Figure 3: Schemes of climatically and environmental behavior of the Leitat building and its construction-structural systems.

About the construction system is important to notice that the architecture has been designed to act as an interface between the weather conditions outside and inside, not as a watertight barrier, but as a membrane that filters and shares the conditions of the environment. The metal structure acts as an integrated beam lattice, which support the floors composed of a single span that embraces from facade to façade. The front has an enclosure that efficiently meets all physical requirements of the building - structure, filtering natural light, privacy and service-provider.

Going further the structural system and its relationship with the HVAC (Heating, Ventilation and Air Conditioning System) of the building is important to specify: It’s a system that uses the thermal mass in precast hollowcore concrete floors to provide low energy heating and cooling. This system is integrated with the structure of a building.
The last part of the ductwork system for the supply air consists of hollow core concrete slabs instead of traditional steel ducts. It uses the thermal storage capacity of the building’s structural mass to regulate the internal temperatures. The effectiveness of the building's thermal mass is enhanced by passing supply air through the slab before it enters the room. The slabs work as heat exchangers between the supply air and the rooms. The floor-ceiling slabs serve many purposes: Besides from being the structural floor it conveys fresh air into the building and it serves also as an energy store.

These planks of the system heat the room in two ways:

Radiation: The concrete planks act as lukewarm radiators. They cover the whole ceiling, and has a capable of heating the whole room. In summer, they act as cooling radiators.

Air feed: The slabs are kept at around room temperature by passing air slowly through them. Some of this air is vented into the room to provide heating and cooling and ventilation. Stale air is sucked out of the room into the slabs

Particularly on this building due the specific functional and quality air conditions required for a laboratory (with high standard of air cleanliness and filtration) the Leitat Building is important to highlight that the system this duct system is integrates on the acclimatization and ventilation system at the last part of the circuit. The air that has been supplied at the interior space using common steel conducts after cool or heats the space is collected by openings that conduct the air into the interior of the concrete floor and flows across the hollow cores until the Exterior Treatment Unity.

Moreover the benefits already explained the use of the floor as a ducting system provides an easy location of its apertures. This option simplify a regularly distribution
on the space allowing and easy space adaptability, an low velocity air movement (with specific value for laboratory uses and high head exchange ratio between the air ant the floor itself).

Figure 5: Image of the building interior during execution works pointing out the holes at the precast concrete floor.

Figure 6: Image of the building interior on the uses phase. Pointed out are seen the filters and elements that finalize the floor holes and become the air apertures. Also as has seen on the photo the floor is painted and not has any suspended ceiling in order to increase the temperature exchange between the floor and the indoor space.

Finally we could highlight that the technological research on the field of concrete has to provide important advances on the environmental behavior of our buildings. Future buildings will be excellent test field in order to guarantee the functionality of these solutions.
Figure 7: Photo of the building structure during its execution.

Figure 8: Photo of the building with all the construction works finalized.

FUTURE PROJECTS DEVELOPMENT

As it was already summarize on the research background nowadays already are on the market technological solutions for the use of the concrete floor as a head exchanger linked with a HVAC system. This technology is already used for precast concrete solutions and in situ concrete solutions.
The technology uses the thermal mass of the concrete in order to reduce the energy peak demand and soft the maximum a minimum indoor temperatures. Moreover is crucial the technological link between the concrete floor, indoor aesthetical appearance and HVAC systems designs.

Related with this there are three important tasks to do in order to increase the use of this solution:

1. The development of a specific concrete formulation in order to increase its energy transmission to activate the maximum floor’s surface. (Precast or in situ solutions)
2. The improvement of a specific concrete composition in order to guarantee the hygienic requirements and the biocidal properties of the interior hollowcore flooring where these air flows.
3. The development of more laboratory test and real building tests. (precast or in situ solutions) that combine all the installation system in one solution with aesthetical quality.

For this reason a future research project has been designed in order to achieve the necessary result to ensure de development of an optimal technical and commercial Solution of: Precast Energy Slabs (PES) system.

Going further on that is important to specify that all this process will be focus on the improvement of the concrete formulation to improve the system. However nowadays there is not the possibility to present the detailed description of the research technical bases the majority of the solutions will focus on the incorporation of the additive at the conventional concrete. This additive will be based on: Aluminum particles (in order to increase the concrete thermal conductivity). Dioxide Titanium nanoparticles (in order to embody at the concrete cleanness air capabilities.

**Figure 8**: Figure with both effects that should be included on the structural concert floor (Cleaning, heat transmission.)
Below is described the methodology established for this research that will bring future results during the next 24 months.

Methodology for the Precast Energy Slabs (PES) system Research project:

1. Requirements & Specifications.
   - Analyse European and member state building codes and adapt them to Precast Energy Slabs (PES) system.
   - Define the requirements & specification of the system.

2. Development of elements composing the Precast Energy Slabs (PES) system.
   - Develop breakthrough technologies to be included in the Precast Energy Slabs (PES) structural and energy system.
   - Develop a mechanically and thermally efficient support system.
   - Develop and specific concrete formulation to ensure the maximization of the head exchanger between the air and the concrete slaps.
   - Develop and specific concrete formulation to ensure the no pollution of the air inside the precast slaps.
   - Develop different technologies integrated within the façade and the HVAC in order to integrate all the system.

4. Prototype & testing
   - Construct prototypes of the retrofitting system and perform laboratory testing to ensure it complies with the Construction Products Directive (CPD).
   - Perform experimental prototype validation prior to demonstration in real buildings:
     - Validate in laboratory and external tests the development made in WP3.
     - Assembling and of a prototype on an external platform in order to make a first feedback to designers
     - Prototype testing in an exterior experimental platform in order to evaluate the functionality and to measure the efficiency

5. Demonstration, monitoring & validation

The main goal is to demonstrate the performance of technologies developed in Precast Energy Slabs (PES) project, in a holistic way, in real. This real installation will allow:

   - Development the architectural-structural project design for the demonstrator.
   - Installation and commissioning the whole system in the demonstrator.
   - Installation of the post-intervention measurement in the demonstrator.
   - Evaluation of the performance of the retrofitted building through the monitored results.
CONCLUSIONS

It is important to conclude that the use of concrete on the structural floor of new buildings even if they are already interesting experiences has still a lot of research potential related with the energetically behavior of these buildings.

As it was described further studies on the development of a specific concrete formulation in order to increase its energy transmission to activate the maximum floor’s surface and the improvement of a specific concrete composition in order to guarantee the hygienic requirements and the biocidal properties of the interior hollow core flooring where these air flows are necessary to achieve the overall potential of this materials.

Moreover any basic research has to be follow up to a technical development, prototype testing and demonstration project in order to validate any future solution.

REFERENCES

Probe Team Building Elisabeth fry building Services Journal 4/98
REDUCING ENERGY NEEDS IN RESIDENTIAL BUILDINGS IN THE SPANISH CLIMATE THROUGH AN INNOVATIVE DAILY STORAGE BASED SOLUTION

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Key words: Energy saving heating and cooling needs, thermally active walls, building thermal inertia, concrete walls.

Summary. The framework of the research presented in the paper is a project oriented to promote the use of concrete solutions in buildings based on maximizing the benefits of its thermal inertia for both heating and cooling periods.

The constructive solution developed has configurations, one for summer (cooling mode) and another for winter (heating mode). In the cooling mode, the constructive solution is similar to a ventilated facade that is formed by a thermally insulated outer layer of concrete, an intermediate air layer and an inner layer of concrete. The inner layer is cooled at night by forced ventilation using an outdoor-outdoor scheme. The heating mode is reached by the addition of a crystal layer in front of the outer of concrete, creating an attached solar space.

The aim of this paper is to show the results about the potential of special concrete walls as solutions to reduce energy demand in residential buildings by heat storage and thermal offset in Spanish Mediterranean climates.

The simulations show that the use of this element is very satisfactory to reduce energy demand in residential buildings in the Spanish climate. The experiments have taken place during 2014. We have obtained important conclusions in the design and possible results with these solutions in real buildings.

1. INTRODUCTION

The Spanish Technical Building Code (2006) has been updated in 2013 [3] with a remarkable increase in the level of energy efficiency of buildings with respect to the levels established in 2006.

Nearly zero-energy concept requires increasing the level of performance of the building envelope and systems, and additionally a higher contribution of renewable energy. Meeting the "20-20-20" targets [4] for reduction of CO2 emissions necessarily involves a drastic reduction of energy consumption in buildings.

However the particular climatic conditions of Andalusia make it very special since there is a wide thermal radiation in summer and also in the cold winter seasons that are not exploited by existing buildings [5] in the rest of Europe this circumstance is given, and therefore have not developed studies in this regard.

To reduce energy consumption and therefore CO2 emissions, it is previously necessary to reduce energy demand through passive building insulation systems. Energy demand can be also reduced through thermal inertia [6] as a thermal energy storage system. Concrete buildings façades are are particularly appropriate.

In summer, the concrete building facades can be used as heat sinks. The aim is to cool the inner layer of concrete moving outdoor air through the air layer during night taking advantage of the low nigh-time air temperatures. The cool stored is released to the interior spaces when the maximum peak load of the space takes place.

In winter, the inner layer of concrete can be heated moving hot air coming from the attached solar space created by adding an external glass to the solution. The heat stored is released to the interior space during evening and early night hours when heating is typically required.

The walls of a building should have a double thermal paper: one related to the resistance to heat transfer and the other related to its power and return heat to accumulate in dynamic regime. The latter is the most overlooked when designing buildings and even usually not considered in the regulations relating to the construction of these enabling energy savings can not fully exploit attempts.

A simple definition of thermal inertia would say it's the ability to keep the mass of thermal energy received and gradually releasing it. Because of this ability, taking into account the thermal inertia of the walls of a building, can be decreased the need for air conditioning, with the consequent reduction of energy consumption and pollutant emissions.

The thermal inertia improves energy performance of buildings because it allows the damping of the temperature variation and the phase of the internal temperature relative to the outside.

For a situation with a high external temperature and solar radiation, the temperature outside the enclosure rises producing a heat transfer to the interior of the building. The evolution of the temperature of the outer face has a maximum (maximum amplitude) in a particular time of day depending on the location and orientation of the enclosure. This wave is damped outside temperature, in amplitude, crossing the enclosure, also emerging a time lag between the instants at which a peak temperature is produced. The effect of phase shift and damping allows the building to stay longer in the comfort zone without additional energy expenditure allowing free savings because they are inherent in the material. [7]

The physical characteristics of the concrete gives it a high thermal inertia, which allows to predict optimum energy performance of the building in the event that this material forms the
inner core (structure) and external (walls and roofs) thereof. The use of concrete as façade cladding and covered in the building:

- Reduce the energy consumption of heating.
- Softens variations in internal temperature.
- Delay maximum temperatures in offices and commercial buildings to the exit of the occupants.
- Reduces peak temperatures (maximum and minimum) and can make the air conditioning unnecessary.
- Maybe employed with nocturnal ventilation to eliminate the need for cooling during the day.
- Makes better use of sources of low temperature heating such as heat pumps for underfloor heating.

Overall the effect of thermal inertia in enclosures is a variable not usually considered in the design of the building.

In addition to its modelling difficult for designers and specifies, calculation tools were not sensitive to this parameter and the knowledge of its potential benefits have not been adequately considered by the technical and scientific community.

The main objective of the project is to parameterize the fundamental variables that characterize the thermal inertia of buildings overlooking substantially improve treatment procedures for calculating the thermal performance of buildings. This would be instrumental to value the role of concrete solutions as part of improving energy efficiency.

This setting will also allow designers can, easily, estimate the energy savings from the thermal inertia of buildings with contour and concrete structure.

2. MATERIALS AND METHOD

This study developed and adapted for specific reference models inertial phenomena based on the contour and the structure of the building.

These models allow us to model the phenomena of inertia that are incorrectly processed in simulation programs, including:

- Interaction of inertia and solar radiation (that influences the use of free solar gains in winter and undesirable modulation of solar charging in cooling mode).
- Treatment of inertia night ventilation strategies to pre-cool the structural elements of the building and reduce cooling requirements of the next day.

Reference models are based on analytical and numerical behaviour of the phenomena studied based on balance equations and heat transfer. The model used in this article is based on the previous developed by Ruiz-Pardo [8]. These models can be used independently for energy characterization applications and in turn are the starting point for other more simplified models that can be integrated into the simulation tools.

Later models have been developed specific reference to phenomena of inertia based on innovative elements of the building envelope.

These two solutions including:
Glazed Spaces. Act as greenhouses and in principle have no connection to the building. The space between the exterior cladding and glazed element is heated, the heat stored inside wall, it will go after giving the interior space throughout the day.

Thermally active walls. The enclosure is cooled using low outside air temperatures overnight. The inner leaf of the wall is cooled and then go into space yielding along the next day.

This innovative element is a combination of the mechanisms of Trombe-Mitchell and Constantine walls. It can be classified as an opaque solar facade, in the group of active elements according to the classification presented by [9].

The variables studied in models that are the thickness of the inner layer of concrete (10-15-20 cm.), the thickness of the internal air chamber wall (2-5-10 cm.) and the air speed walking the chamber (0.5-1-2 m/s). System has been optimized for winter or summer depending on location within the existing climate variability in the autonomous community of Andalusia, of which has focused on Granada, Seville and Cadiz for their thermal differences winter / summer.

In the coming months is to validate previous models by testing on the test cells are described in the following section. They are reinforced concrete cubicles 3.0 x 3.0 x 3.0 m. to which he included a special front face south to investigate several special construction solutions:

A planned two sheets of concrete and an air facade, which will be circulated in the summer evenings by cold air extractors (see Figure 1) is included first.

![Figure 1: Module test for the analysis of the influence of nocturnal cooling of the concrete wall through the chamber ventilated by forced ventilation.](image1)

Secondly, is studied the influence of the thickness of the glass chamber. Includes a glass cubicle in the main face, facing south, you can travel around by the width of the inner chamber between 0.10 and 0.50 meters, as shown in Figure 2.

![Figure 2: Test module for analyzing the influence of thickness of the chamber.](image2)
3. RESULTS AND DISCUSSION

In order to select a proper design of the innovative element, several simulations were performed with different designs in the three selected localities. In Sevilla and Cádiz the design was optimized for summer period because in these cities the cooling demand is dominant. In Granada the design was optimized for winter.

The process performed was the following in the three localities:

- Nine options were initially tested; this was the result of three air velocities in the air layer and three thicknesses of the inner wall. The results of these cases are shown in Table 1.
- From the results, the option with highest energy savings for the selected season is selected.
- For the selected option, some variations in the operational conditions are proved in order to maximize its behavior for winter season if the selected design was optimized for summer. The opposite procedure is performed if the optimization was performed for winter.

For the three cities considered, it was found that the optimal design was the same. This result was not expected, since they are very different climates and in addition, for Granada the design was optimized for heating season. The only difference in the design is that for the case of Cádiz, the optimum temperature for allowing ventilation is 25°C and not 23 as in the cases of Seville and Granada. The main descriptive parameters of the optimal design are shown in Table 1.

### Table 1. Studied solutions to find a proper design of the innovative element in Seville

<table>
<thead>
<tr>
<th>Inner wall thickness</th>
<th>Air layer velocity</th>
<th>Air layer convective heat transfer coefficient (air in circulation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>solution 5</td>
<td>15 cm</td>
<td>1 m/s</td>
</tr>
</tbody>
</table>

### Table 2. Gross heating gains for the innovative element in the three studied cities. Negative values means heating losses (desirable behaviour in cooling season) and positive values means heating gains (desirable behaviour in heating season). All values are in kWh/m²

<table>
<thead>
<tr>
<th>City</th>
<th>Summer</th>
<th>Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Monthly heat losses kWh/ m²</td>
<td>Total heat losses kWh/ m²</td>
</tr>
<tr>
<td></td>
<td>month</td>
<td></td>
</tr>
<tr>
<td>Seville</td>
<td>Jun</td>
<td>-7.57</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-3.04</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>-2.51</td>
</tr>
<tr>
<td>Cadiz</td>
<td>Jun</td>
<td>-19.81</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-9.71</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>-6.54</td>
</tr>
<tr>
<td>Granada</td>
<td>Jun</td>
<td>-32.08</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-20.77</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>-19.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The overall results for these three cities are shown in Table 2. It can be seen that the innovative element performs cooling in summer and heating.

The results of Table 2 show that in the three localities studied, the innovative element removes heat in summer and introduces heat in winter. In addition when it is compared with a conventional wall, its benefit is increased since the conventional wall has the opposite behaviour of the innovative element, namely heating in summer and cooling in winter.

To illustrate the behaviour of the innovative element in comparison with a conventional wall, in the figure 3, the temperature profiles are shown for the interior layer of the innovative element and for a conventional wall. These profiles were calculated for one typical day of the cooling season and for a typical day of the heating season.

Figure 3 shows that the amplitude of the temperature oscillation obtained in the innovative element, is much lower than that obtained in the conventional wall. Additionally, it is seen how in the cooling season, the temperatures of the innovative solution are always lower than the indoor temperature (assumed as 25°C), while the conventional wall is oscillating around that temperature. In the heating season, temperatures of the innovative element remain above the indoor temperature, while in the conventional wall; temperatures are always below the indoor temperature.

The expected savings for the innovative element are obtained from the comparison with a conventional wall. The results of this comparison are shown in Table 3.

<table>
<thead>
<tr>
<th></th>
<th>Summer</th>
<th>Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Innovative</strong></td>
<td><strong>conventional</strong></td>
<td><strong>difference</strong></td>
</tr>
<tr>
<td>element</td>
<td>wall</td>
<td>(savings)</td>
</tr>
<tr>
<td>Heat gains</td>
<td>Heat gains</td>
<td>kWh/m$^2$</td>
</tr>
<tr>
<td>Seville</td>
<td>-13.1</td>
<td>12.6</td>
</tr>
<tr>
<td>Cadiz</td>
<td>-36.1</td>
<td>4.2</td>
</tr>
<tr>
<td>Granada</td>
<td>-72.9</td>
<td>3.4</td>
</tr>
</tbody>
</table>

In the comparison shown in table 3, it can be seen that in Granada (the coldest locality)
savings in both, heating and cooling, are higher than in the other two localities. This is because in summer, in Granada where a larger value of cooling is provided by the innovative element while the conventional wall is almost neutral. By contrast, in winter, in Granada where innovative element provides the least amount of heat input, however, as the conventional wall shows high losses, the difference between the heat input of the innovative element and the heat losses of the conventional wall, is greater than in the other two locations.

4. CONCLUSIONS

The results show that the innovative element provides cooling in summer and heating in winter. In the three locations where the simulation were performed, energy savings are high, being highest in the coldest locality, but more simulations are required to establish if this is a continuing trend.

The levels of expected savings in both, heating and cooling show a promising performance in energy savings. For these reason the investigation of this element is justified to be continue in an experimental set up, in order to confirm the actual potential is as high as predicted.

Acknowledgements

The authors would like to thank the FEDER of European Union for financial support via project “Análisis del comportamiento energético de los cerramientos de hormigón en base a la maximización de las ventajas derivadas de su inercia térmica”of the “Programa operativo FEDER de Andalucia 2012-2014”. We also thank all Agency of Public Works of Andalusia Regional Goberment staff and reserchers for their dedication and professionalism.

REFERENCES

REFUSE CORK AS LIGHTWEIGHT AGGREGATE FOR MORE SUSTAINABLE MASONRY UNITS

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Key words: Refuse cork, lightweight aggregate, concrete, masonry units

Abstract. This communication constitutes an exploratory study about the use of refuse cork as aggregate in concrete mixtures by partially replacing natural aggregates. Granulated refuse with size between 3.2 to 5 mm has been employed for partially replace natural aggregates in different dosages in concrete specimens and masonry units. Different properties such as hardened density, compressive strength, thermal conductivity and resistance to thermal shock of the casted specimens have been evaluated. Concrete and concrete blocks obtained from natural aggregates have been used as reference. Hardened density, compressive strength and thermal conductivity present a decrease for an increase of the refuse cork dosage, and therefore, this organic aggregate allows to make concrete lighter and less thermal conductor.

1 INTRODUCTION

Currently there is an increasing awareness about environment and sustainable development due to the high consumption of natural resources and pollution [1]. Disposal of solid waste generated from agricultural and industrial production is another global serious problem. Moreover it has been reported that the world’s population will increase until 6-9 billion by the year 2050 and this will result in a substantial increase in the demand for water, energy, food, common goods and services. The construction industry will also grow up. The demand for concrete is expected to reach 18 billion tons [2] and as a consequence a considerable amount of natural resources will be required for producing construction materials. The great
advantage of concrete is that it is possible to choose its constituents and to modify them in order to optimize final material properties. Since aggregate make up about 60-80% of concrete volume, utilization of industrial and agricultural waste components, as whole or partial replacement of conventional aggregates, contributes to energy saving and conservation of natural resources [3]. It also solves disposal problem of wastes and helps environmental protection, making concrete industry more environmentally friendly and sustainable [4].

The use of industrial or agricultural waste as aggregate can provide an alternative to conventional methods for the production of lightweight concrete [5]. In the last years, numerous studies and researches have been carried out regarding this matter [6-11]. Lightweight aggregates are porous and granular materials with a typical bulk density lower than 2000 kg/cm³ [7] that may have different values depending on aggregate nature (organic or inorganic) and their source (natural or artificial) [12-18]. They lighten the weight of the structures and contribute to thermal and acoustic insulation of buildings. Thus materials such as expanded perlite or vermiculite present bulk density values between 30 and 240 Kg/m³ [12-13], expanded polymers have lower values (20-30 Kg/m³) [14-15] and agro-wastes in the range of 450-550 Kg/m³ [3, 16]. Granulated cork has been reported to have bulk density values of 104-171 Kg/m³ [17-18].

Cork is a light, renewable and biodegradable material with a cellular structure that absorbs CO₂ in its natural production process. It is extracted from the bark of the cork oak tree (Quercus Suber L) every 9-12 years as large rectangular planks. Cork oak woodlands constitute species-rich ecosystems with other ecological functions such as soil conservation and carbon dioxide retention. Cork extraction is a practice that allows to get a valuable product and to maintain these biological systems.

Portugal, Spain and Argelia are the main cork producers, although cork oak trees are also grown in Morocco, Tunisia, Italy and France. The world’s cork production is estimated at 340000 tons per year. One significant amount of harvested cork (20-25%) is discarded or remain under-utilized because the granules have a high density or are of very small dimensions or both [10]. This material, considered refuse cork, is used as agglomerated cork.

Cork’s cells are limited by a cell wall whose constituents are: suberin (40%) with a hydrophobic character, lignin (20-22%) which provides rigidity to cork, tenacious and resistant polymers such as hemicellulose (11%) and cellulose (9%) and extractives, aliphatic compounds, phenols and triterpenes (15-20%). Due to its cellular structure cork has a low bulk density (< 300 K/m³) and viscoelastic behaviour. Its tensile strength is 1,1 MPa and is compressive strength 12,5 MPa. Moreover cork shows low thermal conductivity and good sound absorption. This material, in a granular shape, has been incorporated in concrete as lightweight aggregate improving [17-19] its density and thermal conductivity, although a reduction of concrete’s mechanical properties has been observed. However, there is not evidence of granulated refuse cork being included as aggregate in concrete production.

This communication analyses the sustainability in terms of lightening a concrete made by a partial replacement of natural aggregate by cork refuse in different proportions and improving its thermal behavior. A preliminary study of different concrete mixtures containing refuse granules was carried out, in order to select that commercially viable according to lightening the concrete with a moderate strength capacity reduction. Once more suitable mixtures for the manufacture of concrete are selected, studies related to their thermal conductivity in laboratory were performed. Finally, laboratory results were validated on an industrial scale by
manufacturing masonry units within the range of substitutions that were selected as commercially viable.

2 EXPERIMENTAL PROGRAM

2.1 Refuse cork characterization

Refuse cork granules for this study were supplied by Eurocork S.L., Spain (Figure 1). They resulted from a classification process, based on density and particle size, addressed to select the best quality cork granules for cork stoppers manufacturing. Cork planks were boiled with water vapor before being grounded, in order to wipe out microorganisms and to remove microflora.

![Granulated cork](left) and granulated refuse cork (right).

Scanning electron microscopy (SEM) studies were carried out with the aim of studying refuse cork microstructure and its surface chemical composition. A scanning electron microscope Hitachi S-4800, connected to an energy-dispersive X-ray (EDX) analyzer (Bruker XFlash 5010) was used under high vacuum conditions. Refuse cork surface was coated with gold.

Particle density and water absorption were determined according to the methodology described in the standard UNE-EN-1097-6:2014 [20], Annex C. Porosity was determined as the ratio between volume of void-space and the bulk volume of material. The standard methodology UNE-EN1097-3:1999 [21] was used to calculate bulk density and voids.

A granulometric analysis of the granulated refuse cork was performed according to the method described in the standard ISO 2030:1990 [22] and using a 1000 g sample.

2.2 Materials and mix design

For the concrete formulation cement, natural aggregate, cork refuse granulate and water were used and their characteristics are presented.

Cement: CEM II/A-L 42.5 R. Produced by A. G. Cementos Balboa S.A. in Spain.
Natural aggregate: particle sizes used in mixtures were 4/8 mm and 2/5 mm as coarse aggregate and 0/4 mm as sand.
Cork refuse: described above.
Water: tap water

Four different “recycled” concrete mix proportions and a reference concrete were
designed. Every mix design keep constant the sum of the volumes of natural aggregate plus cork refuse granules. The analyzed percentage of substitution in volume of total natural aggregate by cork refuse granules were (C1: 1.9%, C2: 3.5%, C3: 5.3%, C4: 7.1%) although in this phase only 2/5 aggregate was replaced. The concretes mix designs are shown in Table 1.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement kg/m³</th>
<th>Sand (0/4 mm) kg/m³</th>
<th>Stone (2/5 mm) kg/m³</th>
<th>Refuse cork (3,2/5 mm) kg/m³</th>
<th>Stone (4/8 mm) kg/m³</th>
<th>Water kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>182</td>
<td>1365</td>
<td>331</td>
<td>331</td>
<td>218</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td>182</td>
<td>1362</td>
<td>297</td>
<td>8</td>
<td>330</td>
<td>218</td>
</tr>
<tr>
<td>C2</td>
<td>182</td>
<td>1361</td>
<td>264</td>
<td>15</td>
<td>330</td>
<td>218</td>
</tr>
<tr>
<td>C3</td>
<td>181</td>
<td>1359</td>
<td>231</td>
<td>23</td>
<td>329</td>
<td>217</td>
</tr>
<tr>
<td>C4</td>
<td>181</td>
<td>1357</td>
<td>197</td>
<td>31</td>
<td>329</td>
<td>217</td>
</tr>
</tbody>
</table>

Masonry units were cast in a precast company at full scale and in real production conditions, in a mechanical mixer, Leblan CT 90. Refuse cork granulated was added at the same time than natural aggregates. Water was added during the mixing process in order to keep constant the plasticity. The mixing time was the required to get a homogeneous mixture. Masonry units were finally obtained after a vibro-compression procedure.

Table 2: Masonry unit mix design.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement kg/m³</th>
<th>Sand (0/4 mm) kg/m³</th>
<th>Stone (2/5 mm) kg/m³</th>
<th>Refuse cork (3,2/5 mm) kg/m³</th>
<th>Stone (4/8 mm) kg/m³</th>
<th>Water kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>BR</td>
<td>184</td>
<td>1378</td>
<td>334</td>
<td>-</td>
<td>334</td>
<td>210</td>
</tr>
<tr>
<td>BC1</td>
<td>184</td>
<td>1372</td>
<td>244</td>
<td>20</td>
<td>333</td>
<td>210</td>
</tr>
<tr>
<td>BC2</td>
<td>182</td>
<td>1284</td>
<td>243</td>
<td>41</td>
<td>330</td>
<td>208</td>
</tr>
<tr>
<td>BC3</td>
<td>182</td>
<td>1234</td>
<td>198</td>
<td>60</td>
<td>329</td>
<td>208</td>
</tr>
</tbody>
</table>
2.4 Concrete and masonry unit characterization

*Scanning electron microscopy (concrete and masonry units)*

Microstructure and chemical microanalysis of C3 and BC1 concrete specimens and masonry units were performed by the SEM-EDX methodology at 28 days were considered to this test. Microstructure in each specimen or unit was determined in three areas and qualitative information on elemental distributions was obtained by line-scanning. Dried samples were coated with gold and analyzed under high vacuum conditions.

*Hardened density and compressive strength (concrete and masonry units)*

Hardened density and compressive strength tests were measured in cube specimens (160x40x40 mm³) at the age of 28 days in accordance with UNE- EN1015-10:2000 [23] and UNE-EN 1015-11:2000 [24]. Regarding masonry units at 7 and 28 days similar tests were conducted in compliance with UNE-EN 772-13:2001 [25] and UNE-EN 772-1:2011 [26].

*Thermal conductivity (concrete and masonry units)*

Thermal resistance of the concrete and unit masonry was evaluated based on the apparatus and procedure similar to that described in ISO 13787:2003 [27]. A quick thermal conductivity meter apparatus (Kemtherm QTM-D3) was used. In a previous step it was necessary to stabilize the samples moisture content to 60% at 20 ºC.

*Resistance to ageing by thermal shock*

The resistance of the concrete and unit masonry was measured using an adaptation of the procedure in UNE-EN 14066:2003 [28]. Samples were submitted to 20 cycles.

3 RESULTS AND DISCUSSION

3.1 Granulated refuse cork characterization

*Scanning electron microscopy*

In a refuse cork sample, two different surfaces may be distinguished, each one with a different color: a clearer one similar to cork surface and a darker bark called “raspa”. SEM analyses difference their microstructures (Figure 2). Thus micrographs 2a and 2b show a homogeneous cell structure, uniform and orderly, where cells are walled in. The cell walls show some ripples. However micrographs 2c and 2d corresponding to the bark surface show a heterogeneous and poorly defined cellular structure, with enclosed irregular cells, and arranged without the characteristic order of cork cells.
Figure 2: Refuse cork surface micrographies. Clearer surface: a) magnification of 250x; b) magnification of 300x. Darker Surface: c) magnification of 700x; d) magnification of 1000x.

Regarding EDX analysis, the two different surfaces described above have a very similar atomic composition, although a few differences have been also found (Table 3). Both surfaces have carbon and oxygen as major elements, as it could be expected since they are the main components of suberine, lignine, cellulose and hemicellulose. Other minor elements which are present in them are Ca, K and Al. But, on the other hand, cork like surface have some chlorine in his composition, and bark surface have some silicon traces between its components.

Table 3: Refuse cork surface EDX analysis.

<table>
<thead>
<tr>
<th>Element</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>76.94</td>
</tr>
<tr>
<td>C</td>
<td>21.89</td>
</tr>
<tr>
<td>Cl</td>
<td>0.40</td>
</tr>
<tr>
<td>K</td>
<td>0.34</td>
</tr>
<tr>
<td>Ca</td>
<td>0.27</td>
</tr>
<tr>
<td>Al</td>
<td>0.17</td>
</tr>
<tr>
<td>Si</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Element</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>76.83</td>
</tr>
<tr>
<td>C</td>
<td>22.50</td>
</tr>
<tr>
<td>Cl</td>
<td>-</td>
</tr>
<tr>
<td>K</td>
<td>0.39</td>
</tr>
<tr>
<td>Ca</td>
<td>0.18</td>
</tr>
<tr>
<td>Al</td>
<td>0.05</td>
</tr>
<tr>
<td>Si</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Density, absorption y granulometric analyses

It can be seen that the refuse cork granules can be clearly considered as lightweight aggregates, but if compared with high quality cork granules [17, 18] they are heavier, although absorption values are much lower (Table 4). Microstructure and chemical composition seem to influence. Refuse cork granule sizes were between 3,2 and 5 mm.

Table 4: Densities.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Cork</th>
<th>Refuse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apparent particle density (kg/m³)</td>
<td>182.6</td>
<td>573.6</td>
</tr>
<tr>
<td>Particle density, oven-dried basis</td>
<td>152.9</td>
<td>388.8</td>
</tr>
<tr>
<td>Particle density, saturated and surface-dried basis</td>
<td>317.4</td>
<td>712.2</td>
</tr>
<tr>
<td>Water absorption (%)</td>
<td>107.6</td>
<td>83.19</td>
</tr>
<tr>
<td>Bulk density</td>
<td>74.97</td>
<td>208.6</td>
</tr>
</tbody>
</table>
3.2 Concrete characterization

Scanning electron microscopy

Micrographies show that in concrete at 28 days, refuse cork appears to be well integrated into the cement paste, which is adhered to the cork refuse cell wall. It is possible even to find some cement paste inside cells (Figure 3). The organic material keeps its cellular morphology and cell wall does not seem to be damaged. The cement paste has a dense and compact appearance.

![Figure 3: SEM micrographs of refuse cork within cement paste in C3 concrete samples (28 days)](image)

EDX line scanning through the contact zone between a refuse cork granule and cement paste allowed us to have a qualitative idea of elements distribution between the two different materials. Concrete C3 sample analyses show that in a 125 µm distance, a progressive decrease of carbon atomic percentage and an increase of calcium, silicon and magnesium are observed from the cork refuse to the cement paste zone (Figure 4).

![Figure 4: EDX line scanning of the cork refuse-cement paste contact zone in C3 (28 days)](image)

Physical properties

Results of density, compressive strength at 28 days, thermal conductivity and resistance to ageing by thermal shock of concrete are presented in Table 5.
The density of the control specimen was 2170 kg/m³ and decreased with an increasing of the percentage of granulated refuse cork. When a 7% of natural aggregate is replaced by granulated refuse cork, the obtained concrete may be classified as lightweight, having densities below 2000 kg/m³ and a resistance decreased by 50%. In view of the results obtained concrete may be classified as lightweight, having densities below 2000 kg/m³ and a resistance decreased by 50%. In view of the results obtained C2 and C3 admixtures seem to be more effective.

The results in Table 6 reveal that mixtures containing 3.5% and 5.5% refuse cork reduce the thermal conductivity by a 19% compared to the control concrete.

Regarding ageing by thermal shock, the replacement of 2/5 mm aggregate by granulated refuse cork in the concrete admixtures significantly improved the concrete performance under wet and dry conditions.

### 3.3 Masonry unit characterization

**Scanning electron microscopy**

Microographies of masonry units BC1 at 28 days age also show a good adhesion between cork refuse and cement paste. The refuse cork granule is well embedded and connected to the matrix. Although the cellular structure of refuse cork granules is well preserved, cells shape appears deformed, it is elongated in one direction (Figure 5).

![Figure 5: SEM micrographs of refuse cork within cement paste in BC1 masonry units (28 days)](image)

Line scanning in the contact zone between a refuse cork granule and the cement paste in masonry units shows similar results that in concrete (Figure 6). Carbon atomic percentage decreases along a 150 µm distance, whereas calcium, silicon and magnesium ratios increase. The atomic percentages variation of aluminum and sulfur seems to show a maximum in this...
area, which could be related to some AFt phases composed of calcium, sulfur and aluminum.

![Image](image.png)

**Figure 6:** EDX line scanning of the cork refuse-cement paste contact zone in BC1 (28 days)

*Net and gross density*

BC1, BC2 and BC3 presented lower net and gross density values than BR, which is a reference block made only with natural aggregate (Table 6). This decrease is in the 4-14% range for both parameters.

<table>
<thead>
<tr>
<th>Block</th>
<th>Net density (Kg/m³)</th>
<th>Relative range (%)</th>
<th>Gross density (Kg/m³)</th>
<th>Relative range (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BR</td>
<td>2180</td>
<td>1,18</td>
<td>1010</td>
<td>1,17</td>
</tr>
<tr>
<td>BC1</td>
<td>2080</td>
<td>0,83</td>
<td>960</td>
<td>0,83</td>
</tr>
<tr>
<td>BC2</td>
<td>2000</td>
<td>0,89</td>
<td>920</td>
<td>0,89</td>
</tr>
<tr>
<td>BC3</td>
<td>1890</td>
<td>1,98</td>
<td>875</td>
<td>1,97</td>
</tr>
</tbody>
</table>

Decreasing of BC1 density with respect to BR density is less than the obtained for concrete C3 with respect to R. The BC2 sample, with a higher aggregates replacement than C3 (9.8% and 5.3% respectively) has a density similar. The manufacture process by energetic vibro-compression may remove some air and therefore it may cause a lower lightening respect to densities obtained in laboratory concretes.

*Thermal conductivity*

The decrease of the thermal conductivity in the concrete blocks with respect to reference block was in the range 7% -39%. The thermal conductivity of the blocks with granulated refuse cork decreases as the amount or organic material increases in their composition (Figure 7(a)). This decrease is more pronounced for replacements with refuse cork between the corresponding to BC1 and BC2 probably due to a higher proportion of refuse cork granules in the mixture and higher porosity values. In addition, removing part of the aggregate size fraction 0/4mm in BC2 and BC3 blocks could also have contributed to this further decline in the conductivity in these blocks. It is known that the grains of sand have a marked influence on the conductivity to act as thermal bridges that facilitate the passage of heat.

*Compressive strength*

BC1 block, despite the decrease in strength that experiments with respect to BR, keep
acceptable values (Figure 7(b)). In the case of the blocks with different natural aggregate sizes substituted and higher amounts of refuse cork, an increase of the amount of cement or the use of additives can mitigate the decrease in compressive strength.

![Figure 7(b)](image)

**Figure 7:** Thermal conductivity (a) and compressive strength (b) of masonry units with different refuse cork percentages.

**Resistance to ageing by thermal shock**

This durability test does not produce cracks or breaks in the concrete with refuse cork blocks, although they present a deteriorated appearance and a different color after it (Figure 9). Regarding mass variation only is observed in the reference block. Mass of BC1 and BC2 remains unchanged. However, compressive strength decreases in all samples. This decline is more important in blocks with refuse cork than in the reference block and could be due to the elimination of some air from inside the refuse cork cells.

![Figure 9](image)

**Figure 9:** BR-BC1-BC2 after wet and dry conditions.

4 CONCLUSIONS

The aim of the present work is to demonstrate the advantage of refuse cork-blocks compared to traditional blocks, regarding their thermal properties which are improved with the use of refuse cork in concrete admixtures. Besides that, refuse cork and cork granulates are natural products, resulting as by-products from the cork stoppers industry and their use has environmental advantages, helping to preserve cork oak woodlands and their ecosystems.
The reduction of energy consumption in the building sector is an important measure to help reduce energy dependency. For this purpose, the thermal behaviour of buildings envelope needs to be improved and appropriate energy performance requirements for technical building systems should be established.

Respecting other properties, concrete specimens and blocks are enlightened with the amount of refuse cork replacing natural aggregates, however mechanical strength tends to decrease if more refuse cork dosage is used, despite the fact that microscopy studies have shown a good affinity between refuse cork and cement paste. An effort to use some kind of cork granulates treatment should be planned if the target is to optimize concrete or masonry unit strength, although this is not a crucial factor for insulation concrete or no structural unit masonry.

REFERENCES


RETROFITTING WITH AN IAB CONCEPT: A SUSTAINABLE SOLUTION

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Key words: Integral Abutment Bridges, Bearings, Expansion joints, Retrofitting, Sustainability

Abstract. Integral abutment bridges [IAB] represent an alternative structural solution often better than conventional solutions for bridges with span lengths less than 60 meters. The benefits of this typology, compared to simply supported bridges (“conventional bridge” as used in this paper) are reduction of inspection and maintenance costs due to the absence of expansion joints and bearings; reduction of the failure risk under seismic loads and an improvement in terms of flood water management. The purpose of this work is to evaluate the benefits of retrofitting an existing bridge, converting it into an integral abutment bridge. This procedure will produce more sustainable bridges. This work also presents some methods to assess sustainability in civil constructions to compare the advantages of retrofitting a simply supported bridge. A practical example is provided.

1 INTRODUCTION

Sustainable structures meet the needs of the present without compromising the ability of future generations to meet their own needs [1].

A sustainable design demands as little as possible from nature, material and energy resources in all stages of its service life: conception, design, construction, service and maintenance.

IAB’s meets many of these requirements, for example: it is possible to reduce the necessary material for the superstructure and the work at construction necessary to installa devices that allow the free expansion of the superstructure.

There are many techniques to materialize an integral connection between deck and substructure in a bridge but the most commonly used one is connecting with a cast of concrete [1].

In the authors’ opinion, the principal advantages of IAB, compared to a conventional bridge are the increase of durability, and the reduction or completely eliminating the maintenance of bearings and expansion joints, according to the idea that “The only good joint is no joint” [1].
In the case of concrete structures, critical factors are represented from deformations and cracks related to the imposed actions which could occur due to a new structural configuration; a careful evaluation and control of these aspects for example with some numerical models or some parameters can ensure less maintenance of an IAB.

The purpose of this work is to evaluate the benefits of retrofitting of existing bridges into IAB.

2 BRIEF DESCRIPTION OF CONVENTIONAL BRIDGES AND INTEGRAL ABUTMENT BRIDGES

An Integral Abutment Bridge is a structure without expansion joints at the ends of the deck (monolithic connection with abutment) and, consequentially, the superstructure is directly connected to the substructure. Fig. 1 shows a simply supported bridge and an Integral Abutment Bridge.

![Figure 1: Scheme of a.) a simply supported bridge with expansion joints and b.) an Integral Abutment Bridge.](image)

There are many conventional bridges built all over the world and many of them are now affected by durability problems related to expansion joints and bearings [1]. The people in charge of the maintenance of these structures need to perform continuous inspections and maintenance plans, increasing the cost of the bridges.

Simply supported bridges are statically determinate structures and thermal variations do not induce important stresses. However, if periodic maintenance and inspections are not adequately planned, an accumulation of extraneous material into the joints could block the free expansion of the structure, facilitating the rise of parasitic stresses neglected in design of the structure. Furthermore a bad maintenance of joints could cause discomfort for the users driving over the bridge.
Another critical problem related to deck’s joints is represented by corrosion of the anchor blocks and the steel reinforcement of the structure. This aspect is particularly important in regions where de-icing salts are used, which accelerates the process of corrosion.

3. ADVANTAGES OF RETROFITTING WITH A JOINT LESS BRIDGE CONCEPT

3.1 Simply supported into Integral abutment bridge.– Durability problems.

IAB’s represent a favorable solution for the construction of new bridges, in addition they could be an interesting solution, for retrofitting existing structures. The key issue in transforming an existing simply supported bridge into a joint less bridge is to achieve a monolithic connection between superstructure and abutment. Fig. 2 shows one schematic representation of a full integral abutment bridge.

In addition, changing from a simply supported beam into a “frame” improves the behaviour of the bridge. At the ultimate limit state a redistribution of internal stress occurs, in particular the bending moments from the mid-span to the connections between the superstructure and substructure of the bridge. This reduces the risk of brittle failure and increases the global ductility of the structure.

![Figure 2: Schematic representation of an IAB - abutment connections (eg.: [2]).](image)

At the service limit state, the structure experience an increase of stiffness with a consequential reduction in the deformations and vibrations.

It is thus evident that retrofitting an existing bridge into an IAB has many benefits and it is
a good strategy to build a “new bridge” with almost 100% of the existing parts of the old bridge, with a reduced economical investment and prolonging the service life of the structure.

3.2 Problems related to expansion joints: seismic risk and water management

Simply supported bridges are a conventional solution for the construction of structures with a reduced span, however, due to its statically determinate scheme this bridge typology is particularly vulnerable to horizontal movement of the deck during earthquake. There are many examples of failures of unseated deck caused by earthquakes as can be seen in fig. 3. In order to face these kinds of failures it is necessary to increase the redundancy level of the structure, materializing solid connections between the main elements of the bridge. Thus, retrofitting these elements into IAB’s reduces the risk of seismic failure. Although, the dynamic behaviour of the structures is normally improved it is necessary to perform simulations to assess the response of the structure to different scenarios.

![Figure 3: Unseating of a simply supported bridge caused by an earthquake (Chile) [3].](image)

Other important aspect is the management of drained waters. If the drainage system is not properly maintained or if it is directly connected to the abutments (as in the case of bridges that cross roads), drained water could cause deterioration of the elements of the infrastructure as seen in fig. 4. Furthermore, heavy rain could produce an overburden of water with consequent erosion of the fill behind abutments and in extreme cases the stability of the bridge could be compromised. Naturally, in a structure without joints it becomes easier to perform a drainage system capable to optimally manage, recycle and dispose of runoff waters.

![Figure 4: Effect of flood water on the deterioration of an abutment and a pier.](image)
4. CASE STUDY: RETROFITTING OF A SIMPLY SUPPORTED BRIDGE INTO IAB

The case study is a bridge with a 20 meters span length and 33.5 meters wide. The superstructure is composed by prestressed concrete box cells and the substructure is composed by an abutment 3 meters high and the foundation piles beneath it are 18 meters long. Since the opening of this bridge, expansion joints have been affected by durability problem and in order to reduce the maintenance cost it was decided to retrofit this bridge from a simple supported bridge into an IAB.

![Section and longitudinal view of the case study](image)

**Figure 5:** Section and longitudinal view of the case study

A finite element model [4] composed by frame and shell elements was made in order to represent all of the components of the bridge. Particular attention was placed to model the soil-structure interaction [5]. The results showed the suitability of the deep foundations to carry the imposed actions on the structure, for instance temperature gradients. All of these phenomena are carefully checked during design stages.

4.1 Rating system for sustainable bridge engineering

There are many works on the assessment of sustainability of construction works [6-8]. A sustainable design should take into account the social goal, the environmental goal and the economical goal to produce a sustainable design.

In order to evaluate the improvement of the sustainability of a conventional Bridge retrofitted into an IAB the rating system proposed by Hunt et al. [6] is used.

![Sustainable design in bridge engineering](image)

**Figure 6:** Sustainable design in bridge engineering [9]
4.2 Sustainability improvement of a bridge retrofitted into an IAB

It is possible to quantify the sustainability improvement of existing bridges retrofitted into IAB’s. Initially, it is necessary to choose significant parameters, which will represent the maintenance work necessary to be done on the structure. The parameters taken into account in this paper are:

- Sustainable site,
- Water used and quality,
- Energy and transportation,
- Material and sources,
- Potential benefits,

For any of these parameters, it is possible to evaluate different aspects and obtain a corresponding score.

The goal of the rating system proposed by Lauren Hunt [6] is to encourage and recognize a good sustainable bridge design by providing cost effective, measurable guidelines promoting sustainable design concepts that would foster positive impacts on the natural and cultural environment. A few minor adjustments in the ratings were made by the authors.

*Sustainable site:* according to the main idea of this topic, the closure of the joints reduces the amount of water that goes directly into the abutment. Consequently, the erosion of the backfill is controlled, and both, maintenance and the probability of structure collapse are reduced.

<table>
<thead>
<tr>
<th>Table 1: Sustainable site</th>
<th>S.S</th>
<th>I.A.B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Does the bridge control in some way sedimentation and erosion phenomena?</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Does the bridge connect two well established places?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Was the bridge conceived to minimize the delays of general public?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Does the bridge replace or improve an existing structure?</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Can this bridge replace other small bridges possibly less functional?</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>0</strong></td>
<td><strong>2</strong></td>
</tr>
</tbody>
</table>

*Water use and quality:* The closure of expansion joints also ensures the protection of elements behind the superstructure and the infrastructure for example, anchor blocks. Moreover, this condition allows to verify where the water flows and to conduct it to an appropriate drainage system.
Table 2: Water used and quality

<table>
<thead>
<tr>
<th>Water used and quality (2 points)</th>
<th>S.S</th>
<th>I.A.B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Does the bridge change the performance of flood water?</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Did you use non potable water during stage construction?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subjects about runoff of water (what system, where, etc.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>0</td>
<td>0.67</td>
</tr>
</tbody>
</table>

Energy and transportation: In simply supported bridges, some of the most critical elements are bearings and expansion joints. Their removal is direct reduction of the monitoring necessary to control the health of the bridge. In addition, the total time and energy necessary to perform the connection between the superstructure and substructure is less compared to the continuous replacement of bearings and expansion joints.

Table 3: Energy and transportation

<table>
<thead>
<tr>
<th>Energy and transportation (9 points)</th>
<th>S.S</th>
<th>I.A.B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remote health monitoring sensors to reduce frequency of visual inspection</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Easy to inspect</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Does the bridge installation reduce the average miles traveled?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridge built using accelerated bridge technologies or techniques to minimize construction activities and energy.</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Is the bridge efficient enough to carry out the traffic?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The automatization on toll sign messages</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Does the bridges have sidewalks and or bicycle lanes?</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>0</td>
<td>3</td>
</tr>
</tbody>
</table>

Material and sources: An existing bridge is “recycled” to obtain a new bridge. Due to this intervention it is foreseeable to reduce the material necessary to replace the joints and bearings. The substructure, if it meets the stresses imposed by the new static scheme, is also reused in almost 100%. Another important aspect is related to the concrete necessary for the retrofitting. Usually it is made by local materials, in contrast to expansion joints and bearings that, in many cases, must be imported.

Table 4: Material and sources

<table>
<thead>
<tr>
<th>Material and sources (6 points)</th>
<th>S.S</th>
<th>I.A.B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Are recycled materials used in the structures?</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Can materials used in the structure be recycled?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduction of required material.</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>If renewal of, are the materials from the old structure reused in the new</td>
<td></td>
<td></td>
</tr>
<tr>
<td>If renewed how much of the original structure is utilized in the new design</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Are materials regionally available or brought in from long distance</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>How much cement did you use (CO2 emissions)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>0</td>
<td>5.83</td>
</tr>
</tbody>
</table>
Potential benefits: The material necessary to make the connections between the suband super structures represent a minimal part compared to material used to build a new bridge. There is also an improvement in terms of comfort of passengers due to elimination of expansion joints.

It is possible to evaluate, from the charts seen in tables 1 to 5, the total score reached by a bridge retrofitted into an IAB. In particular, the case study obtains a score of 13 points out of a total of 25. A summary of the points presented in the rating system are shown in table 6.

<table>
<thead>
<tr>
<th>Potential benefits (3 points)</th>
<th>S.S</th>
<th>I.A.B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridges that utilize fewer raw materials on the job site</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Bridges that utilize less time and energy to construct</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Bridges that meet their own electrical needs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridges that help deal with the coming needs of 21 century travel faster and more efficient</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Bridges that encourage alternate modes of transportation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridges that are able to monitor their own health and alert own critical conditions</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total 0 1.5

5. SUMMARY AND CONCLUSIONS

This work shows the advantages of retrofitting a simply supported bridge, with a span length less than 60 meters, into an Integral Abutment Bridge. This solution presents many advantages in terms of cost, inspection time and maintenance, including a significant improvement in performance of the bridge for static and seismic loads.

The upgrade flow of water inside the abutments is avoided reducing the probability of corrosion.

In order to quantify the benefits obtained with the retrofit, a rating system related to the sustainability of bridges is used.

A real case was presented to study the sustainability and the results show an advantage of the IAB in comparison with the conventional bridge. The principal factors that improved the sustainability of the structure are: the reduction on the maintenance works, that the material used to do retrofitting is recovered from the previous bridge and the reduction of erosion on the slopes near the abutments.
Table 6: Summary of points obtained from the rating system

<table>
<thead>
<tr>
<th>PARAMETERS</th>
<th>No of points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sustainable sites</td>
<td>2.00</td>
</tr>
<tr>
<td>Water used and quality</td>
<td>0.67</td>
</tr>
<tr>
<td>Energy and transportation</td>
<td>3.00</td>
</tr>
<tr>
<td>Material and sources</td>
<td>5.83</td>
</tr>
<tr>
<td>Potential benefits</td>
<td>1.50</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>13.00</strong></td>
</tr>
</tbody>
</table>

ACKNOWLEDGEMENT

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REFERENCES

[9] Yadong Li, Xiun Zhang, Xianzhen Li “School of civil engineering, Southwest Jiaotong University, Chengdu, Sichuan 610031, P.R China.
STUDY OF THE USE OF DIFFERENT CHEMICAL ADMIXTURES IN MORTARS MANUFACTURED WITH RECYCLED SAND FROM CDW

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Key words: Air-entraining admixture, C&DW recycling, Mortar, Recycled aggregates, Water-reducing admixture.

Abstract. Construction and demolition waste (CDW) properly treated and processed in recycling plants can be transformed into recycled aggregates (RA). According to the European Directive 2008/98/EC, the member states must reach a minimum recycling rate of 70% by 2020. To date, there are numerous investigations of using the coarse fraction of RA in different applications, such as roads and concrete production. However, the use of recycled fine aggregate fraction in concrete manufacture has not been widely studied. The incorporation of fine RA decreases the mechanical strength and durability of concrete. The use of chemical admixtures could improve properties of concrete and mortar made with the fine fraction of RA. An alternative would be the use of fine fraction of RA in the manufacturing of masonry mortar. Recent studies demonstrate the feasibility of replacing up to 50% of natural sand by recycled sand in mortar production, although a higher w/c ratio is needed for achieving the same fresh consistency as a result of the increased water absorption of RA. In this work, the effects of two chemical admixtures on the mortar properties made with RA from CDW were analyzed: an air-entrained and a superplasticizer. Fresh and hardened mortar properties were studied using a CEM II/BL 32.5N. A 1:6 volumetric cement-to-aggregate ratio was adopted. The w/c ratio was experimentally adjusted in order to ensure a fresh consistency of 175 ± 10mm. In general, the use of chemical admixtures decreased the w/c ratio. The use of superplasticizer improved the mechanical strength and decreased the setting time and shrinkage. The use of an air-entrained admixture increased the workability and decreased the dry bulk density and compressive strength.
1. INTRODUCTION

Construction and demolition waste (CDW) is one of the largest types of waste generated in developed countries. These wastes are generated within the lifecycle of building and infrastructures, especially during the rehabilitation and demolition phases. CDW is mainly composed of concrete and masonry rubble, which have a high potential for recycling. This waste properly treated in a recycling plant is transformed into recycled aggregate (RA). Two types of RAs can be obtained: recycled concrete aggregates (RCA) and mixed recycled aggregates (MRA). According to the European Directive 2008/98/EC, the member states must reach a minimum recycling rate of 70% by 2020.

Nowadays, there are numerous studies about the use of the coarse fraction of RA (concrete, masonry) in different applications such as road construction [1] and concrete manufacturing [2]. However, the use of the fine fraction of RA has been investigated less than the coarse fraction in concrete manufacturing because of its worse properties, such as a larger amount of cement paste, higher water absorption and more sulphur compounds. The incorporation of fine RCA jeopardizes the mechanical and durability properties of concrete [3]. An alternative to use this fine RA can be its use in the manufacture of masonry mortars [4-6].

Recent research has shown that it is possible to replace up to 50% of natural aggregates by fine recycled concrete aggregates [4, 7] or recycled masonry aggregate [5-6, 8] in mortar production. Workability and shrinkage are the most affected properties. These results were attributed to the higher water/cement (w/c) ratio required during mixing process, due to the higher water absorption of recycled aggregates.

The use of chemical admixtures enables reducing the w/c ratio and improves the mechanical and durability properties of concrete and mortar made with the fine fraction of AR [9]. Toledano-Prados et al. [10] confirmed that the use of superplasticizer improved the behaviour of cement-based materials, increasing fluidity and reducing water demand. Moreover, Traczewska [11] showed that the use of superplasticizer increase the mechanical properties in mortars, besides requiring less water for the same consistency, leading to a shortening of the setting time.

This research studies the effect of two kinds of admixtures (air-entraining and water-reducing) on the following properties of mortars made with fine mixed RA: fresh consistency, workability, bulk density of fresh mortar, dry bulk density, compressive strength and drying shrinkage at 28 days.

2. MATERIALS AND EXPERIMENTAL

2.1. Material characteristics

A fine mixed RA with a particle size 0/4 mm (recycled sand) was used (FRMA). The material was collected from a recycling plant (GECORSA SL) located in Cordoba (South of Spain). The mortar mixes were made with 100% of FRMA.

The main components according to UNE-EN 933-11:2009 were: 38.2% of concrete rubble and masonry mortar; 31.5% of unbound natural aggregates; 28.9% of ceramic particles, 1.1% of bituminous materials; 0.15% of gypsum; 0.01% of glass and other 0.14% (wood, plastic, etc).
Figure 1 shows the particle size distribution of FRMA according to UNE-EN 993-1:2006, which approaches the theoretical curve of Faury.

![Particle size distribution of FRMA](image)

The density and water absorption was carried out according to UNE-EN 1097-6 (Table 1).

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Standard</th>
<th>FRMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle dry density $\rho_{rd}(g/cm^3)$</td>
<td>UNE-EN 1097-6/00</td>
<td>2.402</td>
</tr>
<tr>
<td>Water absorption (%)</td>
<td>UNE-EN 1097-6/00</td>
<td>4.025</td>
</tr>
</tbody>
</table>

As binder Portland cement type CEM II/BL 32.5N was used, the cement was supplied by COSMOS SA. Two kinds of admixtures were used: an air-entraining admixture (AEA) type "Mastercast 934" and a superplasticizer type "Mastercast 205 MA", both supplied by the Company Badische Anilin Und Soda Fabrik (BASF). Table 2 shows the properties of both admixtures: Mastercast 205 MA and Mastercast 934.

<table>
<thead>
<tr>
<th>Mastercast 205 MA</th>
<th>Mastercast 934</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary function</td>
<td>Superplasticizer mortar</td>
</tr>
<tr>
<td>Secondary function</td>
<td>Cohesive.</td>
</tr>
<tr>
<td>Secondary effect</td>
<td>Risk of disintegration</td>
</tr>
<tr>
<td>Physical appearance</td>
<td>Beige powder</td>
</tr>
<tr>
<td>Apparent density, 20°C</td>
<td>0.870 – 0.970 g/cm³</td>
</tr>
<tr>
<td>PH</td>
<td>6 ± 1</td>
</tr>
<tr>
<td>Chlorides</td>
<td>&lt; 0.1%</td>
</tr>
</tbody>
</table>

MasterCast 205 MA is a powder admixture particularly suitable for self-compacting mortar production. High fluidity without exudation and high quality of mortar can be achieved without compaction. This admixture is characterized for providing an excellent workability. It is especially recommended for pumping mortar.
MasterCast 934 is an admixture designed to improve the workability and docility of industrial mortars, even with dry crushing and inapposite grading aggregates. This admixture produces a plasticizer / air-enthraining effect that improves plasticity of mortars in the fresh state, reduces the amount of mixing water required and increases the mechanical strength and improves the durability of mortar against freeze-thaw cycles.

2.2. Mortar mixture proportions

A volumetric proportion of 1:6 (cement: recycled aggregate) was used. The FRMA was mixed at laboratory temperature and moisture (2%). It was not pre-saturated or oven dried before use.

Table 3 shows the mortar mixes composition and the nomenclature used for the 3 types of mortars tested: the reference mortar (MRII), the mortar made with AEA MasterCast 205 MA (MRII1) and the mortar made with superplasticizer MasterCast 934 (MRII4). For each type of mortar, the maximum dosage recommended by the manufacturer was used: 0.8% for the superplasticizer (MasterCast 205 MA) 2.5% of the weight of cement.

The mixing water was adjusted experimentally to achieve a consistency of 175 ± 10 mm. Mortars that did not meet this value were rejected. The use of admixtures reduces water requirement in the mortar, which leads to a lower w/c ratio, especially in the case of superplasticizer (MasterCast 205 MA), in which a decrease of nearly 27% was observed.

<table>
<thead>
<tr>
<th>Mortar Type</th>
<th>RA</th>
<th>CEM</th>
<th>Mastercast 934</th>
<th>Mastercast 205 MA</th>
<th>Water</th>
<th>w/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRII</td>
<td>3335</td>
<td>658</td>
<td>-</td>
<td>-</td>
<td>858</td>
<td>1.3</td>
</tr>
<tr>
<td>MRII1</td>
<td>3335</td>
<td>658</td>
<td>5.26</td>
<td>-</td>
<td>788</td>
<td>1.2</td>
</tr>
<tr>
<td>MRII4</td>
<td>3335</td>
<td>658</td>
<td>-</td>
<td>16.45</td>
<td>628</td>
<td>0.95</td>
</tr>
</tbody>
</table>

2.3. Tests performed

Fresh mortar was characterized by consistency (EN 1015-3: 1999), bulk density (EN 1015-6: 1998) and workability (EN 1015-9: 2000).

The hardened mortar was characterized by the dry bulk density (UNE-EN 1015-10:2000), compressive strength (EN 1015-1011: 1999) at 28 days and drying shrinkage (UNE 83831: 2010). Drying shrinkage strains were measured at 2, 7, 14 and 28 days on prismatic specimens of 40 mm × 40 mm × 160 mm. To obtain mean values, four repetitions were made.

3. RESULTS AND DISCUSSION

3.1. Evolution of consistency over time

Table 4 shows the fresh mortar consistency over time. It was measured every 15 min until a runoff of 100 mm (diameter of the cone). It was observed that the mix MRII4 reached this
value in less time (30 min) compared to other mixes. These results were attributed to the reduced amount of mixing water and the fact that the FRMA used was not pre-saturated.

No study to show that the admixture MasterCast 205 MA accelerates the setting of cement was carried out.

### Table 4: Consistency

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>MRII</th>
<th>MRII1</th>
<th>MRII4</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>175.5</td>
<td>169.3</td>
<td>170.3</td>
</tr>
<tr>
<td>15</td>
<td>154.2</td>
<td>153.8</td>
<td>115.4</td>
</tr>
<tr>
<td>30</td>
<td>131.6</td>
<td>142.2</td>
<td>103.6</td>
</tr>
<tr>
<td>45</td>
<td>121.3</td>
<td>126.6</td>
<td>--</td>
</tr>
<tr>
<td>60</td>
<td>111.7</td>
<td>115.9</td>
<td>--</td>
</tr>
<tr>
<td>75</td>
<td>105.9</td>
<td>110.4</td>
<td>--</td>
</tr>
<tr>
<td>--</td>
<td>--</td>
<td>90</td>
<td>105.8</td>
</tr>
</tbody>
</table>

**3.2. Workability**

The mortar MRII4 mixed with superplasticizer showed shorter workability (Table 5), approximately 72% lower than the reference mix MRII and 87% lower than the mortar MRII1 mixed with AEA. These results are consistent with those obtained in the consistency test (Table 4). The use of AEA (MasterCast 934) increases the workability by almost 120% with respect to the reference mortar MRII, which is consistent with the characteristics of this admixture. This fact is positive since the workability is one of the most affected properties from the use of fine RA.

### Table 5: Workability

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRII</td>
<td>87</td>
</tr>
<tr>
<td>MRII1</td>
<td>191</td>
</tr>
<tr>
<td>MRII4</td>
<td>24</td>
</tr>
</tbody>
</table>

**3.3. Density of fresh and hardened mortar**

The use of superplasticizer (MasterCast 205 MA) slightly increases the bulk density of fresh and hardened mortar, while the use of AEA (MasterCast 934) decreases the bulk density compared to the reference mortar (Table 6). Ma et al. [12] confirmed that increasing the dosage of superplasticizer produces a higher bulk density.
Table 6: Bulk density of fresh and hardened mortar

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Fresh bulk density (g/cm³)</th>
<th>Dry bulk density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRII</td>
<td>1.7</td>
<td>2.01</td>
</tr>
<tr>
<td>MRII1</td>
<td>1.4</td>
<td>1.59</td>
</tr>
<tr>
<td>MRII4</td>
<td>1.75</td>
<td>2.02</td>
</tr>
</tbody>
</table>

3.4. Compressive strength

As shown in Table 7, the mortar MRII4 made with superplasticizer (MasterCast 205 MA) achieved nearly twice the compressive strength of the reference mortar. This can be due to the lower w/c ratio and higher bulk density achieved. Tkaczewska [11] stated that the use of superplasticizer increases the mechanical strength in mortars. Moreover, when adding the AEA (MasterCast 934), mechanical strength compared to reference mortar decreased by 64%.

Table 7: 28-day compressive strength

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Mean (MPa)</th>
<th>SD(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRII</td>
<td>9.8 ± 0.60</td>
<td></td>
</tr>
<tr>
<td>MRII1</td>
<td>3.5 ± 0.89</td>
<td></td>
</tr>
<tr>
<td>MRII4</td>
<td>19.4 ± 1.21</td>
<td></td>
</tr>
</tbody>
</table>

3.5. Drying shrinkage

The use of superplasticizer (MasterCast 205 MA) decreases the drying shrinkage compared to the reference mortar (Figure 2). This is attributed to the lower w/c ratio.

However, Ma et al. [12] showed that drying shrinkage strains in mortar made with superplasticizer was apparently greater than those measured for the reference mortar, and increases with an increasing dosage of superplasticizer.

The use of AEA slightly increases the drying shrinkage due to the lower mechanical strength of this mortar.
4. CONCLUSIONS

The addition of an air-entraining admixture in mortars made with fine recycled aggregates from CDW had the following consequences on the mortar's properties:

- w/c ratio decreased up to 8%;
- consistency was maintained for longer;
- workability increased up to 120% (191 min);
- bulk density of fresh mortar decreased up to 18%;
- dry bulk density of hardened mortar decreased up to 21% at 28 days;
- 28-day compressive strength decreased up to 64%;
- 28-day drying shrinkage increased up to 13%.

The addition of an superplasticizer admixture in mortars made with fine recycled aggregates from CDW had the following consequences on the mortar's properties:

- consistency changed quickly;
- workability decreased up to 72% (24 min);
- bulk density of fresh mortar increased up to 3%;
- dry bulk density of hardened mortar was not changed significantly;
- compressive strength increased up to 98% at 28 days;
- 28-day drying shrinkage decreased up to 36%.

Acknowledgment

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5. REFERENCES


SUSTAINABILITY FEATURES OF AN ELEVATED ROAD CORRIDOR UNDER CONSTRUCTION IN AN URBAN ENVIRONMENT

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Key words: Elevated Road Corridor, Sustainability Features

Abstract. In the new millennium, Delhi, the capital city of India has experienced a phenomenal growth in vehicular population resulting in choking of the city roads all over Delhi. In order to reduce the travelling time between distinct far ends, express corridors have been planned by Govt. of Delhi. One such corridor is South-East Delhi corridor popularly called as Elevated Road Project over Barapulla Nallah [1]. This is conceived in three phases having nodal locations as Mayur Vihar in East Delhi and Aurobindo Marg in South Delhi. This Paper covers sustainability features of the 2nd Phase of the project.

1 INTRODUCTION

The Proposed Barapulla Nallah Corridor has been conceived as East-south corridor as an alternative route to congested section of Ring Road between Sarai Kale Khan and AIIMS that facilitates the immediate need of Commonwealth Games and in long run for the movement of freight, goods, people, and utilities. The alignment that is partly over River Yamuna and partly along Barapulla Nallah (drain) that collects the discharges of other internal, peripheral and trunk drains to further discharge into the Yamuna. Barapulla Nallah in its east west orientation starts from Ring Road near Sarai Kale Khan to Ring Road near INA.

2 EXECUTION OF PROJECT IN 3 PHASES

As defined in the background note itself, the Project has been conceived and its execution planned in 3 phases.

3.1 Phase 1

In phase 1, an Elevated road, 4 Kms long connecting Sarai Kale Khan and Jawahar Lal Nehru Stadium has been completed in recent past in 2010 to facilitate the movement to Jawahar Lal Nehru Stadium for Common Wealth Games 2010 without facing the traffic hassles of Delhi. This Phase was defined as a sustainable corridor due to various features...
qualifying it as supportive to society, environment friendly and economic viable. One of the key feature worth mentioning it saving the eclipsing of a heritage structure namely Khan-e-khan tomb by realigning the corridor to move more than 100 m away from it surroundings and also elevating the deck to 12 m height so that full view from any distance is not disturbed [2]. Moreover the Cantilever segmental construction over the major crossings and railway crossings helped in maintaining the free flow of road and rail traffic [2].

3.2 Phase 2

In Phase 2, the construction of connectivity from Jawahar Lal Nehru Stadium to INA is in progress. In addition to this connectivity, there are certain intermediate loops connecting the elevated road constructed in Phase 1 with the major arterial roads crossing on the way [3]. Also the missing facilities for ascending and descending to the elevated corridor at Sarai Kale Khan are being added in 2nd phase. The works in Phase 2 are expected to be completed and operational by September 2016.

3.3 Phase 3

In phase 3, the major link to connect the elevated corridor with East Delhi at Mayur Vihar after crossing Yamuna is being provided. The work has started in April 2015 ad expected to be operational by end of 2017. This corridor will cross the Yamuna River with an extrados bridge so as to minimize the foundations in the River by providing 128m spans.

4 PHASE 2 FROM JLN STADIUM TO INA

In this Phase the Corridor dropped at Nehru Stadium in the phase 1 is being taken ahead to INA to touch the Aurobindo Marg as well as further elevated to facilitate the traffic from Ring Road to enter this corridor. In addition to this there were certain loops not taken up in Phase 1 due to paucity of time are being added to make the corridor better functional and useful to more number of citizens. In Phase 1 the ascending and descending facilities at Sarai Kale Khan were limited. Now, with the provision of two more loops at this location, the limitation is resolved and the corridor is a more user friendly as it caters to traffic from/to all the directions. There are many more intricacies involved in Phase 2 due to which the distinct types of designing technologies have been adopted as detailed below. In fact the structure provided in this phase covers different types of structures as mentioned in following paras.

4.1 Precast segmental construction

Most of the spans have been designed with pre-cast segmental technique with 3 or 4 spans continuous. In most of the segmental construction works taken in the country, the continuity is limited to deck by providing the continuous reinforcement over simply supported structures so as to have Expansion joints after 3 or 4 spans thus having a better riding quality. One step ahead in this case is that the continuity is not limited to deck, but the entire 3 or 4 span structure is made continuous by use of pre-stressing couplers between pre-stressing cables.
In the first stage of launching for three (or four) span continuous structure, 1¼ span is launched and pre-stressed in the usual manner. The pre-stressing cables can be taken further by joining next set of cables through the pre-stressing couplers and the launching of 2¼ (or 3¼) span is completed and so on till last ¼ span is left over for final launching of a module and finally pre-stressed. This system proves to be economical in comparison to conventional system of providing Deck slab Continuity by about 10%. Moreover, the constructability is also simplified.

4.2 Cantilever construction (CLC) Technique

The corridor is required to cross the Ring road at one location at Sarai Kale Khan. Due to deeper segments, it was not be possible to transfer the segments through the heavy traffic and decision has been taken to substitute from precast to in-situ balanced cantilever structure. Accordingly, this crossing has been constructed as in-situ Cantilever structure technique monolithic with the twin flexible piers.

4.3 Precast Pre-tensioned I Girders

There are certain locations were the radius of the curves is too less to cast the segment with sufficient accuracy and launch them at site as segmental construction. At such locations, the construction has been carried out with pre-cast pre-tensioned I Girders of approximately 20 m Length.

In addition to above locations, the structure meeting Aurobindo Marg for a length of about 300 m will be just over the Nallah. In such a location, there is a limitation of the depth of the structure, as deeper segments will cause obstruction to the free flow of Nallah. At such locations, the structure has been designed as pre-tensioned, pre-cast I-Girders with a pre cast and in-situ slab for the ease of constructability.
4.4 Steel Arch Bridge Structure

During the midst of the construction, it was noticed that DMRC has planned an underground tunnel that was crossing the Barapullah Corridor at grade. Interestingly, their construction was also executed at the same period. It was essential to have a perfect coordination between two executing bodies as well as monitoring of structures during the course of construction. The location of pile and piles caps were to be tailor made along the direction of the Tunnel so as to avoid any conflict between the two structures. Again there is a limitation of depth of the structure to be provided and span of the structure varying from 45-65 m in order to accommodate tunnels between the pile caps. Here steel arch truss was provided. On one carriageway, it is 3 spans of 45 m each, while on the other carriage way it is 2 span of 65 m each.

4.5 In Situ Solid/voided Slab

There are certain locations like a structure passing through Silver Oak Park, the only solution was to negotiate the curve with in-situ solid slab monolithic with the piers due to non standard spans and also where there was a possibility of Channel water rising up to the deck level.

The starting point of Phase II beyond Nehru Stadium was found to be having underground cellular foundation structure provided for Bus depot constructed over the Nallah just abutting the Barapullah Corridor. Since the foundation structure was continuous, it was not possible to dismantle the foundation for boring the piles and therefore, the alignment was to be shifted slightly by about 10 m. This shifting resulted into very limited areas for the placement of foundation and substructure. Accordingly decision was taken to provide structure over single pier for a length of about 200 m. For such a length of corridor, the structure is being provided as in-situ pre-stressed voided slab with shorter spans for making it lighter in weight and casting monolithically with single piers.

5 SUSTAINABILITY CONSIDERATIONS AND ACHIEVEMENTS

(1) The work has been executed without disrupting the traffic by carefully planning the sequence of operations so that the traffic moves unhindered at all times.

(2) Providing helmets, masks, safety goggles, etc. ensured safety of workers during construction. Adequate signage, barriers and persons with flags to control traffic had been provided during construction. Adequate drainage, sanitation, and waste disposal facilities were provided at work places.

(3) The alignment wherever possible has been modified to save maximum number of trees. In complete project of Phase II, only 79 trees have been removed and out of them about 20 trees are being transplanted.

(4) High grade concrete like M60 is being used to save overall cement and in turn reducing CO₂ emission to atmosphere [7,8].

(5) Maximum pre-casting has been done to reduce pollution to city environment and also reducing the construction time period.
(6) A perfect coordination has been done with DMRC to allow the execution of both the projects simultaneously without any adverse impact on other project.

![Figure 3: Traffic moving unhindered during the construction over Ring Road](image)

(7) No disturbance has been caused to the existing utilities.

![Figure 4: Water mains and HT Lines remained intact (not shifted)](image)

(8) All the construction and demolition waste is being taken to recycling plant, where it is getting converted into useful products like bricks, tiles, paver blocks etc. [4,5,6].

6 CONCLUSIONS

- It is true that civil constructions in urban areas are essential for overall development and benefits of the community, but it is successful only if equal importance is given to the environment for a sustainable development. It is essential that every construction activity should be environment friendly as the Environment too has a right to remain protected. Engineering solutions to minimize the Environment impacts and for adopting the mitigation measures are available.

- Adoption of standard and safe construction practices is very much essential particularly in urban environment. It must be ensured at all times that workers adopt best safety protections in their own interest. Protection of Health, Safety and Environment should always be kept as the prime goal.

- The right of respectful living of the residents around the construction sites should not be jeopardized.

REFERENCES


SUSTAINABLE TBM TUNNELS FOR TOMORROW

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Key words: Tunnels, TBM, TMG, TMF, TISB, Sustainability

Abstract. Three innovative and very cost-effective concepts for the construction of tunnels using the TBM (“Tunnel Boring Machine”) technique are presented: the TISB concept for tunnels in soft soil, in seismic areas; and the TMG and TMF concepts, for railway and roadway tunnels, respectively. The added-value obtained with the use of these concepts in the construction of sustainable tunnels is emphasized.

1 INTRODUCTION

Tunnels are becoming increasingly used in the construction of transporting infrastructure, for both railway and roadway network systems.

The TBM (“Tunnel Boring Machine”) technique is the most common, nowadays, for tunnel building, allowing for significant savings in costs and time. In underwater tunnels, the TBM technique also allows for the reduction of the environmental impacts, allowing for non-disturbance of the sea bed.

TBMs are of different types, according to the conditions of the soil to be drilled (EPB, Mix-Shield, Double-Shield, etc). It is becoming common, nowadays, a TBM to build more than 0.3 km of tunnel per month, as average.

When building tunnels with a TBM, the circular cutter head of the front shield excavates the soil while the erector mounts precast segments around the tunnel surface, which are clamped together, forming the lining circular wall of the tunnel (Figure 1). Afterwards, a filling on the bottom of the tunnel is executed, creating a platform for the circulation of the vehicles: trains in the case of railway tunnels, and cars or trucks in the case of roadway tunnels.

Figure 1: Schematic view of a TBM
The precast segments are made of high strength concrete (C40, or higher), with steel reinforcement or mixed with fibbers, having, in general, 1.20-1.60 m width. Its number will be the adequate to form the complete circles with pieces with a specific weight; commonly medium size tunnels have 6-8 segments per circle. The thickness of the tunnel lining will depend on the surrounding acting stresses and on the thrust forces applied by the TBM; it corresponds, in common situations, to about 1/25 of the interior diameter of the tunnel.

2 CHALLENGES

Despite great progress observed in recent times, the TBM technique still faces significant challenges.

This technique is appropriate for tunnels in stiff soil (stiff clay, compacted sand, etc), because the tunnels built this way have their stability assured by the strength of the surrounding soil (the lining precast segments will function mostly as finishing), thus they do not need to have significant strength in both the transverse direction and the direction of the tunnel axis.

In the case of soft soil (mud, soft clay, loose sand, etc), the construction of tunnels with the conventional TBM technique is unreliable because, in the tunnel thus formed, the connections between precast segments are very weak (it is a kind of “LEGO”); hence the strength and the ductility of the tunnel are low and there is the risk of sinking or collapsing, particularly during earthquakes. The soil treatments, used sometimes to help the seismic behavior of the structure, besides being very expensive, generally do not assure the required reliability.

Another important issue is of the number of tubes to form the tunnel. In fact, for both railway and roadway systems, the installation of the two directions of traffic placed side by side in the same tunnel are adequate in short tunnels, only.

In long railway tunnels (typically, the tunnels spanning more than 1 km), for safety reasons [3], instead of a single tunnel, two separated tunnels are in general necessary, each one for a direction of traffic, and complex systems of safety galleries and shafts, for the rescue of the users and the local access of personnel in the event of an accident or of fire. When the access galleries and shafts are not possible to build (e.g., underwater tunnels) a third tunnel is still necessary be adopted.

Thus, in the most recent railway tunnels, such as the Gotthard-Base tunnel (which is being finalized in Switzerland), the longest in the world, 57 km long, solutions with two tubes, each one for a direction of traffic, are being used. Cross-passages regularly spaced and crossovers at certain distances to allow the trains change the lines, as well as access galleries to outside, in several points are also adopted (Figure 2a). Obviously, these solutions are complicated and expensive.

Where the Channel Tunnel (between UK and France) and of the basic proposal for the Gibraltar Strait tunnel, three tubes were adopted, two for each direction of traffic, and a service tunnel placed between the two (Figure 2b). The three tubes are interconnected by a number of cross-passages, regularly spaced, making the construction very complicated and expensive. However, the change of trains from one tube to another is not possible, unless big caverns are built.
In roadway tunnels, the installation of the two directions of traffic in the same tube is also only possible for tunnels with a single lane in each direction. When there are two or more lanes in each direction, the required diameter would become so large that is not possible to be achieved. In any case, for long tunnels (typically, tunnels spanning more than 0.5 km), because of safety reasons [1], the placing of the two-way traffic side by side is always problematic, and two separated tunnels, each one for a direction of traffic, have to be adopted so that, for ventilation and smoke removal purposes, the air will circulate in one direction only, the direction of the traffic. In addition, evacuation routes and access galleries have also to be built along the tunnel, to allow for access to the interior of the tunnel and the evacuation of people in case of an accident or of fire inside the tunnel.

This is the case of the tunnel of the south by-pass of the M30 motorway, in Madrid, Spain, in which, two large tubes 15.2 m diameter, were adopted [4] (Figure 3). Although that solution allows for the installation of additional facilities, the use of two tubes is very expensive and the impacts on the ground surface, during construction, are very much aggravated.

In order to overpass the limitations referred above, three innovative concepts: the TISB, the TMG and the TMF concepts, were recently developed.
3 TISB, TMG AND TMF CONCEPTS

3.1 TISB concept

The TISB (“Tunnel of Improved Seismic Behavior”) concept constitutes an innovative solution for tunnels of the roadway and the railway types, built with the TBM technique, when the referred tunnels are executed in soft soil (e.g., mud), in seismic areas, allowing for the tunnel to be provided with the adequate strength and ductility under earthquakes [5]. It also allows for the strengthening of existing tunnels, using them as exterior formwork for the execution of the interior strengthening. The TISB concept is illustrated in Figure 4.

![Detailed perspective of the tunnel](image1)
![Perspective of the complete solution](image2)

**Figure 4: Illustration of the TISB concept**

The tunnel is formed by two concentric tubes; an exterior tube (3), which is a conventional TBM tunnel, and an interior tube (4), which is executed later on, inside the exterior one. The exterior tube (3) is formed by precast segments mounted by the TBM, while the interior tube (4) is cast inside the exterior tube (3), using the latter as exterior formwork. Inside the thickness of the interior tube (4) longitudinal reinforcement bars (7) and transverse reinforcement bars (8) are laid, both in two layers, which are confined by confinement bars, in order to provide the tunnel with the adequate strength and ductility.

If there will be a significant variation on the vertical actions (by the increasing or the diminishing of the weight of the upper soil, for example), the tunnel will be provided with intermediate supports, regularly spaced along the axis of the tunnel. These supports are constituted by groups of piles with great horizontal deformability and ductility, which are anchored in large concrete blocks (5) evolving the exterior tube (3), and in the stiff soil below, allowing for the horizontal movements of the tunnel during earthquakes.

The concept allows for the obtaining of monolithic structures (there are no joints) with great resistance on both longitudinal and transversal directions and great ductility under earthquakes. They will also be very effective if liquefaction and cyclic mobility phenomena occur. Furthermore, the structures will present great structural redundancy, which can be helpful in the case of scenarios not foreseen during the design phase.
3.2 TMG concept

The TMG (“Tunnel Multi Gallery”) concept allows with a single tube the realization of railway tunnels with completely independent but interconnected directions of traffic, and the installation of appropriate means for easy local access and the evacuation of people, in the case of accident or fire inside the tunnel [6]. The TMG concept is illustrated in Figure 5.

After executing the circular wall (1) through the TBM, a slab (3), placed slightly above the base of the tunnel and the entire width, and a vertical wall (2), arranged in the middle of the tunnel and its entire height, are built so as to create two independent railway galleries, disposed side by side (4) (5), one for each line, and a service gallery (6) below.

In both sides of the tunnel, vertical access galleries (7), regularly spaced, provided with escape doors (8) and equipped with stairs, are arranged, allowing for the safe passage of people to the service gallery (6), in case of accident or of fire inside the tunnel.

Inside the service gallery (6), emergency vehicles (9) of monorail type are installed, in order to provide local access to the personnel and to allow for the evacuation of people to outside. In the vertical wall (2) large openings can be arranged (10), regularly spaced along the length of the tunnel and provided with fire doors, which will be opened to allow the trains to pass from one to the other gallery, in the case one of the lines became out of service.

A variant B to the basic concept can also be adopted, in which the vertical access galleries (7), instead of being placed on the outer side of the railway galleries, are placed in the middle of the tunnel, at the separating wall (Fig 5).

![Figure 5: Illustration of the TMG concept](image-url)
### 3.3 TMF concept

The TMF (“Tunnel Multi Floor”) concept allows with a single tube the realization of two-way separated roadway tunnels, with two identical roadway galleries, isolated and independent, and the installation of appropriate means for easy local access and the evacuation of people, in the case of accident or of fire inside the tunnel [9]. The TMF concept is illustrated in Figure 6.

![Perspective of the tunnel](Image)

![Current cross-section of the tunnel](Image)

**Figure 6**: Illustration of the TMF concept

After the execution of the circular wall (1) by the TBM, two slabs (2) (3) are built, at its full width, one placed roughly at half the height of the tunnel and the other placed slightly over the base of the tunnel, in order to form three superimposed galleries, isolated and independent: two roadway galleries (4) (5), one for each direction of traffic, and a service gallery (6) below.

In one of the sides of the tunnel, vertical access galleries (7), regularly spaced and provided with escape doors (8) in both roadway galleries, are arranged. The vertical access galleries (7) are provided with stairs (10), allowing for easy passage of people between the roadway galleries (4) (5) and the service gallery (6) below.

Inside the service gallery (6), emergency vehicles (9) of monorail type are installed, to provide local access to the personnel and to allow for the evacuation of people to outside of the tunnel in case of accident or of fire inside the tunnel.
4 FEATURES

4.1 Alternative solution for the Gibraltar Strait tunnel

Based on TISB and TMG concepts, an alternative solution to the basic proposal for the Gibraltar Strait tunnel (see Figure 2b) was developed [8]. This tunnel will link Cape Malabata, near Tanger, north of Marocco, with Punta Paloma, at west of Tarifa, south of Spain. It will be 39 km long, of which 28 km under the sea, and goes very deep, until elevation -400 [2] (Figure 7). The maximum slopes of the tunnel will be of 2.5%. The maximum speed of the trains will be of 120 km/h.

The geological environment consist mainly of formations called flysches, consisting of clayey materials and sandstone, which are overlaid by post-flyshes formations, consisting of shallow granular deposits. On the marine investigations it was discovered the existence of two very deep parallel paleo-channels, in the middle of the sea, about 1 km wide each, mostly composed of deposits of clayey brescia and bioclastic sand, which are much weaker materials. The area is very seismic-prone, and severe earthquakes have been known to occur.

In the alternative solution the tunnel is formed by a single tube 11.0m inner diameter, inside of which an intermediate concrete slab and a central separating wall are built, creating two independent and isolated railway galleries, placed side by side, and a service gallery below. The slab is supported on both sides of the circular wall and on two longitudinal concrete walls (Figure 8a). The cross-section of each railway gallery is about 36 m², the same that of the railway tubes of the basic proposal (7.5m inner diameter each).

In the zones of the tunnel where it goes through the paleo-channels the cross-section is enhanced with an interior vault in concrete cast-in-situ, that completes the interior ring (Figure 8b), which is provided with longitudinal and transversal reinforcements dully confined by confining reinforcements, in order to provide those parts of the tunnel with adequate ductility under earthquakes.

Vertical access galleries are arranged in the middle of the tunnel (Figure 8c), with escape doors at both sides and provided with stairs, spaced 400m, to provide safe passage of people between the rail galleries and the service gallery.
The service gallery is equipped with emergency vehicles of the EMEV type (“Emergency Monorail Electric Vehicles”), to provide local access to the personnel and the evacuation of the users to outside in the case of accident or of fire inside the tunnel (Figure 8).

4.2 Proposal for a roadway tunnel under the Tagus river in Lisbon

Based on TISB and TMF concepts, a solution for the construction of the Algés-Trafaria tunnel, under the Tagus river in Lisbon, Portugal, was developed [7, 8 and 10]. This tunnel will allow for the closing of the regional inner ring of Lisbon, being located west of the existing 25 de Abril (suspension) bridge. The speed of the vehicles will be limited to 80 km/h.

The site is characterized by the existence of alluvial deposits of large thickness along the river bed, composed of various complexes of mud and sands, which extends from elevation -20 until elevation -75 (Figure 9). Underlying the alluvial deposits there are bed-rock formations composed of basalts and limestone, which extend through the north bank. On the south bank there are Miocene formations composed mainly of sands and clays. It is also to be noted the very prone-seismic conditions of the area (to be remembered the 1755 Lisbon earthquake, one of the most destructive in history).
In the proposed solution the tunnel is a single tube 5.1 km long, formed by two concentric tubes: an exterior tube, which is a conventional TBM tunnel, and an interior tube, in reinforced concrete cast in-situ (Figure 9).

The maximum slope of the tunnel is 4.8%. At its deepest part, the tunnel will be between elevations -35 and -50, and a minimum soil overburden of 11m will be guaranteed. Thus, in the zones where the natural soil is deeper than elevation -24m (in some particular zones the natural soil is at elevation -27m), a rock-fill will be installed before the passing of the TBM.

The cross-section of the tunnel is illustrated in Figure 10. The inner diameter is 13.6m. The thickness of the precast tube is 0.55m and that of the cast in-situ tube is 0.3m. Adopting grout injections of 0.15m thickness between the precast segments and the soil, the total thickness of the wall of the tunnel will thus be 1.0m. The outer diameter of the tunnel will then be 15.6m, the same that of the largest existing TBM tunnel (Sparvo tunnel, Italy).

![Figure 10: Proposal for the Algés-Trafaria tunnel, in Lisbon - Cross-section](image)

Inside the tunnel two concrete slabs are built, allowing for the installation of two superimposed roadway galleries (each one for a direction of traffic) with two lanes each, 3.5m wide and 4.8m high, and a service gallery below 2.0m high (Figure 10a). In a second phase (if the traffic will justify it), both roadway galleries can be modified for 3 lanes each, 3.3m wide, being the exterior ones limited to vehicles up to 3.5m high. The upper slab is supported laterally in corbels cast jointly with the inner tube, while the lower slab is supported in the inner tube and on two longitudinal concrete walls.

The roadway galleries are provided with escape doors spaced 400m, placed in one of the sides of the tunnel, which give access to the service gallery below, through vertical access galleries, provided with stairs (Figure 10b).

The service gallery is equipped with emergency vehicles of the EMEV type (“Emergency Monorail Electric Vehicles”), to allow for access of the personnel to the interior of the tunnel.
and the evacuation of the users to outside in the case of accident or of fire.

The vertical access galleries will be built by locally dismounting the precast segments of the exterior tube and casting new concrete walls in-situ. In the cases where there will be water pressure around the tunnel, grout injections or the local freezing of the soil will allow for the development of the works in safe conditions.

5 CONCLUSIONS

As shown, the concepts TISB, TMG and TMF are innovative developments that may provide obtaining very economical and reliable solutions in the construction of tunnels using the TBM technique.

These concepts can thus be a great added value when applied in the construction of rail and road networks sustainable worldwide, significantly improving safety and reducing drastically the costs, including through a large reduction in the consumption of materials.

REFERENCES

Sustainable Technology for PC Grout Infill

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Key words: PC grout, Super-low viscosity, Infill technology, CO₂ emission

Abstract. In this study, the effect of the execution methods on the infill performance of super-low viscosity type of PC grout was examined in the following three cases: (1) general method; (2) vacuum suction method from the end of the sheath tube; (3) vacuum suction method from the end part and intermediate parts of the sheath tube. The test results showed that: (1) regardless of the execution methods, the super-low viscosity type of PC grout causes the flow-ahead phenomenon; (2) the vacuum suction only from the end part of the sheath tube makes a large void in the vicinity of the starting point of the down slope; (3) the vacuum suction from the end and intermediate parts of the sheath tube makes the infilling of PC grout 100%, and (4) the use of vacuum pumps has an extremely small effect on the amount of CO₂ emission. The super-low viscosity PC grout infilling technology with vacuum suction from the end and intermediate parts of the sheath tube enables efficient and reliable grouting. This is a sustainable technology to ensure the long-term safety, serviceability and environmental performance of a PC structure.
1 INTRODUCTION

The prestressed concrete (PC) member of post-tension system is casted and cured with concrete, and PC tendons in its prearranged sheath tube are tensioned and anchored, and then the sheath tube is grouted. PC grout is conducted to protect PC tendon from corrosion by keeping it in alkaline environment and to ensure the bond of the PC tendon to surrounding concrete. As the infill of PC grout greatly influences the serviceability and mechanical performances of a PC structure, it is significant to assure the infill performance and to enhance the efficiency in the grouting executions.

There are two types of PC grout: high viscosity type and super-low viscosity type. The high viscosity type grout has an excellent material segregation resistance. The high viscosity type of PC grout needs high pressure in the grouting execution. In addition, sometimes PC grout gets blocked in the sheath tube [1]. On the other hand, the super-low viscosity type of PC grout can be executed efficiently with the low pressure and the risk to getting blocked is low. However, there is a possibility that the void in the down slope area of the sheath tube is formed due to ‘flow-ahead phenomenon’ [2].

In this study, the effect of the execution methods on the infill performance of super-low viscosity type of PC grout was examined. Furthermore, CO₂ emissions for each of the execution methods were calculated.

2 OUTLINE OF EXPERIMENT

In this study, full-scale PC grout infill test was conducted to confirm, when the super-low viscosity type of PC grout was infilled, influences of the presence and position of vacuum suction over its execution quality.

2.1 Shape of the sheath tube

Figure 1 shows the shape of the sheath tube used. The sheath tube was 10.8m in horizontal length and had 10 degrees in declivity where it had height difference of approximately 1.0m. The tube was an 80mm-inner-diameter waveform polyethylene sheath, and it had 12 PC steel strand wires inside, each of which was 15.2mm in diameter. The cross-section area of the sheath tube was 5027mm², and that of the PC steel strand wires was 1664.4mm². Therefore, the void content of the sheath tube was 66.9%. Accordingly, the amount of grout infilled was 36.6ℓ. In addition, the position of the intermediate parts of the sheath tube conforms to the method shown in the reference [3].

![Figure 1: Shape of the sheath tube](image-url)
2.2 Mix proportion and production of PC grout

The super-low viscosity PC grout used in this study was a premixing type of cement and chemical admixture. The PC grout contained approximately 1.0% chemical admixture to the mass of cement. Table 1 shows the mix proportion of the PC grout. The target value of flow time is the intermediate value of the values specified in the super-low viscosity PC grout (3.5 ~ 6.0s).

The super-low viscosity PC grout was produced by a grout mixer of 100ℓ capacity. After the water, cement and chemical admixture was inserted into the grout mixer, it was mixed for 1.5 minutes.

Table 1: Mix proportion of super-low viscosity PC grout

<table>
<thead>
<tr>
<th>Water-powder ratio (%)</th>
<th>Quantity of material per unit volume of PC grout (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water</td>
</tr>
<tr>
<td>38</td>
<td>539</td>
</tr>
</tbody>
</table>

2.3 PC grout infill methods

The super-low viscosity PC grout infill test was conducted for three cases. Figure 2 shows the placement of equipment for each case.

Figure 2: Placement of equipment
Case 1 shows that no vacuum suction is conducted when the PC grout is infilled. And Case 2 shows a method of reducing pressure in the sheath tube by vacuum suction from the end of the sheath tube. In addition, Case 3 shows a pressure-reducing method by vacuum suction from the end and intermediate parts of the sheath tube. In any cases, the infill speed of the super-low viscosity PC grout is 15ℓ/ min.

In the case of vacuum suction, the PC grout was infilled after the pressure in the sheath tube was reduced to -0.085MPa. In Case 2, a vacuum gauge was positioned at the intermediate part (2). With the vacuum gauge, changes in the pressure-reducing state at the bending-down part in the sheath tube were measured, after the flow-ahead phenomenon occurred and the infilling of the PC grout reached at the end part of the sheath tube. A void in the sheath tube was also measured after PC grout hardening. The measuring point was set between the two intermediate parts (1) and (2).

3 RESULTS AND DISCUSSION

3.1 Quality of PC grout

Figure 3 shows the flow time for each case. In all of these cases, the flow time of the PC grout was within the specified value. Figure 4 shows the air content. The air content of the PC grout was the range of 0.1 to 0.9%.

3.2 The visual confirmation of flow direction

Figure 5 shows the visual confirmation result of flow direction of the infilling PC grout. In all three cases, the super-low viscosity PC grout caused a flow-ahead phenomenon regardless of the presence and position of vacuum suction. The measured result of negative pressure in the sheath tube at the intermediate part (2) for grout infilling (Case 2) shows that the PC grout gradually came close to the atmospheric-pressure situation from the time it reached the end part. This estimates that vacuum suction only at the end part does not sufficiently obtain the combined effect of a vacuum pump at the bending-down part.
3.3 The surface porosity of the PC grout

Table 2 shows the void rate of super-low viscosity PC grout in the sheath tube. To obtain the void rate for each case, the void area formed in the sheath tube is divided by the cross-sectional area of the tube. The void area is the mean value of 10 values obtained between the two intermediate parts. The largest void rate is 2.7% when the vacuum suction was conducted only at the end part (Case 2); the void rate was 1.1% when no vacuum suction was conducted (Case 1). On the other hand, no void occurred when the vacuum sanction was conducted at the intermediate parts as well as at the end part (Case 3). From these results, it can be clearly seen that the super-low viscosity PC grouting is an effective execution method of enhancing the PC grout infilling performance when the vacuum suction was conducted at both the end part and intermediate parts, and that the vacuum suction only at the end part increases more risk of infilling failure than no vacuum suction at all.

The results above suggest that the super-low viscosity PC grout infilling with vacuum suction conducted at the end and intermediate parts of the sheath tube enables efficient and reliable grouting, and it is a sustainable technology to ensure the safety and serviceability of a PC structure.

Table 2: The surface porosity of the PC grout

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vacuum suction</td>
<td>No vacuum suction</td>
<td>End part</td>
<td>End part and intermediate parts</td>
</tr>
<tr>
<td>Void situation</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
<td><img src="image3.png" alt="Image" /></td>
</tr>
<tr>
<td>Void rate</td>
<td>1.1%</td>
<td>2.7%</td>
<td>0%</td>
</tr>
</tbody>
</table>
4 ENVIRONMENTAL ASSESSMENT

This study calculates the amount of CO₂ emissions during the material production and execution for the three methods mentioned above. The inventory data used are from “Environment text of concrete (Draft) [4].” Table 3 indicates the unit-based CO₂ emissions in this study.

Table 3: CO₂ emission unit

<table>
<thead>
<tr>
<th>Items</th>
<th>Unit-based CO₂ emission</th>
</tr>
</thead>
<tbody>
<tr>
<td>Production stage</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>0.348kg-CO₂/m³</td>
</tr>
<tr>
<td>Cement</td>
<td>769kg-CO₂/t</td>
</tr>
<tr>
<td>PC steel</td>
<td>1320kg-CO₂/t</td>
</tr>
<tr>
<td>Sheath tube</td>
<td>0.91kg-CO₂/kg</td>
</tr>
<tr>
<td>Execution stage</td>
<td></td>
</tr>
<tr>
<td>Electricity</td>
<td>0.41 kg-CO₂/kWh</td>
</tr>
</tbody>
</table>

4.1 CALCULATION METHOD

Table 4 shows the consumption amount of materials and electricity. The consumption amount of materials is calculated from materials necessary for filling the inside of a 33.6ℓ sheath tube (shown in Figure 1) with PC grout, a 10.8m PC steel, and a sheath tube. Chemical admixtures are excluded from the calculation objects.

The consumption amount of electricity during the execution is calculated from a grout mixer (200V, 14.8A), a filling pump (200V, 16.4A), and a vacuum pump (200V, 6.5). In the Case 2 method, the vacuum pump was manually stopped because PC grout already reached the end part due to the flow-ahead phenomenon when a third amount of PC grout was infilled; while the vacuum pump, in the Case 3 method, was operated from the start of the PC grout infilling through the finish.

Table 4: Consumption amount of materials and electricity

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Consumption amount of materials</th>
<th>Consumption amount of electricity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water (m³)</td>
<td>Cement (t)</td>
</tr>
<tr>
<td>Case 1</td>
<td>0.020</td>
<td>0.052</td>
</tr>
<tr>
<td>Case 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.2 CO₂ EMISSION

Table 5 shows the amount of CO₂ emission for each method. The total amount of CO₂ emission occurring during the material production and the execution was within the range of 236.052kg to 236.074kg, and over 99% of the total amount occurred during the material production. Clearly, the use of vacuum pump had an extremely small effect on CO₂ emission amount.

Figure 6 and Figure 7 show a breakdown of the amount of CO₂ emitted during the material
production and the execution in the Case 3 method. Approximately 80% of the CO₂ emission amount during the material production was emitted from PC steel, while approximately 51% of the amount during the execution was from filling pumps.

Table 5: CO₂ emission

<table>
<thead>
<tr>
<th>Case No.</th>
<th>CO₂ emission of production stage (kg)</th>
<th>CO₂ emission of execution stage (kg)</th>
<th>Total (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water</td>
<td>Cement</td>
<td>PC steel</td>
</tr>
<tr>
<td>Case 1</td>
<td>0.007</td>
<td>39.938</td>
<td>187.608</td>
</tr>
<tr>
<td>Case 2</td>
<td>0.007</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td>0.022</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5: CO₂ emission breakdown (production stage)  Figure 6: CO₂ emission breakdown (execution stage)

5 CONCLUSIONS

This study concludes as follows:
- Super-low viscosity PC grout causes a flow-ahead phenomenon regardless of the presence and position of vacuum suction. In addition, the presence or absence of vacuum suction does not affect the flow direction of super-low viscosity PC grout.
- When vacuum pumps are also used with the infilling of super-low viscosity PC grout, vacuum suctioning only at the end part of the sheath tube creates a large void around the start position of the bending-down part.
- When super-low viscosity PC grout is infilled with the use of vacuum pumps, vacuum suctioning at both the end part and the two intermediate parts enables to significantly reduce the size of a void in the sheath tube, compared with vacuum suctioning only at the part.
- In the super-low viscosity PC grout infilling execution, most CO₂ is emitted during the material production. In addition, the use of vacuum pumps has an extremely small
effect on the amount of CO₂ emission.

REFERENCES


TRC MULTILAYER PRECAST FAÇADE PANEL: STRUCTURAL BEHAVIOR IN FREEZING-THAWING CONDITION

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Key words: textile reinforced concrete (TRC), sandwich beams, freezing-thawing cycles, durability, tensile behavior, bending behavior

Abstract. This paper presents an experimental investigation carried out at Politecnico di Milano in order to study the mechanical behavior of both textile reinforced concrete (TRC) and textile reinforced concrete sandwich solution when exposed to an increasing number of freezing and thawing cycles (up to 500, +4/-18°C) considering both un-cracked and pre-cracked initial conditions. An analytical model that accounts material non-linearity was used to investigate the effect of freezing and thawing on the sandwich behavior. Concerning sandwich solution, experimental results showed a negligible effect on both initial stiffness and maximum bearing capacity of the solution, but a significant reduction of the ductility.

1 INTRODUCTION

In Europe, about 40% of the total energy consumption is due to residential and commercial buildings; in particular, the 70% of this energy is used for heating. To significantly reduce this consumption, it is necessary to energy retrofit existing buildings, because of their large impact on the phenomenon.

In this perspective, in the framework of a European project to improve sustainability in existing buildings (EASEE), a multi-layer prefabricated façade panel is proposed (maximum dimension 1.50 x 3.30 m²). The panel is characterized by an internal expanded polystyrene layer (EPS), 100 mm thick, used to transfer distributed shear stresses, and by two external layers made of textile reinforced concrete (TRC) 10 mm thick. The main advantages of the solution are: the low impact on occupant life, the possibility to obtain the desired finishing and a good durability. This latter aspect is particularly important, especially considering a residual expected building life of at least 30 years. Durability deals both with winter and summer conditions.

In the paper, the behavior of the adopted sandwich solution when subjected to freezing and thawing cycles is investigated, while further investigations concerning the effect of sun radiation are still in progress. Freezing and thawing can affect the cement paste [1, 2, 3, 4] and, therefore, the bond between fabric and mortar, which is the main mechanism governing the tensile behavior of TRC. In addition, problems can occur at the interface between TRC...
and EPS. Among cement based composites, few investigations of freezing-thawing behavior can be found for fiber reinforced concrete [5, 6], and no information is available for the mechanical performances of TRC in these conditions.

The paper focuses on an experimental investigation carried out at Politecnico di Milano in order to study the mechanical behavior of sandwich beams when exposed to an increasing number of freezing and thawing cycles (up to 500, +4/-18°C) considering both un-cracked and pre-cracked initial conditions. Previous tests were performed on specimens made of similar TRC in order to evaluate how the exposure to freezing-thawing conditions could affect the tensile behavior of external layer of the sandwich. The results of that campaign are summarized in Figure 1 and are presented and deeply discussed in [7]. These results showed that two different phenomena seem to govern the behavior of TRC: the damage due to thermal cycles, and the matrix self-healing and late hydration due to the permanence in water of the material.

![Figure 1](image-url): Tensile behavior of TRC specimens subjected to freezing-thawing cycles: nominal stress vs. normalized displacement curves and crack pattern for un-cracked (a, b) and pre-cracked (c, d) specimens. [7]
2 EXPERIMENTAL PROGRAM

The experimental campaign investigates the behavior of sandwich beams tested according to a four point bending scheme after the exposure to an increasing number of freezing-thawing cycles. The beams are characterized by two external TRC layers, 10 mm thick, connected by an insulation layer of expanded polystyrene foam 100 mm thick. The beam has a nominal size equal to 550 x 150 x 120 mm³.

2.1 Materials

The expanded polystyrene foam used is commercially known as EPS250 and is characterized by a compressive strength of 0.25 MPa reached at a strain equal to 10% [8]. According to experimental results, the behavior in compression is elasto-hardening, with a Young modulus - estimated as the initial slope of the σ-ε curve - equal to 13.7 MPa. The uniaxial tensile behavior is elasto-brittle, with a higher strength in tension rather than in compression; the measured average tensile strength is equal to 0.39 MPa. Concerning shear behavior, the maximum shear stress measured is equal to 0.16 MPa, and the shear modulus – defined as the initial slope of the τ-γ curve – is equal to 5.04 MPa.

TRC is obtained reinforcing a high strength fine grain mortar with an alkali-resistant glass fabric, manufactured by means of a leno-weave technique and covered with an epoxy coating. The fabric used as reinforcement, whose geometrical and mechanical characteristics are collected in [9], is characterized by a nominal strength in the warp direction of 820 MPa. The warp of the glass fabric is aligned with the beam longitudinal direction.

The cementitious matrix used is characterized by a water to binder ratio equal to 0.225 and by a superplasticizer to cement ratio equal to 9.3%. The maximum aggregate size selected is equal to 1 mm. These properties guarantee a high flow capability and, hence, a good bond between matrix and fabric and the possibility to cast the mortar in pressure. The in-pressure casting technique is adopted in order to minimize the voids (defects) in the mortar and to enhance the bond between TRC layers and EPS, also because the insulating material alone is used to transfer the shear between the external TRC layers. At 28 days, the mortar is characterized by a cubic compressive strength equal to 73 MPa obtained as an average value on four nominally identical specimens. At about 150 days, the cubic compressive strength measured testing 32 specimens is found to be 95.1 MPa (STD = 5.8%); a similar enhancement in the mortar mechanical properties was also found by Brameshuber et al. [10].

2.2 Curing and thermal treatment

48 hours after the in-pressure casting, specimens were demoulded and then cured in air for at least 28 days. The air condition was that of a laboratory with temperature ranging between 18 and 22 °C during winter and between 22 and 27 °C during summer; the related relative humidity ranges respectively between 40-60% and 50-80%.

Afterwards, the specimens were treated in a climatic chamber following a thermal cycle described in procedure A of ASTM International C666/C 666M [11]. The range of temperature varied between +4 and -18°C with both cooling and heating rate equal to 11 °C/h and a 30 minutes rest phase both at +4°C and at -18°C. Only one TRC layer of each specimen was immersed in water, icing and de-icing during thermal cycles (Figure 2(a)); this choice is due to the fact that, in a real panel applied on a façade, only the external TRC layer is exposed.
to rain. Specimens exposed to 150 and 500 cycles are considered; in particular, the following scenarios are taken into account: un-cracked specimens subjected to 150 cycles (2 nominally identical specimens); un-cracked specimens subjected to 500 cycles (3 nominally identical specimens); pre-cracked specimens subjected to 500 cycles (pre-cracking phase performed before the exposure to freezing-thawing cycles, 3 nominally identical specimens).

As reference, specimens not treated with freezing-thawing cycles are taken into account (see a detailed description of the experimental campaign and results in [9]).

Each specimen is identified through the following notation: letter "S", that indicates that small sandwich beams are involved (according to [9]); a letter, that specifies if the specimen was un-cracked (U) or pre-cracked (C) when thermally treated; a number, that stands for the number of cycles the specimen was exposed and another number that denotes the nominally identical specimens (e.g. S_U500_2 stands for small sandwich beam, un-cracked specimen treated through 500 cycles, specimen number 2).

![Figure 2. Sandwich beam: set-up during freezing-thawing cycles (a) and during tests (b).](image)

### 2.3 Test set-up

Once performed the thermal cycles, specimens are tested according to a four point loading scheme considering the test set-up shown in Figure 2(b). An electromechanical press INSTRON 5867 with a maximum load capacity of 30 kN is used. The sandwich beams are loaded keeping face down the TRC layer previously immersed in water.

In the case of pre-cracked specimens, this TRC layer is also the one that presented cracks due to pre-cracking. Hence, this loading condition is the most critical as the most damaged TRC layer (immersed in water during cycles and also pre-cracked in "C" specimens) is loaded in tension. The pre-cracking phase is performed using the same test set-up adopted for the four point loading tests (Figure 2(b)) and loading the beams up to a load equal to 3.5 kN. This load was selected considering that, at the Serviceability Limit State, a maximum wind pressure equal to 1.50 kN/m² could act on the panel (according to [12], assuming a building 30 m tall, placed in an area characterized by unfavorable wind condition - zone 7). Assuming a panel 1.5 m wide and 3 m high and considering the elastic solution proposed by Bares [13] for a plate supported on four points at the corners, the maximum acting specific moment is equal to 1.76 kNm/m at the corner. In order to apply on the specimen cross-section the same bending moment, a pre-cracking load equal to 3.5 kN is computed considering the specimen
width (0.15 m) and the set-up in Figure 2(b) characterized by a lever arm of 0.15 m. During pre-cracking, only one crack formed in each specimen; this crack was not visible by visual inspection, thus indicating that its width is lower than 50 μm.

The tests are displacement controlled and are performed by imposing a machine cross-head displacement (stroke) rate that is initially equal to 1e-3 mm/s and then, after cracking, is increased up to 4e-3 mm/s.

3 EXPERIMENTAL RESULTS

The results of the experimental campaign are collected in Figure 3 in terms of load versus stroke curves respectively for un-treated specimens (a), un-cracked specimens tested after 150 freezing-thawing cycles (b), un-cracked specimens tested after 500 freezing-thawing cycles (c) and pre-cracked specimens tested after 500 freezing-thawing cycles (d). In Figures 3(a), 3(c) and 3(d) a good repeatability is obtained, as the curves are characterized by a relatively low scatter. Only in Figure 3(b), a larger scatter is observed; however, the average behavior can be superimposed to the curves of un-treated specimens (Figure 3(a)).

Figure 3. Behavior of sandwich beams subjected to four-point load test: un-treated beams (a), un-cracked beams after 150 cycles (b), un-cracked beams after 500 cycles (c) and pre-cracked beams after 500 cycles (d).
Comparing the load-stroke response of specimens subjected to freezing-thawing cycles with the behavior of un-treated specimens, it is clear that the exposure to freezing and thawing phenomenon affects mainly the ductility of the beams rather than the performance in terms of initial elastic global response and maximum load reached. The pre-cracking does not seem to affect the sandwich beam behavior.

In all cases, the multi-cracking phenomenon is experienced by TRC layers; the multi-cracking of the bottom TRC layer is also evidenced by the high crack opening displacement measured. Looking both at the curves shown in Figure 3 and at the crack patterns, it is possible to note that both the global behavior and the failure mechanisms of treated specimens are comparable with those of specimens not subjected to freezing-thawing cycles.

Looking at the graphs, it is clear that, up to a displacement of 45 mm, the behavior is not affected by freezing-thawing phenomenon. These results are extremely important as they demonstrate that the panel is not expected to show a loss of performance due to freeze-thaw attack at Serviceability Limit State.

To investigate the influence of thermal cycles on the sandwich beam response, the variation with cycle number of the main parameters that characterize the behavior is shown in Figure 4. In all the graphs, each un-filled marker represents the value assumed by the considered parameter for each specimen, normalized by the average value registered in the zero cycles case; the average of these points is represented by filled markers. Circular markers refer to un-cracked specimens, while rhomboid markers refer to pre-cracked specimen. The considered parameter are: - the maximum load achieved ($P_{\text{max}}$); the corresponding displacement ($\text{stroke}_0$); the secant bending stiffness ($EI$) defined as the slope of the bending moment vs. nominal curvature curves at a moment level equal to 0.3 kNm and the percentage variation of the mass after the exposure to the thermal cycles ($\Delta m/m_i$; $\Delta m =$ difference between the final and the initial mass, $m_i =$ initial mass).

The latter data are available only in the case of 500 cycles. It is worth noting that the mass variation is measured by weighing specimens after cycles: it includes the water absorption occurred just from the bottom TRC layer, that was immersed in water during the thermal treatment. Looking at subfigure (b) it is confirmed that the exposure to freezing-thawing cycles affects mainly the ductility of the specimens: in fact, a reduction of the normalized ultimate displacement down to 0.5 is registered when the thermal treatment is performed.

On the other hand, other parameters such as the maximum load achieved (subfigure (a)) and the bending stiffness measured (subfigure (c)) are scantily affected by freezing-thawing phenomenon: minimum normalized value registered equal to 0.86 and 0.76 respectively.

It is interesting to observe a stiffness recovery passing from 150 to 500 cycles, thus indicating a possible activation of self-healing and late hydration phenomena due to a longer permanence in water. This bond strength recovery was also observed in TRC specimens treated with freezing-thawing cycles and tested in tension (Figure 1(c)).

It is also interesting to note that, in case of 500 cycles, the bending stiffness of pre-cracked (C) specimens is even slightly higher than the bending stiffness of un-cracked (U) specimens, thus confirming the activation of self-healing and late hydration phenomena, made easier by the presence of cracks during the thermal cycles, which facilitated the penetration of water. The larger water absorption of pre-cracked specimens, which is partially related to self-healing, is also clearly visible in the mass growth shown in Figure 4(d).
Isabella G. Colombo, Matteo Colombo and Marco di Prisco

Figure 4. Normalized sandwich beam parameters: (a) peak load vs. number of cycles, (b) ultimate displacement vs. number of cycles, (c) bending stiffness vs. number of cycles and (d) mass variation vs. number of cycles

2 ANALYTICAL MODELING OF EXPERIMENTAL RESULTS

The analytical model proposed by Stamm and Witte [14] for sandwich beams is used to model the specimen behavior. Non-linear behavior of materials is introduced following the approach proposed by Shams et al. [15]. An elasto-perfectly plastic (τ-γ) relationship is adopted for EPS; the values of shear modulus and yielding shear stress were determined through experimental tests performed on polystyrene specimens and are respectively equal to 5.04 MPa and 0.16 MPa. The beam is divided into a finite number of elements characterized by the same length and both axial (EA) and bending (EI) stiffness are assigned to each element accounting non-linearity according to a secant stiffness approach. The differential equations of Stamm and Witte’s model are solved by using constant and homogeneous equivalent stiffness properly defined for each layer. The flow chart of the analytical model is proposed in Figure 5.
At the beginning, the initial elastic stiffness of both TRC layers and the core initial shear modulus $G$ are imposed. The load, initially null, is increased by $\Delta P$ and the values of the vertical displacement $w$, the rotation $\gamma$, the bending moments $M_s$, $M_o$ and $M_u$ are computed according to Stamm and Witte’s model as a function of the coordinate $x$ along the beam length. Subscript “s” refers to the sandwich action, while “o” and “u” refer respectively to the top and the bottom TRC layer.

Once the material behavior becomes non-linear, the stiffness of each element are computed by means of a secant approach referring to generalized constitutive laws ($M-\varphi$ and $N-\varepsilon$) for the TRC layers and to a shear ($\tau-\gamma$) constitutive relationship for the EPS. Stamm and Witte’s differential equations ask for unique stiffness for each layer; in order to define equivalent stiffness to be used, the local stiffness evaluated for each element are weighted according to different response parameters. In particular, equivalent values of axial and bending stiffness of TRC are evaluated by using the vertical displacement $w$ as weighting function, while equivalent shear modulus of EPS considers shear deformation as weighting function. In Figure 6(a) the analytical responses are compared with the average experimental curves of un-treated sandwich specimens (“S_av”), un-cracked beams tested after 150 cycles (“S_U150_av”), un-cracked beams tested after 500 cycles (“S_U500_av”) and pre-cracked specimens tested after 500 cycles (“S_C500_av”).

In particular, two analytical curves are plotted: “S_S&W_0” refers to un-treated specimens, while “S_S&W_U500” refers to specimens tested after 500 freezing-thawing cycles. The latter takes into account the effect of freezing and thawing on textile reinforced concrete and is obtained penalizing first cracking strength, initial and final stiffness, peak strength and peak normalized displacement of TRC layers according to the results presented.
in [7]. In Figure 6(b) the analytical load-displacement prediction obtained for a full-scale panel subjected or not to freezing-thawing cycles is shown (“R_S&W_U500” and “R_S&W_0” curve respectively). A simply supported 3 x 1.5 m² panel with the same stratigraphy of the sandwich beams is taken into account; the lever arm is assumed equal to 1.1 m. Looking at Figure 6 it is possible to state that the reduction of TRC properties due to freezing-thawing cycles does not affect the global response of sandwich beams, whose behavior is governed by the shear deformability of EPS. On the other hand, this reduction has great influence on the failure load of full-scale panels, in which bending action is more relevant than shear (reduction of about 20% of the peak load after 500 cycles).

It is worth to note that the analytical results for sandwich beams are not able to catch the reduction of ductility due to thermal treatment (Figure 6(a)). Taking into account the negligible effect of freezing and thawing on the behavior of EPS [17], it is possible to assume that this reduction is related to a decay of bond properties at the TRC/EPS interface.

A degradation of the interface could also affect the behavior of a full-scale panels exposed to freezing and thawing cycles, thus determining a global response that stops at a peak load lower than 30 kN (see “R_S&W_U500” curve, Figure 6(b)).

![Figure 6. Comparison between load-displacement curves: analytical and experimental average curves for small beam (a) and analytical prediction for a full-scale panel (b).](image_url)

**5 CONCLUSIONS**

Basing on experimental and analytical results shown in the paper, it is possible to draw the following conclusions:
- freezing-thawing phenomenon does not affect initial stiffness and maximum bearing capacity of the sandwich solution, while a significant reduction of ductility is observed;
- the stiffness recovery and mass growth observed for specimens treated with 500 cycles indicate a possible activation of self-healing and late hydration phenomena due to a long permanence in water;
- the performance decay experienced by TRC layers (Figure 1) does not affect the
mechanical sandwich behavior for small element, mainly subjected to shear action; the ductility reduction seems to be related to an interface degradation mainly due to the specimen permanence in water during cycles.

REFERENCES
ALKALI-SILICA RESISTANCE OF COAL BOTTOM ASH MORTARS

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Key words: Bottom ash, fly ash, cement constituent, alkali-silica reaction, durability.

Abstract. It is well-known the efficiency of the fly ash as cement constituent thanks to many studies carried out in the past. However, its small amount produced currently leads to a more limited use. In these days, coal bottom ash is dumped in many cases as a waste generating an environmental problem. This paper contributes to improve the knowledge about this material and check its viability in order to be used mixed with fly ash or alone as a new Portland cement constituent.

In order to check the durability of ground coal bottom ash when is part of a Portland cement, some alkali-silica reaction tests were performed. These test methods provide a means of assessing the potential alkali reactivity of mortars made using blended hydraulic cements.

Blended cements with coal bottom ash, coal fly ash or a mix of them presented a better resistance in this type of durability tests with regard to the reference mortars without any addition.

Summing up, it can be highlighted that coal bottom ash could be used in common Portland cements alone or mixed in an optimized percentage with fly ash.

1 INTRODUCTION

An important aspect of concrete durability its resistance to the alkali-silica reaction (ASR) which is a deleterious process. The alkali–silica reaction (ASR) is a chemical reaction that is produced between amorphous silica present in some reactive aggregates and sodium, potassium and hydroxyl ions, among others present in the concrete pore solution. The alkali-silica reaction (ASR) produces a gel which expands with the absorbed water. This reaction evolves in some steps [1, 2, 3]. The first step consists of the scission of siloxane bonds (Si-O-Si) in aggregate by hydroxyl ions to generate alkali silicate gel and silicic acid (silanol bonds).

$$\equiv Si-O-Si \equiv + R^+ + OH^- \rightarrow \equiv Si-O-R \equiv + \quad H-O-Si \equiv$$ (1)
R$^+$ is the alkali ion, sodium or potassium, i.e. Na$^+$ and K$^+$. The silicic acid reacts with further hydroxyl ions forming an amorphous and hygroscopic alkali-silica gel.

$$= Si-O-H + R^+ + OH^- \rightarrow = Si-O-R = + H_2O$$

(2)

The second step is the alkali-silica gel expansion due to the water absorption. The expansive pressure generated by the hydrated alkali silicate promotes aggregate cracking and, therefore, the surrounding concrete.

$$= Si-O-R + nH_2O \rightarrow = Si-O -(H_2O)_n + R^+$$

(3)

The hydrated alkali-silica gel diffuses away from the aggregate surface into concrete and cement paste pores. It also can react with Ca$^{2+}$ present in the cement paste to form an alkali-calcium hydrate gel as shown in equation 4. Thus, the alkali-silica reaction produces two types of gel component: non-swelling calcium-alkali silicate-hydrate and swelling alkali-silica-hydrate gels.

$$= Si-O -(H_2O)_n + R^+ + \frac{1}{2}Ca^{2+} \rightarrow = Si-O + \frac{1}{2}Ca^{2+}+(H_2O)_n + R^+$$

(4)

It is well-known that at least three conditions have to be satisfied for the concrete alkali-silica reaction (ASR) to take place. First, an amorphous silica aggregate must be in the concrete, second, a high alkali amount in the pore solution and third a high humidity level in the environment much be maintained along the time in order to ensure more than 80% relative humidity in the concrete pores [4].

Some supplementary cementitious materials (SCMs) are quite effective in mitigating ASR damage [5, 6]. Supplementary cementitious materials (SCMs) mitigate ASR, among other effects, reducing the concentration of alkalis in the pore solution by replacing cement or by alkali binding of the additional pozzolanic C-S-H. The C–S–H gel produced in the pozzolanic reaction has a lower C/S than the C–S–H gel formed in the Portland cement hydration, therefore a higher alkali binding capacity. Some authors have reported that the most efficient supplementary cementitious materials (SCMs) with regard to reducing the alkali-silica reaction expansion are silica fume and metakaolin, followed by low calcium fly ash and blast-furnace slag [7, 8].

The presence of Al in C–S–H gel can improve the alkali binding capacity of C-S-H gel as suggested by Hong and Glasser [9]. The origin of the aluminum in the pore solution could be explained because the alumina present in coal fly ash could be dissolved into the pore solution and further incorporated into the C-S-H gel by means of the cement hydration and pozzolanic reactions to form a C–A–S–H gel. By contrary, the presence of aluminum in the pore solution could reduce the silica dissolution rate [10, 11], because the aluminum can be adsorbed on the silica surfaces forming an alumino-silicate layer [12, 13].

Another mechanism by which some supplementary cementitious materials (SCMs) can suppress expansion due to alkali silica reaction is the reduction of concrete permeability and diffusivity due to the fly ash pozzolanicity and the modification of ASR gel properties. In particular, high CaO/Na$_2$O$_e$ gels have lower swelling capacity than low CaO/Na$_2$O$_e$ gels [14]. Therefore, coal fly ash can reduce the swelling capacity of concrete alkali-silica reaction.
2 EXPERIMENTAL DETAILS

2.1. Raw materials

A CEM I 42.5 N cement according to the European standard EN 197-1:2011 [15] was used to prepare some blended cements made of coal fly ash and coal bottom ash. A German standard sand (NORMSAND) and distilled water were used to make testing mortars. A 5% of the normalized sand was replaced by an opal aggregate (from Segovia, Spain) with 4.5-5mm size.

Coal fly ash (CFA) and coal bottom ash (CBA) were provided by ENDESA-Carboneras. They came from South African hard coal (90%) and Colombian hard coal (10%). Coal bottom ash was ground to a fineness similar to that of the Portland cement (5-8% residue on 45 µm sieve). Coal fly ash was used as it was provided. The chemical composition of the coal fly ash, coal bottom ash and reference cement is shown in Table 1.

Table 1: Chemical compositions of raw material: Cement CEM I 42.5 N, coal bottom ash (CBA) and coal fly ash (CFA) (as percentage).

<table>
<thead>
<tr>
<th>Chemical Contents (%)</th>
<th>Cement</th>
<th>CBA</th>
<th>CFA</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.R.</td>
<td>2.15</td>
<td>81.24</td>
<td>76.00</td>
</tr>
<tr>
<td>L.O.I.</td>
<td>3.93</td>
<td>1.85</td>
<td>3.63</td>
</tr>
<tr>
<td>SO₃</td>
<td>3.14</td>
<td>0.15</td>
<td>0.37</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>3.85</td>
<td>25.55</td>
<td>26.66</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.43</td>
<td>5.86</td>
<td>4.72</td>
</tr>
<tr>
<td>CaO</td>
<td>57.16</td>
<td>7.07</td>
<td>5.55</td>
</tr>
<tr>
<td>MgO</td>
<td>1.54</td>
<td>1.28</td>
<td>1.33</td>
</tr>
<tr>
<td>SiO₂</td>
<td>19.04</td>
<td>48.12</td>
<td>46.84</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.17</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>0.07</td>
<td>0.96</td>
<td>1.03</td>
</tr>
</tbody>
</table>

*Insoluble residue was determined by the Na₂CO₃ method (EN 196-2:2005).

2.2 Cement design

A Portland cement (CEM I 42.5 N) was partially replaced by coal fly ash, coal bottom ash or mixes of them in order to prepare cements with similar compositions to CEM II/A-V, CEM II/B-V and CEM IV/A (V) standardized cements [15]. Coal fly ash and coal bottom ash mixes were coded as β, γ, δ, λ and Ω as shown in Table 2. The reference cement without any addition is coded as α.

The properties of these prepared cements were compared to those of CEM I 42.5 N. Chemical analyses of SiO₂, Al₂O₃, Fe₂O₃, CaO, MgO, SO₃, TiO₂, P₂O₅, K₂O and Na₂O were performed by XRF with a Bruker S8 Tigger 4kW model. Loss on ignition (LOI) and insoluble residue (IR) were determined according to EN 196-2:2005 [16, 17].

Table 3 shows the chemical composition determined by XRF of manufactured cements CEM II/A-V, CEM II/B-V y CEM IV/A (V), compared with the reference cement CEM I 42.5 N α. The alkalis in the elaborated cements are represented as Na₂Oₑ content (Na₂Oₑ)=Na₂O + 0.658*K₂O.

When the amount of coal ashes in cement increases, the CaO content decreases. Coal
bottom ash cements have a higher content of CaO than coal fly ash cements. As expected, the more amount of coal fly ash or coal bottom ash in the cement, the higher SiO2 content.

### Table 2: Coal bottom ash (CBA), coal fly ash (CFA) and cement (CEM I 42,5 N) mixes.

<table>
<thead>
<tr>
<th>Cement</th>
<th>Raw material</th>
<th>Coal fly ash + coal bottom ash mix codification</th>
<th>α</th>
<th>β</th>
<th>γ</th>
<th>δ</th>
<th>λ</th>
<th>Ω</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I</td>
<td>Fly ash</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom ash</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>100%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/A-V</td>
<td>Fly ash</td>
<td>10%</td>
<td>9%</td>
<td>8%</td>
<td>5%</td>
<td>0%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom ash</td>
<td>0%</td>
<td>1%</td>
<td>5%</td>
<td>10%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>90%</td>
<td>90%</td>
<td>90%</td>
<td>90%</td>
<td>90%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V</td>
<td>Fly ash</td>
<td>25%</td>
<td>22.5%</td>
<td>20%</td>
<td>12.5%</td>
<td>0%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom ash</td>
<td>0%</td>
<td>2.5%</td>
<td>5%</td>
<td>12.5%</td>
<td>25%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>75%</td>
<td>75%</td>
<td>75%</td>
<td>75%</td>
<td>75%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A (V)</td>
<td>Fly ash</td>
<td>35%</td>
<td>31.5%</td>
<td>28%</td>
<td>17.5%</td>
<td>0%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom ash</td>
<td>0%</td>
<td>3.5%</td>
<td>7%</td>
<td>17.5%</td>
<td>35%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cement</td>
<td>65%</td>
<td>65%</td>
<td>65%</td>
<td>65%</td>
<td>65%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 3: Chemical composition determined by XRF of manufactured cements CEM II/A-V, CEM II/B-V y CEM IV/A (V), compared with the reference cement CEM I 42,5 N α and Na2Oe calculation.

<table>
<thead>
<tr>
<th>Cements</th>
<th>CaO (%)</th>
<th>SiO2 (%)</th>
<th>Al2O3 (%)</th>
<th>Fe2O3 (%)</th>
<th>SO3 (%)</th>
<th>MgO (%)</th>
<th>TiO2 (%)</th>
<th>P2O5 (%)</th>
<th>K2O (%)</th>
<th>Na2O (%)</th>
<th>Na2Oe (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I-α</td>
<td>65.98</td>
<td>16.07</td>
<td>4.07</td>
<td>2.23</td>
<td>3.88</td>
<td>3.22</td>
<td>0.22</td>
<td>0.11</td>
<td>1.08</td>
<td>0.32</td>
<td>1.03</td>
</tr>
<tr>
<td>CEM II/A-V β</td>
<td>57.73</td>
<td>20.69</td>
<td>8.34</td>
<td>2.61</td>
<td>3.83</td>
<td>2.63</td>
<td>0.50</td>
<td>0.22</td>
<td>1.10</td>
<td>0.34</td>
<td>1.06</td>
</tr>
<tr>
<td>CEM II/A-V γ</td>
<td>59.13</td>
<td>20.04</td>
<td>7.57</td>
<td>2.57</td>
<td>3.93</td>
<td>2.78</td>
<td>0.45</td>
<td>0.19</td>
<td>1.13</td>
<td>0.34</td>
<td>1.08</td>
</tr>
<tr>
<td>CEM II/A-V δ</td>
<td>58.34</td>
<td>20.38</td>
<td>7.84</td>
<td>2.60</td>
<td>3.95</td>
<td>2.75</td>
<td>0.46</td>
<td>0.22</td>
<td>1.13</td>
<td>0.34</td>
<td>1.08</td>
</tr>
<tr>
<td>CEM II/A-V λ</td>
<td>58.69</td>
<td>20.16</td>
<td>7.75</td>
<td>2.69</td>
<td>3.90</td>
<td>2.71</td>
<td>0.47</td>
<td>0.21</td>
<td>1.10</td>
<td>0.32</td>
<td>1.04</td>
</tr>
<tr>
<td>CEM II/A-V Ω</td>
<td>58.19</td>
<td>20.13</td>
<td>7.58</td>
<td>2.71</td>
<td>3.84</td>
<td>2.73</td>
<td>0.50</td>
<td>0.19</td>
<td>1.06</td>
<td>0.31</td>
<td>1.01</td>
</tr>
<tr>
<td>CEM II/B-V β</td>
<td>48.84</td>
<td>25.15</td>
<td>12.33</td>
<td>2.94</td>
<td>3.35</td>
<td>2.37</td>
<td>0.76</td>
<td>0.36</td>
<td>1.09</td>
<td>0.32</td>
<td>1.04</td>
</tr>
<tr>
<td>CEM II/B-V γ</td>
<td>49.06</td>
<td>25.14</td>
<td>12.24</td>
<td>2.87</td>
<td>3.38</td>
<td>2.46</td>
<td>0.74</td>
<td>0.35</td>
<td>1.05</td>
<td>0.32</td>
<td>1.01</td>
</tr>
<tr>
<td>CEM II/B-V δ</td>
<td>49.52</td>
<td>24.91</td>
<td>11.97</td>
<td>2.90</td>
<td>3.37</td>
<td>2.47</td>
<td>0.73</td>
<td>0.34</td>
<td>1.05</td>
<td>0.30</td>
<td>0.99</td>
</tr>
<tr>
<td>CEM II/B-V λ</td>
<td>49.51</td>
<td>25.11</td>
<td>11.77</td>
<td>3.04</td>
<td>3.32</td>
<td>2.48</td>
<td>0.79</td>
<td>0.33</td>
<td>1.01</td>
<td>0.28</td>
<td>0.94</td>
</tr>
<tr>
<td>CEM II/B-V Ω</td>
<td>51.45</td>
<td>24.05</td>
<td>10.80</td>
<td>3.17</td>
<td>3.41</td>
<td>2.48</td>
<td>0.75</td>
<td>0.28</td>
<td>0.99</td>
<td>0.29</td>
<td>0.94</td>
</tr>
<tr>
<td>CEM IV/A(V) β</td>
<td>41.41</td>
<td>29.10</td>
<td>15.84</td>
<td>3.09</td>
<td>2.92</td>
<td>2.17</td>
<td>0.97</td>
<td>0.48</td>
<td>1.06</td>
<td>0.31</td>
<td>1.01</td>
</tr>
<tr>
<td>CEM IV/A(V) γ</td>
<td>43.73</td>
<td>27.83</td>
<td>14.49</td>
<td>3.06</td>
<td>3.12</td>
<td>2.22</td>
<td>0.91</td>
<td>0.44</td>
<td>1.05</td>
<td>0.31</td>
<td>1.00</td>
</tr>
<tr>
<td>CEM IV/A(V) δ</td>
<td>43.00</td>
<td>28.25</td>
<td>14.93</td>
<td>3.15</td>
<td>3.04</td>
<td>2.19</td>
<td>0.95</td>
<td>0.47</td>
<td>1.01</td>
<td>0.31</td>
<td>0.97</td>
</tr>
<tr>
<td>CEM IV/A(V) λ</td>
<td>45.28</td>
<td>27.42</td>
<td>13.95</td>
<td>3.32</td>
<td>3.05</td>
<td>2.28</td>
<td>0.92</td>
<td>0.42</td>
<td>1.02</td>
<td>0.28</td>
<td>0.95</td>
</tr>
<tr>
<td>CEM IV/A(V) Ω</td>
<td>45.61</td>
<td>27.20</td>
<td>13.20</td>
<td>3.54</td>
<td>3.06</td>
<td>2.35</td>
<td>0.94</td>
<td>0.36</td>
<td>0.96</td>
<td>0.26</td>
<td>0.89</td>
</tr>
</tbody>
</table>

### 2.3 Alkali-silica reaction testing

The potential reactivity of aggregates was evaluated in mortar prisms according to UNE 146508:1999 EX [18] equivalent to ASTM C-1260-14 [19]. Three mortar prisms measuring 2.5 x 2.5 x 28.5 cm³ were prepared mixing 400 grams of the elaborated cement and 900 grams of aggregate using a water-cement ratio equal to 0.47 by mass.

Special moulds were used with a stainless steel gauge stud into both ends of the longitudinal section of the prism. The effective gauge length was 254 mm according to UNE 80113:2013 [20].
After being demoulded, the prisms were stored for twenty-four hours at 80±2°C immersed in water. Their initial lengths were measured. Then, the prisms are submerged into a 1N NaOH solution at 80±2°C for a further 14 days. Readings were taken from 2 to 90 days. Mortar expansion was calculated, and the average expansion of the three prisms after fourteen days of exposure is given as the result.

\[
\text{Expansión (\%)} = \frac{(L_n - L_0)}{L_c} \times 100
\]  

Where:
- \(L_n\) Length at test age.
- \(L_0\) Initial length after 24 hours in water at 80±2°C
- \(L_c\) Specimen length (according to the Spanish standard UNE 80113:2013, \(L_c=254\) mm).

After fourteen days, an expansion less than 0.10% is an indication of a non-reactive aggregate and cement combination; expansions over 0.20% indicate a potentially reactive aggregate and expansions between 0.10% and 0.20% are considered as non-defined results, therefore, prisms length measurements must be taken up to 28 days. When the expansion is greater than 0.20% after 28 days, the aggregate is considered potentially reactive.

2.4 Mortar microanalysis

The microstructure of mortars was analyzed by a scanning electron microscopy equipped with energy dispersive x-rays analysis (SEM/EDX) with a JEOL JSM-5400 model, equipped with three detectors, one of secondary electrons, another backscattered electron and another of dispersive energy EDX Oxford EDS Link model. Test samples were embedded in a low viscosity resin, cut and polished, in order to obtain a suitable observation surface.

3 RESULTS AND DISCUSSION

3.1. Alkali-silica reaction

The performance of mortars containing different cement mixtures against alkali-aggregate reaction was studied. In Figures 1 and 2 results of expansion up to 28 and 90 days are collected. In general, it is noted that the larger percentage of ash in cement less expansion occurs in mortars.

After 14 days, all the mortars having 35% of ash presented an expansion below 0.10%. In the case of the mortars having 25% of ash, only were below this limit mortars with the highest level of ash (80-100%), i.e. mortars CEM II/B-V γ, CEM II/B-V β y CEM II/B-V δ. Therefore, mortars with 35% fly ash, bottom ash or any mixture of both of them, are able to avoid the expansion due to the aggregate reactivity at 14 days of testing.

Mortars made of CEM II/B-V λ y CEM II/B-V Ω, presented expansion results between 0.10 y 0.20% at 14 days, and over 0.20% at 28-days. Thus, they do not have enough capacity to avoid the expansion produce by a potentially reactive aggregate.

All the mortars with 10% of ashes, CEM II/A-V, had expansion values over 0.20% at 14 days. Moreover, expansion was also measured at 90 days. Only 35% ash mortars remained expansion values below 0.20%. No differences between bottom ash and fly ash were recorded.
The use of bottom ash and coal fly ash cements reduces similarly the expansion with regard to the ordinary Portland cement without ash addition. The more coal bottom ash or coal fly ash amount in the cement, the less expansion in the mortar. Cement with a 10% of either bottom ash or fly ash (CEM II/A-V) reduce the mortar expansion 34% at 28 days and 29% at 90 days. Cement with a 25% of either bottom ash or fly ash (CEM II/B-V) reduce the mortar expansion 84% at 28 days and 71% at 90 days. Cement with a 35% of either bottom ash or fly ash CEM IV/A (V) reduce the mortar expansion 94% at 28 days and 84% at 90 days (Table 4). The expansion decrease is due to an effect beyond mere dilution since the expansion decrease is proportionally higher than the level of ash content in cement.
### Table 4: Expansion of different elaborated cements at 28 and 90 days.

<table>
<thead>
<tr>
<th>Cement Type</th>
<th>Expansion (%)</th>
<th>28 days</th>
<th>Expansion (%)</th>
<th>90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I-α</td>
<td>1.09</td>
<td>1.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/A-V β</td>
<td>0.72</td>
<td>0.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/A-V γ</td>
<td>0.68</td>
<td>0.81</td>
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</tr>
<tr>
<td>CEM II/A-V δ</td>
<td>0.80</td>
<td>1.02</td>
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</tr>
<tr>
<td>CEM II/A-V λ</td>
<td>0.90</td>
<td>1.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/A-V Ω</td>
<td>0.73</td>
<td>0.81</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V β</td>
<td>0.16</td>
<td>0.29</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V γ</td>
<td>0.13</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V δ</td>
<td>0.14</td>
<td>0.32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V λ</td>
<td>0.19</td>
<td>0.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM II/B-V Ω</td>
<td>0.18</td>
<td>0.39</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A(V) β</td>
<td>0.06</td>
<td>0.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A(V) γ</td>
<td>0.09</td>
<td>0.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A(V) δ</td>
<td>0.10</td>
<td>0.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A(V) λ</td>
<td>0.08</td>
<td>0.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEM IV/A(V) Ω</td>
<td>0.08</td>
<td>0.19</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 3.2. Microanalysis

Figures 3 and 4 show the composition of C-S-H gel in cement pastes at 28 and 90 days obtained by energy dispersive X-ray (EDX). The diagrams represent the Si/Ca ratio against the Al/Ca ratio. It is noted that the composition of C-S-H gel has a Ca/Si ratio ranging from 1.8 to 3.9. In particular, the values of Ca/Si ratio to the CEM I-α ranging from 2.5 to 3.0, the CEM II/A-V (about 2.7) and CEM II/B-V (about 2.5). Conversely, mortar pastes with higher ash content CEM IV/A (V) have much lower ratios, up to 1.8. This is due to the consumption of Ca(OH)$_2$ in the pozzolanic reaction and the dilution effect.

![Figure 3](image_url)

**Figure 3**: Effect of C-S-H gel composition in mortars at 28 and 90 days.
Table 5: Ca/Si and Al₂O₃/CaO composition of C-S-H gel of different elaborated cements at 28 and 90 days.

<table>
<thead>
<tr>
<th>Cement Code</th>
<th>CaO/SiO₂ C-S-H gel (28 days)</th>
<th>Al₂O₃/CaO C-S-H gel (28 days)</th>
<th>CaO/SiO₂ C-S-H gel (90 days)</th>
<th>Al₂O₃/CaO C-S-H gel (90 days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I-α</td>
<td>2.72</td>
<td>0.038</td>
<td>2.78</td>
<td>0.029</td>
</tr>
<tr>
<td>CEM II/A-V β</td>
<td>2.91</td>
<td>0.052</td>
<td>2.54</td>
<td>0.070</td>
</tr>
<tr>
<td>CEM II/A-V γ</td>
<td>2.71</td>
<td>0.081</td>
<td>2.68</td>
<td>0.072</td>
</tr>
<tr>
<td>CEM II/A-V δ</td>
<td>2.53</td>
<td>0.058</td>
<td>2.79</td>
<td>0.062</td>
</tr>
<tr>
<td>CEM II/A-V λ</td>
<td>2.44</td>
<td>0.057</td>
<td>2.91</td>
<td>0.049</td>
</tr>
<tr>
<td>CEM II/A-Ω</td>
<td>2.6</td>
<td>0.062</td>
<td>3.25</td>
<td>0.055</td>
</tr>
<tr>
<td>CEM II/B-V β</td>
<td>2.27</td>
<td>0.085</td>
<td>1.89</td>
<td>0.086</td>
</tr>
<tr>
<td>CEM II/B-V γ</td>
<td>2.77</td>
<td>0.091</td>
<td>2.5</td>
<td>0.110</td>
</tr>
<tr>
<td>CEM II/B-V δ</td>
<td>2.86</td>
<td>0.081</td>
<td>2.71</td>
<td>0.095</td>
</tr>
<tr>
<td>CEM II/B-Ω</td>
<td>1.98</td>
<td>0.118</td>
<td>2.57</td>
<td>0.077</td>
</tr>
<tr>
<td>CEM II/B-Ω</td>
<td>2.47</td>
<td>0.094</td>
<td>2.43</td>
<td>0.089</td>
</tr>
<tr>
<td>CEM IV/A(V) β</td>
<td>2.01</td>
<td>0.137</td>
<td>1.95</td>
<td>0.116</td>
</tr>
<tr>
<td>CEM IV/A(V) γ</td>
<td>2.44</td>
<td>0.143</td>
<td>2.25</td>
<td>0.106</td>
</tr>
<tr>
<td>CEM IV/A(V) δ</td>
<td>3.16</td>
<td>0.071</td>
<td>2.18</td>
<td>0.122</td>
</tr>
<tr>
<td>CEM IV/A(V) λ</td>
<td>1.93</td>
<td>0.017</td>
<td>2.57</td>
<td>0.173</td>
</tr>
<tr>
<td>CEM IV/A-Ω</td>
<td>2.35</td>
<td>0.103</td>
<td>2.4</td>
<td>0.093</td>
</tr>
</tbody>
</table>

With respect to the Al/Ca ratio, the values range from 0.025 to 0.21. A direct relationship was found between Al/Ca ratio and the ash content. The reference cement CEM I-α has the lowest Al/Ca ratio (about 0.033), followed by the CEM II/A-V (about 0.062), the CEM II/B-V (about 0.10) and finally, CEM IV/A(V) has the highest values of Al/Ca ratio ranging from 0.1 to 0.2. The Al amount in the C-S-H gel increases with the ash proportion in the cement.

The main mechanism explaining the effect of fly ash on alkali-aggregate reaction expansion is based on the availability of alkalis in the pore solution reduction and on the C-S-H gel capacity to incorporate alkalis into its structure.

The efficiency of fly ash in lowering the alkalinity of the pore solution is strongly influenced by the fly ash chemical composition itself. Therefore, mentioned efficiency is related to the low alkali and calcium contents and high silica content of fly ash. Some research works show that Class F fly ash is more effective in preventing alkali-aggregate reaction than Class C fly ash due to its chemical composition [21, 22].

Figure 4: Effect of Na₂Oₑ, CaO and SiO₂ on 28 and 90 days expansion due to alkali-silica reaction.
Thomas developed an empirical relationship between the chemical composition of the fly ash and ASR expansion \([(10\cdot\text{Na}_2\text{O}_e + (4.45\cdot\text{CaO})/\text{SiO}_2)] [21]. In this work, a relationship between chemical composition of cements and alkali-silica expansion was studied. Taking into account CaO, Na$_2$O$_e$ and SiO$_2$ data from Table 3 and 28-days and 90-days expansion from Table 4, a relationship between chemical composition of cements and alkali-silica expansion was found at 28-days and 90-days (Figure 4). An expansion below 0.1% at 28-days and below 0.2% at 90-days only occurs in cements with 35% coal fly ash or coal bottom ash, CEM IV/A (V), which have the lowest CaO (lower than 46%) and alkali values (lower than 1.0% Na$_2$O$_e$). On the contrary, they present the highest content of SiO$_2$ (higher than 27%). Therefore, it was confirmed that expansion decreases when CaO and Na$_2$O$_e$ decreases. Also, the expansion decreases when the SiO$_2$ content increases in the cements.

A threshold level of 0.20 M [23] or slightly less than 0.25 M [24], of hydroxyl ions in the pore solution is proposed to be necessary to initiate concrete alkali-silica reaction. Hydroxyl concentration could be estimated by applying the empirical equation developed by Helmuth [25] (Equation 5) or using the equation developed by Nixon and Page [26] (Equation 6).

\[
[\text{OH}^-] = \frac{0.339 \cdot \text{PC}_{\text{alk}} + 0.022 \pm 0.06}{\text{w/c}} \text{ mol/l} \tag{5}
\]

Where:  
\[\text{[OH}^-]:\text{ hydroxyl ion concentration in the pore solution.}\]  
\[\text{Na}_2\text{O}_e: \text{ the alkali content of the cement expressed as } \% \text{ Na}_2\text{O}_e.\]  
\[\text{w/c: water-to-cement ratio.}\]

\[
[\text{OH}^-] = 0.7 \cdot [\text{Na}_2\text{O}_e] \text{ mol/l} \tag{6}
\]

Where:  
\[\text{[OH}^-]:\text{ hydroxyl ion concentration in the pore solution.}\]  
\[\text{Na}_2\text{O}_e: \text{ the alkali content of the cement.}\]

Incorporating the data from Table 3 in equations 5 and 6, it could be possible to predict the average value of hydroxyl ion concentrations in mol/l in the pore solution (Table 6).

**Table 6:** Hydroxyl ion concentrations in pore solution of different elaborated cements.

<table>
<thead>
<tr>
<th>\text{CEM I}</th>
<th>\text{CEM II/A-V}</th>
<th>\text{CEM II/B-V}</th>
<th>\text{CEM IV/A (V)}</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.78</td>
<td>0.80</td>
<td>0.75</td>
<td>0.74</td>
</tr>
<tr>
<td>0.72</td>
<td>0.74</td>
<td>0.69</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Hydroxyl concentration in the pore solution estimated by mentioned equations for CEM IV/A(V) cements is about 0.7 mol/l.

The alkali-binding capacity of C-S-H gel is governed by the Ca/Si ratio due to surface charges on C-S-H gel. Binding is produced at acidic silanol (Si-OH) sites. Then, the C-S-H gel retains more alkalis when the Ca/Si decreases, because the concentration of acidity of silanol groups increase in this case. C-S-H gel is negatively charged when the Ca/Si ratio decreases; therefore, its capacity to bind cations is increased [27]. The alkali-binding capacity could be increased by introducing alumina into C-S-H gel to form C-A-S-H gel. [8, 28].

In this study, CEM IV/A(V) cements have the lowest Ca/Si ratio at 28-days and 90-days, but the highest Al/Ca ratio, therefore, the trend cited in the literature is followed.
5 CONCLUSION

Based on the results obtained in the present work it is concluded that coal bottom ash (CBA) perform in a similar way in comparison to coal fly ashes (CFA). Coal bottom ash and coal fly ash used as cement constituents in proportions of 10%, 25% y 35% decrease the alkali-silica expansion compared with a common Portland cement CEM I without any addition. The more proportion of ashes in the cement, the less expansion is recorded. Ash contents above 35% by weight in the cement lead to a high alkali-silica resistance in mortars.

Also, the higher CaO, Na$_2$O$_e$ content and the lower SiO$_2$ content of the cements, the lower expansion. The cement chemical composition of mentioned parameters could be used to provide an indication of the potential expansion in particular couples aggregate-cement.

As the Ca/Si ratio decreases and Al/Ca increases in the C-S-H gel, the alkali binding capacity increases. Then, the more ashes are in the cement, the lower amount of available alkalis in the cement pore solution.

Summing up, coal bottom ash alone or mixed with coal fly ash is suitable to be used in cement production. Finally, it can be concluded that coal bottom ash could be a good new main constituent in the common Portland cements.

ACKNOWLEDGEMENTS

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CONCRETE CRACKING IN MARINE MICROCLIMATES

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Key words: Concrete cracking, corrosion initiation, corrosion propagation, micro-climates, marine environment

Abstract. Understanding the mechanism of corrosion-induced concrete cover cracking is important for durability forecasting of reinforced concrete structures. Accelerated corrosion tests have provided very useful information on corrosion propagation and residual life stages, but tests and data from natural corrosion are required to validate and corroborate previous findings. Concrete cylinders cured for one day, with different water/cement (w/c) ratios, exposed at three natural sites, 50 m, 100 m and 780 m from the seashore (three micro-climates) in the Port of Progreso, Yucatan, Mexico, were used to measure corrosion parameters such as time-to corrosion initiation, apparent corrosion rate, surface crack propagation, rebar radius loss to generate concrete cover cracking, and rebar pit depth due to natural corrosion. A slight but clear correlation between data of concrete cracking from the three micro-climates was found. The w/c ratio of concrete was the most important parameter in the durability performance of concrete in such tropical environment. Empirical correlations between natural and accelerated corrosion tests were also obtained to corroborate data available in the literature regarding residual life of corroding reinforced concrete structures.
1 INTRODUCTION

Corrosion of reinforcing steel is one of the most important causes of reinforced concrete structures degradation [1]. In fact, humidity and chloride concentrations gradient in the air due to distance from the seashore enhance such degradation [2]. This changes in microclimates have led to establish empirical relationships between corrosion-induced cross-section loss of the reinforcing steel, and concrete damage symptoms such as cracking and spalling, to determine service life reductions of reinforced concrete structures, and have been presented elsewhere [3]. The study of any possible relationship between concrete cover cracking and reinforcing steel (rebar) corrosion parameters will help structural inspectors to define possible repair strategies.

On the other hand, reinforced corrosion research in natural environment exposure is not commonly found in the literature due to complexity on data analysis [4-5], which may include various parameters involved in this corrosion phenomenon [6]. Then, a serious concern is to find possible relationships between accelerated and natural corrosion tests in terms of corrosion-induced variables such as concrete surface crack width, rebar radius loss, and pit depth, etc. This could help to predict cross section loss or remaining load capacity, if crack width is known, among other prediction possibilities.

The objective of this work is to provide information and discuss empirical correlations between accelerated and natural corrosion tests, where data from three marine exposure sites where evaluated (50 m, 100 m and 780 m away from the seashore) and compared with accelerated test results from the literature.

2 EXPERIMENTAL PROCEDURE

2.1 Materials and Sample dimensions

ASTM Type I ordinary Portland cement (OPC) and crushed limestone aggregates were used for fabrication of concrete cylinders (diameter ~ 75 mm, height~150 mm). Plain concrete cylinders were used to determine 28-day compressive strength and chloride penetration during the experimental period. Typical no. 3 deformed steel rebar (~0.4 wt% carbon content) was used to simulate reinforcement. Rebar sections were weighed before cylinder casting, and section ends were protected against corrosion with epoxy and adhesive tape. This limits the exposed area inside the concrete specimen and isolates any rebar areas directly exposed to the environment. Rebar sections were positioned axially within the cylinders and extended the entire length [7]. A 50-mm activated titanium rod (ATR), characterized elsewhere was used as a reference electrode for electrochemical measurements [7-8]. For this study, it was calibrated vs. a saturated calomel electrode (SCE). An epoxy coating layer was applied to the cylinder’s ends to delimit concrete and steel exposure area. Concrete mixture proportions and physical/mechanical properties of the hardened concrete have been published elsewhere [7-10]. A total of 120 cylinders were tested at three natural marine micro-climates (50, 100 and 780 m away from the seashore) having two cylinders from each combination of plain and reinforced concrete for a total of 90 plain concrete and 90 reinforced concrete cylinders.
To study the effect of concrete’s w/c ratio on its service life, concrete was made using five w/c ratios: 0.76, 0.70, 0.53, 0.50, and 0.46. Curing time (CT) was tested at 1, 3 and 7 days, with two cylinders per each combination of w/c ratio and CT. This investigation only presents results from 1-day CT. Other results regarding other CT’s will be presented in the near future.

2.2 Environmental exposure.

Specimens had been exposed to the marine environment of Progreso’s Port, located at the north coast of the Yucatan Peninsula (21°18’ N, 89°39’ W), in Mexico, and is characterized as having a tropical humid climate. The carbon steel corrosivity category in this particular marine environment is higher than C5 (environment with t₄ time of wetness, or time that relative humidity is ≥ 85% and temperature > 30°C and ≤ 60°C, between 2,500 h/y and 5,500 h/y; atmospheric chloride concentration 300 mg/m²d to 1,500 mg/m²d) with an average annual temperature of 26°C and average annual relative humidity of 79% [11-12].

Specimens were placed at about 50 m, 100 m, and 780 m away from the seashore (AFS), on top of a three different one-story building roof (approximately 4m above ground level). Both specimen types (plain and reinforced concrete cylinders) were oriented vertically. Chloride was not added to the concrete mixtures prior to specimen casting to ensure that any chloride contamination was due to natural exposure during the experimental period.

2.3 Corrosion measurements.

Rebar electrochemical properties were measured using an external conductive rubber counter electrode tightly attached to the concrete cylinders with aluminum clamps for each test. Electrochemical measurements were taken with a commercially available potentiostat/galvanostat/ZRA, and included corrosion potentials (E⁺⁺⁺) and apparent corrosion rates (i⁺⁺⁺), determined by using the polarization resistance (Rp) technique [13-14]. These measurements were taken every two to three months during the entire experimental natural exposure period, which was up to 60 month for the 50 m AFS exposure site, 195 months for the 100 m AFS exposure site, and 240 months for the 780 m AFS exposure site.

At the end of the experimental period, in each exposure site, all specimens were examined carefully for signs of deterioration (i.e., crack formation, corrosion stains, and delaminations). Crack morphology (i.e., position, length and width) was recorded using a crack comparator card. The steel rebar was then removed from the concrete cylinder to determine final mass and calculate gravimetric loss. Average corrosion penetration or radius loss \( x_{Aver} \) was estimated with the following equation:

\[
x_{Aver} = \frac{10 \Delta W_G}{\rho \cdot \pi \cdot \phi \cdot L_{Effec}}
\]

Where \( \Delta W_G \) is gravimetric mass loss (in grams); \( \rho = 7.86 \text{ g/cm}^3 \); and \( L_{Effec} \) is the actual rebar
length affected by corrosion (in cm). Depending on the recorded crack width, estimated radius loss was defined as $x_{\text{crit}}$ (crack width $\leq 0.1$ mm) or $x_{\text{Aver}}$ (crack width $>0.1$ mm). Finally, pit depth was estimated using a magnifying lens and a caliper. Detailed descriptions of these parameters are available elsewhere \cite{15}.

Corrosion of the taped rebar ends was adjusted by assuming a modified total length (effective length) of the anodic zone, $L_{\text{Effec}}$. Therefore, $L_{\text{Effec}}$ was obtained by measuring the corroded rebar area with a caliper and magnifying lens (X20) before cleaning off the corrosion products.

Once a pit was located, the caliper was used to measure its deepest point. Caliper position was then changed to one end of the rebar (that closest to the measured pit) and the rebar diameter was recorded to create a pre-corrosion baseline. Finally, pit depth was estimated as the difference between the baseline measurement and pit depth measurement.

3 RESULTS

3.1 Reinforcing steel apparent corrosion rate for cylinders exposed at 50 m AFS.

Figure 1 shows average (of two specimens) $i_{\text{corr}}$ values as a function of exposure time for the specimens at the 50 m AFS exposure site, for all w/c ratio concretes and 1-day CT. As observed, all w/c ratio concretes showed changes of $i_{\text{corr}}$ values from passive to active corrosion ones (higher than 0.5 $\mu$A/cm²) at different time periods \cite{16}.

![Figure 1: Average apparent $i_{\text{corr}}$ values as a function of exposure time, for specimens at 50 m AFS and 1 day CT.](image-url)
3.2 Reinforcing steel apparent corrosion rate for cylinders exposed at 100 m AFS.

Figure 2 shows average (of two specimens) \(i_{\text{corr}}\) values as a function of exposure time for the specimens at the 100 m AFS exposure site, for all w/c ratio concretes and 1-day CT. The first 20 months of exposure, the five w/c ratio concretes presented below active corrosion threshold values (between 0.1 to 0.5 μA/cm\(^2\))\(^{[16]}\). It was until 24 months of exposure that 0.70 and 0.76 w/c ratio concretes showed higher \(i_{\text{corr}}\) values, thus such specimens were retrieved for crack appearance survey and rebar autopsy (as explained above). Depassivation for 0.53 w/c ratio concrete specimens occurred after 46 months of exposure, and the exposure time lasted up to 192 months. For 0.50 and 0.46 w/c ratio concretes depassivation occurred after 120 months and 64 months of exposure, respectively.

![Figure 2: Average apparent \(i_{\text{corr}}\) values as a function of exposure time, for specimens at 100 m AFS and 1 day CT.](image)

3.3 Reinforcing steel apparent corrosion rate for cylinders exposed at 780 m AFS.

Figure 3 shows average (of two specimens) \(i_{\text{corr}}\) values as a function of exposure time for the specimens at the 780 m AFS exposure site, for all w/c ratio concretes and 1-day CT. It is clearly to observe that depassivation for 0.76 and 0.70 w/c ratio concrete specimens initiated after 84 months of exposure. For 0.53 and 0.46 w/c ratio concretes depassivation occurred after 136 months. At the moment, apparent \(i_{\text{corr}}\) value for 0.50 ratio concrete is still not showing active corrosion\(^{[16]}\).
Figure 3: Average apparent $i_{corr}$ values as a function of exposure time, for specimens at 780 m AFS and 1 day CT.

Information described at this point was similar and reproducible for the other two CT’s (3 and 7 days) but will be discussed in the near future. As observed from Figures 1 through 3, electrochemical results presented well defined trends: higher w/c ratio concretes presented higher $i_{corr}$ values. But additional analysis is required with the data obtained from field measurements to obtain other important parameters as, for example, the corrosion initiation times as a function of w/c ratio and distance from the seashore, corrosion degradation parameters i.e., rebar corrosion penetration, pitting distribution and concrete surface cracking distress, as discussed as follows.

4 DISCUSSION

4.1 Accumulated apparent corrosion rate, $i^A_{corr}$

From $i_{corr}$ values as a function of time plots, it is possible to estimate Tuutti’s service life stages, initiation ($T_1$) and propagation ($T_2$), [17] for each w/c ratio concrete. However, this is difficult to visualize with the way the results are showed. Thus, additional $i_{corr}$ data processing was considered to compare each w/c ratio concrete types, which includes the accumulated $i_{corr}$ estimates as a function of time. These $i^A_{corr}$ estimates were calculated as the integral of the $i_{corr}$ vs. time results ($\int i_{corr} dt$). Results of these $i^A_{corr}$ estimates are shown in Figures 4, 5 and 6 for each exposure site, and w/c ratio (1-day TC).

The three figures include arrows representing the time of corrosion activation for each w/c ratio concrete. It is clear the performance of $i^A_{corr}$ estimates as a function of concrete types and presented similar trend than the proposed by Tuutti’s two stage durability model (1982). At the beginning, $i^A_{corr}$ vs t data presented a small slope (m1) with time and when the reinforcing steel begin to corrode, the slope of $i^A_{corr}$ vs. t switched to a higher value (m2) indicative of the end of
the corrosion initiation stage (T₁) in Tuutti’s durability model, and the starting point of the corrosion propagation stage (T₂).

**Figure 4:** Typical $i^A_{corr}$ estimates as a function of exposure time, for specimens at 50 m AFS, and 1-day CT.

**Figure 5:** Typical $i^A_{corr}$ estimates as a function of exposure time, for specimens at 100 m AFS, and 1-day CT.
Figure 6: Typical $i^{A}_{\text{corr}}$ estimates as a function of exposure time, for specimens at 780 m AFS, and 1-day CT.

Another important trend observed with the experimental results is the differences between $T_1$ stage for 50 m, 100 m and 780 m AFS exposure sites. For 50 m AFS exposure site (the closest from seashore), $T_1$ estimates were always smaller than the ones obtained at the 100 m and 780 m AFS exposure sites, for all w/c ratio concretes. For example, concretes with higher w/c ratio (0.76, 0.70 and 0.53) reached typical $T_1$ estimates between 2.5 and 6 months at the 50 m AFS exposure site, compared with ~ 48 months for the 100 m AFS exposure site. At the 780 m AFS exposure site, the 0.70 and 0.76 w/c ratio concrete specimens showed $T_1$ estimates of 54 months, followed by the 0.53 w/c ratio specimens with 96 months. On the other hand, lower w/c ratio concretes (0.50 and 0.46) reached $T_1$ estimates between 10 and 22 months, at the 50 m AFS exposure site; in comparison with 108 and 60 months, for w/c ratio specimens of 0.50, and 0.46, respectively, for the 100 m AFS exposure site; and 132 months for 0.46 w/c ratio at 780 m AFS exposure site. The 0.50 w/c ratio concrete specimens at 780 m AFS exposure site are still not showing any corrosion distress up to date.

At the moment, the data is still in a scrutinized analysis process to determine the slope values for each one of the two exposure sites, three CT’s, and five w/c ratios, and will be presented in the near future.

4.2 Experimental determination of $x_{\text{Aver}}$ and $\text{PIT}_{\text{Max}}$

Average corrosion penetration depth estimates, $x_{\text{Aver}}$, from Equation (1) using the gravimetric mass loss, are listed in Table 1 for all exposure sites (50 m, 100 m and 780 m). As observed from Table 1, data from 0.46 and 0.50 w/c ratio concretes presented the lowest $x_{\text{Aver}}$ values (median $x_{\text{Aver}} \sim 0.22$ mm for 50 m exposure site, $x_{\text{Aver}} \sim 0.32$ mm for 100 m exposure site, and $x_{\text{Aver}} \sim 0.23$ mm for 780 m exposure site), following data from 0.53 and 0.7 w/c ratio concretes (median $x_{\text{Aver}} \sim 0.25$ mm for 50 m exposure site, $x_{\text{Aver}} \sim 0.29$ mm for 100 m exposure site and $x_{\text{Aver}} \sim 0.27$ mm for 780 m exposure site). For 0.76 w/c ratio, $x_{\text{Aver}}$ was $\sim 0.35$ mm, which was the highest of all...
concrete mixtures tested at the 50 m exposure site, but for the 100 m exposure site, no cracks were observed up to date. For the case of the 780 m AFS exposure site, $x_{\text{Aver}} \sim 0.32$ mm similar to the one obtained at the 50 m AFS exposure site. Again, no correlation was observed between CT and $x_{\text{Aver}}$ for the three exposure sites.

Maximum pit depths ($\text{PIT}_{\text{max}}$) obtained in this investigation are listed in Table 1 for the three exposure sites (50 m, 100 m and 780 m AFS) for rebar nominal diameter of 10 mm. As observed from these results, no differences were observed between $\text{PIT}_{\text{max}}$ results from the three exposure sites. Data from 50m AFS exposure site ranged between 0.6 mm and 1.5 mm (average 1.03 mm); data from 100 m AFS exposuresite ranged between 0.8 and 1.0 mm (average 0.9 mm); and data from 780m AFS exposure site ranged between 0.24 and 4.0 mm (average 1.98 mm), but there are still some cylinders that are been exposed at the present date with no corrosion distress observed (concrete cracking or corrosion stains).

### 4.3 Corrosion-induced crack propagation in concrete

It is well known that the rebar radius gradually decreases as its corrosion increase, and its corrosion products produce a higher volume than the original steel \cite{18, 19}. This increment in volume is responsible of concrete cracking because concrete’s tensile strength is lower than the volumetric expansion stresses produced by formed oxides.

At the beginning, one or more cracks are generated at the steel/concrete interface and then propagate radially until reaching concrete surface\cite{15}. There are several studies that have tried to correlate radius loss with concrete surface crack widths (CW) \cite{5, 19, 22}, being most of them from accelerated corrosion tests. The accelerated tests ranged from anodic current/potential application, and/or the addition of certain quantities of chloride to concrete during mixing.

There have been some concerns about crack generation and rebar radius loss ($x_{\text{Aver}}/r_0$) in naturally exposed environments. This works intend to correlate some data obtained from this investigation with natural exposure during several years, and available data using accelerated corrosion tests.
Table 1: Experimental results of specimens exposed at 50, 100 and 780 m AFSand 1-day CT.

<table>
<thead>
<tr>
<th>Distance from the seashore (m)</th>
<th>Specimen number</th>
<th>w/c ratio</th>
<th>( \Delta W_G ) (g)</th>
<th>( L_{Effec} ) (mm)</th>
<th>Average radius loss ( x_{Aver} ) (mm)</th>
<th>Maximum crack width, ( CW_{Max} ) (mm)</th>
<th>Maximum pith depth, ( PIT_{max} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>505</td>
<td>0.46</td>
<td>7.67</td>
<td>156.6</td>
<td>0.21</td>
<td>0.80</td>
<td>2.16</td>
</tr>
<tr>
<td></td>
<td>506</td>
<td>0.46</td>
<td>9.06</td>
<td>148.7</td>
<td>0.26</td>
<td>1.50</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td>361</td>
<td>0.50</td>
<td>8.69</td>
<td>134.1</td>
<td>0.28</td>
<td>1.00</td>
<td>2.92</td>
</tr>
<tr>
<td></td>
<td>362</td>
<td>0.50</td>
<td>4.37</td>
<td>129.3</td>
<td>0.14</td>
<td>1.00</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td>289</td>
<td>0.53</td>
<td>9.02</td>
<td>144.0</td>
<td>0.27</td>
<td>-</td>
<td>2.16</td>
</tr>
<tr>
<td></td>
<td>290</td>
<td>0.53</td>
<td>6.37</td>
<td>127.8</td>
<td>0.21</td>
<td>0.60</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>181</td>
<td>0.70</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.50</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>182</td>
<td>0.70</td>
<td>10.86</td>
<td>180.0</td>
<td>0.26</td>
<td>-</td>
<td>1.65</td>
</tr>
<tr>
<td></td>
<td>73</td>
<td>0.76</td>
<td>8.86</td>
<td>165.7</td>
<td>0.23</td>
<td>-</td>
<td>2.49</td>
</tr>
<tr>
<td></td>
<td>74</td>
<td>0.76</td>
<td>16.41</td>
<td>201.0</td>
<td>0.35</td>
<td>0.80</td>
<td>2.79</td>
</tr>
<tr>
<td>100</td>
<td>510</td>
<td>0.46</td>
<td>7.17</td>
<td>98.0</td>
<td>0.31</td>
<td>0.80</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td>509</td>
<td>0.46</td>
<td>14.12</td>
<td>183.0</td>
<td>0.33</td>
<td>0.80</td>
<td>2.22</td>
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<tr>
<td></td>
<td>293</td>
<td>0.53</td>
<td>9.75</td>
<td>135.0</td>
<td>0.31</td>
<td>1.00</td>
<td>1.67</td>
</tr>
<tr>
<td></td>
<td>294</td>
<td>0.53</td>
<td>9.03</td>
<td>145.0</td>
<td>0.27</td>
<td>1.00</td>
<td>1.56</td>
</tr>
<tr>
<td></td>
<td>185</td>
<td>0.70</td>
<td>7.26</td>
<td>123.0</td>
<td>0.25</td>
<td>1.00</td>
<td>1.07</td>
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<tr>
<td></td>
<td>186</td>
<td>0.70</td>
<td>12.24</td>
<td>169.0</td>
<td>0.31</td>
<td>0.80</td>
<td>1.89</td>
</tr>
<tr>
<td>780</td>
<td>513</td>
<td>0.46</td>
<td>3.84</td>
<td>77</td>
<td>0.21</td>
<td>0.55</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>514</td>
<td>0.46</td>
<td>4.86</td>
<td>83</td>
<td>0.25</td>
<td>0.55</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>297</td>
<td>0.53</td>
<td>5.60</td>
<td>140</td>
<td>0.17</td>
<td>1.00</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>298</td>
<td>0.53</td>
<td>5.10</td>
<td>106</td>
<td>0.21</td>
<td>0.24</td>
<td>2.50</td>
</tr>
<tr>
<td></td>
<td>189</td>
<td>0.70</td>
<td>8.91</td>
<td>125</td>
<td>0.30</td>
<td>1.50</td>
<td>1.61</td>
</tr>
<tr>
<td></td>
<td>190</td>
<td>0.70</td>
<td>12.15</td>
<td>132</td>
<td>0.39</td>
<td>4.00</td>
<td>1.77</td>
</tr>
<tr>
<td></td>
<td>81</td>
<td>0.76</td>
<td>9.64</td>
<td>132</td>
<td>0.31</td>
<td>4.00</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td>82</td>
<td>0.76</td>
<td>9.66</td>
<td>126</td>
<td>0.33</td>
<td>4.00</td>
<td>1.62</td>
</tr>
</tbody>
</table>

Figures 7 and 8 were plotted to observe a general trend with the data from accelerated and natural corrosion tests separately. It shows a slight but clear correlation between \( CW \) and average \( x_{Aver}/r_0 \) for data from accelerated tests from several authors, and natural tests from this research. The empirical correlation shown in Figures 7 and 8 are similar to that from other investigations, being \( CW_{Max}=21.8 \cdot x_{Aver} \) with \( R^2=0.46 \) [19-20].

Although most of the data fell inside the cloud of data from other authors, it is also clear that natural environment measurements show similar tendency and larger scatter between the data than the accelerated corrosion data. The slope for natural corrosion data (16.6) is slightly higher than for accelerated corrosion (15.3), but they are from the same order of magnitude. It is also observed that generation of a crack with accelerated tests requires less corrosion products than for the natural case. This could be explained by the fast accumulation of corrosion products on the pores during the accelerated tests, which generate tensile stresses between reinforcement and 10370.
concrete, inducing cracks more easily. It is also reported that acidification of the reinforcement surface because of corrosion oxides/hydroxides can produce a bond loss and then could generate earlier cracks [5].

Figure 7: $CW_{\text{Max}}$ vs $x_{\text{Aver}}/r_0$ as for accelerated tests [24, 19, 22, 25-27]

Figure 8: $CW_{\text{Max}}$ vs $x_{\text{Aver}}/r_0$ as for natural tests [5, 20, 27]
It is also observed from natural test data in Figure 8 that more corrosion products are needed for crack appearance. This is due to the slower corrosion process and longer periods of time that allow a migration of corrosion products from the rebar to the concrete surface through the pores. Therefore, more corrosion products are needed to produce surface cracks. Figure 8 also shows an apparent limit value for $x_{\text{Aver}}/r_0$ which is 0.1. This might be due to the fact that $x_{\text{Aver}}/r_0$ does not increase at the beginning of the process, similar as crack width does. At the beginning when the rebar starts to corrode, the formation and propagation of more than one crack close to the rebar/concrete interface, may affect a possible linear relationship between CWMax and $x_{\text{Aver}}/r_0$. As a consequence, when $x_{\text{Aver}}/r_0$ reaches values of 0.1, the crack width might be more influenced by the distribution of variable tensile and compressive stresses at the rebar/concrete interface due to crack opening or closing during variable corrosion products formation along the rebar, instead of having only tensile stresses due to corrosion product expansion only. Previous results have considered that $x_{\text{Aver}}/r_0 \sim 0.1$ produce load capacity loss of concrete structural elements up to 30% and 50% for generalized corrosion and localized corrosion, respectively [20].

![Figure 9: PITMax vs Aver empirical correlation for accelerated and natural corrosion tests](image)

4.4 Corrosion-induced pit formation at the corroding rebar

After determining mass loss, each rebar was further inspected for evidence of pitting. All pit depths were measured and recorded using a caliper, as explained in the experimental procedure section. Table 1 lists the values for the three exposure sites: 50 m, 100 m and 780 m. Results
from this table and previous investigations were plotted as a correlation between $x_{\text{Aver}}$ and maximum pit depth, $\text{PIT}_{\text{Max}}$ (Figure 9).

With the information presented in this section, it is possible to obtain an empirical relation between $x_{\text{Aver}}$ and $\text{PIT}_{\text{Max}}$. This relation is presented as the discontinuous line in Figure 9, which represents the equation $\text{PIT}_{\text{Max}} = 7.8 \cdot x_{\text{Aver}}$ with a correlation coefficient $R^2 = 0.74$. This empirical relation shows that $\text{PIT}_{\text{Max}}$ values are approximately eight times the value of $x_{\text{Aver}}$, similar to findings reported in previous references [29] with $\text{PIT}_{\text{Max}} \sim \alpha \cdot x_{\text{Aver}}$ where $\alpha \sim 10$, where also the accelerated corrosion tests were included in Figure 9 and the obtained empirical correlation.

5 CONCLUSIONS

Based on the information obtained from this investigation, the following conclusions were obtained: Electrochemical monitoring of naturally exposed reinforced concrete cylinders at three different exposure sites determine important differences between corrosivity parameters, even though the exposure sites were located at the same port city, Progreso, Yucatan, Mexico: the closest is the exposure site to the sea shore, the faster was the corrosion activation of the reinforcing steel and the concrete cracking due to the corrosion products formed at the rebar/concrete interface.

Between the parameters chosen in this investigation regarding concrete quality, the water to cement ratio (w/c) is the most important of all, giving better performance in the tropical marine environment of Progreso, Yucatan, Mexico, concretes with w/c ratios less than 0.50.

This investigation has experimentally established the correlation between visible corrosion degradation (maximum crack widths at the concrete surface, $\text{CW}_{\text{Max}}$) and rebar radius loss ($x_{\text{Aver}}/r_0$) using naturally exposed reinforced concrete cylinders: $\text{CW}_{\text{Max}} \sim 16.56 x_{\text{Aver}}/r_0$ for natural corrosion.

Also an empirical correlation was obtained from corrosion-induced pit depths: $\text{PIT}_{\text{Max}} \sim 7.776 x_{\text{Aver}}$, corroborating empirical calculations from previous investigations using accelerated corrosion tests.

Small differences between crack opening and extension results in accelerated and natural corrosion tests were observed, but with other corrosion parameters such as mass loss and pit depth, trends were similar, supporting the use of such constant current accelerated method for remaining life forecast.

REFERENCES


CORROSION CRACK PATTERN AT EARLY AGES DUE TO THE PRESSURE RUST LAYER IN REINFORCED CONCRETE
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Key words: Corrosion, crack pattern, concrete, porous network, durability.

Abstract. Corrosion of a reinforcement bar leads to expansive pressure on the surrounding concrete that provokes internal cracking and, eventually, spalling and delamination. The aim of this paper is to study the mechanical effects of the rust layer in the surrounding concrete of the steel rebar at the initial stages of the corrosion. At the beginning of the corrosion the rust products penetrate the porous network of the concrete and the pressure is mitigated. The numerical and analytical modelling of the cover cracking due to corrosion of concrete usually assume that the pressure of the rust products of the rebar on concrete is delayed due to their penetration of the porous network. The assessment of this delayed time is based on empirical data. There is insufficient published research focused on the initial stage of the propagation of the rust products in the porous structure of the surrounding concrete. The aim of this work is to study thoroughly the penetration of the rust products in the porous network combined with the assessment of the strain and stresses of the concrete, near to the rebars, at the beginning of the corrosion process. A concrete with silica fume has been cast, which has a closer and narrower porous network in comparison with a conventional concrete. An accelerated corrosion test was carried out. The penetration of the rust layer in the concrete was observed and measured by means of scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDS). In addition, the strain of the concrete due to the corrosion of the rebar was assessed by means of strain gauges. The increasing of the strain in the concrete is observed from the beginning of the test. A good relationship between the velocity of the penetration of the rust products in the porous network and the delaying of the cracking pressure in concrete has been observed.
1 INTRODUCTION

Corrosion of reinforcement bars is a primary cause in the deterioration of concrete structures which may lead to failure and even collapse. In any such case, the cost of the associated repair and maintenance is enormous (for example, Kirkpatrick et al., 2002). Chloride-induced corrosion of the reinforcing steel is known to be a major cause of premature rehabilitation of reinforced concrete structures.

The present work is focused on chloride attack and distinguishes the two distinct phases customarily adopted in the literature (Tuutti, 1982): the initiation stage, until the aggressive species concentration around the reinforcement bar reaches a threshold value, necessary to trigger active corrosion; and the propagation stage, in which rust generation induces tensile stresses in the concrete cover. Once the chloride concentration threshold (for example, Angst et al., 2009) has been reached in the rebar surroundings, the subsequent oxide generation not only implies a steel section reduction and degradation in the steel-concrete adherence, but also increases the total equivalent rebar section. This is because rust products have a four-to-six-time higher volume than that of steel (Tuutti, 1982; Liu, 1998; Marcotte, 2007). Consequently, expansive stress appears in the concrete, causing cracking, spalling or delamination of the cover. (Zhao, Wu, & Jin, 2013).

This work sets out to integrate the two phases (initiation and propagation) widely cited in the literature and usually analysed separately. An overview of the previous proposals leads the reader to the general conclusion that service life ends when steel depassivates (the initiation phase) (Andrade et al., 1993).

When the corrosion of the concrete has started, it is accepted that the process has three stages: (1) a first stage in which rust fills the porous zone around the steel/concrete interface; (2) a second one in which the rust starts to exert stress on the surrounding concrete until the first crack initiates; and (3) a third one when the process arrives at a last stage in which the rust fills the cracks as they are being created. Recently, authors such as Zhao et al. (Zhao, Wu, & Jin, 2013) have suggested that the penetration of the corrosion products into the porous zone of concrete and the formation of a corrosion layer at the steel/concrete interface might proceed simultaneously after the initiation of the steel corrosion. In addition, they estimate that rust might not penetrate the corrosion-induced cracks before the cracks reach the concrete surface. These last points still cause controversy.

The aim of this work is to study the first stage of the process in which the corrosion products penetrate the concrete porous network around the reinforcement. This zone is named by Michel et al. (Michel, Pease, Geiker, Stang, & Olesen, 2011) as the corrosion-accommodating region (CAR) and its thickness ranges from 0.002 to 0.120mm. A concrete with silica fume (SF) was cast which has a closer and narrower porous network in comparison with a conventional concrete. An accelerated corrosion test was carried out. The penetration of the rust layer in the concrete was observed and measured by means of scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDS). In addition, the strain of the concrete due to the corrosion of the rebar was assessed by means of strain gauges. The relationship between the velocity of the penetration of the rust products in the porous network and the delay of the cracking pressure in concrete has been observed.
2 EXPERIMENTAL PROCEDURE

2.1 Materials, mix proportioning and specimens

The component materials included Portland cement type CEM I 52.5 R (UNE EN 197-1:2000) and SF as a mineral addition (10% of cement weight and two as efficiency factor). The water/binder ratio was 0.45. The water included a chloride content of 3% of cement weight. The aggregates used were siliceous gravel and sand with, respectively, a maximum size of 16 mm and 4 mm. Table 1 shows the mix proportioning.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Water/binder ratio</th>
<th>Binder (kg/m³)</th>
<th>Aggregate(Kg/m³)</th>
<th>Admixture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFC</td>
<td>0.45</td>
<td>280</td>
<td>35*</td>
<td>552.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225.4</td>
<td>907.8</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
</tr>
</tbody>
</table>

Table 1. Mix proportioning (*Efficiency factor silica fume is taken 2)

The compressive strength of SFC at 28 days was 52 MPa, measured on 300x150mmØ cylinder specimens, according to EN-12390-3. In addition, the depth of penetration of water under pressure test, according to EN-12390-8 was carried out, with the values of the penetration being 20 mm (mean value) and 30 mm (maximum penetration).

With the aim of avoiding preferential attacks, an insulating tape of epoxy resin was placed at the concrete–air interphase at the rebar, in order to keep away from corrosion initiators and propagating points. Initially, rebars of Ø20 mm were covered entirely with epoxy resin. Then, a machining was applying along 100mm at 30mm away from the button of the specimen, reducing the diameter up to 12 mm as shown in Figure 1.

In order to study the evolution of the corrosion product around the steel rebars regarding distinct ages, seven concrete specimens with an embedded steel bar were manufactured. The specimens were cast in moulds of 150x150x150 mm³.

2.2 Accelerated corrosion tests and test programme

The concrete-rebar system was externally short-circuited (Figure 3). A constant anodic current was applied to increase the velocity of corrosion. The electric circuit joined the steel rebar (anode) with a lead sheet placed on the surface of the slabs acting as a cathode. The uniform distribution of the electric current was achieved by immersing the specimen to a depth of 1cm in a water container. With the aim of ensuring the needed moisture supply, water was continuously poured into the container in order to prevent drying.

During the process, current passing through each of the rebars was recorded by using a digital multimeter, performing periodic readings and correcting any electricity drop by the electric power supply potential variation.
The mean density of the current in each bar was approximately $100 \mu \text{A/cm}^2$. After checking electrical resistance in the entire batch of specimens, a difference of resistance was observed at the specimen with strain gauges as it had been cast in two stages. From this observation, two independent circuits were assembled. The first one comprised only the specimen with strain gauges, whereas the second one with included the rest of the specimens by using a parallel system. The samples for the imaging were disconnected and extracted at 0, 1, 2, 5, 8 and 14 days. At the end of the test, the amount of current delivered was 4838.4 coulombs.

![Figure 2. Schematic diagrams of the cut specimen](image)

![Figure 3. Slab assembly during an accelerated corrosion test](image)

### 2.3 Sample preparation for imaging

The evolution of the corrosion and growth of the rust layer were observed by means of SEM and EDS. The specimens were cut at distinct ages. The cut procedure is shown in Figure 2. Prismatic samples with a $28 \times 28 \text{mm}^2$ base, containing the rebar, were cut. Then, slices were obtained for SEM observation. The slices were impregnated with a low-viscosity epoxy resin. Previously, the specimens had been placed in a vacuum chamber. The epoxy was hardened for 48 hours. All the specimens were polished by using a lapping and polishing machine with silicon carbide abrasive papers from grades 240 to 1200 lubricated with water. Polishing was performed through use of wenol metal polish to improve the imaging quality. A 5nm thick layer of gold (measured with a film thickness monitor) was sputtered onto the polished surface.
2.4 SEM imaging

The main objective was to observe the areas where the corrosion product had penetrated the cement paste and obtain measurements to study their size, distribution, and composition. A field-emission scanning electron microscope (JEOL Superprobe JXA-8900 M) operated in the backscattered electron (BSE) mode was used to study the microstructure at the steel-concrete interface. A set of images was taken at randomly selected locations on the polished sections of the plain concrete prisms. The reference region of each image had a 400µm length and 300µm width, with a resolution of 0.42 µm/pixel (150x). Energy dispersive spectrum analysis (EDS) was applied to assess the content of O, Fe, Si and Ca in the selected areas on an analytical line across the steel/concrete interface and mapping.

2.5 Strain gauge

The characteristics of the strain gauges used to measure the strains of the concrete around the rebar are given in Table 2.

<table>
<thead>
<tr>
<th>Type BHM</th>
<th>Operating temp. range compensated (ºC)</th>
<th>Resistance (Ω)</th>
<th>Gauge factor</th>
<th>Dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-LY41-10/120</td>
<td>-10/ +45</td>
<td>120</td>
<td>2.07</td>
<td>Measuring grid</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Measuring grid carrier</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10x5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8 x 18</td>
</tr>
</tbody>
</table>

Table 2. Strain gauges characteristics

The positioning of strain gauges is an important aspect to be considered (Figure 4). Other authors (Yüzer, Aköz, & Kabay, 2008) have located the gauges in the outside surface of the cylinder specimen, where the distance to the rebar may in many cases produce complications. As an alternative, in this work the gauges were situated around the corroded rebar as closely as possible. In order to situate the gauges inside the concrete specimen, it was cast in two stages. After hardening the first stage, the four gauges were glued on the concrete surface, forming a right angle between each one. After that, the second stage was performed that guaranteed a homogenous expansion. Figure 4 shows the strain gauges positioned around the rebar after casting the first stage.

3. RESULTS

Distinct comparative methods were performed to visualise the progress of the corrosion layer on the rebar surface at early ages.
3.1 Microstructure characteristics

Mapping was adopted to observe the typical rust distribution at the steel/concrete interface. Figure 5 shows the amount of Fe by using a colour code. The areas with high levels of Fe are shown in purple and the rust layer is represented in green.

The distributions of Fe and O were analysed by EDS along an analytical line, as shown in Figure 5. The horizontal axis represents the distance from the starting point of the analytical line, whereas the vertical axis shows the count of photoelectrons per second. Based on these results, the thickness of the rust layer can be determined.

An increase in the thickness of rust as a function of time was analysed as shown in Figure 6. Based on these results, a linear growth is observed from the second day by the following equation:

\[ y = 4.8x - 1 \]

A slice of each specimen analysed was scanned with a conventional flatbed scanner at 1200 dpi to produce a colour image. The image was enhanced by using the level of contrast and colour saturation to improve the visualisation of the rust layer. An example of the crack patterns obtained is shown in Figure 7. This image was obtained from the scanning of the slice SFC14 where cracks start. In the figure, the trajectory of the crack is marked with a white line to enable a better view.
Figure 5. BSE image at the steel/concrete interface (C, concrete; MS, Millscale; S, Steel) and analytical lines across the interface.
Figure 5 (cont). BSE image at the steel/concrete interface (C, concrete; MS, Millscale; S,Steel) and analytical lines across the interface.
Figure 6. The thickness of rust from time ($R^2=0.97$)

Figure 7. Crack patterns for the two bars

3.2 Experimental displacement field measurements at steel/concrete interface with strain gauges

The strain field surrounding the corroded rebar was recorded by an acquisition data system, as Figure 8 shows.

Figure 8. Data acquisitions from strain gauges
4 DISCUSSION

To describe the behaviour of the concrete cover, the thick-walled-cylinder model (Timoshenko & J.N. Goodier, 1970) is used. As long as the hoop stresses remain below the tensile strength, a linear elastic stress state is present. When the circumferential stress reaches the tensile strength, one or more radial cracks start to grow and the response of the concrete to the internal pressure becomes non-linear. For the last stage the total elongation of a circular fibre (Equation 1) with radius \( r \) consists of the total width of the fictitious cracks and the elastic deformation of the concrete between the cracks.

\[
\frac{\Delta R}{r} + \frac{n w}{2\pi r} = \varepsilon
\]

where

- \( w \) is the width of crack
- \( r \) is the radius
- \( \Delta R \) is the radial displacement
- \( n \) is the number of cracks
- \( \varepsilon \) is the circumferential strain

Due to the test setup a length of 4 mm had to be added to rebar radius because of the location of the strain gauge, such as Figure 9 shows.

Figure 9. Sketch of the distance between rebars and strain gauges

Figure 5 shows the EDS analysis results for the selected analytical line at the steel/concrete interface. Slices SFC0 and SFC1 did not show any steel corrosion. The thickness of the mills was measured for the rest of the cases studied (SFC2, SFC5, SFC8 and SFC14). The values found were, respectively, 0.01, 0.02, 0.045 and 0.063. For these slices, a gradual increase of the thickness was observed in all the cases except for SFC14, where the increase was steeper than the others.

In order to find the expansive pressure, the circumferential strain (\( \varepsilon \)) was measured by means of strain gauges. The results for the ages one, two, five, eight and 14 days were, again respectively. 0.22, 0.23, 0.52, 0.81 and 1.78 mm/m. The results reveal that the expansion pressure started immediately after the initiation of steel corrosion. Taking into account only the first four ages, the growth of circumferential strain was lineal and had a mild slope. Nevertheless, from this point until the end the growth of the circumferential strain became exponential.

In the first stage of the corrosion model (Liu, 1998) it is assumed that corrosion products fill the porous zone around the steel/concrete interface before starting the expansive pressure. According to recent research (Zhao, Wu, & Jin, 2013), the penetration of corrosion products of the porous zone of concrete and formation of the corrosion layer at the steel/concrete interface might proceed simultaneously after the initiation of steel corrosion.
In line with such an approach, a yield factor \( f \) was included in Equation 2 in order to approach the true radial displacement \( \Delta R_{\text{true}} = \Delta R f \) of the rebar, which is responsible for the expansive pressure.

\[
\frac{\Delta R f}{r_1} + \frac{n w}{2\pi r_1} = \varepsilon
\]

(2)

The yield factor was obtained based on the hypothesis of the non-existence of cracks \( (w=0) \) in the concrete around the rebar by means of Equation 3.

\[
f = \frac{\varepsilon r_1}{\Delta R}
\]

(3)

The range of ages where there were no cracks was obtained by comparing the calculated stress \( (\sigma) \) with the tensile strength \( (f_{ct}) \), which had an adopted value of 4 MPa. In this work Hooke’s law was adopted to calculate the stress as a function of the strain \( (\varepsilon) \) measured by the strain gauges. Table 3 shows the calculated stress and the \( f \) factor.

<table>
<thead>
<tr>
<th>( \sigma ) (MPa)</th>
<th>( \sigma &gt; f_{ct} ) cracks</th>
<th>( f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFC0 0.0</td>
<td>No crack</td>
<td>0</td>
</tr>
<tr>
<td>SFC1 0.0</td>
<td>No crack</td>
<td>0</td>
</tr>
<tr>
<td>SFC2 2.32</td>
<td>No crack</td>
<td>0.23</td>
</tr>
<tr>
<td>SFC5 10.68</td>
<td>Confined crack*</td>
<td>0.26</td>
</tr>
<tr>
<td>SFC8 20.34</td>
<td>Confined crack*</td>
<td>0.18</td>
</tr>
<tr>
<td>SFC14 35.80</td>
<td>Crack</td>
<td>0.28</td>
</tr>
</tbody>
</table>

*The crack was not found

Table 3. The relation between the current stress \( (\sigma) \) and the tensile strength \( (f_{ct}) \). On the basis of the above in the slices SFC2, SFC5, SFC8 there was not any possibility of finding cracks. As might have been expected, a yield factor smaller than the value was found. The values of this factor were, respectively, 0.23, 0.26 and 0.18. For such a reason, it may be suitable to adopt a value of 0.2 to adjust the values of \( \Delta R \), obtained previously. Therefore, from Equation 2 and the yield factor previously obtained, the width of the crack has been deduced by assuming the formation of only one crack (for the age of 14 days) with the resulting value being 0.0326 mm.

Unfortunately, given that two of the four strain gauges stopped working at 300 hours, the information for the later ages is incomplete. Considering that Gauge 5 and Gauge 7, a significant increase of slope was observed at 450 hours. From that moment, as the porous zone was considered to be full of corrosion products, the formation of corrosion products was translated into an expansion pressure and spread through the cracks.

5 CONCLUSIONS

In concordance with the results obtained, a good relationship between data obtained by the strain gauges and those extracted from SEM image is observed. Furthermore, an approached value of width of crack is obtained with both procedures.

For future work, the research could be continued by increasing the time corrosion of the rebars beyond 500 hours. In a second stage, plans to take the research further entail studying the behaviour under
corrosion with other types of concrete (in which the microstructural characteristics would be distinct), in order to provide a comparison with the results previously obtained.

6 REFERENCES


DURABILITY OF SUSTAINABLE TERNARY BLENDED CONCRETE CONTAINING BLAST FURNACE SLAG AND LIMESTONE FILLER
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Key words: Durability, Mechanical Strength, Blast Furnace Slag, Limestone Filler.

Abstract. In the present paper, the physical-mechanical and durability properties of a concrete designed according to sustainable parameters (64% OPC + 30% BFS + 6% LF) is compared with a reference concrete (100% OPC). Results show that the ternary concrete is able to achieve similar mechanical properties than the reference at 28 days but overpass the OPC at longer curing ages. The reason of this is attributed to the generation of a more refined pore structure, outcome of the differences in hydration evolution. The durability performance depends on the nature of the aggressive under consideration: 1) Water capillary absorption coefficient is lower in ternary blended concrete as consequence of the refinement of the pore structure, 2) Carbonation rate of ternary blended concrete is higher regardless of the curing length, 3) The chloride transport is delayed with ternary blended concrete, confirmed with chloride diffusion coefficient (migration) and after natural diffusion penetration in saturated conditions.

1 INTRODUCTION

The partial replacement of Ordinary Portland Cement (OPC) by mineral additions is one of the main ways to increase the sustainability in the concrete construction industry as it reduces the clinker production and consequently, the generation of greenhouse gases. Furthermore, the replacement of OPC by mineral additions gives added-value to industrial by-products.

The use of mineral additions in cement production has technical, economic and environmental consequences [1-4]. Concerning technological aspects, the mechanical strength is affected in different ways depending on the type of mineral addition employed. High content of a mineral addition with pozzolanic capacity, as fly ash (FA), or hydraulic, as blast furnace slag (BFS), delay the developing of compressive strength [3, 5]. Limestone filler (LF) shows negative or positive contribution depending on the content [6]. Recent studies combining mineral additions have demonstrated that they can reduce the delay in mechanical growing if optimal proportions are used [1, 6-11], but also affects the composition of OPC employed [1, 6, 12, 13]. However there is still a lack of knowledge related to the performance of these new sustainable concretes.

Regarding the contribution of the use of one mineral addition in the OPC concrete
durability, the experience shows that the LF reduces the capillary absorption capacity if a refinement of porosity takes place [7], and also, it can reduce the carbonation rate too [14]. The partial substitution of a mineral addition, like BFS and FA to OPC contribute to reduce the chloride transport [4, 8, 15], while carbonation rate is faster due to the lower OPC content [4, 16-18]. However, the effect on durability of combined mineral additions with OPC is still under question due to the lack of experience [7, 19, 20]. It is known that the synergistic effect of two mineral additions in OPC based materials promotes different microstructural evolution than in binary mixes [11, 13] and this must also have contribution in the durability performance of the resulting concretes.

The aim of this paper is to understand the technical and durable performance of a concrete designed with sustainability criteria with respect to a traditional OPC concrete. The study focuses on the analysis of combined properties: compressive strength, porosity, water absorption, natural carbonation and chloride penetration. The effect of the curing age is also evaluated.

2 EXPERIMENTAL PROCEDURE

Ordinary Portland Cement, CEM I 42.5 R-SR type has been used to prepare the binders. The chemical composition is included in Table 1. The mineralogical composition of OPC from Bogue [21] (after the correction with 4% of CaCO3) indicates 4% content in of C3A. Blast furnace slag (BFS) and limestone filler (LF) were used to prepare the ternary blend; so their chemical composition is also included in Table 1. OPC has lower alkali content than mineral additions (Na2Oeq.: OPC=0.4, BFS=2.3, LF:0.7). Al2O3 is also higher in the mineral additions than in OPC.

A 100% OPC was considered as reference and the blend of 64% OPC + 30% BFS + 6% LF represents the ternary blend, CEM II/C-M type designed according to EN 197-1 [22].

<table>
<thead>
<tr>
<th></th>
<th>Na2O</th>
<th>K2O</th>
<th>CaO</th>
<th>SiO2</th>
<th>Al2O3</th>
<th>Fe2O3</th>
<th>MgO</th>
<th>SO3</th>
<th>LoI</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>0.18</td>
<td>0.34</td>
<td>60.30</td>
<td>17.40</td>
<td>4.68</td>
<td>5.08</td>
<td>1.78</td>
<td>3.17</td>
<td>4.2</td>
</tr>
<tr>
<td>LF</td>
<td>0.55</td>
<td>2.65</td>
<td>26.09</td>
<td>21.39</td>
<td>7.75</td>
<td>2.57</td>
<td>5.80</td>
<td>0.05</td>
<td>30.1</td>
</tr>
<tr>
<td>BFS</td>
<td>0.42</td>
<td>0.49</td>
<td>45.50</td>
<td>36.57</td>
<td>10.39</td>
<td>0.29</td>
<td>7.46</td>
<td>0.05</td>
<td>NA</td>
</tr>
</tbody>
</table>

Table 1: Chemical composition (wt %) of the raw materials

Table 2 includes the nominal compositions of the designed concretes that are identified as 100OPC and 64OPC30BFS6LF. The water/binder ratio was varied to obtain similar consistency in the fresh state in both concretes. The characteristics measured in the fresh state were as follows: density (2.12 g/cm³ for 100OPC and 2.13 for the blend), consistency (6.5cm for 100OPC and 7.0cm for the blend) and air content (3.0% OPC and 2.1% blend). It is remarkable that even with a lower w/b ratio; the ternary blended concrete had a slightly higher consistency in the fresh state.

Cylindrical and cubic samples were fabricated according to the requirements of the tests.
Samples were cured in a chamber at 21±2°C and 98±2% RH for different ages from 7 to 90 days.

### Table 2: Nominal composition of concretes

<table>
<thead>
<tr>
<th></th>
<th>100OPC</th>
<th>64OPC 30BFS6LF</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (Kg/m³)</td>
<td>385</td>
<td>247</td>
</tr>
<tr>
<td>BFS (Kg/m³)</td>
<td>-</td>
<td>116</td>
</tr>
<tr>
<td>LF (Kg/m³)</td>
<td>-</td>
<td>23</td>
</tr>
<tr>
<td>Water (Kg/m³)</td>
<td>237</td>
<td>228</td>
</tr>
<tr>
<td>Water/binder</td>
<td>0.61</td>
<td>0.59</td>
</tr>
<tr>
<td>Medium aggregate 6-12mm (Kg/m³)</td>
<td>842</td>
<td>843</td>
</tr>
<tr>
<td>Sand 0-6mm (Kg/m³)</td>
<td>802</td>
<td>803</td>
</tr>
</tbody>
</table>

The compressive strength was measured in cylindrical samples of 15x30 cm after 7, 28 and 90 days of curing. The results refer to mean values of three samples. Mercury intrusion porosimetry was used to determine pore size distribution after 28 days of curing. The water absorption was measured in cylindrical slices of 7.5x5 cm (cut from 7.5x15 samples) after 28 and 90 days of curing following the testing procedure described in UNE 83982. Carbonation test were carried out after 28 and 90 days of curing. Cylindrical samples of 7.5x15cm were cut in two halves of 7.5x7.5cm. The samples were exposed to the outside atmosphere on Madrid environment although sheltered from the rain. The penetration depth was determined after 1, 2 and 3 years of exposure using phenolftalein indicator and following the requirements of UNE 112011. The chloride penetration has been tested using two methodologies: 1) Migration tests according to the procedure described in UNE 83987. Slices of 1cm depth were cut from 7.5x15cm samples of both concretes after 28 and 90 days of curing. Only slices with cut surfaces were used. 2) Cubic samples of 10x10 cm cured during 28 and 90 days were saturated under vacuum in water for 24 hours. The lateral and bottom surface were painted with epoxy resin in order to allow the chloride ingress to occur only through the exposed surface. A ponding containing 0.5M NaCl was located on the surface and maintained during 18 months. After powder sampling of 2 mm thickness were taken up to 1 cm depth and total chlorides were analyzed. For total Cl determination the powdered samples were dissolved in acidic medium and analyzed with silver nitrate by potentiometric titration.

### 3. RESULTS AND DISCUSSION

#### 3.1 Building up of technological properties in ternary blended concrete

Interesting information is pointed out analyzing the evolution of compressive strength of concretes, as deduced from Figure 1. At 7 days of curing, ternary blended concrete shows a 20% delay in mechanical strength developing, that is well compensated at 28 days. At longer
curing ages, 90 days, the plain OPC concrete is stabilized while the ternary concrete has a significant strength gain, 20% higher. This response of ternary blend concrete with respect mechanical strength building up is associated to differences in the reactivity of mineral additions with respect OPC, as the initial lower reactivity of BFS respect to that of OPC [5]. The contribution of LF is expected to take place at early ages favoring the nucleation of OPC hydration, although the substitution of OPC by BFS (30%) cannot be fully compensated at 7 days as also found in [11]. The OPC has almost completely reacted after 28 days of hydration reason for the stabilization of mechanical strength growth. By contrast, BFS generates additional hydration products (mainly C-S-H gel) at more advanced ages that contribute to densification the concrete and increase on compressive strength.

![Figure 1: Evolution of compressive strength in 100OPC and 64OPC30BFS6LF.](image)

The mechanical strength evolution with binder hydration advance should have a relationship with the concrete pore structure. Analyzing the net of pores of Figure 2 after 28 days of hydration it can be stressed out clear differences between both concretes.

The ternary blend concrete develops a more refined pore size structure, concentrating most of the pores <0.05μm, probably as consequence of the delayed C-S-H formation from BFS hydration inside the initial hydrates from OPC [23, 24]. However in OPC concrete after 28 days of curing most of the pores are distributed in a wider region of pores, from 0.5 to 1 μm. These differences in pore structure are not already affecting the mechanical strength as it can be at long term ages when BFS continue hydrating and refining more the pore structure.

3.2 Durability performance of ternary blended concrete

The weakness of a concrete with respect to durability performance it is well associated to its ability to avoid water penetration. The pore structure refinement in ternary blended concrete should positively contribute to control the penetration of water through capillary pores, as can be deduced from Figure 3. In the case of 100OPC concrete the curing time (28
or 90 days) has not significantly influenced the water suction capacity of the concrete, indicating that the pore structure of the OPC concrete does not suffer significant changes at longer curing ages. In this sense, in the 100OPC concrete most of the hydration processes have been completed at 28 days, in accordance with [13]. This agrees with the stabilization of the compressive strength after 28 days shown in Figure 1.

Figure 2: Pore size distribution in 100OPC and 64OPC30BFS6LF concretes at 28 days of curing.

Figure 3: Water absorption of 100OPC and 64OPC30BFS6LF concretes. Effect of curing time.
Concerning ternary blended concrete the refinement of pore structure in Figure 2 agrees with the lower water capillary suction measured. In fact, this beneficial influence is even more important at longer curing time. The most probable reason for this positive effect is the advance in hydration of blast furnace slag, confirmed in [13], that also leads to lower total porosity and probably a more refined pore structure, as found in case of mortar [11]. All these phenomena finally contribute to the retardation of water absorption.

Additionally the sorptivity coefficients of both concretes after different curing times have been calculated and they confirm the differences in the water absorption curves of Figure 3. The Table 3 compiles the capillary absorption coefficient and effective water porosity. Both concretes show lower values after 90 days of curing. However, the results indicate that the calculated coefficients are always lower in the ternary blended concrete, both at 28 and 90 days of curing. Moreover, the relative difference between the obtained values is higher after 90 days of curing. Thus, although the long term curing contributes positively in both concretes, the effects are higher in the ternary one. Accordingly the water porosity is lower in the blended concrete after 90 days of curing.

<table>
<thead>
<tr>
<th></th>
<th>28 days curing</th>
<th>90 days curing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm/min$^{0.5}$)</td>
<td>(%)</td>
</tr>
<tr>
<td>100OPC</td>
<td>0.123</td>
<td>12.10</td>
</tr>
<tr>
<td>64OPC30BFS6LF</td>
<td>0.084</td>
<td>12.19</td>
</tr>
</tbody>
</table>

The corrosion of reinforcements is considered one of the reasons that affect more the durability of concrete structures. Carbonation of concrete cover and chloride penetration up to the rebar level are considered the main reasons for corrosion initiation. Figure 4 shows the evolution of the carbonation depth in concretes exposed to the natural atmosphere of Madrid but sheltered from the rain. Regardless of the curing time, the carbonation depth is higher in ternary blended concrete. The refinement in pore structure of ternary blended concrete respect to OPC concrete is not able to decrease the carbonation rate of ternary blended concrete, even after long curing time. The partial substitution of OPC by mineral additions is expected to be the cause that better explains the acceleration in carbonation with a lower portlandite formation in spite of the higher C-S-H expected from the hydration of BFS, as found in [13].

Therefore, the inclusion of these mineral additions causes an increasing risk on corrosion initiation of reinforcements due to the lower carbonation resistance of ternary blended concrete. The carbonation coefficients calculated after 3 years of exposure in OPC and ternary blended concrete (with 28 days of curing) give values of 1.1 mm/month$^{0.5}$ for the former and 1.4 for the last.

The positive effect of BFS in retard the Cl transport is particularly important when it is mixed with a low C$_3$A content OPC (as the used in this work) due to the C$_3$A plays a
significant role in binding the chloride ions [25, 26]. In this sense, the BFS addition can compensate the low C₃A content of the OPC thus increasing the chloride binding phenomena. In addition, these results can be also enhanced by the porosity and pore size distribution formed (Figure 2), which leads to a reduction in the mobility of the chloride ions [27, 28].

Figure 4: Carbonation depth in 100OPC and 64OPC30BFS6LF concretes exposed to Madrid atmosphere sheltered from the rain. Effect of curing time

Figure 5: Chloride migration coefficient in 100OPC and 64OPC30BFS6LF. Effect of curing time.

The binding capacity of the cement contributing to the transport of chlorides can be
deduced from the comparison of the amount of chlorides penetrated, up to one cm depth, after natural exposure in saturated Cl natural transport conditions. From Figure 6 it can be deduced that after 28 days curing the chloride penetrate faster as consequence of lack in hydrating of BFS and lower OPC content in the mix, although this effect decays with the penetration depth. However the beneficial contribution of BFS is clearly observed after 90 days of curing when the hydration of BFS has been clearly initiated and contributing to binding of Cl.

![Figure 6](image_url)

**Figure 6**: Chloride penetration up to 1 cm cover in 100OPC and 64OPC30BFS6LF exposed to chloride transport in natural saturated conditions.

The chloride accumulated up to 1 cm cover, Table 4, in 100OPC is quite independent of the curing time (between the two ages studied). By contrast, in the ternary blend the content is significantly lower with 90 days of curing. In fact, their values in the samples cured during 90 days are 25% lower than the values obtained in the reference concrete. The advance in hydration of BFS has contributed to the binding of chlorides in the ternary blended concrete.

<table>
<thead>
<tr>
<th></th>
<th>Mass of Cl (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>28 days</td>
</tr>
<tr>
<td>100OPC</td>
<td>0.86</td>
</tr>
<tr>
<td>64OPC30BFS6LF</td>
<td>0.94</td>
</tr>
</tbody>
</table>

**Table 4**: Mass of Cl accumulated up to 1 cm cover in 100OPC and 64OPC30BFS6LF after 18M exposure to Cl transport under natural saturated conditions. Effect of curing time

The contribution of the different parameters analyzed with respect to durability of concretes considering the 28 days cured concretes are compiled in Table 5. Concerning the indirect
durability indicators as water porosity and water capillary suction the ternary concrete can be classified to have high-medium durability performance according to [29, 30]. For the transport of chlorides also the ternary concrete improves the durability by 25%, and classifies as high durability, as suggested in [30, 31], that becomes very high after long curing (90 days) while does not change for OPC concrete. On the contrary lower resistance to carbonation of ternary blended concrete respect to OPC is deduced.

<table>
<thead>
<tr>
<th>Indicator durability</th>
<th>100OPC</th>
<th>DP</th>
<th>64OPC30BFS6LF</th>
<th>DP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity accessible to water (%)</td>
<td>12.1</td>
<td>Medium (12-14) [30]</td>
<td>12.2</td>
<td>Medium (12-14) [30]</td>
</tr>
<tr>
<td>Capillary absorption coefficient (mm/min^0.5)</td>
<td>0.12</td>
<td>Medium (0-2-0.1) [29]</td>
<td>0.08</td>
<td>High (&lt;0.1) [29]</td>
</tr>
<tr>
<td>AppCI Diff Coefficient D_appmig *10^{-12} (m^2/s)</td>
<td>3.62</td>
<td>High (1-5) [30,31]</td>
<td>2.68</td>
<td>High (1-5) [30,31]</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

- The compressive strength of the designed ternary concrete shows improvements respect to the 100OPC concrete after 28 days of hydration and a more refined pore structure.
- Regarding the durability properties, the designed ternary blends shows lower capillary absorption coefficient and lower chlorides transport in samples cured for 28 or 90 days. In both cases, the longer curing/hydration has more positive effect in the ternary concrete. On the contrary, the carbonation depth is increased in the ternary concrete due to the significant partial substitution of Portland cement by mineral additions (BFS and LF).
- The ternary concrete shows not only ecological benefits, but also durability resistance improvements increasing the sustainability of the concrete.

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pp. 137-163.
EFFECT OF PHASE CHANGE MATERIAL ON TEMPERATURE SHIFTING OF CONCRETE PANELS

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Key words: Phase Change Material, Temperature Shifting, Lightweight Concrete

Abstract. This paper describes the experimental results on effect of phase change material (PCM) on the thermal behavior of concrete panels. The investigation was carried out on lightweight aggregate concrete. The PCM is impregnated into lightweight aggregates and incorporated into concrete during the mixing process. The experiments were conducted using a temperature chamber constructed at King Mongkut’s University of Technology North Bangkok (KMUTNB). Four thermocouples were installed inside the panel and in the chamber to monitor real time temperature change. During the experiment, the heat is supplied to the panel until the mid-plate temperature reaches the 65°C and then ceased. Experimental results indicated that the existence of phase change material does have markedly effect on the temperature movement inside the panel before and after the peak. For example, prior to the peak, more time is required to drive the temperature inside the PCM panels to hit the target temperature of 65°C than the non-PCM panels. Also after the peak, the heat tended to remain at the core of the PCM panels longer. And since the heat is trapped at the core location, this caused the temperature at the inner surface and inside the chamber of the PCM panels lower than the non-PCM panels.
1. INTRODUCTION

Some concrete structures may require to preventing heat from entering the vicinity or escaping to the surrounding areas. Conventionally, there are two ways to achieve that, the first is to increase the duration of heat transfer and the second is to slow down the rate of heat transfer. The first one is obtained by increasing structure thickness, while the latter one can be lower by increasing thermal insulation of material.

Beside from those two, there is also a concept of heat capturing. This heat capturing concept can be achieved through the use of phase change material (PCM). PCM are the materials capable of changing their phase from solid to liquid (or liquid back to solid) at a certain level of temperature. During the phase changing, a certain amount of energy (heat) is taken or released into the surrounding environment [1-3].

In the liquidation process, the temperature of PCM also increases gradually with the increasing ambient temperature. When the temperature reaches the PCM’s melting point, a certain amounts of heat is absorbed at near constant temperature until all PCM transform to the liquid phase. On the other hand, when the ambient temperature decreases, the solidification begins and the stored latent heat is released. The ability of PCM to absorb heat without significant rise of temperature has been proved to increase heat storage inside the structures and slow down the rate of heat transmission (shifting the temperature peak period) [4-20].

There are three groups of phase change materials [6]. Organic group refers to materials like paraffin and some fatty acids. Inorganic group refers to materials like salt hydrate and metallics. Eutectic group refers to a compound between at least two or more components, each of which melts and freezes congruently forming a mixture of the component crystals during crystallization [7].

In construction material applications, the organic PCM such as paraffin is commonly used. The advantages of paraffin include cost effectiveness, wide melting point range, high inertness, high stability (less volume change) and durability (ability to remain effective after numbers of temperature cycles without segregation or efficiency degradation) [5]. In early applications, the paraffin particles were directly mixed with concrete as part of the constituent materials. However, after subjecting several temperature cycles, the surface leakage was often found.

To solve this, the encapsulating technology was introduced. In encapsulation, the paraffin is contained inside closed sphere capsules and then mixed with concrete [8-12]. This technique is quite effective in preventing leakage. However, there are some drawbacks like strength reduction, cost effectiveness and manufacturing process. Cabeza et al [13] conducted actual field tests using commercial type microencapsulated PCM capsule. The results showed improvement in thermal storage of walls with encapsulating PCMs in comparison with conventional concrete wall without phase change materials which leads to an improved thermal inertia as well as lower inner temperatures. Castellón et al. [14] analyzed and compared a concrete cubicle with microencapsulated PCM with a cubicle with no PCM. They also included the effect of ventilation by opening and closing windows at certain times of the day. Their results showed the reduction of daily temperature oscillations up to 4°C and the shift in peak temperatures of the PCM cubicle to later hours. Similarly, there are some experimental work investigating the application of PCM in different component of the house.
such as brick wall [15,16], sandwich panels [17,18], floor and ceiling free-cooling [19] systems.

In this study, instead of using commercial type PCM microencapsulation, the lightweight aggregates impregnated with PCM are introduced. The impregnation is carried out using combinations of heat and pressure. The process is expected to be more efficiency in paraffin insertion, more cost effective and simpler to manufacture. The PCM aggregates are then used in the concrete mixture by partially replacing parts of normal lightweight aggregates from 25% to 100% by volume. The concrete samples are tested in temperature chamber to investigate the effect of PCM on temperature change and movement inside and outside the chamber.

2. EXPERIMENTAL PROCEDURE

2.1 Materials

Materials used in this study consist of:

- Portland Cement Type I
- Fine aggregate: River sand
- Coarse aggregate: Lightweight aggregate (Table 1)
- Phase change material: Paraffin (Table 2)

<table>
<thead>
<tr>
<th>Properties (unit)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kg/m3)</td>
<td>732</td>
</tr>
<tr>
<td>Percent of voids (%)</td>
<td>72</td>
</tr>
<tr>
<td>Bulk specific gravity (Dry)</td>
<td>1.08</td>
</tr>
<tr>
<td>Bulk specific gravity (SSD)</td>
<td>1.25</td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>1.30</td>
</tr>
<tr>
<td>Percent Absorption (%)</td>
<td>16</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Properties (unit)</th>
<th>Standard</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Melting Point (°C)</td>
<td>ASTM D87</td>
<td>57.2-59.9</td>
</tr>
<tr>
<td>Oil Content (%)</td>
<td>ASTM D721</td>
<td>0.4</td>
</tr>
<tr>
<td>Penetration at 25 °C</td>
<td>ASTM D1321</td>
<td>10.0-17.0</td>
</tr>
<tr>
<td>Color</td>
<td>ASTM D156</td>
<td>30</td>
</tr>
<tr>
<td>UV Absorbtion</td>
<td>FBA 178.3710</td>
<td>1.3</td>
</tr>
</tbody>
</table>

2.2 PCM Aggregates preparation

To prepare the PCM aggregates, the aggregates are dried in the oven for 24 hours, submerged in liquid paraffin and put into an autoclave chamber for at least 3 hours. The chamber temperature and pressure is set at 120°C and 15 psi (103.5 kPa), respectively. The impregnation target is set at 16 to 17 %. To ensure impregnation target achievement, samples
are taken out from the container every hour to measure weight difference before and after autoclaving (Fig. 1).

2.3 Experiment

Using aggregates from 2.2, test samples are cast using the mix proportion given in Table 3. The normal lightweight aggregates are gradually replaced by the PCM aggregates at 4 different proportions: 25, 50, 75 and 100% by volume. Three tests are carried out: density test (ASTM C138), compression test (EN12390), and heat insulation test.

![Figure 1: Aggregate’s impregnation level vs. time](image)

**Table 3 Mix Proportions per cubic meter**

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>C (kg)</th>
<th>FA (kg)</th>
<th>W (kg)</th>
<th>LA (kg)</th>
<th>PCM Ag (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100L/0P</td>
<td>477</td>
<td>669</td>
<td>200</td>
<td>488</td>
<td>0</td>
</tr>
<tr>
<td>75L/25P</td>
<td>477</td>
<td>669</td>
<td>200</td>
<td>366</td>
<td>122</td>
</tr>
<tr>
<td>50L/50P</td>
<td>477</td>
<td>669</td>
<td>200</td>
<td>244</td>
<td>244</td>
</tr>
<tr>
<td>25L/75P</td>
<td>477</td>
<td>669</td>
<td>200</td>
<td>122</td>
<td>366</td>
</tr>
<tr>
<td>0L/100P</td>
<td>477</td>
<td>669</td>
<td>200</td>
<td>0</td>
<td>488</td>
</tr>
</tbody>
</table>

The heat insulation is carried out using the heat insulating chamber constructed at KMUTNB (Fig. 1). The 800x800x2000 mm. chamber is constructed of steel plates and covered inside with insulating sheets. The heat source (1500 W spotlight) and the specimen supporter (made of double layer wooden boards) are located as shown in Fig. 2. The 400x400x50 mm specimens are cast with three thermocouples embedded inside at the outer surface, the mid panel and the inner surface. Another thermocouple is located inside the chamber (300 mm away) to measure room temperature’s change. The temperature change is measured and collected automatically every 5 seconds using a data acquisition system connected to a laptop. To begin the test, the heat is supplied to the panel until the core temperature (mid-plate thermocouple) reads 60±3°C and then stopped. The measurement continues for about 5000 seconds after the peak temperature is reached.
3. RESULTS AND DISCUSSION

3.1 Properties of PCM aggregates

The present of paraffin inside the aggregates cause the reduction of absorption value from 16.2% to 0.20% (Fig. 5) due to the effect of paraffin filling permeable voids and lowering porosity. With lowering porosity, the dry specific gravity of the PCM aggregates also increases from 1.062 to 1.300. The specific gravity of PCM aggregates also appeared to be constant regardless of the exposed moisture condition.

<table>
<thead>
<tr>
<th>Properties</th>
<th>LA</th>
<th>PCM</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absorption</td>
<td>16.2</td>
<td>0.20</td>
<td>-98.8</td>
</tr>
<tr>
<td>Specific Gravity (Dry)</td>
<td>1.062</td>
<td>1.300</td>
<td>22.5</td>
</tr>
<tr>
<td>Specific Gravity (SSD)</td>
<td>1.234</td>
<td>1.303</td>
<td>5.6</td>
</tr>
<tr>
<td>Specific Gravity (Apparent)</td>
<td>1.282</td>
<td>1.304</td>
<td>1.7</td>
</tr>
</tbody>
</table>

3.2 Properties of lightweight concrete panel

3.2.1 Compressive strength and absorption

Results from both compression and absorption tests are plotted together in Fig. 2. The compressive strength is found to increase with the increasing PCM aggregate. This is contrary to the concrete mixed with PCM microcapsules in which the compressive strength reduction is commonly found due to the decreasing interfacial bond strength of the sphere microcapsules. In our PCM aggregates, the roughness of aggregate’s surface maintains the interfacial bond strength and the increasing density of aggregate allow the strength to increase. For the absorption, it is found to decrease with increasing PCM aggregate content. This is because the PCM aggregates are less porous and by replacing normal lightweight aggregates with PCM aggregates, the overall absorption gradually decreases.

![Fig. 2 Relationship between compressive strength and absorption](image)
3.2.2 Temperature patterns

The temperature changes at different locations of each concrete type are as shown in Fig. 3a-3e. Prior to the peak (during heat supply), the temperature tends to increase with the exposure time at all locations though as different rates. At the outer surface and the core, the rate of temperature change (Table 5) tends to slow down with the increasing PCM aggregate content. The rate of temperature change reduces from 1.63 to 1.43°C/min and 1.21 to 0.95°C/min as the PCM aggregate replacement ratio increases from 0% to 100% at the outer surface and the core location, respectively.

Table 5 Rate of Temperature Change (°C/min)

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Location</th>
<th>Outer Surface</th>
<th>Core</th>
<th>Inner Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Prior to the peak</td>
<td></td>
<td>After the peak</td>
</tr>
<tr>
<td>100L/0P</td>
<td></td>
<td>1.63</td>
<td>1.21</td>
<td>-0.376</td>
</tr>
<tr>
<td>75L/25P</td>
<td></td>
<td>1.61</td>
<td>1.19</td>
<td>-0.352</td>
</tr>
<tr>
<td>50L/50P</td>
<td></td>
<td>1.60</td>
<td>1.17</td>
<td>-0.333</td>
</tr>
<tr>
<td>25L/75P</td>
<td></td>
<td>1.51</td>
<td>1.16</td>
<td>-0.329</td>
</tr>
<tr>
<td>0L/100P</td>
<td></td>
<td>1.43</td>
<td>0.95</td>
<td>-0.323</td>
</tr>
</tbody>
</table>

After the peak (no heat input), the temperature decreases with time also at different rates. The panels with PCM aggregates tend to release temperature at slower rate than those without PCM aggregates. The rate of temperature reduction at the outer surface and the core location reduces from -0.376 to -0.323 °C/min and -0.218 to -0.191 °C/min, respectively, with the increasing PCM aggregate ratio from 0% to 100%. This indicates the ability of PCM to capture or contain heat and slow down the rate of heat transfer.

The temperature movements of different panel types at the core and inner surface are compared in Fig. 4. Considering the core location (Fig. 4a), the peak temperature is controlled at about 63°C in order to observe to duration to reach peak temperature of each panel. Results show that the duration to reach peak temperature gradually shifted to the right with the increasing PCM aggregates content. This means more time is required to drive to temperature to the peak for PCM panels than for non-PCM panel.

For the inner surface, comparing at the same time, both temperatures curves and peak temperature of the PCM panels appear to be lower than the non-PCM panel (maximum of about 4°C difference) (Fig. 4b). These findings support the previous discussion that the PCM panels allow the heat to travel at slower rate due to the ability to capture and delay heat during the phase changing process. The temperature inside the chamber also affects by the temperature at the inner surface location. With less heat travels through the thickness, the chamber temperature become lower (Fig. 4c).
Fig. 3 Temperature changes at different locations of panel type (a) 100L/0P, (b) 75L/25P, (c) 50L/50P, (d) 25L/75P, (e) 0L/100P
4. CONCLUSION

The use of PCM impregnated aggregate to partially replace the regular lightweight aggregates can improve the thermal properties of concrete significantly. The improvements are observed in the form of 1) the shifting of time to reach peak temperature and 2) to temperature difference (lower) at different locations of the panels. The maximum shifting duration found in this study is about 870 seconds or 14.5 minutes. The maximum temperature difference found in this study is about 4°C and 0.71°C at the inner surface and inside the chamber, respectively.
REFERENCES


REPLACEMENT OF STEEL WITH GFRP AS INTERNAL REINFORCEMENT FOR CORROSION-FREE REINFORCED CONCRETE STRUCTURES

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Keywords: Corrosion; deflection; GFRP bars; GFRP spirals; reinforced concrete; tension stiffening; compression.

Abstract.

Steel-reinforced concrete structures are prone to corrosion that cost billions of dollars in repair and maintenance every year in addition to loss of productivity. In 2010, the total annual cost of corrosion globally was estimated at US $2.2 trillion which amounted to about 3% of the world’s GDP. Obviously, an alternative is needed to build sustainable infrastructure. Glass fibre reinforcement polymer (GFRP) bars present a feasible and cost effective solution to the steel corrosion problem in structures. Despite many advantages of GFRP, most designers are still reluctant to use GFRP as the main reinforcement in concrete members due to its different behavior than steel and a lack of experimental data.

The aim of this paper is to develop a better understanding of the overall behaviour of GFRP as internal reinforcement so that they have better confidence in using it as a sustainable material. Significant results from an extensive experimental program underway at the University of Toronto are briefly presented. The part of the program discussed in this paper involved testing of 40 GFRP reinforced beams, 60 GFRP direct tension specimens and 20 GFRP confined columns in which the behavior of GFRP-reinforced concrete in flexure, shear, tension and compression was investigated. A new tension-stiffening model was developed that significantly improved capability for the prediction of deflection and stiffness of GFRP-reinforced concrete beams. Another significant conclusion drawn from this research is that GFRP spirals not only can be used efficiently as primary lateral reinforcement in columns but also confine the concrete core more effectively than steel spirals. GFRP bars were also found to resist about 700 MPa compression which is about 60% of their tensile capacity.
1. INTRODUCTION

The aging and deterioration of the world’s reinforced concrete infrastructure has long been identified as a major engineering challenge for the foreseeable future. While this deterioration can be attributed to several causes, the corrosion of the internal steel reinforcement is a principle factor. The total annual cost of corrosion worldwide in the year 2010 was estimated at USD $2.2 trillion which amounts to approximately 3% of the world’s GDP [1]. The costs associated with corrosion can be unquantifiable when loss of life occurs. In the catastrophic failure of the Laval Overpass in Quebec Canada, corrosion of the steel reinforcement was identified as a potential contributing factor towards the structural failure that resulted in loss of life [2].

With advances in polymer technology, Fibre Reinforced Polymer (FRP) bars have emerged as a viable sustainable solution to the problem of steel corrosion in newly built and rehabilitated structures. Although a variety of FRP bars are commercially available, glass fibre reinforced polymer (GFRP) reinforcement is one of the more cost effective products with significant cost advantage over stainless steel. While engineers have gained much confidence in using GFRP externally to retrofit systems to restore the structural performance and providing new corrosion resistant systems, there is still hesitancy associated with the use of internal GFRP bars as an alternative to steel reinforcement in the case of new construction.

Significant advances in research in the field of the GFRP-RC have been made in the past decades with the pioneering research linked with groups like ISIS Canada Network Association [3] and have resulted in several design recommendations and guidelines for designing GFRP reinforced concrete members [3-6]. However, in spite of these advances, most designers are still hesitant to use GFRP as the main reinforcement in RC members primarily due to a lack of test data and analytical procedures compared with that available for steel-reinforced structures.

Although GFRP is a viable alternative to steel, it presents its own challenges in terms of having different properties than steel. One of the design challenges related to GFRP is the relatively low modulus of elasticity compared with conventional steel which influences the cracking behaviour and results in much larger deflections in GFRP-RC flexural members. This results in GFRP reinforced concrete design being largely controlled by service conditions that makes it essential to predict the deflection in GFRP reinforced members more accurately. The lower modulus of elasticity can also adversely affect the shear response by widening cracks and resulting potentially in reducing the efficiency of shear reinforcement.

The behavior of GFRP bars in compression is an even more unexplored territory. The majority of the design codes in North America either have no provisions regarding the application of GFRP in compression members [4] or explicitly prevent designers from using GFRP in compression members [6]. This is mostly due to a lack of sufficient data on the behavior of GFRP-RC columns and the uncertainty in the response of GFRP bars in compression. However, there are several potential reasons in using GFRP in columns; the leading one being that GFRP spirals can be a solution in preventing concrete cover deterioration in columns. The spalling of concrete cover occurs due to the expansion resulting from steel spiral corrosion, which consequently results in a drop of load carrying capacity. The structural integrity of the column is also significantly compromised due to the loss of confinement.

The research presented here is part of a large program at the University of Toronto investigating the performance of GFRP bars as internal reinforcement in an assortment of concrete structures. Initially, work commenced with the evaluation of high strength GFRP bars [7]. Because of the anchorage concerns for high strength GFRP bars, Vint and Sheikh [8] tested a variety of bent, straight and anchored GFRP products for bond and development performance.
Getzlaf [9] investigated the flexure behaviour of GFRP reinforced concrete including the deformability of balanced, under- and over-reinforced beams presenting new methods for evaluating member deformability. Johnson [10] extended this work by testing large scale beams and providing a base to determine member shear strength, a critical requirement for understanding member based deformability. Kharal [11] investigated the serviceability characteristics of GFRP-RC beams and investigated the role of tension stiffening. Tavassoli et al [12, 13] studied the behavior of GFRP as internal reinforcement in columns. This paper integrates the above mentioned studies to provide a broad view of the developments in this field and aims at creating a better understanding of the overall behaviour of GFRP-RC.

2. BEHAVIOUR OF GFRP-RC BEAMS

2.1 Phase 1 – Flexure

One of the more contentious topic in GFRP-RC design is regarding its deformability and performance in flexure. In order to investigate the flexural behaviour of GFRP-RC beams more thoroughly, 15 GFRP-RC beams (200 x 325 x 3650 mm) were tested [9]. One of the major contributions of the research included a new evaluation of the moment of inertia to better predict flexural deflection by modifying the ACI 440.1R [6] formulation of the effective moment of inertia to better correlate with the test data (equation 1). The optimal results were found when the parameter $\gamma_d$ is taken as 0.8.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1.0 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \gamma_d I_{cr} \leq I_g$$

This study also presented an in-depth discussion into issues relating to both the performance and deformability of GFRP RC members. Initially, GFRP-RC structures were not capacity designed in shear to achieve the desired flexural capacity and failure mode. Hence a new method was proposed in which the influence of shear could potentially be incorporated. The new proposed formulation for the effective moment of inertia was included in the equations for the member deflection deformability. The research concluded that the evaluation of the shear behaviour was critically important to predict member response.

2.2 Phase 2 – Flexure and Shear

In this second phase of the beam tests, the behaviour of shear- and flexure-critical GFRP-RC beams was investigated. The investigation consisted of constructing, testing, and analysis of twenty-four large scale GFRP beams [10]; double the size in height and width of those tested in Phase 1. Tests were planned to correlate the results of phase one into the behaviour of large scale reinforced concrete specimens. The study was also designed to complement the previous beam tests by investigating the influence of the type of transverse reinforcement. Types of transverse reinforcement used in this program was consistent with those tested by Vint and Sheikh [8].

All the specimens had the same overall dimensions of 400 mm x 650 mm x 3650 mm and a longitudinal reinforcement ratio of 1%. The typical specimen geometry without shear reinforcement and the test set-up is shown in Figure 1. It should be noted that the load was applied monotonically and testing was displacement controlled.
Table 1 gives the summary of the experimental results of an example specimen JSV40-NT and Figure 2 shows the load versus deflection response of a few beam specimen tests in comparison with JSV40-NT, all of which had concrete strength of 40 MPa, GFRP bar type B1 with sand-coated surface profile and all other parameters constant. The alphanumerical characters NT, 22B, 40B and 50B in the specimen name indicate, respectively, no transverse reinforcement, 0.22%, 0.40% and 0.50% transverse reinforcement provided by bent bar stirrups.

The test results demonstrated that the traditional definition of ductility did not appropriately explain the GFRP-RC beams behaviour. The load versus deflection responses in Figure 2 clearly show that GFRP-RC beams underwent significant deformation that resulted in their ability to absorb energy; this proves that a lack of ductility in GFRP does not necessarily translate to brittle behaviour. Thus the results of the research revealed that GFRP-RC structures have the potential to be designed for a non-brittle failure.

The choice of transverse reinforcement was found to have considerable impact on both the flexural failure load and ultimate beam deflection. Additionally, prior to failure, wider cracking was visible for GFRP-RC beams with relatively lower shear reinforcement indicating serviceability issues. This can be seen from the results of the specimen JSV40-NT in which relatively wide flexural cracking started occurring at a load of 149 kN and inclined shear cracks were first visible at 300 kN. At 327 kN, shear failure took place with one dominant shear crack.

Table 1: Summary of JSV40-NT Experimental Results

<table>
<thead>
<tr>
<th></th>
<th>JSV40-NT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Cylinder Strength, ( f'_c ) (MPa)</td>
<td>40</td>
</tr>
<tr>
<td>Longitudinal Stiffness, ( EA ) (MN)</td>
<td>170.4</td>
</tr>
<tr>
<td>Cracking Load, ( P_{crack} ) (kN)</td>
<td>149</td>
</tr>
<tr>
<td>Failure Load, ( P_{fail} ) (kN)</td>
<td>327</td>
</tr>
<tr>
<td>Peak Deflection, ( \delta_{fail} ) (mm)</td>
<td>7.6</td>
</tr>
<tr>
<td>Peak Upper Layer Longitudinal Bar Strain at Failure (x 10(^{-3}))</td>
<td>2.75</td>
</tr>
<tr>
<td>Peak Lower Layer Longitudinal Bar Strain at Failure (x 10(^{-3}))</td>
<td>3.18</td>
</tr>
<tr>
<td>Crack Angle at Failure (°)</td>
<td>41</td>
</tr>
</tbody>
</table>
In the analytical part of this study, the effectiveness of predicting the GFRP-RC beam behaviour in comparison to the experimental results was investigated. A 2-Dimensional nonlinear finite element analysis program named VecTor2 [14] was used to simulate the experimental beam results. In order to improve the accuracy of the analytical predictions, Vint and Sheikh [8] GFRP bond model was adopted in the analysis. Various tension stiffening models were utilized in the analysis to evaluate the effectiveness of each on the overall response of the members. Figure 3 shows the comparison of the experimental versus analytical results in terms of load versus deflection for specimen JSV40-NT.

It was determined that when tension stiffening was not at all considered in the analysis, the results obtained showed very soft and unreasonable beam response. Results from the other specimens also corroborated this behaviour. Thus it was concluded that in GFRP-RC flexural members, the consideration of tension stiffening for deflection calculation is much more important than steel-RC members. This is plausible since GFRP has approximately three times lower stiffness than steel.
It was then decided to consider Benz 2003 (B-03) and Collins and Mitchell (C-87) tension stiffening models to study the effect of each on the specimen behavior. While this resulted in comparatively better results, neither of the models predicted reasonably accurate results. The mid-span deflection was underestimated for most of the load range irrespective of the tension stiffening model used; this under prediction of deflection is specifically undesirable at service loads. Also neither of the models influenced the post-cracking stiffness since the models do not take into account the reduced GFRP modulus that affects the concrete-to-bar bond and crack widths which are critical to tension stiffening.

Available analytical procedures were found to be inaccurate because either they did not take tension stiffening into account or the deflection was derived based upon tension stiffening formulations for steel-reinforced concrete. The accuracy of the prediction of deflection depends on the determination of accurate effective moment of inertia which, in turn, is dependent upon tension stiffening behavior. The phenomenon of tension stiffening in GFRP reinforced concrete structures must, therefore, be accurately taken into account in the analysis. It was thus decided to carry out an experimental study involving direct tension tests to accurately evaluate the tension stiffening behaviour of GFRP-RC.

3. TENSION STIFFENING BEHAVIOUR IN GFRP-RC

The testing program consisted of 52 GFRP reinforced concrete and 8 steel reinforced concrete specimens. All the uniaxial test specimens consisted of a concrete prism that was square in cross-section and 1000 mm long cast around a single reinforcing bar. The dimensions of the square cross-sections were either 100 x 100 mm or 150 x 150 mm. In case of steel reinforcement 10M and 15M bar sizes were used and for GFRP reinforcement the bar sizes were 13 mm, 16 mm and 19 mm. Each steel- and GFRP-RC specimen configuration was repeated in duplicate to measure the degree of variation between the test results. The test parameters investigated included bar diameter, reinforcement ratio, concrete strength and bar type. The three bar types used were made by different manufacturers and their main difference lied in the bars’ surface profiles; the first bar was helically wrapped with fibers, the second was heavily sand-coated and the last one had ribbed bar surface. A typical set-up of a direct tension specimen and the corresponding GFRP-RC response is shown in Figure 4.

Figure 4. (a) Typical direct tension specimen test set-up; (b) Typical GFRP-RC specimen response
The predictions from the currently available models (Figure 3) clearly showed a need for the development of a more accurate GFRP-RC tension stiffening model. Prior to attempting the development of such a model, it was thought prudent to first identify the various parameters that affect tension stiffening. After making various comparisons between specimens, it was determined that reinforcement ratio, concrete strength and bar diameter had no significant effect on tension stiffening if shrinkage was taken into account in determining the behaviour of specimens. However, GFRP bar type was found to influence tension stiffening to some extent. GFRP members were also found to exhibit greater tension stiffening than steel-RC [11]. This is because the concrete contribution is proportionally higher in GFRP-RC members than steel-RC members due to the low stiffness of GFRP reinforcement. Also, in steel-RC specimens the effect of tension stiffening is minimal after steel yields at a strain of about 0.002 while in GFRP-RC specimens, tension stiffening effect is present till much higher strain values, approximately 0.014-0.015, since there is no yielding in GFRP bars.

The proposed model accounted for the parameters that affected the tension stiffening behavior of GFRP-RC specimens. Using the proposed model, the average concrete tensile strength for GFRP reinforced specimens after cracking can thus be determined by the following proposed relationship.

\[ f_c = \frac{f'_t}{1 + \left( \frac{E_b}{200000} \right) \cdot \beta_1 \cdot (\varepsilon_{cf} - \varepsilon_{cr})} \]  

where \( f_c \) is the average concrete tensile strength after cracking; \( f'_t \) is the concrete tensile strength taken as \( 0.38(f'_c)^{0.5} \); \( \gamma \) is a factor based on surface profile of the reinforcing bar; \( \varepsilon_{cf} \) is the average net concrete strain; \( \varepsilon_{cr} \) is the concrete tensile cracking strain; \( E_b \) is the modulus of elasticity of the GFRP bar (MPa); and \( \beta_1 \) is a constant fixed at 1400. The factor \( \gamma \) depends on the GFRP bar type and is taken as 1.5 for helically wrapped bars, 1.0 for sand-coated bars and 0.8 for ribbed bars for the best correlation with the test data.

The proposed model’s application to the specimens tested as part of this study resulted in more accurate predictions than those obtained from any of the existing models. The analytical results from the proposed model have been compared with the experimental results for a representative specimen in Figure 6. The alphanumerical characters in the specimen names can be explained as follows: C30-19V-100(1) means concrete strength of 30 MPa, 19 mm bar size of Type V GFRP in a 100 mm square prism, 1 denotes the first specimen in the duplicate set of two specimens. Type V GFRP bars had sand-coated surface. The predicted response for the GFRP-reinforced specimens was found to be more accurate than that from any of the currently existing models for all bar types, concrete strengths and reinforcement ratios.
Figure 6. Predicted response of tension stiffening by the proposed model for $\rho_b = 3\%$ and reinforced with GFRP bar with sand-coated surface profile (Type V)

After the validity of the GFRP tension stiffening model had been proven, it was used to predict the GFRP-RC beam behaviour again by using VecTor2 program. The only parameter that was varied from the previous analysis was the proposed tension stiffening model. The predicted result for the load versus deflection response of the beam JSV40-NT utilizing the proposed tension stiffening model is shown in Figure 7. It can be seen that the new model’s prediction is considerably more accurate than all the prior predictions (Figures 3 and 7); not only does the analytical ultimate deflection more closely simulate the experimental value but the accuracy of post-cracking stiffness prediction has also improved.

![Figure 7. VecTor2 prediction by proposed tension stiffening model for Specimen JSV40-NT](image)

4. GFRP BARS IN COMPRESSION AND GFRP-RC COLUMNS

An experimental program was undertaken to evaluate the behavior of GFRP bars in compression. A total of 34 bar specimens of 25 mm GFRP bar were tested under direct compression. To establish the relationship between length and strength, the specimen length was varied between 50 mm and 600 mm. The average compressive modulus of elasticity of bars for all the specimen lengths was determined to be approximately 60 GPa almost equal to the modulus of elasticity determined for the same bars in tension.

The compressive strength versus the unbraced length for the 25 mm GFRP bars is shown in Figure 8. The maximum compressive strength was observed to be approximately 730 MPa in GFRP bars with nominal un-braced length of up to 230 mm; this is approximately 60% of the bar tensile strength. The failure mode was a combination of buckling and material failure when the bar length varied between 230 mm and 315 mm. For the nominal un-braced length larger than 315 mm, the failure occurred by buckling with the ultimate compressive strength being an inverse function of the un-braced bar length.
Based on the encouraging results obtained from the direct compressive tests of GFRP bars, it was decided to test GFRP reinforced concrete columns under combined axial, shear and flexural loads [12, 13]. A total of twenty columns, reinforced internally with GFRP, were tested under constant axial load while subjected to cyclic lateral displacement excursions that simulated earthquake loading. All the columns tested in this study comprised of a 356 mm diameter circular column connected to a 480×700×800 mm reinforced concrete stub. The column part represented the part of a column or a structural member in a regular building frame between the section of maximum moment and the contra-flexure point. The stub, cast integrally with the column, represented a discontinuity like a footing or a beam column joint. The specimen with the test set-up and the corresponding lateral loading cycles is shown in Figure 9.

![Figure 8. Compressive strength of GFRP bars as affected by unbraced length](image)

Figure 8. Compressive strength of GFRP bars as affected by unbraced length

The main variables investigated were the type of transverse reinforcement and their sizes, the volumetric ratio and spacing of transverse reinforcement, longitudinal bar type (steel vs GFRP) and the level of axial load. The first nine specimens had GFRP longitudinal bars and spirals, the last eight specimens contained steel longitudinal bars and GFRP spirals. The column
size, concrete strength, and amount and distribution of longitudinal bars were kept constant in order to make direct comparisons to evaluate a single variable.

The behaviour of columns in the form of shear vs. tip deflection and moment vs. curvature was studied in detail. The behavior of a column in which six 25 mm diameter GFRP bars were used as longitudinal reinforcement and 12 mm spirals with 160 mm pitch was used as lateral reinforcement can be seen in Figure 10 [12]. A stable column behaviour with a large energy dissipation capacity and high level of deformability can be observed despite large spiral spacing. The lateral GFRP provided continuous support even at large deformations until the columns failed by a combination of crushing and buckling of the longitudinal bars in compression accompanied by the crushing of the concrete core in the most damaged zone. Columns with 50 mm spiral pitch were significantly more ductile and had much larger energy dissipation capacity [13]. The results showed excellent response for GFRP bars in compression as well as GFRP spirals as lateral reinforcement.

The columns that contained GFRP longitudinal bars and spirals were also compared with the conventional steel columns. The main difference was the softer response of the GFRP-reinforced columns which results in larger P-delta moment and reduces the maximum shear load capacity and increases deformability relative to the steel reinforced columns.

In order to improve the soft column response, eight more specimens containing steel longitudinal bars and GFRP spirals were tested. It was hypothesized that the longitudinal conventional steel bars would increase the stiffness and the strength of the column to the desired level while the GFRP transverse reinforcement would provide the confinement and corrosion resistance as desired. The comparison made between the behaviour of columns in Figures 10 and 11 shows that the columns with steel bars are less deformable but have higher strength and stiffness in comparison to the GFRP reinforced columns. Column P28-LS-12-160 had six 25 mm steel longitudinal bars while P28-C-12-160 contained six 25 mm GFRP bars. All the other parameters between the two columns were maintained the same with the only difference being the type of longitudinal reinforcement.
The main advantage of GFRP spirals over steel spirals was observed for the strain range beyond steel yielding. Steel spirals are very efficient up to a strain of 0.002 when they yield. However, beyond the yield strain, the steel spirals are unable to effectively confine the concrete core. GFRP has linear elastic behaviour until rupture at a strain of about 0.02. Therefore, GFRP lateral reinforcement has the capability of continually providing resistance with the same stiffness, against the expansion of the concrete core, beyond the steel yield strain. This was corroborated by the results since column failure was initiated mostly due to the crushing of concrete core and/or yielding of the longitudinal steel bars. Only in a few columns and that also at the last stages of the column’s life was the rupture of GFRP spirals was observed.

CONCLUDING REMARKS

In this paper, selected results are presented from an extensive research program, which involves investigation of GFRP bars as internal reinforcement in concrete structures. The structures investigated included beams, slab and columns. Hence the performance of GFRP bars was investigated in compression, tension, shear and cyclic loads. The main purpose of this investigation was to gain a better understanding of the overall behaviour of GFRP reinforced specimens under various load applications. This research aims to help engineers gain more confidence in using GFRP-RC as a sustainable alternative to steel-RC structures. The following conclusions can be drawn from this research.

Beams reinforced with GFRP longitudinal bars and stirrups displayed excellent behaviour under shear and flexural loads. GFR-reinforced concrete design is largely controlled by service limit conditions due to the lower stiffness of GFRP which makes it crucial to accurately predict the deflection in GFRP reinforced members. The analytical member response from a nonlinear finite element analysis using the existing tension stiffening models for concrete was found to overestimate the stiffness and underestimate the ultimate mid-span deflection of beams. A detailed experimental and analytical study was carried out on the tension stiffening of concrete due to GFRP based on which a model was developed which predicted the behaviour of beams significantly more accurately than the existing models.

In monotonic compression, GFRP bars were able to resist stress levels in excess of 700 MPa which was higher than 60% of their tensile capacity. The concrete columns that were reinforced with GFRP longitudinal bars and spirals and tested under constant axial load and lateral displacement excursions simulating earthquake forces displayed large amount of deformability and energy dissipation capacity. The overall strength and ductility of GFRP confined columns was either similar or better than that of steel confined columns. The use of GFRP longitudinal bars resulted in a softer member response due to the low stiffness of GFRP.
columns in comparison with steel bars. Due to the linear elastic behaviour of GFRP up to a rupture strain of approximately 2%, the concrete core was confined and longitudinal bars supported more effectively than they would be by steel spirals. This is mainly due to the fact that steel stiffness drops significantly after yield at a strain of 0.2%. An optimum and sustainable solution against corrosion of reinforcement in columns, for both non-seismic and seismic regions, appears to be the use of longitudinal steel bars and GFRP spirals.

ACKNOWLEDGEMENT

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REFERENCES


Abstract

The combination of steel and concrete in the forming of reinforced concrete is the most frequently used construction material in the world. Due to the increasing awareness of the impact on the environment of the production processes of materials and construction of infrastructure, the life span of the most important civil structures has recently been increased by up to 100 years. Among the durability issues that can appear in the material during such long periods, the corrosion of steel is one of the most relevant. An option to avoid such problems entails reducing the steel elements embedded in concrete, whether in the form of fibres or re-bars, or substituting them for other materials. The development of polyolefin fibres has become one of the most attractive options available in carrying this out.

In order to assess the suitability of polyolefin fibres as a substitution for steel fibres, a thorough experimental campaign has been carried out. In addition, a hybrid fibre-reinforced concrete with addition of both steel and polyolefin fibres has also been tested. Compressive strength, indirect tensile strength and modulus of elasticity tests have been performed. Moreover, fracture tests have been conducted in order to evaluate if the properties provided by the fibres enable a reduction or total substitution of steel-bar reinforcement.

The results have shown that the simultaneous use of short steel-hooked fibres and longer polyolefin ones is particularly well suited for structural applications where only a reduced amount of cracking is allowed, enabling a reduction of the use of steel and therefore decreasing environmental impact. In addition, the presence of polyolefin fibres improves the toughness and ductility of the PFRC, with it being well suited for its use in construction. The study also provides values of the contribution of the fibres to the residual strength of concrete which deliver relevant information for the structural designer.

1 INTRODUCTION

Through the development of steel reinforced concrete (RC), concrete technologies steered towards a position of supremacy in construction, playing a key role in the construction of infrastructure in modern society [1]. The good performance of steel combined with concrete is widely known and its use in building industry has been profuse. However, steel is not only highly corrosive in nature but also expensive for purchasing, storing and handling. Therefore, several durability issues have concerned engineers and industry. In recent years, the efforts of the plastic industry have allowed the production of a new generation of polyolefin-based synthetic macro-fibres that are inert in an alkaline environment and provide concrete with
structural capacities to substitute traditional concrete steel reinforcement [2, 3]. Polyolefin fibres have good tensile properties, abrasion resistance and excellent endurance in chemically aggressive environments which, added to a low cost, place them as an alternative to steel solutions when meshes or fibres are used [4]. Polyolefin fibre-reinforced concrete (PFRC) has considerable residual tensile strengths [5, 6, 7, 8, 9, 10] and a lower weight in comparison with steel fibres. Both researchers and the construction industry have made significant advances in the applications of plastic fibres to reinforce concrete [11]. Mainly due to the lower cost of the material and the lack of corrosion when subjected to hazardous environments, the use of this type of fibres has become attractive [12, 13].

Additionally, PFRC has multiple sustainability benefits that have justified new research, given that in both use and end-of-life cycles the material has a limited impact compared with the common practice of using steel reinforcing mesh or steel fibres [14]. The lower dosages in terms of the weight needed to reach similar strengths reduce the costs of transport and thus the carbon footprint of the life cycle of the material. Derived from the production methods, significant decreases of carbon emissions compared with the production of steel can also be found in the literature [15]. Plastic fibres can be directly mixed with concrete without clustering problems and with a reduced impact on workability. Even by using ready-mix trucks, published research has shown that the loss of fibres is somewhat limited in comparison with steel fibres [16]. In addition, the handling of this type of fibres is safer and lighter and saves time in operations, such as preparation and placing of the wire mesh. These aspects permit continuous production of concrete setting with a consequent reduction of labour costs to about half of those in the case of using steel [17]. Hence, PFRC has become an appealing solution that provides certain additional advantages if the whole life cycle is considered [18].

Beyond such reductions of the total cost of the material and environmental benefits, there are some mechanical topics to be considered. Among them, one of the most important is the control of the cracking processes which is directly related with durability of conventional reinforced concrete (RC). Even though previous works have reported that macro-synthetic fibres can effectively control plastic shrinkage cracking, the weak point of PFRC appears for the initial crack openings usually related to service limit states (SLS). In one sense, the limitations at such cracking stages are focused on avoiding brittleness. In another, the crack width in SLS is related with the penetration of water and other components that can both deteriorate concrete and accelerate corrosion of the steel reinforcement.

The intention of this study is to discuss the influencing parameters of the behaviour of PFRC and in particular the possible benefits in terms of sustainability. The mechanical properties and fracture behaviour of PFRC have been assessed to sustain the proposed substitution of steel-bar reinforcement. Furthermore, with the aim of producing a high-performance concrete in terms of sustainability, the crack openings at SLS have been improved by combining a small amount of 35mm-long steel-hooked fibres with a medium amount of 60mm-long polyolefin fibres. Given that the latter provides an efficient use of the materials, as well as improved mechanical behaviour in both service and failure states, it assures an improved durability for the concrete piece.

2 CONCRETE PRODUCTION AND FRESH-STATE PROPERTIES

The component materials included Portland cement type EN 197-1 CEM I 52.5 R-SR 5
and a mineral admixture of limestone used as a micro-aggregate. They have a specific gravity and Blaine surface of, respectively, 2700 kg/m³ and 400-450 m²/kg. The calcium carbonate content of the limestone powder was higher than 98%, and less than 0.05% was retained in a 45 µm sieve. A polycarboxylate-based superplasticizer named Sika Viscocrete 5720 with a solid content of 36% and 1090 kg/m³ density was employed. The mixtures were made with siliceous aggregates composed of two types of gravel with a size of 4-8mm and 4-12 mm and sand of 0-2 mm. The maximum aggregate size was 12.7 mm. Polyolefin embossed straight fibres with a chemical surface treatment were employed with two lengths of 60 and 48mm. Figure 1 shows the main characteristics of the fibres. For the paste design, the cement required was at least 375 kg/m³ with 1.25% of cement weight of Sika Viscocrete-5720 admixture and a water-to-powder ratio of 0.50. The limestone powder addition remained at 200 kg per m³ of concrete. The proportions of the aggregates were those obtained for the maximum dry packing density.

On the basis of the self-compacting concrete (SCC) mix proportioning, a conventional concrete (VCC) was designed with the challenge of providing analogous fracture energy to the plain concrete. Nonetheless, analysis should evaluate the energy added by fibres and the structural behaviour in the post-cracking region of the curves. The mix proportioning of both types of concrete, SCC and VCC, can be seen in Table 1.

![Figure 1: Properties and visual aspect of polyolefin macro-structural fibres](image)

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Cement</th>
<th>Limestone powder</th>
<th>SP (%CEM)</th>
<th>Water</th>
<th>Sand</th>
<th>Grit</th>
<th>Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC</td>
<td>375</td>
<td>200</td>
<td>1.25 %</td>
<td>187.5</td>
<td>918</td>
<td>245</td>
<td>367</td>
</tr>
<tr>
<td>VCC</td>
<td>375</td>
<td>100</td>
<td>0.75 %</td>
<td>187.5</td>
<td>916</td>
<td>300</td>
<td>450</td>
</tr>
</tbody>
</table>

Through use of SCC and VCC and with the aim of comparing their behaviour, two batches of each mixture with 0, 3, 4.5, 6 and 10kg/m³ of 60mm-long polyolefin fibres were produced. The five SCC mixtures were respectively named: SCC, PFR-SCC3, PFR-SCC4.5, PFR-SCC6 and PFR-SCC10. The corresponding five mixtures that used VCC were, consequently, named as VCC, PFR-VCC3, PFR-VCC4.5, PFR-VCC6 and PFR-VCC10 (see Table 2). One more mixture was manufactured with 10kg/m³ that also used a bond improver admixture named Sikatell 250 (see reference [5]). The latter was named PFR-SCC10M.
Two mixtures were defined in the initial step with the aim of comparing the results of PFRC with those using 35mm-long steel-hooked fibres added to the SCC formulation. These two mixtures were labelled as SFR-SCC26 and SFR-SCC39. The SFR dosages were chosen to coincide in terms of volume fraction with the relative proportions of PFR-SCC3 (0.33%) and PFR-SCC4.5 (0.49%), as can be seen in Table 2. Finally, a hybrid mixture was defined, using 26 kg/m³ of steel fibres plus 4.5 kg/m³ of polyolefin fibres added to the SCC formulation. The latter is described in more detail in Section 5 of this paper.

With each type of concrete, two prismatic specimens of 600x150x150mm³ and nine cylindrical specimens with a height of 300mm and diameter of 150 mm were cast. All the specimens were stored at room temperature for the first 24 hours and then kept in a climatic chamber at 20ºC and 95% of relative humidity for 28 days until the age of testing.

Concerning the pouring methods, in the SCC mixes all the specimens were filled in a single pour from one of the sides of the mould and compacted and levelled off by the action of their own weight. Those specimens prepared with VCC were filled following the procedure stated in the standards [19, 28] and compacted for 10 seconds by means of a vibratory table. Both consolidation processes might affect the distribution and orientation of the fibres and, as a consequence, the fracture behaviour of the specimens.

The fresh-state behaviour of SCC mixes was assessed by using the slump flow test [20] and the V-funnel test [21]. The results can be seen in Figure 2. The tendencies were as expected: a decrease of slump flow spread as the amount of fibres increased and an increment of the emptying times of the V-funnel proportionally with the fibre dosage. In addition, the influence of steel-hooked short fibres in mixture SFR-SCC26, which corresponds in volume fraction to the formulation PFR-SCC3, revealed the good performance of the polyolefin fibres. The latter can be easily observed in Figure 2 and be explained by the non-rigid nature of the fibres.

![Figure 2: Fresh-state results of SCC with several fibre dosages.](image)

Regarding VCC mixes, their fresh-state properties were addressed by means of the slump test [22]. In order to maintain the fresh properties similar for all the formulations the amount...
of superplasticizer used was increased for VCC10 from 0.75% to 0.82% with respect to cement weight. The results were in all the VCC mixes around 150 mm of descent of the mass.

3 EVALUATION OF THE MECHANICAL PROPERTIES

The compressive strength, modulus of elasticity and indirect tensile strength tests were performed by using the cylindrical specimens. All these tests were carried out by following some of the most employed standards [23, 24, 25], using three specimens at 28 days of age. The mean values can be seen in Table 2. Regarding the modulus of elasticity of the SCC mixes, there was not a clear tendency. Conversely, it would seem that the addition of fibre contents above 3 kg/m³ might worsen the compaction of the concrete and consequently reduce the test results. Analogous analysis could be observed in the VCC mixes. Compressive strength and indirect tensile strength showed no variations in the values out of the typical experimental scatter, although some slight reductions were noticed for the highest fibre dosages (10 kg/m³).

Table 2: Mechanical properties

<table>
<thead>
<tr>
<th>Fibre Content (kg/m³)</th>
<th>Volume fraction, ( V_f ) (%)</th>
<th>( E ) (GPa) (c.v.)</th>
<th>( f_{ck} ) (MPa) (c.v.)</th>
<th>( f_{ct} ) (MPa) (c.v.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFR-VCC</td>
<td>-</td>
<td>34 (0.04)</td>
<td>34 (0.04)</td>
<td>3.9 (0.10)</td>
</tr>
<tr>
<td>PFR-SCC</td>
<td>-</td>
<td>36 (0.03)</td>
<td>39 (0.01)</td>
<td>3.8 (0.14)</td>
</tr>
<tr>
<td>PFR-VCC3</td>
<td>3</td>
<td>30 (0.03)</td>
<td>31 (0.02)</td>
<td>3.6 (0.03)</td>
</tr>
<tr>
<td>PFR-SCC3</td>
<td>3</td>
<td>36 (0.06)</td>
<td>39 (0.01)</td>
<td>4.0 (0.09)</td>
</tr>
<tr>
<td>PFR-VCC4.5</td>
<td>4.5</td>
<td>31 (0.07)</td>
<td>31 (0.07)</td>
<td>4.2 (0.12)</td>
</tr>
<tr>
<td>PFR-SCC4.5</td>
<td>4.5</td>
<td>32 (0.01)</td>
<td>38 (0.06)</td>
<td>4.2 (0.20)</td>
</tr>
<tr>
<td>PFR-VCC6</td>
<td>6</td>
<td>30 (0.03)</td>
<td>33 (0.04)</td>
<td>3.8 (0.07)</td>
</tr>
<tr>
<td>PFR-SCC6</td>
<td>6</td>
<td>32 (0.02)</td>
<td>37 (0.02)</td>
<td>4.2 (0.20)</td>
</tr>
<tr>
<td>PFR-SCC6-48</td>
<td>6</td>
<td>32 (0.02)</td>
<td>35 (0.05)</td>
<td>4.5 (0.08)</td>
</tr>
<tr>
<td>PFR-VCC10</td>
<td>10</td>
<td>28 (0.04)</td>
<td>28 (0.03)</td>
<td>4.1 (0.06)</td>
</tr>
<tr>
<td>PFR-SCC10</td>
<td>10</td>
<td>30 (0.02)</td>
<td>37 (0.02)</td>
<td>4.6 (0.09)</td>
</tr>
<tr>
<td>SFR-SCC-26</td>
<td>26</td>
<td>34 (0.01)</td>
<td>41 (0.02)</td>
<td>5.3 (0.15)</td>
</tr>
<tr>
<td>SFR-SCC-39</td>
<td>39</td>
<td>32 (0.01)</td>
<td>43 (0.01)</td>
<td>6.2 (0.07)</td>
</tr>
</tbody>
</table>

In order to assess the durability performance, permeability tests according to the standard [26] were performed. As can be seen in the results shown in Figure 3, the presence of fibres did not influence the connected porosity network of concrete. The value of penetration of water under pressure of all the VCC mixes was greater than the equivalent formulation of SCC in all the mixes. The mixes made with SCC were suitable for use in extremely harmful environments in accordance with EHE-08 [27]. VCC mixes were apt for use in medium-hazardous environments (according to EHE-08) due to a more permeable porous structure. The same conclusions were found for plain and fibre-reinforced mixes. Therefore, it could be
argued that the presence of the fibres had hardly any influence on the VCC and SCC results.

**Depth of penetration under water pressure (mm)**

![Diagram showing depth of penetration under water pressure for different materials and environments.]

Figure 3: Depth of penetration under water pressure (mm)

### 4. RESIDUAL LOAD-BEARING CAPACITY

The residual load-bearing capacity is the basis of the structural character of fibre reinforced concrete (FRC). In the fracture tests, the residual strengths obtained are the reference values to be used in the structural design and are critical in providing an alternative for conventional bar reinforcement and steel fibres alike. Therefore, this section offers significant results that help to assess this topic. Fracture results showed that polyolefin fibres provide structural capacities to concrete and, hence, that their contributions can be considered to reduce other reinforcements with reduced costs and the already mentioned environmental and durability benefits. The tests carried out to design SFRC elements can also be used to characterise the fracture behaviour of PFRC. These are EN 14651 [28], RILEM TC 162-TDF [19], ASTM C 1609/C 1690M-07 [29] and RILEM TC 187 SOC [30].

Previous research [5, 8] has shown that PFRC provides fracture curves with three main turning points: the load at the limit of proportionality \( L_{LOP} \), the minimum post-cracking load \( L_{MIN} \) and the maximum post-cracking load-bearing capacity \( L_{REM} \). The conclusions provided by those studies showed that there was a slight influence of the polyolefin fibres on the values of \( L_{LOP} \). Nevertheless, the values of post-cracking loads were mostly governed by the presence of fibres. The residual load values can be transformed into strength values by expression (1), allowing their use in the structural design as recommended in the regulations [27, 31]. In the expression, \( f_{R,j} \) stands for strength, \( S \) for the span, \( b \) for the depth of the beam and \( h_{sp} \) for the ligament length.

\[
f_{R,j} = \frac{3 L_{R,j} S}{2 b h_{sp}}
\]  (1)
The definition of a structural fibre type slightly varies from the EN 14889-2 [32] and Model Code [31] or EHE-08 [27]. For SLS, the value of strength of the FRC at a crack opening of 0.5mm \( (f_{R1}) \) is used and for ultimate limit state (ULS) the value obtained at 2.5mm of crack opening \( (f_{R3}) \) is accepted. These strengths could be assumed as the contribution of the fibres in the structural design, if \( f_{R1} \) surpasses 40% of \( f_{LOP} \) and \( f_{R3} \) is higher than 20% of \( f_{LOP} \). These requirements in terms of a proportion of \( f_{LOP} \) contrast with the minimum fixed value of 1.5 MPa proposed by EN 14889-2 [32] for \( f_{R1} \).

Figure 4(a) shows the mean load-deflection curve of three specimens of each type of PFRC. Figure 4(b) shows the results of SFR-SCC with, respectively, 26 and 39 kg/m³ of steel-hooked fibres. For the initial crack openings, steel-hooked fibres lessen the drop of strength once the limit of proportionality is surpassed. It should be noted that the higher elasticity modulus and the mechanical anchorage of the hooks might improve the behaviour in the very first crack openings.

Table 3 shows both the average values of \( f_{R1} \) and \( f_{R3} \) and the strength at the crack opening 3.5mm \( (f_{R4}) \). This average value was obtained from two specimens of size 600x150x150 mm. In the table, the percentage with respect to \( f_{LOP} \) is also shown.

![Figure 4](image_url)

**Figure 4:** Fracture results using the same mix proportioning of SCC: (a) PFR-SCC; (b) SFR-SCC.

It should be noted that meeting the requirements in the values of \( f_{R1} \) in PFRC was not possible with dosages lower or equal than 6kg/m³. Such values were close to 1.5 MPa, although those percentages did not reach 40% of \( f_{LOP} \). However, the values in the table highlight that even the lowest dosage of polyolefin fibres exceeded the values required by the standards of exceeding 20% of \( f_{LOP} \) at \( f_{R3} \). Furthermore, it was found that in order to reach \( f_{R1} \) strengths higher than 40% of \( f_{LOP} \), 48mm-long fibres seemed to improve the values for the initial crack openings compared with the longer fibres. It was also noticeable that all the strengths \( f_{R4} \) were higher than \( f_{R3} \), showing that taking the value of \( f_{R3} \) does not reflect all the possible benefits of the fibres. As a final argument, the dosage of 10 kg/m³ fully complied with the standards and showed improvements when using SCC.
Table 3: Residual strengths obtained from flexural tensile strength tests [10, 33] following EN 14651 [28]

<table>
<thead>
<tr>
<th>Residual strength (MPa)</th>
<th>$f_{\text{LOP}}$</th>
<th>$f_{R1}$</th>
<th>% $f_{\text{LOP}}$</th>
<th>$f_{R2}$</th>
<th>% $f_{\text{LOP}}$</th>
<th>$f_{R4}$</th>
<th>% $f_{\text{LOP}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>VCC3</td>
<td>4.81</td>
<td>0.93</td>
<td>19%</td>
<td>0.96</td>
<td>20%</td>
<td>0.97</td>
<td>21%</td>
</tr>
<tr>
<td>SCC3</td>
<td>5.21</td>
<td>0.93</td>
<td>18%</td>
<td>1.15</td>
<td>22%</td>
<td>1.27</td>
<td>26%</td>
</tr>
<tr>
<td>VCC4.5</td>
<td>4.74</td>
<td>1.06</td>
<td>22%</td>
<td>1.40</td>
<td>29%</td>
<td>1.56</td>
<td>33%</td>
</tr>
<tr>
<td>SCC4.5</td>
<td>5.23</td>
<td>0.95</td>
<td>18%</td>
<td>1.25</td>
<td>24%</td>
<td>1.32</td>
<td>25%</td>
</tr>
<tr>
<td>VCC6</td>
<td>4.41</td>
<td>1.57</td>
<td>36%</td>
<td>2.38</td>
<td>54%</td>
<td>2.61</td>
<td>59%</td>
</tr>
<tr>
<td>SCC6</td>
<td>5.09</td>
<td>1.39</td>
<td>27%</td>
<td>2.03</td>
<td>40%</td>
<td>2.25</td>
<td>44%</td>
</tr>
<tr>
<td>SCC6-48*</td>
<td>4.57</td>
<td>1.50</td>
<td>33%</td>
<td>2.21</td>
<td>48%</td>
<td>2.24</td>
<td>49%</td>
</tr>
<tr>
<td>VCC10</td>
<td>4.21</td>
<td>1.98</td>
<td>47%</td>
<td>2.87</td>
<td>68%</td>
<td>3.05</td>
<td>72%</td>
</tr>
<tr>
<td>SCC10</td>
<td>5.22</td>
<td>2.41</td>
<td>46%</td>
<td>3.87</td>
<td>74%</td>
<td>4.16</td>
<td>80%</td>
</tr>
</tbody>
</table>

*PFRC using 48mm-long polyolefin fibres. The rest of the PFRC formulations contain 60mm-long polyolefin fibres

5 DESIGNING A COMBINATION OF STEEL FIBRES WITH POLYOLEFIN FIBRES

The performance previously presented suggested that a combination of fibres could take advantage of the best performance of each of the two types of fibres tested. This can be more easily understood by observing Figure 4 and comparing the shapes of the PFRC and SFRC. SFRC curves showed a scattered fall of strength due to the rectification of the hooks. PFRC revealed a remarkable performance of the polyolefin fibres in advanced deformations, with the figure highlighting the remarkable fall of strength of PFRC for small deformations. A combination of both materials and shapes could provide a stable post-cracking response and strength closer to the value in limit of proportionality. Therefore, the optimisation of the materials would reach to a more sustainable structural concrete.

As a starting point, the maximum volume fraction occupied by all fibres, providing structural capacities without a significant loss of workability was considered to be around 1%. Should such a volume fraction need to be divided by polyolefin and steel fibres, two of the previously studied [5] dosages were chosen: the lower dosage of SF (0.33%) and an intermediate of PF (PFR-SCC4.5 means 0.49% in volume). Thus, the hybrid mixture named as H1 was defined. The same type and number of specimens used for the previous types of concrete were produced in two batches. Table 4 shows the results of the fresh-state and mechanical properties tests.

Even with the total volume fraction used of 0.82%, the results showed that the mixtures kept the self-compacting properties. It is worth noting that the use of steel fibre affected to a similar extent the fresh-state results. The latter shows the good performance of the polyolefin fibres in this regard. The compressive strength and elasticity modulus were analogous to the previous results. Conversely, the indirect tensile strength tests showed that the combination of fibres improved the results.
Table 4: Fresh-state results and mechanical properties of the mixture H1

<table>
<thead>
<tr>
<th>Slump test</th>
<th>SCC</th>
<th>S26</th>
<th>P4.5</th>
<th>H1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{500}$ (s)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>4.0</td>
</tr>
<tr>
<td>$d_t$ (mm)</td>
<td>655</td>
<td>570</td>
<td>600</td>
<td>565</td>
</tr>
<tr>
<td>V-funnel</td>
<td>8</td>
<td>14</td>
<td>11</td>
<td>14</td>
</tr>
<tr>
<td>Elasticity Modulus (GPa)</td>
<td>35.8</td>
<td>33.7</td>
<td>31.2</td>
<td>33.0</td>
</tr>
<tr>
<td>$f_{ck}$, 28 days (MPa)</td>
<td>39</td>
<td>41.7</td>
<td>38.5</td>
<td>36.5</td>
</tr>
<tr>
<td>$f_{ct}$ indirect (MPa)</td>
<td>3.8</td>
<td>5.3</td>
<td>4.2</td>
<td>5.4</td>
</tr>
</tbody>
</table>

Concerning the fracture behaviour, the curves obtained can be seen in Figure 5. By analysing such curves, an improved analysis of the residual capacities of the hybrid mixture H1 can be performed. As expected, the hybrid mixture H1 obtained a higher residual load-bearing capacity than the mixtures with only one type of fibre. Beyond that, the algebraic sum of the post-cracking curves of the mono-fibre mixtures (SFR-SCC26 + PFR-SCC4.5) was computed in order to proceed to a detailed comparison. With such a procedure, synergies appeared in the curves when compared with the experimental curve of the combined use of the two types of fibres. Translating this to figures, Table 5 shows the residual strengths and the fracture energy of the mixtures. As can be observed, the residual strengths $f_{R1}$ and $f_{R3}$ were higher than 90% of $f_{LOP}$. That is to say, they remained as remarkable values with a significantly stable response. The analysis of fracture energy was performed at a deflection of 8 mm. Table 5 shows that H1 provided a 10% increase of fracture energy when compared with the algebraic sum of the fracture energies of SFR-SCC26 and PFR-SCC4.5.

Figure 5: Fracture results of H1 and the sum of the residual contributions of mono-fibre reinforced mixtures
Table 5: Residual strengths of H1 and fracture energy at 8 mm

<table>
<thead>
<tr>
<th>Mixture</th>
<th>$f_{LOP}$ (MPa)</th>
<th>$f_{R1}$ (MPa)</th>
<th>%</th>
<th>$f_{R3}$ (MPa)</th>
<th>%</th>
<th>$G_f$ (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC</td>
<td>5.03</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>130</td>
</tr>
<tr>
<td>SFR-SCC26</td>
<td>6.05</td>
<td>3.81</td>
<td>63%</td>
<td>3.45</td>
<td>57%</td>
<td>2621</td>
</tr>
<tr>
<td>PFR-SCC4.5</td>
<td>5.73</td>
<td>1.15</td>
<td>20%</td>
<td>1.66</td>
<td>29%</td>
<td>1846</td>
</tr>
<tr>
<td>H1</td>
<td>5.48</td>
<td>4.95</td>
<td>90%</td>
<td>5.36</td>
<td>98%</td>
<td>4931</td>
</tr>
</tbody>
</table>

The synergies obtained were also explained with a fracture surface analysis. The combination of the two types of fibres produced an increase in the orientation factor compared with the individual results of both types of fibres, as can be seen in Table 6. The description of the procedure to obtain $\theta$ and its definition can be studied in more detailed in references [6, 33, 34, 35].

Table 6: Orientation factor

<table>
<thead>
<tr>
<th>Mixture</th>
<th>$\theta$ (Orientation factor)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFR-SCC26</td>
<td>SF 0.68</td>
</tr>
<tr>
<td>PFR-SCC4.5</td>
<td>- 0.61</td>
</tr>
<tr>
<td>H1</td>
<td>0.75 0.63</td>
</tr>
</tbody>
</table>

6. SUMMARY OF CONCLUSIONS

This study has presented a complete and extensive analysis of the properties of polyolefin fibre reinforced concrete (PFRC), mainly focused on the residual strengths. The main purpose was to optimise a combination of polyolefin fibres with steel fibres that may fully exploit the advantages of each fibre material and shape. This improved material meets the requirements of the standards, reduces durability problems of the steel fibres and improves the utilisation of the materials.

The simultaneous use of short steel-hooked fibres and longer polyolefin ones is particularly well suited for structural applications where only a reduced amount of cracking is allowed. In addition, the hybrid-fibre-reinforced concrete enables a remarkable reduction of the use of steel. Hence, derived from the production methods, significant reductions of carbon emissions can be achieved. Additionally, the lower dosages in terms of the weight reduce the costs of transport and thus the carbon footprint of the life cycle of the material. Therefore, beyond other benefits such as safer handling or the ease of continuous productions, the reduction of the amount of steel required decreases the impact on the environment.

Regarding the use of only polyolefin fibres in concrete, the paper has shown that such fibres are especially suitable for use when self-compacting properties are required. Moreover,
these do not produce significant changes in the mechanical properties of the material. On the contrary, the presence of fibres improves the toughness and ductility of the PRFC and allows strains higher than those conventionally considered in civil engineering structures. As an example of these virtues, PFRC was found to be well suited for its use in construction, especially in applications such as pavements, light precast elements and tunnel linings and, among others, water pipelines. On this ending note, the study provides the structural designer with values of the residual strength of PFRC.

Acknowledgements

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THE DAMAGE OF CALCIUM SULFOALUMINATE CEMENT PASTE PARTIALLY IMMERSED IN Na₂CO₃ SOLUTION

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Key words: chemical sulfate attack, physical sulfate attack, concrete, calcium sulfoaluminate cement

Abstract: In the presented paper, the tests were designed to offer indirect evidences for the physical sulfate attack on ordinary concrete. The calcium sulfoaluminate (CSA) cement paste was partially exposed to 10% Na₂CO₃ solution under condition of 20°C and RH 60%. The microanalysis results showed that Na₂CO₃ crystallization did not occur in the CSA cement paste and the Na₂SO₄ crystallization, the product of chemical reaction between CSA cement paste and Na₂CO₃, caused the cracks formation at the edge of specimens. The Na₂CO₃ crystallization occurred after the chemical reactions disappeared. As comparison, it can be confirmed that the physical sulfate attack or sulfate crystallization also cannot occur in the ordinary concrete due to the chemical reactions between Portland cement paste and sulfate.

1 INTRODUCTION

Sulfate attack on concrete is considered as a typical chemical corrosion of cement, causing the decomposition of cement hydration products. However, when the concrete element is partially exposed to sulfate environment, the similar salt weathering of porous materials occurs that the upper part of concrete in contact with air is severely damaged but with the sound part of concrete surrounded by sulfate ions. People generally indicates that the concrete is damaged by physical sulfate attack or sulfate salt crystallization distress causing the structures failure not the decomposition of hydration products. Therefore, there is viewpoint that the physical sulfate attack cannot be regarded as sulfate attack on concrete.

However, according to author’s previous researches [1], it seems that the main reason causing concrete damage is not the physical sulfate attack but still the chemical sulfate attack. In the paper [2], author indicated that the sulfate salt crystallization cannot occur if there is chemical reaction between sulfate and hydration products of cement. When Portland cement paste was partially exposed to 10% Na₂SO₄ solution under the condition of 20°C and RH 60%, the specimens were damaged after 5months’ exposure. The micro-analysis results showed that the ettringite and gypsum were identified as the evidences of chemical sulfate attack [3-4]. However, the similar test
results showed that the CSA cement paste specimens were quickly damaged just after 7 days exposure. Na$_2$SO$_4$ could be clearly identified as the evidence of sulfate salt crystallization [5]. The reason for the quick failure is that the hydration products of CSA cement are stable in the Na$_2$SO$_4$ solution.

The hydration products of CSA cement contain rich ettringite, C-S-H and Al gel. The effect of carbonation on the stability of CSA cement paste is a problem, and ettringite decomposes to gypsum, calcium carbonate, and alumina gel after carbonation, as described in the following equation [6-7]:

$$3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O} + 3\text{CO}_2 = 3\text{CaCO}_3 + 3(\text{CaSO}_4 \cdot 2\text{H}_2\text{O}) + \text{Al}_2\text{O}_3 \cdot x\text{H}_2\text{O} + (26-x)\text{H}_2\text{O}$$

(1)

If CSA cement paste is immersed in Na$_2$CO$_3$ solution, the chemical reaction cannot be avoided. However, a test can be designed as following: the CSA cement paste specimens are divided into two parts, (1) one part will be immersed in the 10% Na$_2$CO$_3$ solution at 40°C until the finish of chemical reaction between CSA cement paste and Na$_2$CO$_3$. Then, the paste specimen will be partially immersed in 10% Na$_2$CO$_3$ solution again under the exposure condition of 20 °C and 60% RH. (2) as reference, the other uncontaminated CSA paste specimens will be also partially exposed to 10% Na$_2$CO$_3$ solution under the same exposure condition. The comparison between the above two concrete performances can offer more evidences to disclose the so-called physical sulfate attack on concrete from a different perspective.

2. EXPERIMENTS

2.1 Raw materials

Low alkalinity sulfoaluminate cement, complying with JC/T 659—2003 was used in the experimental program. The chemical composition of cement is given in Table 1.

| Table 1: Chemical composition of low alkalinity sulfoaluminate cement (% by mass) |
|---------------------------------|-------|-------|-------|-------|-------|-------|-------|
| CaO    | SiO$_2$ | Fe$_2$O$_3$ | MgO  | Al$_2$O$_3$ | SO$_3$ | TiO$_2$ |
| Cement | 41.51   | 6.30   | 4.26  | 0.80   | 28.07 | 13.26 | 2.20 |

2.2. Specimens preparation, curing and testing

Plain cement paste with a W/C ratio of 0.45 was mixed. The fresh paste were cast in a mould with size of 10*40*150 mm covered with a plastic sheet in a moist room for 24 h. The specimens were demoulded and stored in water at 20 ± 1 °C. At the age of 7 days, all specimens were dried in a climate box at 20 ± 3 °C and 60 ± 5% RH for 7 days. After drying, the CSA cement paste specimens are divided into two parts, (1) three specimens were immersed in the 10% Na$_2$CO$_3$ solution at 40°C until the finish of chemical reaction between CSA cement paste and Na$_2$CO$_3$. Then, the specimens were partially immersed in 10% Na$_2$CO$_3$ solution again under the exposure condition of 20 °C and 60% RH, (2) the other three uncontaminated CSA paste specimens were partially exposed to 10% Na$_2$CO$_3$ solution under the same exposure condition.
2.3. Visual observation, XRD, IC

The damaged specimens were visually observed and analyzed by XRD analysis. The samples were vacuum dried with silica gel at 20 °C. The specimens did not touch solution and water any more after removal from the solution. The SO$_4^{2-}$ concentration was measured by IC (Ion Chromatography) The substance on the surface of the specimens was cleaned by a soft brush or a thin blade.

3. RESULTS AND DISCUSSION

3.1 CSA cement paste exposed to 10% Na$_2$CO$_3$ solution

After 3 months exposure to 10% Na$_2$CO$_3$ solution, the CSA cement paste specimen observation is shown in Figure 1. It can be found that the evaporation zone of specimen was covered with white crystals. Several cracks occurred at the edge of specimen, and small pieces of paste can be easily taken off from the specimen. After the paste parts in the solution and evaporation zone were broken off respectively, two different zones in the section were observed the outer zone with light color and the inner zone with dark color. The products of small pieces of paste and white crystals were identified by XRD analysis (shown in Figure 2). The products of outer zone (shown in Figure 3) and inner zone (shown in Figure 4) were also identified by XRD analysis.

Figure 1: Visual observation of CSA paste specimen partially immersed in Na$_2$CO$_3$ for 3 months
It is very interesting to find that the white crystals were Na$_2$SO$_4$·10H$_2$O although the CSA cement paste was partially immersed in Na$_2$CO$_3$ solution. CaCO$_3$ was the main product in the small paste pieces.
Figures 3 and 4 show the XRD patterns of products in the outer zones of paste in the solution and evaporation zone. Obviously, the products in the two zones were the same, containing rich CaCO$_3$ and Al(OH)$_3$.

Based on the results shown in Figures 2, 3 and 4, it can be conformed that (1) Na$_2$CO$_3$ crystallization did not appear in the evaporation zone paste, (2) in the evaporation zone the same chemical reaction between Na$_2$CO$_3$ solution and hydration products of CSA cement paste occurred as in the solution as following:

$$3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O} + 6\text{Na}^+ + 3\text{CO}_3^{2-} = 3\text{CaCO}_3 + 3\text{Na}_2\text{SO}_4 + \text{Al}_2\text{O}_3 \cdot x\text{H}_2\text{O} + (32-x)\text{H}_2\text{O}$$

(2) CaCO$_3$ is more indissolvable than CaSO$_4$, therefore CaCO$_3$ was the final generation in the solution compared to the carbonation reaction shown in equation 1. the concentration of SO$_4^{2-}$ reached 51.67g/l (about 7.6% Na$_2$SO$_4$) in the solution after 3 months exposure according to the IC analysis result. (4) the crystallization distress of the Na$_2$SO$_4$ caused the cracks shown in Figure 1. (5) the peaks of Al(OH)$_3$ in Figure 4 are higher than them in Figure 3. The reason is that the Na$_2$CO$_3$ concentration of pore solution in the evaporation zone was higher than 10% due to wick action [8].

The above results support that Na$_2$CO$_3$ crystallization cannot occur in the porous CSA cement paste due to the chemical reactions between it and the hydration products. Another experiment was designed that the CSA cement paste was fully immersed in the Na$_2$CO$_3$ solution until the chemical reaction finish. Then, the paste was partially immersed in Na$_2$CO$_3$ solution again to check if Na$_2$CO$_3$ crystallization can occur in the paste.

3.2 CSA cement paste after chemical reaction exposed to 10% Na$_2$CO$_3$ solution
According to the observation in Figure 1, the light color part means the chemical reaction occurrence in the paste. After the color of paste section totally became light, the specimens were partially immersed in 10% Na₂CO₃ solution again. It is interesting to found that the specimen was damaged just after 10 days exposure shown in Figure 5. The successive paste layers were formed at the top of specimen. This appearance is a typical crystallization distress on porous materials. The XRD analysis on damaged paste clearly proved that the Na₂CO₃ crystallization distress was the mechanism for the formation of successive paste layers.

![Visual observation of reacted CSA paste specimen partially immersed in Na₂CO₃ for 10 days](Figure 5)

**Figure 5**: Visual observation of reacted CSA paste specimen partially immersed in Na₂CO₃ for 10 days

![XRD pattern of paste layers](XRD_pattern)

**Figure 6** XRD pattern of paste layers

Combined with the above two tests’ results, the viewpoint, that the salt crystallization distress cannot occur in the cement paste if there is chemical reaction between paste and salt, can be further confirmed. Therefore, when the Portland cement concrete element is partially buried in the sulfate environment, the mechanism for the failure of concrete in the evaporation zone cannot be simply attributed to
sulfate crystallization distress due to the chemical reaction between sulfate and hydration products.

5 CONCLUSIONS

The conclusions can be deduced as follows:

(1) \( \text{Na}_2\text{CO}_3 \) crystallization cannot occur in the CSA cement paste partially immersed to \( \text{Na}_2\text{CO}_3 \) solution.

(2) The paste damage is caused by \( \text{Na}_2\text{SO}_4 \) crystallization that is one of the chemical reaction products between CSA cement paste and \( \text{Na}_2\text{CO}_3 \).

(3) \( \text{Na}_2\text{CO}_3 \) crystallization can occur in the paste only when the chemical reactions disappear.

(4) Compared to ordinary concrete partially buried in sulfate environment, the failure of concrete in the evaporation zone cannot attributed to the physical sulfate attack or sulfate crystallization distress. It should be still the chemical sulfate attack.

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THE PARADOX OF HIGH PERFORMANCE CONCRETE USED FOR REDUCING ENVIRONMENTAL IMPACT AND SUSTAINABILITY INCREASE

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Key words: sustainability, high performance concrete.

Abstract. The concern about the depletion of natural resources and environmental pollution is increasing in the construction sector. Paradoxically, the society and the main control organizations of sustainability indexes in construction have given little importance to the concrete, which is the industrial product most widely used on the planet, to improve the quality of life of society. Despite the concrete structures are rarely considered in established sustainability programs in construction, several studies have been developed in order to quantify and minimize their environmental impacts, which are related to the structural design and the physical and mechanical properties of the concrete. In this context, this article points out some concepts of environmental management and proposes, with significant examples, the observation of new and interesting sustainability criteria for the use of structural concrete, demonstrating the employment advantage of high strength concrete.

1 INTRODUCTION

Current models of social, economic and industrial development, based on the fast and growing consumption of natural resources, when mismanaged, could result in degradation and environmental pollution. According to Mehta & Monteiro [1], decisions aiming results exclusively in the short term and simplistic goals are contributing to further aggravate the global situation.

On the other hand, in recent decades, it was noticed a growing awareness of society with regard to the limitation of natural resources and the need to adopt practices with less environmental impact and the search for a development model that is more sustainable.

Sustainable development must act in three dimensions [2], as illustrated in Fig. 1:

1. environmental: in order to find a balance between protecting the physical environment and its resources and use these resources so that the planet to continue providing an acceptable quality of life for human beings;
2. economic: requiring the development of an economic system that facilitates access to resources and opportunities, promoting prosperity within what is
environmentally possible and without violating basic human rights;
3. social: seeking the development of fair societies that allow the human development and ensure opportunities for personal improvement and acceptable quality of life.

![Sustainability dimensions](translated from ELKINGTON [3]).

In the construction industry, a sector that, according to Valdés [4], employs 7% of the world population, uses 2/5 of all energy produced in the world and consumes 50% of the natural resources of the earth's crust, the concrete is the material that occupies the most prominent position, being the manufactured product most used by society, with current global consumption estimated at 19 billion tons per year [1]. The authors emphasize that, with the exception of water, there is no other material consumed in such quantity per capita.

Therefore, the study of the concrete in the context of sustainable construction becomes every day more crucial. This article retrieves the concepts of environmental management and proposes the observation of new sustainability criteria in the design and construction of reinforced concrete structures, employing, paradoxically, high strength concrete with high cement consumption.

2 ENVIRONMENTAL MANAGEMENT

When the chase for economic development started to seriously jeopardize ecosystems and human health through pollution and natural resources depletion, the issue of sustainability has surfaced. Since then, several conferences focusing on this topic took place in order to draw prescriptive standards up, starting in 1992 with the RIO-92 Conference.

In response to this global demand for a more reliable, aware and fair environmental management, the Technical Committee 207 for ISO (TC 207), in 1994, developed the ISO 14000 series of standards, which proposed the concept of Life Cycle Assessment - LCA. This concept involves analyzing and determining the environmental impacts of products or
services at all stages of their life cycle: acquisition of raw materials, production, use and after-use treatment, recycling, until final disposal of this product or waste resulting from service.

Through the Life Cycle Assessment it is also possible to produce Environmental Product Declaration (Environmental Product Declaration - EPD), considered one of the best and most complete references of sustainability today.

The EN 15804:2012 establishes three types of EPD, described below, based on the life cycle stages covered in the study of each product or service (Fig. 2):

1. **cradle to gate**: mandatory, only comprehends the production stage (supply of materials, transportation, manufacturing and associated processes);
2. **cradle to gate with options**: optional, comprehends the stage of production and other additional stages, chosen by the product or service provider;
3. **cradle to grave**: optional, involves production processes, installation, use, maintenance, repair or replacement, demolition, treatment for reuse, reconstruction, recycling and final disposal, considered the most correct option but more laborious analysis.

![Figure 2: Stages of the life cycle of a product or service (adapted from EN 15804:2012 [5]).](image)
Yet, according to EN 15804:2012 [5], environmental impact indicators, quantified in different categories at each stage of EPDs involve the analysis of the following listed parameters:

a. global warming potential, in kg of CO₂ equivalent;
b. the stratospheric ozone layer depletion potential, in kg of equivalent CFC 11;
c. ground and water acidification potential, in kg of equivalent SO₂;
d. eutrophication potential, in kg of (PO₄)₃⁻ equivalent;
e. tropospheric ozone formation potential, in kg of equivalent ethylene;
f. abiotic resources (elements) depletion potential, in kg of equivalent Sb;
g. abiotic resources (fossil fuel) depletion potential, in MJ.

In addition to these indicators of environmental impact, the same standard also sets parameters describing the use of natural resources (renewable or not), energy and water.

The EPDs are valid for 5 years and after this period, must be reviewed and verified. It is not necessary to recalculate them if the underlying information does not present substantial changes. If either environmental impact indicators suffer change of at least 10% (for more or less), the EPD should be updated.

Also in the context of sustainable buildings, various international organizations, aiming to encourage this construction segment, created certifications to enhance and guide the transformation of conventional projects in environmentally friendly projects. Among these certifications, can be highlighted the LEED (Leadership in Energy and Environmental Design), Casa Azul Seal (Brazilian certification system developed by Caixa Econômica Federal), HQE (Haute Qualité Environnementale), BREEAM (Building Research Establishment Environmental Assessment Method) and DGNB (German Sustainable Building Council).

However, the evaluation criteria of the certifications, mistakenly, do not contemplate the use of high-strength concrete in the positive composition of the scores.

Although it seems a paradox, throughout this article, the many advantages involved in the adoption of this type of concrete will be presented, aiming to support the proposal for its inclusion in certification systems.

3 SUSTAINABLE CONCRETE?

As already discussed, concrete is the building material widely used worldwide, mainly due to its resistance, flexibility, durability, easy implementation and low cost. However, as with any other product to be used in construction, production of concrete and its components (especially cement) requires energy, consumes water and results in CO₂ generation [6].

According to Isaia and Gastaldini [7], cement production consumes 5.5GJ energy and releases around of 1ton of CO₂ per ton of clinker. Levy [8] points out that cement production is responsible for about 6% to 7% of total CO₂ emissions in the world. Brazil, with annual production of 40 million tons of cement and about 16 million tons of clinker, contributes with about 1% to 2% of total carbon emissions in Brazil.
The environmental impact of the concrete, however, is not caused only by the cement. For the production of concrete, materials and non-renewable natural resources such as sand and gravel are also used, bordering the amount of consumption of 12 billion tons annually. Considering the impact of exploration, processing and transport of this raw material, it is observed that all specific concrete manufacturing process seems to adversely affect the environment [9], although its essential for the improvement of human life quality through building houses, bridges, roads, viaducts, harbors, water and wastewater treatment plants, schools, hospitals, etc.

The concrete industry also uses large amounts of potable water, about 1 trillion liters each year, just as concrete water content, to which are added large portions of concrete mixers and equipment wash water and curing water of the concrete [10].

Thus, in order to reduce the consumption of potable water, natural resources and energy, reducing environmental impact, there is a need to consider the service life, durability and strength of concrete structures, taking into account the long-term (more than 50 years) of constructions in concrete.

However, according to EN 15804:2012 [5], only the cradle to gate life-cycle assessment is currently recommended and even required in certain circumstances. Therefore, the concreting service companies, for example, are concentrating on accomplishing it, discouraging, mistakenly, the use of high-strength concrete. Corresponding to this mistaken view, the environmental impacts exposed in Tab. 1 refers only to the production of ready-mix concrete.

Table 1: Environmental impacts for 1m³ of ready-mixed concrete produced by concreting services company Allied Concrete [11].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Impact / m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (MPa)</td>
<td>20  25  30  35  40  45  50</td>
</tr>
<tr>
<td>Global warming potential (kg CO₂ eq)</td>
<td>333  366  395  445  513  539  609</td>
</tr>
<tr>
<td>Stratospheric ozone layer depletion potential (kg CFC 11 eq 10⁻⁵)</td>
<td>1,10  1,19  1,26  1,39  1,57  1,56  1,83</td>
</tr>
<tr>
<td>Ground and water acidification potential (kg SO₂ eq)</td>
<td>1,42  1,56  1,68  1,89  2,18  2,29  2,60</td>
</tr>
<tr>
<td>Eutrophication potential (kg (PO₄)³⁻ eq)</td>
<td>0,339  0,372  0,402  0,451  0,519  0,548  0,620</td>
</tr>
<tr>
<td>Tropospheric ozone formation potential (kg ethylene eq)</td>
<td>0,068  8  0,075  6  0,081  8  0,092  0,106  0,112  0,127</td>
</tr>
<tr>
<td>Potential depletion of non-fossil resources (kg Sb eq·10⁻⁴)</td>
<td>1,10  1,18  1,25  1,41  1,44  1,49  1,83</td>
</tr>
<tr>
<td>Potential depletion of fossil resources (MJ)</td>
<td>3080  3390  3650  4090  4700  4900  5570</td>
</tr>
</tbody>
</table>

However, this incomplete and misplaced analysis does not reflected the overall environmental impacts of a concrete structure. Contrary to this view, Levy [8] explains that
the high performance concrete (HPC) have a more compact pore structure and, in its formulation, superplasticizers additives and mineral additions that react with the free lime and improve its strength. Thus, in these HPCs, the water/cement ratio is lower than in a conventional concrete, which increases their mechanical properties and durability.

The superior mechanical behavior of HPCs also allows significant reductions of structural sections, generating large cement, steel, water and aggregates economy, as well as possible economic gain by increase in useful areas for use, rental and parking on enterprises.

An important example of the application of this concept in Brazil is the colorful HPC with $f_{ck} 80\text{MPa}$, used in *e-Tower* building in São Paulo, which enabled a significant reduction in the area occupied by columns in parking areas [12], as shown in Fig. 3.

![Figure 3: Project design of the e-Tower: existing columns on the original design with $40\text{MPa } f_{ck}$ (90cm x 100cm) and modified columns with $80\text{MPa } f_{ck}$ (60cm x 70cm)](image)

This modification (reduction in the size of the columns) made the project compatible with the architectural requirements and made it possible to meet the criteria of a sustainable structure. Under the service life and sustainability point of view, one of the main deleterious mechanism of a concrete structure is the corrosion of steel, very unlikely phenomenon in structures built with high strength concrete.

All carbon steel is eternally protected by a high alkaline environment with a pH greater than 11.5. This fact is legitimately observed in the case of chloride free Portland cement concrete structures, since the hydration products of the curing reaction between the anhydrous grains of cement and water release large quantities of Ca(OH)$_2$, NaOH and KOH, which are strong bases [13], able to chemical and effectively protect steel from deleterious corrosion.

This protection capability by passivation can be lost over time due to various actions of which the most important are the chloride penetration and the reaction of carbon dioxide $\text{CO}_2$
with the hydration products of these alkalis, resulting in low alkaline salts, a phenomenon known by carbonation of concrete.

With the increase in concrete strength, there is an important reduction of the risk associated with the reinforcement corrosion, given the high difficulty of aggressive agents penetration. According to Levy [8], with smaller and not connected pores, the high strength concrete is less subject to the action of aggressive agents present in the atmosphere and water, which increases its durability and hence service life of the structure.

From the point of view of sustainable construction, some important parameters were achieved with this design change: increase of the service life, reduction of natural resources use, environmental impacts, energy and the total volume of the work concrete (even with a cement consumption per cubic meter of concrete top to the original design of concrete - with $f_{ck}$ 40MPa).

Specifically about the elevation of service life, some standardized values from enshrined bibliographies\textsuperscript{1} were adopted to illustrate the magnitude of the growth, as shown in Table 2, where it can be seen an increase of ten times on the service life of project.

Table 2: Data collected in the case study on the concrete pillars of the building and high strength-Tower, for the growth of life.

<table>
<thead>
<tr>
<th>Element</th>
<th>Design characteristic cover ((1)) (cm)</th>
<th>Carbonation constant adopted((2)): (k_{CO2}) (\text{mm/ year}^{1/2})</th>
<th>Estimated design service life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural column (90cm x 100cm) with $f_{ck} = 40$MPa</td>
<td>3,0</td>
<td>2,45</td>
<td>150</td>
</tr>
<tr>
<td>Structural column (60cm x 70cm) with $f_{ck} = 80$MPa</td>
<td>3,0</td>
<td>0,77</td>
<td>1500</td>
</tr>
</tbody>
</table>

\(1\) It was considered as the design characteristic cover within the tolerance of the NBR 6118:2014, i.e. the minimum admitted covering.

\(2\) This value was adopted depending on the Prática Recomendada do IBRACON only for the purpose of demonstrating that the structure service life increases tenfold when the concrete strength changes. It is noteworthy, however, that these coefficients were estimated.

As for saving natural resources, it was found that there was a considerable reduction of all materials used in the concrete composition and design of columns with $f_{ck} = 80$MPa, compared to the $f_{ck} = 40$MPa. The volume of aggregates was reduced by 70%, while the cement in 20%, according to Table 3.

\textsuperscript{1} The value of the carbonation coefficient was estimated based on literature: Prática recomendada IBRACON – Comentários Técnicos NB-1, produced in 2003. The adopted model has been simplified based on the structure deterioration mechanism of carbonation, through the formula: \(e = k_{CO2} \sqrt{t}\), where \(e\) is the concrete cover in cm, \(k_{CO2}\) is the carbonation constant in cm/year\(^{1/2}\) and \(t\) is the time, in years.
Table 3: Data collected in a study about e-Tower high-strength concrete columns, referring to the reductions of materials and concrete.

<table>
<thead>
<tr>
<th>Material</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>70%</td>
</tr>
<tr>
<td>Gravel</td>
<td>70%</td>
</tr>
<tr>
<td>Cement</td>
<td>20%</td>
</tr>
<tr>
<td>Water</td>
<td>53%</td>
</tr>
<tr>
<td>Steel</td>
<td>43%</td>
</tr>
<tr>
<td>Formwork</td>
<td>31%</td>
</tr>
<tr>
<td>Concrete</td>
<td>53%</td>
</tr>
</tbody>
</table>

Confirming this view, Couto [14] studies showed that the design of buildings of 41 floors with high strength concrete is advantageous, both from an economic and sustainable point of view, compared with those designed with conventional concrete. It was observed that it is possible to obtain a reduction of approximately 11% in the overall cost of the structure, changing the strength from a $f_{ck}$ 25MPa to a $f_{ck}$ 50MPa.

So, with holistic and long-term vision, it is necessary to perform the concrete life cycle analysis taking into account the whole life cycle of this material and not only from the "cradle to gate".

Bento e Rossignolo [15] shows in their doctoral research the importance of a holistic view, result of a cradle to grave analysis of a residential building hypothetically designed, consisting of eight floors and a ground floor type. Were analyzed in this study the strength classes with $f_{ck}$ of 25MPa, 30MPa, 35MPa, 40MPa, 45MPa and 50MPa, as explained in Table 4.

Table 4: Overall balance of impact categories with results obtained for each structure strength [15].

<table>
<thead>
<tr>
<th>Impact category</th>
<th>C25</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eutrophication (g NO$_3^-$)</td>
<td>(2,33 · 10$^3$)</td>
<td>Larger (2,43 · 10$^3$)</td>
<td>(2,15 · 10$^3$)</td>
<td>Minor (2,09 · 10$^3$)</td>
<td>(2,10 · 10$^3$)</td>
<td>(2,13 · 10$^3$)</td>
</tr>
<tr>
<td>Photochemical ozone formation (g C$_2$H$_4$ eq)</td>
<td>(4,03 · 10$^4$)</td>
<td>Larger (4,10 · 10$^4$)</td>
<td>(3,83 · 10$^4$)</td>
<td>Minor (3,55 · 10$^4$)</td>
<td>(3,61 · 10$^4$)</td>
<td>(3,72 · 10$^4$)</td>
</tr>
<tr>
<td>Consumption of material resources (kg)</td>
<td>Larger (6,93 · 10$^3$)</td>
<td>(6,56 · 10$^2$)</td>
<td>(5,43 · 10$^2$)</td>
<td>(5,00 · 10$^2$)</td>
<td>Minor (4,95 · 10$^2$)</td>
<td>(4,90 · 10$^2$)</td>
</tr>
<tr>
<td>Consumption of energy resources (kWh)</td>
<td>(4,20 · 10$^1$)</td>
<td>Larger (4,30 · 10$^1$)</td>
<td>(4,01 · 10$^1$)</td>
<td>Minor (2,17 · 10$^1$)</td>
<td>(2,20 · 10$^1$)</td>
<td>(2,26 · 10$^1$)</td>
</tr>
</tbody>
</table>
Table 4: Overall balance of impact categories with results obtained for each structure strength [15]. [conclusion]

<table>
<thead>
<tr>
<th>Impact category</th>
<th>C25</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ecotoxicity (m³ compartment)</td>
<td>(2.54 · 10²)</td>
<td>Larger (2.64 · 10²)</td>
<td>(2.20 · 10²)</td>
<td>Minor (2.10 · 10²)</td>
<td>Minor (2.10 · 10²)</td>
<td>Minor (2.10 · 10²)</td>
</tr>
<tr>
<td>Global warming (g CO₂ eq)</td>
<td>Larger (6.75 · 10³)</td>
<td>(6.67 · 10³)</td>
<td>(6.12 · 10³)</td>
<td>(5.74 · 10³)</td>
<td>Minor (5.71 · 10³)</td>
<td>Medium (5.67 · 10³)</td>
</tr>
<tr>
<td>Human toxicity (m³ compartment)</td>
<td>Larger (1.80 · 10⁸)</td>
<td>(1.78 · 10⁸)</td>
<td>Minor (1.68 · 10⁸)</td>
<td>(1.76 · 10⁸)</td>
<td>(1.76 · 10⁸)</td>
<td>(1.76 · 10⁸)</td>
</tr>
<tr>
<td>Acidification (g SO₂•eq)</td>
<td>(9.52 · 10¹)</td>
<td>Larger (1.00 · 10²)</td>
<td>(9.19 · 10¹)</td>
<td>Minor (8.90 · 10¹)</td>
<td>(9.03 · 10¹)</td>
<td>(9.23 · 10¹)</td>
</tr>
<tr>
<td>Waste (kg)</td>
<td>Larger (2.30 · 10⁻¹)</td>
<td>Larger (2.30 · 10⁻¹)</td>
<td>(1.90 · 10⁻¹)</td>
<td>(1.70 · 10⁻¹)</td>
<td>Minor (1.68 · 10⁻¹)</td>
<td>Minor (1.68 · 10⁻¹)</td>
</tr>
</tbody>
</table>

This study considered the section reduction in two stages, in C35 and C45 classes. It was concluded that, in this case, the strength class with less environmental impact would be the C40. Thereafter, increasing the resistance would not result in a significant decrease of resistant section of flexed elements and it would not be advantageous for the loading determined initially.

The study of Schmidt and Teichmann [16], aiming the construction of a bridge with lattice structure in prestressed concrete (work of art), also concluded that the ultra high performance concrete (UHPC), with \( f_{ck} = 200\text{MPa} \), proves to be much more sustainable than the conventional concrete, resulting in a lower consumption of raw materials and energy, as shown in Tab. 5.

Table 5: Demand for materials (in tons) and energy (in MJ) for the construction of a bridge with lattice structure with conventional concrete and high performance concrete [16].

<table>
<thead>
<tr>
<th>Material</th>
<th>C25/C30</th>
<th>High performance ( f_{ck} ) 200MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>120</td>
<td>98</td>
</tr>
<tr>
<td>Aggregates</td>
<td>620</td>
<td>170</td>
</tr>
<tr>
<td>Water</td>
<td>60</td>
<td>21</td>
</tr>
<tr>
<td>Silica fume</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td>Steel (passive reinforcement)</td>
<td>70</td>
<td>22</td>
</tr>
<tr>
<td>Steel Fibers</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>Steel (active reinforcement)</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>Total energy (MJ)</td>
<td>2.050.256</td>
<td>1.148.517</td>
</tr>
</tbody>
</table>

Indeed, it is clear that the use of high performance concrete (paradoxically, with higher cement consumption per cubic meter) brings significant advantages, not only with respect to
mechanical properties, but also in environmental and sustainable aspect. However, as already explained, these benefits appear only in the long term and over a review of the service life of the material, when a full analysis (cradle to grave type) is performed.

Besides the importance of high resistance in reducing total input demand, it is also important to investigate the specific emissions by concreting service companies in m$^3$ of the product supplied. The concrete generally have CO$_2$ at around 7% to 15% of the produced concrete mass [6], depending on the designed proportions.

These values are closely related to the clinkering of used cement, the optimization of the production process of the plants and also the quantity of cement per m$^3$ calculated for the concrete mixes. Along with analysis of the total energy used for the entire structure, a proper study and the optimization of the inputs of the m$^3$ of concrete can also help to mitigate their environmental impact.

In the case of simplistic analysis of 1m$^3$ concrete alone, the environmental impact can be evaluated through the concept of yield, expressed by the ratio of compressive strength (MPa) / cement consumption (C$_{cim}$). The fact that the efficiency of a concrete is closely linked to the amount of cement required to achieve the desired strengths is evident. For Helene and Tutikian [17], the yield has a great peak for each concrete mix and must be studied through the dosage diagram, in order to obtain the most sustainable concrete, which must also be an economically viable solution.

![Dosage diagram of Portland cement concrete.](image)

Figure 4: Dosage diagram of Portland cement concrete.

That said, studies of Boggio [18] bring an assessment of the efficiency factor or concrete yield calculated for concrete with strengths between 20MPa and 40MPa, dosed according to IBRACON method (Table 6).
Table 6: Efficiency factor or concrete yield evaluation for different strengths (adapted from Boggio [18]).

<table>
<thead>
<tr>
<th>Strengths $f_{c28}$ (MPa)</th>
<th>Efficiency factor $f_{c28}/C_{cim}$ (MPa) ( \frac{kg/m^3}{kg/m^3} )</th>
<th>$C_{cim}/f_{c28}$ (MPa) ( \frac{kg/m^3}{kg/m^3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0,082</td>
<td>12,20</td>
</tr>
<tr>
<td>25</td>
<td>0,091</td>
<td>10,93</td>
</tr>
<tr>
<td>30</td>
<td>0,099</td>
<td>10,10</td>
</tr>
<tr>
<td>35</td>
<td>0,109</td>
<td>9,18</td>
</tr>
<tr>
<td>40</td>
<td>0,109</td>
<td>9,14</td>
</tr>
<tr>
<td>80*</td>
<td>0,174</td>
<td>5,75</td>
</tr>
</tbody>
</table>

* Data collected in the study of high strength concrete columns of e-Tower building.

Given the importance of concrete in constructive chain and at the development of a nation and given the benefits of high strength concrete employment, it is paradoxical to use consumption of cement as environmental degradation index, because the correct would be thinking about the construction life cycle with a global and holistic view, not only about cement consumption of concrete.

4 FINAL CONSIDERATIONS

As shown, even having a higher consumption of cement per m³, and hence, a larger amount of CO₂ emission per m³, the reduction in the volume of concrete and a considerably increased service life justify the use of high strength concrete from the point of view of sustainability. As pointed out by Hajek, Fiala and Novotna [19], using materials with better physical and mechanical characteristics is a realistic mean of achieving substantial advantages from the perspective of materials and energy savings, allowing, in the case of concrete, the preparation of designs with optimized sections, increased durability and strengths and, ultimately, the generation of less environmental impact. In this context, it is suggested that the use of high-strength concrete also compose the score of existing seals for sustainable construction, such as LEED, Casa Azul Seal, HQE, BREEAM and DGNB, considering that its employment offers high performance in environmental quality, productivity, global economy of materials and resources.

REFERENCES


A STUDY ON THE CRACK DISTRIBUTION AND
CHARACTERISTICS OF A CONTINUOUSLY REINFORCED
CONCRETE PAVEMENT

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Key words: CRCP, Concrete, Pavements, Environment, Shrinkage, Cracks

Abstract: Rigid pavements are used throughout the continuously expanding infrastructure in South Africa owing to their durability and their minimal maintenance requirements. As budgets for pavement construction decrease, with clients still wanting the same quality product, the need has arisen to develop more accurate models to better predict pavement performance during its lifetime.

Crack distributions in rigid pavements significantly affect the durability of the pavement, but little is known about the exact effect that material properties, loads and environmental factors play in the spacing of cracks. The design that is applied considers only single mix proportions and environmental conditions to deliver a final design life. But, mix proportions and environmental conditions do however vary during construction and during the pavement’s life.

Climatic conditions and the material properties of concrete are the two most important factors affecting the performance of continuously reinforced concrete pavement (CRCP), with each site having its own micro environment (that changes hourly), and considerable variance in the material used in each concrete batch. It remains the case that even if a design is executed with averages in mind, temperature, wind, humidity and other governing factors change hourly and are different for each month of the year.

This paper focusses on the collection of actual material, climatic and crack spacing data throughout construction as well as a year after, and reports on actual observations. All of the collected data was then analysed by using a model setup by Associate Professor L.J.M. Houben of TU Delft, which calculates the induced and accrued stress at short time intervals throughout the life. The final output of this model is the predicted crack spacing for a given environmental and material set.

The most significant contribution of this paper would be to compare results from this mechanical model to that of standard regression models as well as illuminating shortfalls of both.
1 INTRODUCTION

To date, a variety of pavement types have been developed to suit particular needs such as higher traffic volumes, speed of construction, heavy vehicle loads and maintenance, to name just a few. One such innovation is CRCP, which allows minimal to no maintenance, has a long design life and can carry large volumes of traffic and heavy loads. The fact that this pavement is designed for minimal maintenance makes it a very attractive option, even considering the initial capital expenditure, for clients that cannot allow long duration closures for maintenance of certain portions of the road.

Rigid pavements are used throughout the continuously expanding infrastructure in South Africa because of their durability and their minimal maintenance requirements.

Various studies have been done on the performance of CRCP and factors that influence it. Climatic conditions and the material properties of the concrete are the two most important factors affecting CRCP performance, with each site having its own environment, and considerable variance in the material used in each concrete batch. It remains the case that even if a design is executed using average values, temperature, wind, humidity and other governing factors change hourly and are variable for each month of the year.

Thus, no matter how good the model, variation in environmental conditions will never be accurately modelled for a design, and the end product will inevitably differ from what was originally envisaged. There is a need to close the loop between design and actual performance of CRCP, thus refining the design model.

In general, crack distributions of less than 0.5 m will result in loss of stiffness and an increase in stress of the subgrade support, whereas when cracks are too far apart the crack widths become excessive, allowing ingress of water into the pavement layer, which may also lead to failures.

With a better understanding of the uncertainty in design predictions, better confidence can be achieved in the final output of the design. With better confidence the safety factors built into designs could be reduced, bringing down cost of CRCP whilst delivering a good product to the client.

The focus of this research is to determine the difference between the actual design crack distribution and crack widths to determine whether the normal design procedures cater for these variations during the construction process as well as the pavement’s life.

The research was only done on a single type of mix, with variations in proportions. The influence of different types of aggregate, cement, sand etc. was not taken into account. The degree of how the steel influenced the crack distributions as well as the crack widths was taken into account by adjusting the stiffness of the concrete rather than considering friction and increased tensile stresses due to elongation.

2 DESIGN AND AS-BUILT PARAMETERS

The study was conducted on a newly constructed weighbridge in the Gauteng province of South Africa. This weighbridge was previously constructed with asphalt, but because of severe failures had to be reconstructed with a more durable surfacing (CRCP). The weighbridge consists of two sections in the two directions of travel, namely north- and southbound. The pavement design for the CRCP study area is shown in Table 1.
Table 1: Concrete Pavement Design

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (mm)</th>
<th>Classification (COLTO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface</td>
<td>200</td>
<td>CRCP</td>
</tr>
<tr>
<td>Upper subbase</td>
<td>150</td>
<td>C3</td>
</tr>
<tr>
<td>Lower subbase</td>
<td>150</td>
<td>C4</td>
</tr>
<tr>
<td>Roadbed</td>
<td>300</td>
<td>Rip and re-compact</td>
</tr>
</tbody>
</table>

The original concrete mix design for the CRCP is shown in Table 2.

Table 2: Mix design for desired pavement design

<table>
<thead>
<tr>
<th>Material component</th>
<th>Unit</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>kg/m³</td>
<td>454</td>
</tr>
<tr>
<td>CEM II-AM(A-L) 42.5R</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>litres/m³</td>
<td>188</td>
</tr>
<tr>
<td>Admixture (Plasticizer)</td>
<td>kg/m³</td>
<td>3.62</td>
</tr>
<tr>
<td>Aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19.0 mm stone</td>
<td>kg/m³</td>
<td>1050</td>
</tr>
<tr>
<td>crusher sand</td>
<td>kg/m³</td>
<td>565</td>
</tr>
<tr>
<td>filler sand</td>
<td>kg/m³</td>
<td>150</td>
</tr>
<tr>
<td>Cement aggregate ratio (C/A by mass)</td>
<td></td>
<td>0.257</td>
</tr>
<tr>
<td>Maximum Slump (mm)</td>
<td></td>
<td>75</td>
</tr>
</tbody>
</table>

3 RESEARCH INSTRUMENTATION AND DATA COLLECTION

Temperature and environmental data were captured by means of iButtons installed underneath the CRCP in the middle of the lane and a mobile weather station that was mounted on the traffic control centre next to the CRCP.

The weather station was installed before the start of the CRCP construction and gathered information on temperature, radiation, wind speed and direction, and humidity, every 30 minutes for the duration of the project.

A detailed concrete register was kept of all trucks that poured concrete. This register recorded truck number, delivery slump, pour slump, location poured, and delivery note number. The delivery note received from the batching plant shows the proportions of the material that was added to the concrete mix.

Six compressive-strength cube tests and two flexural-strength tests were done per day.

A visual assessment was conducted on the entire CRCP, where the focus was mainly on crack distributions rather than crack type as described in TRH19. A comprehensive visual assessment was conducted after cracks started to initiate and then again after 28 days to see how the crack distributions changed during that time.
4 DATA ANALYSIS

The main focus of this study was to determine the difference between actual and predicted crack distributions and to see whether a model can be used to accurately determine crack distributions for this project. The steps defined below describe the sequence of events that led to the final conclusions presented in this study.

a. Identify suitable concrete sections that deviated significantly from the design (mix proportions) to allow all further data sets to be summarised accordingly.

b. Gather information with regards to physical properties (stiffness of underlying layers, compressive and flexural strengths and all environmental information).

c. Do visual assessment to determine crack distributions.

d. Develop a mechanical model with these known parameters for each section.

e. Compare actual crack distributions to those of the mechanical model.

f. Conduct an ANOVA analysis to determine their statistical importance.

g. Draw conclusions on differences.

a. Identify concrete sections that deviated from the design

The figures below show the percentage each elements of the concrete mix deviated from the original design in relation to location for the weighbridge on the northern and southern sides.

Figure 1: Northbound: Mix variance vs Stake Value
Nine areas were noted where at least one component varied significantly from the norm, and these are used as the identified sections for the rest of the report.

To compensate for the likelihood that the concrete might have mixed with the batch before and after the specific mix, and to allow a larger section for interpretation, the mix before and after was also assumed to have contributed to the given section. This compensation was done by using the average of these mixes.

**Table 3:** Identified sections that varied from the design

<table>
<thead>
<tr>
<th>Mix</th>
<th>Description</th>
<th>Unit</th>
<th>Stone</th>
<th>Crusher Sand</th>
<th>Plaster Sand</th>
<th>Cement</th>
<th>Water</th>
<th>Admixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>Design Mix</td>
<td>kg/m³</td>
<td>1086</td>
<td>454</td>
<td>188</td>
<td>2.887</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>- 3% aggregate</td>
<td>kg/m³</td>
<td>1050</td>
<td>715</td>
<td>454</td>
<td>188</td>
<td>3.837</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>+ 5% water</td>
<td>kg/m³</td>
<td>197.4</td>
<td>178.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>- 5% water</td>
<td>kg/m³</td>
<td>3.263</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>+ 13% admix</td>
<td>kg/m³</td>
<td>3.263</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>- 8% admix</td>
<td>kg/m³</td>
<td>2.656</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>+ 10% sand</td>
<td>kg/m³</td>
<td>786.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>- 10% sand</td>
<td>kg/m³</td>
<td>643.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>different stone type</td>
<td>kg/m³</td>
<td>1076.7</td>
<td>620</td>
<td>236.7</td>
<td>453.7</td>
<td>167.7</td>
<td>2.407</td>
</tr>
<tr>
<td>9</td>
<td>- 3% water and aggregate</td>
<td>kg/m³</td>
<td>1053.4</td>
<td>182.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3 lists the nine identified sections that deviated from the original design. Only the specific properties that changed are shown, to allow for easier interpretation. The mix proportions varied from a 10% reduction to a 13% increase of a specific quantity.

**b. Gather information with regards to physical properties**

To avoid the expense of a plate bearing test to determine the modulus of subgrade reaction, this modulus can also be determined by indirect means, using the material’s California bearing ratio (CBR) value\(^1\), as shown by Equation (1).
\[ \begin{align*}
    k &= 2.7145 \times 10^{-4} (C1 + C2 \cdot e^{C3} + C4 \cdot e^{C5}) \\
\end{align*} \quad (1)
\]

Where:

- \( C1 = 30 + 3360k_o \)
- \( C2 = 0.3778 (h_f - 43.2) \)
- \( C3 = 0.5654 \ln(k_o) + 0.4139 \ln(E_f) \)
- \( C4 = -283 \)
- \( C5 = 0.5654 \ln(k_o) \)

\( k_o \) = modulus of subgrade/substructure reaction at the top of the underlying layer (N/mm\(^3\))

\( h_f \) = thickness of layer

\( E_f \) = dynamic modulus of elasticity of layer (N/mm\(^2\))

\( k \) = modulus of substructure reaction at top of layer (N/mm\(^3\))

To relate the material results to useful information for analytical purposes, Equation (1) was used to calculate the effective subgrade reaction of the entire stabilised subbase. A statistical summary is shown in Table 4.

The compressive and flexural strengths were gathered from tests done on the concrete as apart from the contractor’s quality control regime. The results of these tests for each of the sections are shown in Table 4.

The compressive and flexural strengths in Table 4 reflect the 28-day strengths and are used in Houben’s mechanical model\[^1\] as the strength at 28 days to do back calculations and forward predictions.

In considering the rate of evaporation for the total duration of construction it must be noted that evaporation is important both as an instantaneous process, considering just the instant when the concrete is cast, and as a continuous process, considering the average evaporation of the concrete over the period during which it sets. The reason for considering these two types of evaporation separately is to see whether the rate of evaporation when the concrete is most susceptible to evaporation plays as large a role as the evaporation from the concrete during the entire setting process.

The evaporation rate was calculated using Equation (2)\[^2\].

\[ ER = 5([T_c + 18]^{2.5} - r[T_a + 18]^{2.5}) (V+4) \times 10^{-6} \quad (2) \]

\( ER \) = evaporation rate (kg/m\(^2\)/h)

\( T_c \) = concrete temperature (°C)

\( T_a \) = air temperature (°C)

\( r \) = relative humidity (%)

\( V \) = wind velocity (km/h)

The variables used to calculate the evaporation were all obtained by means of the weather station that was installed on site except for the concrete temperature, which was obtained from the iButton that was cast into the concrete.

Perrie and Rossman\[^3\] advise that special precautions should be taken when the evaporation rate exceeds 1.0 kg/m\(^2\)/h.

The evaporation rates for all sections are shown in Table 4 below.
Table 4: Subgrade reaction for each mix section

<table>
<thead>
<tr>
<th>Section</th>
<th>k (N/mm$^3$)</th>
<th>Comp (MPa)</th>
<th>Flex (MPa)</th>
<th>Evap rate (kg/m$^2$/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>0.22</td>
<td>59.01</td>
<td>5.88</td>
<td>0.40</td>
</tr>
<tr>
<td>1</td>
<td>0.21</td>
<td>61.75</td>
<td>6.19</td>
<td>0.40</td>
</tr>
<tr>
<td>2</td>
<td>0.19</td>
<td>54.25</td>
<td>5.60</td>
<td>0.30</td>
</tr>
<tr>
<td>3</td>
<td>0.21</td>
<td>60.38</td>
<td>5.41</td>
<td>0.40</td>
</tr>
<tr>
<td>4</td>
<td>0.22</td>
<td>55.25</td>
<td>6.29</td>
<td>0.30</td>
</tr>
<tr>
<td>5</td>
<td>0.27</td>
<td>62.50</td>
<td>6.19</td>
<td>0.40</td>
</tr>
<tr>
<td>6</td>
<td>0.23</td>
<td>62.75</td>
<td>5.77</td>
<td>0.60</td>
</tr>
<tr>
<td>7</td>
<td>0.23</td>
<td>56.75</td>
<td>6.15</td>
<td>0.40</td>
</tr>
<tr>
<td>8</td>
<td>0.22</td>
<td>58.40</td>
<td>5.90</td>
<td>0.30</td>
</tr>
<tr>
<td>9</td>
<td>0.23</td>
<td>59.33</td>
<td>5.47</td>
<td>0.40</td>
</tr>
</tbody>
</table>

c. Actual crack distribution

During the visual assessment of the CRCP the actual crack distributions were measured. Frequency distribution plots of these distributions are shown in Figure 3 and Figure 4.

Figure 3: Northbound section: Actual crack distribution
These figures give a good indication of how crack distribution differed for the two long sections even though each had exactly the same design. For a closer understanding of how these crack distributions were observed, a statistical analysis was done on all sections of CRCP. The results from this analysis are shown in Table 5.

Table 5: Statistical values for crack distributions north- and southbound

<table>
<thead>
<tr>
<th>Statistical variable</th>
<th>Crack distribution (m)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Northbound</td>
<td>Southbound</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>1.7</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Largest</td>
<td>11.3</td>
<td>23.2</td>
<td></td>
</tr>
<tr>
<td>Smallest</td>
<td>0.3</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>90th percentile</td>
<td>2.8</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>10th percentile</td>
<td>0.8</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>St dev</td>
<td>0.9</td>
<td>1.2</td>
<td></td>
</tr>
</tbody>
</table>

From the data in the table it can be noted that, in view of the low standard deviation (around 1), the crack distributions are relatively similar per direction, even though the largest crack distributions varied between 11.3 and 23.2 m and the smallest crack distributions varied between 0.3 and 0.1 m.

A statistical analysis was also done in the same manner on all the identified sections and the results are given in Table 6.

Table 6: Statistical values for crack distributions north- and southbound

<table>
<thead>
<tr>
<th>Statistical variable</th>
<th>Crack distribution (m)</th>
</tr>
</thead>
</table>

Figure 4: Southbound section: Actual crack distribution
Theoretical crack distribution used is the mechanical model set up by Houben\cite{1} to simulate the behaviour of concrete. This model gives variance in crack widths over time, and shows crack distributions and how they originate. This model does not, however, take into account the effect of the steel reinforcing in the concrete. To compensate for this it is important to keep in mind that steel behaves differently to concrete and that the main function of the steel in CRCP is to keep the cracks that develop closer together.

To simulate steel in concrete without changing the main principles of the mechanical model it was decided to change a parameter in the concrete to incorporate the effect of steel in the system. On review of all the characteristics of concrete that this model takes into account it was decided to incorporate additional elasticity in the system, on the grounds that dimensions could not be changed because they are an integral element of shrinkage and changing them would change the system to something that behaves differently; nor could tensile strength be changed as it would only develop in the steel once a crack has occurred. This was done by using the composite beam system explained by Carino\cite{4}.

To incorporate the relationship between the concrete modulus of elasticity and the effect of the reinforcing, a composite beam design was used. Equation (3) shows how the new effective modulus of elasticity was calculated.

\[
A_{\text{comp}} \times E_{\text{comp}} = A_c \times E_c + A_s \times E_s
\]

Where:
- \(A_{\text{comp}}\) = The area of the composite system
- \(E_{\text{comp}}\) = The modulus of elasticity of the composite system
- \(A_c\) = The area of concrete in the system
- \(E_c\) = The modulus of elasticity of the concrete in the system
- \(A_s\) = The area of steel in the system
- \(E_s\) = The modulus of elasticity of the steel in the system

This formula provides insight on how the modulus of elasticity could be modelled for the system. The result was that the reinforcing contributed an additional 5% to 10% to the concrete’s modulus of elasticity.

The original design was run and used as a comparison against the other sections that deviated from the original design mix. The results from this model are shown in Figure 5 to 7.

Once the concrete has been poured, the strength in the concrete gradually increases with
the passage of time. In addition, the concrete goes through the various stages of shrinkage, which in turn produces a build-up in stress. Cracks that develop in concrete allow the concrete to relieve stress, only to let the stress build up again until the concrete strength eventually exceeds the induced stress. Figure 5 shows how tensile strength and tensile stress develop through the life of the concrete without occurrence of cracking. The thick black line in the graph indicates how the tensile stress increases and decreases throughout the life of the concrete. This is however only a theoretical model to indicate a full life cycle of stresses, as a crack would occur when the concrete strength is exceeded by an induced stress.

Figure 5: Tensile strength and tensile stress over time without cracks for the original design

The band of stresses shown Figure 5 is caused by temperature variation. It is clear that the induced stress exceeds the concrete strength early during the life of the concrete, which would have induced a crack. Figure 6 shows the same model with the exception that a crack was allowed, and the entire system thus went through a stress relaxation each time after the crack. Figure 7 was also drawn up to isolate the area between 0 and 1000h (±42 days), to give an improved insight into the four cracks that occur.

Figure 6: Tensile strength and tensile stress over time with cracks for the original design
From Figure 6 and 7 it can be seen that the system went through four stress relaxation phases, with phase 1 to 4 happening at 16, 18, 39 and 929 hours respectively, which implies that cracks occurred at three different stages during the life of the concrete. After the third stress relaxation, however, the stress no longer reached the concrete strength and the system was then balanced.

This information can be related to crack distribution. Figure 8 shows how the crack distributions develop after each stress relaxation, and the distances between the cracks.

The final conclusion from Figure 8 was that when the system has completed all its cycles, to the point where the induced tensile stress never exceeds the concrete strength, the final average crack distance would be 2.7m.

A summary of the results from the mechanical model for the 9 Sections are shown in Table 7.

<table>
<thead>
<tr>
<th>Section</th>
<th>Change in mix</th>
<th>Crack distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Increased aggregate content</td>
<td>3.10</td>
</tr>
<tr>
<td>2</td>
<td>5% increase in water content</td>
<td>2.83</td>
</tr>
<tr>
<td>3</td>
<td>5% decrease in water content</td>
<td>3.20</td>
</tr>
<tr>
<td>4</td>
<td>13% increase in add mix content</td>
<td>2.77</td>
</tr>
<tr>
<td>5</td>
<td>8% decrease in add mix content</td>
<td>3.10</td>
</tr>
<tr>
<td>6</td>
<td>10% increase in sand content</td>
<td>3.20</td>
</tr>
<tr>
<td>7</td>
<td>10% decrease in sand content</td>
<td>2.90</td>
</tr>
</tbody>
</table>
e. Comparison between actual and mechanical results

The crack distributions from actual measurements and measurements from the mechanical model are shown in Table 8 and, together with percentage deviances from the original design crack distributions.

<table>
<thead>
<tr>
<th>Section</th>
<th>Change in mix</th>
<th>Crack distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Different aggregate type</td>
<td>3.00</td>
</tr>
<tr>
<td>9</td>
<td>3% decrease in water and aggregate</td>
<td>3.05</td>
</tr>
</tbody>
</table>

From Table 8 it is clear that the mechanical model was not able to accurately determine the crack distributions as they actually happened, although the table also shows that for 70% of the cases the model was able to determine whether the crack distribution would increase or decrease from the original design.

f. ANOVA analysis to determine statistical importance

This section presents a statistical analysis of all of the data to determine the analysis of variance for the set of data collected. Data from all of the previous sections were used to determine the influence each parameter has on crack distributions and to eventually set up a theoretical model as an equation that can be used to determine whether crack distributions can be accurately determined from the collected data.

The purpose of this ANOVA analysis was to predict a dependent variable based on one or more predictor variables and determine whether those predictor variables are good in predictions.

The predictor variables that were used in the initial analysis are the average variables for each of the determined sections. These variables together with the results are shown in Table 9.
Table 9: Predictor variables for ANOVA analysis

<table>
<thead>
<tr>
<th>Section</th>
<th>Mix Properties (kg)</th>
<th>Subgrade</th>
<th>Strength (MPa)</th>
<th>Evap (kg/m²/hr)</th>
<th>Crack Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stone</td>
<td>Sand</td>
<td>Cement</td>
<td>Water</td>
<td>Admixture</td>
</tr>
<tr>
<td>Original</td>
<td>1086</td>
<td>715</td>
<td>454</td>
<td>188</td>
<td>2.887</td>
</tr>
<tr>
<td>1</td>
<td>1050</td>
<td>715</td>
<td>454</td>
<td>188</td>
<td>3.837</td>
</tr>
<tr>
<td>2</td>
<td>1070</td>
<td>646</td>
<td>454</td>
<td>197.4</td>
<td>3.03</td>
</tr>
<tr>
<td>3</td>
<td>1086</td>
<td>700</td>
<td>454</td>
<td>178.6</td>
<td>2.89</td>
</tr>
<tr>
<td>4</td>
<td>1086</td>
<td>650</td>
<td>454</td>
<td>188</td>
<td>3.263</td>
</tr>
<tr>
<td>5</td>
<td>1086</td>
<td>650</td>
<td>454</td>
<td>188</td>
<td>2.656</td>
</tr>
<tr>
<td>6</td>
<td>1086</td>
<td>786.5</td>
<td>454</td>
<td>188</td>
<td>2.89</td>
</tr>
<tr>
<td>7</td>
<td>1086</td>
<td>643.5</td>
<td>454</td>
<td>188</td>
<td>2.89</td>
</tr>
<tr>
<td>8</td>
<td>1076.7</td>
<td>856.7</td>
<td>453.7</td>
<td>167.7</td>
<td>2.407</td>
</tr>
<tr>
<td>9</td>
<td>1053.4</td>
<td>1606</td>
<td>130</td>
<td>454</td>
<td>182.4</td>
</tr>
</tbody>
</table>

However to determine whether different combinations produce adequate results with a regression analysis all combinations were ran to see how good the data fit. The results from this analysis are shown in Table 10. The “x” indicates whether the parameter was considered for the analysis.

Table 10: Parameter combinations showing how well the data fit

<table>
<thead>
<tr>
<th>Stone</th>
<th>Sand</th>
<th>Cement</th>
<th>Water</th>
<th>Admixture</th>
<th>k</th>
<th>Comp</th>
<th>Flex</th>
<th>Evap</th>
<th>Multiple R</th>
<th>R Square</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td>0.52</td>
<td>0.27</td>
</tr>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>x</td>
<td>x</td>
<td>0.57</td>
<td>0.32</td>
</tr>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td>0.80</td>
<td>0.64</td>
</tr>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
<td>x</td>
<td>x</td>
<td>0.86</td>
<td>0.73</td>
</tr>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>x</td>
<td>x</td>
<td>0.86</td>
<td>0.74</td>
</tr>
<tr>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>x</td>
<td></td>
<td>0.88</td>
<td>0.77</td>
</tr>
</tbody>
</table>

From the data in Table 10 it is deduced that by using the four parameters that had the best correlation to crack distributions, the best fit was obtained. Thus Compressive-, Flexural strength, evaporation and admixtures were used further in the regression model.

The equation that best represents the influence of each of these predictor variables is taken from the first column of Table 11 and is shown by Equation (4).

\[
\text{Crack} = 0.12 \times \text{Admix} + 0.07 \times \text{Comp} + 0.07 \times \text{Flex} - 1.2 \times \text{Evap} - 2.38
\]  

(4)

The residual value in Table 11 shows the difference between the predicted crack distribution, from the determined equation, and the actual crack distribution.

Table 11: Regression outputs
The standard error ranges between 0.009 and 0.229, which implies very good correlation with all of the determined sections.

Equation (4) was very accurate in determining the actual crack distribution, as shown in Table 11, but because it was set up with only a small cluster of data the only way to know how well it correlates to the rest of the project data is by running the analysis with other sections in the project.

Sections were chosen where the most complete data sets were available. The section locations and date and time of casting are shown in Table 12.

**Table 12: Test section location and date and time of casting**

<table>
<thead>
<tr>
<th>Section name</th>
<th>Location</th>
<th>Location</th>
<th>Date cast</th>
<th>Time cast</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Section 1</td>
<td>Southbound</td>
<td>CH 61 - 101</td>
<td>2013/09/03</td>
<td>11:17:00 AM</td>
</tr>
<tr>
<td>Test Section 2</td>
<td>Southbound</td>
<td>CH 253 - 327</td>
<td>2013/09/16</td>
<td>12:01:00 PM</td>
</tr>
<tr>
<td>Test Section 3</td>
<td>Southbound</td>
<td>CH 1218 - 1314</td>
<td>2013/09/06</td>
<td>07:10:00 AM</td>
</tr>
</tbody>
</table>

The mix proportions and the degree of subgrade reaction, together with their strength properties and the evaporation rate, are shown in Table 13.

**Table 13: Physical and environmental properties of the test sections**

<table>
<thead>
<tr>
<th>Section name</th>
<th>Admixture (kg)</th>
<th>Compressive (MPa)</th>
<th>Flexural (MPa)</th>
<th>Evaporation (kg/m²/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Section 1</td>
<td>2.5</td>
<td>72.0</td>
<td>6.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Test Section 2</td>
<td>2.9</td>
<td>56.8</td>
<td>5.9</td>
<td>0.4</td>
</tr>
<tr>
<td>Test Section 3</td>
<td>2.9</td>
<td>61.2</td>
<td>6.1</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The equation from the ANOVA analysis was used and the actual crack distributions were compared to the predicted crack distributions. The results from this, together with the percentage error found, are shown in Table 14.

**Table 14: Actual vs predicted crack distribution for the test sections**

<table>
<thead>
<tr>
<th>Section</th>
<th>Crack distance (m) Actual</th>
<th>Predicted</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.561</td>
<td>1.570</td>
<td>0.009</td>
</tr>
<tr>
<td>2</td>
<td>1.879</td>
<td>1.908</td>
<td>0.029</td>
</tr>
<tr>
<td>3</td>
<td>1.297</td>
<td>1.331</td>
<td>0.033</td>
</tr>
<tr>
<td>4</td>
<td>1.741</td>
<td>1.635</td>
<td>-0.107</td>
</tr>
<tr>
<td>5</td>
<td>1.592</td>
<td>1.483</td>
<td>-0.109</td>
</tr>
<tr>
<td>6</td>
<td>1.840</td>
<td>1.821</td>
<td>-0.019</td>
</tr>
<tr>
<td>7</td>
<td>1.641</td>
<td>1.598</td>
<td>-0.043</td>
</tr>
<tr>
<td>8</td>
<td>1.433</td>
<td>1.420</td>
<td>-0.013</td>
</tr>
<tr>
<td>9</td>
<td>1.596</td>
<td>1.586</td>
<td>-0.010</td>
</tr>
<tr>
<td>10</td>
<td>1.340</td>
<td>1.570</td>
<td>0.229</td>
</tr>
</tbody>
</table>

The standard error ranges between 0.009 and 0.229, which implies very good correlation with all of the determined sections.

Equation (4) was very accurate in determining the actual crack distribution, as shown in Table 11, but because it was set up with only a small cluster of data the only way to know how well it correlates to the rest of the project data is by running the analysis with other sections in the project.

Sections were chosen where the most complete data sets were available. The section locations and date and time of casting are shown in Table 12.

**Table 12: Test section location and date and time of casting**

<table>
<thead>
<tr>
<th>Section name</th>
<th>Location</th>
<th>Location</th>
<th>Date cast</th>
<th>Time cast</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Section 1</td>
<td>Southbound</td>
<td>CH 61 - 101</td>
<td>2013/09/03</td>
<td>11:17:00 AM</td>
</tr>
<tr>
<td>Test Section 2</td>
<td>Southbound</td>
<td>CH 253 - 327</td>
<td>2013/09/16</td>
<td>12:01:00 PM</td>
</tr>
<tr>
<td>Test Section 3</td>
<td>Southbound</td>
<td>CH 1218 - 1314</td>
<td>2013/09/06</td>
<td>07:10:00 AM</td>
</tr>
</tbody>
</table>

The mix proportions and the degree of subgrade reaction, together with their strength properties and the evaporation rate, are shown in Table 13.

**Table 13: Physical and environmental properties of the test sections**

<table>
<thead>
<tr>
<th>Section name</th>
<th>Admixture (kg)</th>
<th>Compressive (MPa)</th>
<th>Flexural (MPa)</th>
<th>Evaporation (kg/m²/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Section 1</td>
<td>2.5</td>
<td>72.0</td>
<td>6.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Test Section 2</td>
<td>2.9</td>
<td>56.8</td>
<td>5.9</td>
<td>0.4</td>
</tr>
<tr>
<td>Test Section 3</td>
<td>2.9</td>
<td>61.2</td>
<td>6.1</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The equation from the ANOVA analysis was used and the actual crack distributions were compared to the predicted crack distributions. The results from this, together with the percentage error found, are shown in Table 14.

**Table 14: Actual vs predicted crack distribution for the test sections**

<table>
<thead>
<tr>
<th>Section</th>
<th>Crack distance (m) Actual</th>
<th>Predicted</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.561</td>
<td>1.570</td>
<td>0.009</td>
</tr>
<tr>
<td>2</td>
<td>1.879</td>
<td>1.908</td>
<td>0.029</td>
</tr>
<tr>
<td>3</td>
<td>1.297</td>
<td>1.331</td>
<td>0.033</td>
</tr>
<tr>
<td>4</td>
<td>1.741</td>
<td>1.635</td>
<td>-0.107</td>
</tr>
<tr>
<td>5</td>
<td>1.592</td>
<td>1.483</td>
<td>-0.109</td>
</tr>
<tr>
<td>6</td>
<td>1.840</td>
<td>1.821</td>
<td>-0.019</td>
</tr>
<tr>
<td>7</td>
<td>1.641</td>
<td>1.598</td>
<td>-0.043</td>
</tr>
<tr>
<td>8</td>
<td>1.433</td>
<td>1.420</td>
<td>-0.013</td>
</tr>
<tr>
<td>9</td>
<td>1.596</td>
<td>1.586</td>
<td>-0.010</td>
</tr>
<tr>
<td>10</td>
<td>1.340</td>
<td>1.570</td>
<td>0.229</td>
</tr>
<tr>
<td>Section name</td>
<td>Crack distribution (m)</td>
<td>Percentage error</td>
<td></td>
</tr>
<tr>
<td>------------------</td>
<td>------------------------</td>
<td>------------------</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Actual</td>
<td>Predicted</td>
<td></td>
</tr>
<tr>
<td>Test Section 1</td>
<td>1.137</td>
<td>1.759</td>
<td>55%</td>
</tr>
<tr>
<td>Test Section 2</td>
<td>1.138</td>
<td>1.406</td>
<td>24%</td>
</tr>
<tr>
<td>Test Section 3</td>
<td>1.180</td>
<td>1.628</td>
<td>38%</td>
</tr>
</tbody>
</table>

The data in Table 14 shows that the predicted crack distribution from the actual data does not correspond well with the actual crack distribution. The percentage error is relatively large indicating that this model is not a good representation of the data available even though the data selected for the model represented the parameters that had the best correlation with actual crack distribution. The reason for this is the small sample of data available and the large variance in actual data reported.

5 CONCLUSION

Although the mechanical model was not able to obtain a very accurate prediction of actual crack distributions, it was able to predict with 70% accuracy whether crack distribution will increase or decrease from the original design, based purely on the environmental conditions and physical properties of the material.

The model does not directly take into account humidity, wind and other environmental factors apart from temperature. Thus, it is not possible to effectively simulate the increase in stress induced during the plastic phase of the concrete’s curing process. Although such factors could be taken into account indirectly by changing the shrinkage properties of the concrete, this would not be helpful for simulation purposes as the consequences would persist throughout the life of the concrete rather than being concentrated in the initial 48 hours when the concrete is most sensitive to these factors.

It is therefore suggested that an additional parameter be created to give better simulation of shrinkage (drying, autogeneous and chemical) during the plastic phase of the concrete in which shrinkage is dependent on (or more sensitive to) environmental conditions other than temperature. This will allow for a more accurate conclusion and will lead to better design outputs.

Further research is suggested to determine these parameters that better simulate the behaviour of the concrete during the plastic phase as well as refining the existing parameters to South African conditions and materials. It is also further suggested that the effect of reinforcing be taken into account in a direct way where the effect of the steel is only taken into account after strain in the steel occurs.

The ANOVA analysis did not yield high accuracy, i.e. approximately 55% of predicting the crack distribution of this CRCP does not take into account the lifecycle of the concrete. The ANOVA analysis used data from the CRCP pavement and reverse engineered an equation to predict the crack distribution, whereas the mechanical model uses first principles and would be applicable in various scenarios.

If this method is to be used in future it is suggested that a uniform mix design be used with tests done on the each parameter of the mix design where only one parameter changes with each test. These results could then be used to determine the extent of influence each parameter has on the concrete’s physical properties which can later be simulated in a trial
section to determine the influence of these physical parameters on actual crack distribution.

REFERENCES


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ASSESSMENT OF FOUR ELECTRICAL MEASUREMENT METHODS FOR ASSESSING THE CHLORIDE RESISTANCE OF CONCRETES

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Key words: Concrete; Chloride permeability; conductivity; pore solution.

Abstract. The present study is an exploratory research that appraise and compare the performance of four rapid tests for assessing the chloride penetration of concrete; Rapid Chloride Permeability Test (RCPT), Rapid Chloride Migration Test (RCMT), Surface Electrical Resistivity (SR) and Modified Rapid Chloride Permeability Test (MRCPT). Due to the concern raised by some scholars about the effect that the conductivity of the pore solution impinges upon the results on some of these rapid tests, silica fume was used as supplementary cementitious material in portions of 7.5% and 15%. All four methods exhibited substantial reduction in the chloride permeability of concrete mixtures containing silica fume compared to the control mixture, at the ages of 28 and 90 days. The increase in performance caused by silica fume usage was significant for RCPT and SR methods, moderate for RCMT and marginal for MRCPT. The results show that the variation in the conductivity of the pore solution significantly alters the results of SR and RCPT tests, and marginally alters those of MRCPT.

1 INTRODUCTION

It is widely known that the penetration of chloride ions into reinforced concrete and the resulting corrosion in the reinforcing bars is a major cause of damage to concrete structures which engenders problems and inflicts costs [1]. This renders imperative to study the resistance of concrete against chloride ion penetration and to measure its diffusion coefficient.

Alters those the diffusion coefficient of chloride ions in concrete can be obtained from tests such as ASTM C1556 [2] and ASTM C1543 [3]. In these tests, the concrete sample is exposed to salt solution, and after a prescribed time, chloride concentration at various depths in concrete is determined by profile grinding. Then the diffusion coefficient is extracted from Fick’s equation [4].

The shortcomings associated with these methods are that they are slow and costly [5]. Consequently, rapid tests are proposed as alternatives to assess the resistance of concrete against penetration of chloride ions. In some tests, such as the RCMT [6], the penetration of
chloride ions is accelerated by an external electric field applied to the sample, and the diffusion coefficient is then measured. In other tests, such as RCPT [7], SR [8] and MRCPT [9], the resistance of concrete against chloride ion penetration is assessed by attributes such as electrical conductivity of concrete.

The electrical properties of cement-based materials have been investigated for nearly a century [10-13]. Electrical measurements can be used to estimate the diffusion coefficients of saturated porous materials [14]. The electrical migration tests, and the conductivity measurements are two most commonly used electrical measurement techniques. Migration tests are used to determine the ionic mobility while conductivity tests are used to estimate the formation factor. Both ionic mobility and formation factor can be related to diffusivity [15].

Although diffusivity and mobility have different mechanisms in terms of the ionic transportation properties, they relate to each other in low ionic concentration levels by the Einstein equation [16] as follows:

$$\mu = zF \frac{D}{RT}$$  \hspace{1cm} (1)

$\mu$, $z$, $F$, $D$, $R$ and $T$ denote the mobility of ion, charge of ion, Faraday's constant, diffusivity of ion, gas constant and absolute temperature, respectively.

Noting that the conductivity of concrete samples affects the results of most rapid tests of chloride permeability [17], the conductivity of the pore solution is then a key determinant in the results of these tests. This problem is particularly salient in studying concrete samples containing supplementary cementitious materials, especially active pozzolans such as silica fume [18]. The underlying reason is that in pozzolanic reactions, OH$^-$ ions of the pore solution which are essential in its conductivity are consumed and the conductivity of the pore solution plummets significantly. Hence, the results of these rapid tests for concrete samples with supplementary cementitious materials may be imprecise and must be appraised with care.

As discussed above, there are different methods to assess the chloride permeability of concrete, each with strengths and shortcomings that are contingent on the application, and must be selected according to the particularities of the intended application [19]. In this research, the performance of the four conventional rapid tests for assessing concrete permeability against chloride penetration (RCPT, SR, MRCPT and RCMT) was studied and compared with the Bulk diffusion method (BD). To this end, a total of 12 concrete mixtures were used and cement was replaced with silica fume in some of these mixtures.

2 EXPERIMENTAL PROGRAM

2.1 Material and mixture proportion

Twelve concrete mixtures were prepared and tested in the laboratory. ASTM C150 type I Portland cement was used for all of the concrete mixtures. The amounts of cement used in the mixtures were 350 and 400 kg/m$^3$. Silica fume was used as supplementary cementitious material with 7.5% and 15% portions. Chemical characteristics of the mentioned materials are shown in Table 1.

In all cases, crushed coarse aggregate with maximum size of 19mm and natural sand as fine aggregates was used. The coarse aggregates have specific gravity and water absorption of 2510 kg/m$^3$ and 1.90% respectively, and the fine aggregate has water absorption of 2.75%
and specific gravity of 2570 kg/m$^3$. Potable water was used for casting the concrete samples, with water to binder ratios of 0.35 and 0.45. Also, the Polycarboxylate Ether (PCE) based superplasticizer with specific gravity of 1.05, solids content of 28% and pH of 6.6 was employed to achieve relatively constant workability. Details of the mixtures are presented in Table 2.

**Table 1:** Chemical and physical characteristics of cement and silica fume

<table>
<thead>
<tr>
<th>Chemical Composition* (%)</th>
<th>OPC</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>65.3</td>
<td>0.36</td>
</tr>
<tr>
<td>SiO$_2$</td>
<td>20.8</td>
<td>94.66</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>4.3</td>
<td>0.31</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>2.2</td>
<td>0.60</td>
</tr>
<tr>
<td>MgO</td>
<td>2.17</td>
<td>0.78</td>
</tr>
<tr>
<td>K$_2$O</td>
<td>0.63</td>
<td>0.22</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>0.36</td>
<td>0.22</td>
</tr>
<tr>
<td>Loss on ignition (%)</td>
<td>0.91</td>
<td>1.77</td>
</tr>
<tr>
<td>Physical properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Specific gravity</td>
<td>3.15</td>
<td>2.14</td>
</tr>
<tr>
<td>Fineness** (cm$^3$/g)</td>
<td>2800</td>
<td>---</td>
</tr>
</tbody>
</table>

* Chemical Composition is specified according to ASTM C114  
** Fineness is determined by blaine apparatus based on ASTM C204

**Table 2:** Mixture proportion of concretes

<table>
<thead>
<tr>
<th>Mix</th>
<th>W/b</th>
<th>Silica Fume (%)</th>
<th>Cement (kg/m$^3$)</th>
<th>Water (kg/m$^3$)</th>
<th>Aggregate (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Coarse</td>
</tr>
<tr>
<td>4045SF0</td>
<td>0.45</td>
<td>0</td>
<td>0</td>
<td>400</td>
<td>180</td>
</tr>
<tr>
<td>4035SF0</td>
<td>0.35</td>
<td>0</td>
<td>0</td>
<td>400</td>
<td>140</td>
</tr>
<tr>
<td>3545SF0</td>
<td>0.45</td>
<td>0</td>
<td>0</td>
<td>350</td>
<td>157.5</td>
</tr>
<tr>
<td>3535SF0</td>
<td>0.35</td>
<td>0</td>
<td>0</td>
<td>350</td>
<td>122.5</td>
</tr>
<tr>
<td>4045SF7.5</td>
<td>0.45</td>
<td>7.5</td>
<td>30</td>
<td>370</td>
<td>180</td>
</tr>
<tr>
<td>4035SF7.5</td>
<td>0.35</td>
<td>7.5</td>
<td>30</td>
<td>370</td>
<td>140</td>
</tr>
<tr>
<td>3545SF7.5</td>
<td>0.45</td>
<td>7.5</td>
<td>26.25</td>
<td>323.75</td>
<td>157.5</td>
</tr>
<tr>
<td>3535SF7.5</td>
<td>0.35</td>
<td>7.5</td>
<td>26.25</td>
<td>323.75</td>
<td>122.5</td>
</tr>
<tr>
<td>4045SF15</td>
<td>0.45</td>
<td>15</td>
<td>60</td>
<td>340</td>
<td>140</td>
</tr>
<tr>
<td>4035SF15</td>
<td>0.35</td>
<td>15</td>
<td>60</td>
<td>340</td>
<td>140</td>
</tr>
<tr>
<td>3545SF15</td>
<td>0.45</td>
<td>15</td>
<td>52.5</td>
<td>297.5</td>
<td>157.5</td>
</tr>
<tr>
<td>3535SF15</td>
<td>0.35</td>
<td>15</td>
<td>52.5</td>
<td>297.5</td>
<td>122.5</td>
</tr>
</tbody>
</table>

2.2 Testing procedure and specimen preparation

All the specimens were casted in three layers; each layer was consolidated on a vibrating table to reduce the air voids. Then, the specimens were kept under a wet towel in the laboratory for 24 hours. After that, they were demolded and cured in lime-saturated water at 23 ± 2 °C until the test day to impede possible leaching of Ca(OH)$_2$ from these specimens.
2.2.1 Surface electrical resistivity test (SR)

Electrical resistivity of concrete is an inherent attribute whose magnitude is contingent upon its ingredients and moisture [20, 21]. Theoretical and empirical studies evince the existence of an underlying relation between electrical resistivity of concrete and the penetration of chloride ions [22]. In general, there is an inverse relation between permeability coefficient of chloride ions and the electrical resistivity of concrete [23]. In a given structure, parts with higher susceptibility against penetration of chloride ions have comparatively lower electrical resistivity. Electrical resistivity is also related to the corrosion rate of bars after the corrosion begins [24]. The flow of ions between anode and cathode on the surface of the bar is a controlling factor for the corrosion rate [15].

One of the most reliable methods for measuring the electrical resistivity of concrete is Wenner four-point method, which is nondestructive, fast and simple [25]. In this method, an array of four equi-distance electrodes is brought into contact with the surface of concrete.

It must be pointed out that the results of SR tests are affected by the chemical compound of the pore solution [26]. This can influence the results of applying these methods in studying concrete samples with supplementary cementitious materials, due to the low concentration of OH⁻ ions in their pore solution. Nonetheless, mainly due to the simplicity and speed of this test, it is suggested within the standards.

In this study, at the age of 28 and 91 days, surface resistivity of three 100×200 mm cylindrical specimens were measured via a four-point Wenner array probe. The set up utilizes four equally spaced surface contacts, where a 13 Hz alternating current is passed through a concrete sample between the outer pair of contacts. The probe array spacing used was 50 mm. The resistivity measurements were taken at four quaternary longitudinal locations of the specimen.

2.2.2 Rapid chloride permeability test (RCPT)

The RCPT test has been standardized in ASTM and AASHTO and is widely used globally [27]. In this method, the measured quantity is the total amount of electric charge passed through saturated concrete samples of 100 mm diameter and 50 mm thickness within a period of 6 hours under 60 V potential difference. Concrete samples are from one side in contact with NaCl solution and from the other with NaOH solution, and by applying potential difference, chloride ions are injected inside the sample through the electric current that ensues.

Although RCPT has garnered much attention due to its relative simplicity, there are concerns raised about it [28]. In RCPT there is no steady state regime for penetration, i.e., to reach the steady state one has to wait long enough so that the chloride ions have reached the other end of the sample starting from the other, and the waiting time is unduly long, rendering it infeasible. Another pitfall associated with the RCPT method is the heat generated throughout the experiment [29]. Also when pozzolans are used, the characteristics of the pore solution undergo changes that render the RCPT results unreliable [17].

In this study, RCPT was carried out in accordance with ASTM C1202 [7]. At the age of 28 and 91 days, three 50 mm thick slices were cut from the middle part of 100×200 mm cylindrical specimens of each mixture. These disks were then conditioned and vacuum
saturated and assembled in the test cell. Finally, direct current (DC) by a constant 60 V potential difference was applied to them for a six-hour period.

2.2.3 Rapid chloride migration test (RCMT)

Among the rapid tests for chloride ion penetration that have proved reliable is Rapid Chloride Migration Test (RCMT) [5]. The general scheme of this method, standardized in NT Build [6] and AASHTO [30], resembles that of RCPT. To avoid temperature increase during the experiment, the applied potential difference is adjusted according to the initial current. The volume of the NaCl solution which is contiguous to the concrete is taken to be large so that the variation in the concentration of chloride ions will be negligible. In this method, to prevent the effects of other ions such as OH⁻ on the results, the penetration depth of chloride ions is measured directly by halving the sample and spraying 0.1 M silver nitrate solution on the freshly split section.

This method was applied to all samples according to NT Build 492 [6] in each age of the experiment. The preparation of samples was identical to that described for RCPT, with the only distinction being that lime-saturated water was used instead of distilled water.

The conditioned specimens are placed in the rubber ring and the clamps are tightened. 0.3 molar NaOH solution is poured into the rubber ring to cover the upper face of the specimen. The assembly is then placed on the special stand inside the plastic box containing 10% solution of NaCl.

Then, via the two electrodes situated at the opposing ends of the sample, a potential difference is applied proportional to the current initially flowed, which propels the chloride ions into the sample.

2.2.4 Modified rapid chloride permeability test (MRCPT)

Another rapid test for assessing the permeability of concrete against chloride ions is the MRCP test, designed at AmirKabir University [9], whose motivating rationale was to remedy the shortcomings concomitant with the RCPT test. To prepare the samples in this method, they are first dried in the oven, and then saturated using 23% NaCl solution.

MRCPT relies on measuring the conduction of concrete which depends on the intrinsic microstructure and amount of charge carriers in the pore solution, mostly the hydroxyl ions. Therefore, by saturating the concrete specimens with highly conductive solution, the variability of pore water becomes much less significant. Different concrete specimens would yield different conductivities because of the differences in their pore structure.

A limitation in the MRCP test stems from slicing smaller size specimens of 100 mm diameter and 25 mm thickness which results in disturbance in the microstructure. Nevertheless, this is regarded as only surface effect. However, concrete samples with small thicknesses cannot accurately emulate real concrete with coarse aggregate. Nonetheless, MRCPT has several advantages over RCPT, one of which is that the current is at the steady state from the inception. The reading of current is observed quickly as the chloride ion flux and no measurements are required at the downstream reservoir.

In this research, 3 cylindrical samples with 100 mm of diameter and 25 mm of thickness are prepared and are placed inside cells, similar to the RCPT method at the ages of 28 and 91 days. Then potential difference of 10 V was applied and the flowed current was measured.
after 1 minute has passed. The conductivity of the samples was calculated using Equation (1), and the results were reported accordingly.

\[ \sigma = \frac{i}{v} \times \frac{t}{A} \]  

(2)

Where \( \sigma \) is the conductivity of the sample [mS/cm], \( i \) is the electric current [mA], \( v \) is the potential difference [V], \( t \) is the thickness of the sample [cm], and \( A \) is the cross section area of the sample [cm²].

3 RESULTS AND DISCUSSION

3.1 SR

The electrical resistivity of concrete depends upon the microstructure of the hardened paste and the chemical compounds of the pore solution [31-33]. Electrical resistivity is actually an indicator of the mobility of ions throughout the concrete matrix.

To provide convenience of comparison between the results obtained from the surface resistivity test with those of other tests, Figures 1 and 2 presents the surface conductivity of the samples, which is numerically the inverse of resistivity.

Figure 1: Results of conductivity (inverse of SR), 28 days

Figure 2: Results of conductivity (inverse of SR), 91 days
As one would expect, in all samples, for equal amount of cementitious material, the reduction in the water-to-cement ratio and consequently, the reduction of capillary pores and the condensation of paste, decreases the conductivity and permeability of concrete.

At a fixed water-to-cement ratio, in all samples reduction in the cement content leads to a decrease in conductivity. It can be justified by noting that reducing the cement content, reduces the volume of paste in the concrete as compared to solid. Furthermore, solid function as insulators in the structure of concrete, hence as the ratio of the insulator phase of concrete to paste increases, conductivity diminishes.

At constant water to binder ratio and cement content, replacing cement with silica fume will remarkably decrease the conductivity of concrete samples. For instance, in “4045” mixture, 15% replacement of silica fume in ordinary Portland cement decreased the surface conductivity of concrete samples by 78% and 90% at the ages of 28, and 91 days, respectively. This reduction can be justified by noting that pozzolanic reactions proliferate the tortuosity in the structure of capillary pores and further condenses the pore structure. Furthermore, as the concentration of OH⁻ ions diminishes in most of these reactions, the conductance of the pore solution diminishes substantially. It is worth mentioning that OH⁻ ions have the key role in the conductance of the pore solution [34]. As a result of these two phenomena, as well as dilution, the conductivity of concrete substantially decreases.

3.2 RCPT

The results of the RCPT for all samples are presented in Figure 3 and 4. The overall trend observed in the results is similar to those of the SR test.

The results illustrate that replacing cement with silica fume reduces the permeability of concrete. Replacement of cement with silica fume results in a 91% reduction in the determinant of the total passed charge in “4045” mixture at the age of 91 days. Generally these reductions are more than that of the SR test; while theoretically they must coincide (the total passed charge equals the area under the current-time curve). The discrepancy arises from the deficiency of RCPT. In RCPT, the current passes through the samples generate the heat which in turn increases the flowing current. Consequently, the difference between the passed charge in samples with silica fume and the control samples is greater than that one would expect to observe solely due to the difference in their conductivity.

![Figure 3: Results of RCPT, 28 days](image-url)
3.3 RCMT

In contrast to RCPT and SR test, in RCMT the penetration depth of chloride ions is measured directly. This is suggestive of the superiority of this method in the assessment of the chloride permeability of concrete, especially when supplementary cementitious materials such as pozzolans are utilized [35]. According to the electro-chemical relation posited in NT Build, the diffusion coefficient of the chloride ions is obtained from the applied potential difference, time, average temperature and penetration depth. The results of this experiment are depicted in Figure 5 and 6. As can be observed, the overall trend of the results is in a good agreement with those of SR and RCP tests.

Although in this method, the difference induced by applying silica fume as compared to control samples is smaller than that of SR and RCPT methods, since this method is less sensitive to variations in the conductivity of the pore solution. The maximum reduction in the migration coefficient resulted by applying silica fume, corresponds to “3545” mixture, are 70% and 80% at the ages of 28 and 91 days, respectively. Overall, this method provides a more potent determinant to assess the chloride permeability of concrete as compared to SR and RCPT methods.
3.4 MRCPT

In this method, it is claimed that by saturating concrete samples with NaCl solutions having high conductance, the effect of the conductance of the pore solution on the results diminishes significantly, and the alteration of the results induced by the variations in the microstructure of capillary pores. Figures 7 and 8 depict the results of MRCPT for all samples.

It can be observed that applying silica fume reduces the conductivity of the samples. However, this reduction is smaller than those observed in the other three tests. The reduction between the differences of the results of the experiments can be ascribed to the saturation of the samples with 23% NaCl solution and the uniformity of the conductivity of the pore solution of the concrete samples under experiment. This corroborates the claim that this method is less sensitive towards the variations in the conductivity of the pore solution in different concretes. The maximum reduction observed in the conductivity of concrete samples by applying silica fume is 57% for the “3545” mixture at the age of 91 days. The results of the MRCPT test can also be analyzed in terms of the effect of the cement content and water-to-cement ratio, in which the observations and discussions would be similar to those presented for the SR test method.
4 CONCLUSIONS

- The results of all test methods illustrated that applying silica fume reduces the chloride permeability of concrete.
- Due to the consumption of OH⁻ ions that ensue in the pozzolanic reactions of the silica fume and consequently the substantial reduction in the conductance of the pore solution, the permeability indicator of concrete samples incorporating silica fume exhibit a substantial reduction when compared to control samples for SR and RCPT methods. This hints at a shortcoming of these methods in assessing the permeability of concretes with active pozzolans.
- The reduction in the permeability of samples with silica fume as compared to control samples is smaller in the RCMT method than those observed for SR and RCPT methods. This can be ascribed to the direct measurement of the penetration depth of chloride ions, as well as saturation of the concrete samples with lime-saturated water. Overall, RCMT yields more reliable results than that of SR and RCPT.
- Among the electrical methods that were studied, the MRCPT has the least sensitivity to the replacement of cement with silica fume. This can be ascribed to the saturation of the samples with 23% NaCl solutions and the uniformity of the pore solutions conductivity of the different concrete samples. It can be contended that MRCPT offers a more transparent account on the performance and permeability of concretes with silica fume compare to RCPT and SR.

REFERENCES


CALCIUM HYDROXIDE CURING FOR ACCELERATED CARBONATION TESTING OF HIGH VOLUME FLY ASH CEMENTITIOUS BLENDS

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Key words: Calcium hydroxide, carbonation, fly ash, durability

Abstract. During wet curing, the alkali concrete compounds, such as calcium hydroxide, can be leached out to the curing solution, due to the pH gradient between concrete and curing solution. In the presence of high volume fly ash cementitious blends, there is a decrease in concrete pH that may further magnify the problem. In this context it was carried out a research in mortars with an original composition of high volume fly ash and calcium hydroxide. These were exposed to water curing and water saturated with calcium hydroxide curing. The results show that the introduction of calcium hydroxide in the curing solution, provides a slightly enhancement of carbonation resistance. Based on the obtained results, the incorporation of calcium hydroxide in the initial composition seems to be useful to develop extra strength to carbonation of high volume of fly ash concrete.

1 INTRODUCTION

Lime is one of the oldest building materials [1], whose first known uses in mortar, date back to the Palaeolithic Age in the Fire Age [2]. Some of known ways of using lime refer to the Neolithic period: lime fragments were found in Nevalı Çorî, in eastern Turkey, referring to 8,000-10,000 B.C.; the oldest mortar made with just lime are in Jericho, Palestine, dating
from 7,000 B.C.; the first uses of hydrated lime in the construction are in the city of Catal Huyuk, South Anatolia, Turkey, dated 6,000 B.C.; and knows the use of lime in stabilization of soils on construction of Shersi pyramid in Tibet, dated 5,000 BC [1-3]. Until the Nineteenth Century, the air lime was the most widely used material as a binder. Was replaced by hydraulic lime and later by the cement. Today, with less expression is used for rehabilitation of old buildings [1, 4]. More recently, mainly in the last decade, the use of hydrated lime with cement has also been declining due to increased use of superplasticizers [5]. Today, knowing that there are large environmental problems connected with the cement industry [6-8], we can use lime to take advantage of some of its properties, reducing the use of cement in the concrete at the same time we add added value on its properties [9].

In this study, we intend to make mortars with cement, high volume of fly ash and hydrated lime. Fly ash, when applied in high volume, have many environmental advantages, but tend to dramatically lower the mechanical and carbonation resistance. Furthermore it is intended to add to hydrated lime on initial mixture and in the curing solution.

Hydrated lime is a dry powder, too thin, obtained from quicklime, consisting essentially of calcium hydroxide, magnesium hydroxide, or both. It is a very versatile material with wide range of applications [3, 5, 9], such construction, treatment, water retainer, asphalt additives, disinfectant, chemical stabilization of soils, fertilizers, etc. Used in concretes and mortars, could increases the yield of the mixtures reducing cement consumption, increase mechanical strength, increase workability, liquids penetration resistance, elasticity, water retention or better surface finish [1, 3, 5, 9-11]. Besides, The lime provides increased mechanical strength due 3 mechanism [10]: carbonation, pozzolanic reactions, and increased solubility of the siliceous materials of the mixture. At least, another advantage may be found in the literature: carbonation reaction is in accordance with Eqn. (1) [4]. It can be seen that the resulting calcium carbonate product shows a mass superior to 21%. In addition, his volume increased 12% [4]. So, it is expected a reduction of voids [1, 4]. For these reasons, it is expected that the use of hydrated lime, increase the density of the matrix, the mechanical strength and durability. Finally, create a functional and sustainable material

\[
\begin{align*}
\text{Ca(OH)}_2 & + \text{CO}_2 \rightarrow \text{CaCO}_3 + \text{H}_2\text{O} + 1522 \text{kJ} \\
1.0 \text{ kg} & \quad 0.59 \text{ kg} \quad 1.351 \text{ kg} \quad 0.243 \text{ kg} \\
0.446 \text{ dm}^3 & \quad 0.33 \text{ dm}^3 \quad 0.498 \text{ dm}^3 \quad 0.243 \text{ dm}^3 \\
P=2440 \text{ kg/m}^3 & \quad P=2710 \text{ kg/m}^3 \quad P=1000 \text{ kg/m}^3 
\end{align*}
\]

2 EXPERIMENTAL PROGRAM

As a starting point for this research, it was considered the equivalent mortar composition of a self-compacting concrete with 500 kg/m$^3$ of cement (C), 731 kg/m$^3$ of aggregate (S) and water-binder ratio (W/B) of 0.25 [12]. Forty five mortar samples were produced by keeping previous that proportion for three distinct groups: cement (C); cement replaced by 50 and 70% of fly ash (FA) by mass of binder (B=C+FA); cement replaced by 50 and 70% fly ash by mass of binder plus hydrated lime (HL) as an aggregate replacement. Besides, three W/B ratio were performed: 0.25, 0.30 and 0.35. Superplasticizer (SP) was used in all mixtures, by 1.70% by mass of the binder. Details of these composition are in Table 1.
Table 1: Tested compositions

<table>
<thead>
<tr>
<th>Designation</th>
<th>W/B</th>
<th>B   [kg/m³]</th>
<th>C   [kg/m³]</th>
<th>FA  [kg/m³]</th>
<th>HL  [kg/m³]</th>
<th>S   [kg/m³]</th>
<th>W   [kg/m³]</th>
<th>SP  [%L]</th>
</tr>
</thead>
<tbody>
<tr>
<td>I 100C</td>
<td>0.25</td>
<td>856</td>
<td>856</td>
<td>0</td>
<td>0</td>
<td>1522</td>
<td>214</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>856</td>
<td>856</td>
<td>0</td>
<td>0</td>
<td>1141</td>
<td>257</td>
<td>1.7</td>
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<tr>
<td></td>
<td>0.35</td>
<td>856</td>
<td>856</td>
<td>0</td>
<td>0</td>
<td>1029</td>
<td>300</td>
<td>1.7</td>
</tr>
<tr>
<td>II 50C+50FA</td>
<td>0.25</td>
<td>856</td>
<td>428</td>
<td>428</td>
<td>0</td>
<td>1151</td>
<td>214</td>
<td>1.7</td>
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<td></td>
<td>0.30</td>
<td>856</td>
<td>428</td>
<td>428</td>
<td>0</td>
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<td>257</td>
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<tr>
<td></td>
<td>0.35</td>
<td>856</td>
<td>428</td>
<td>428</td>
<td>0</td>
<td>928</td>
<td>300</td>
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<td>856</td>
<td>257</td>
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<td>1126</td>
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<tr>
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<td>21.4</td>
<td>1015</td>
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<td>903</td>
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<td>1.7</td>
</tr>
<tr>
<td>IV 50C+50FA+5HL</td>
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<td>856</td>
<td>257</td>
<td>599</td>
<td>42.8</td>
<td>1101</td>
<td>214</td>
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<tr>
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<td>856</td>
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<td>599</td>
<td>42.8</td>
<td>990</td>
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<td>599</td>
<td>42.8</td>
<td>878</td>
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<td>1.7</td>
</tr>
<tr>
<td>V 50C+50FA+10HL</td>
<td>(0.25)</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>85.6</td>
<td>1051</td>
<td>214</td>
<td>1.7</td>
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<tr>
<td></td>
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<td>856</td>
<td>257</td>
<td>599</td>
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<td>940</td>
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</tr>
<tr>
<td>VI 50C+50FA+15HL</td>
<td>(0.25)</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>128</td>
<td>952</td>
<td>214</td>
<td>1.7</td>
</tr>
<tr>
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<td>599</td>
<td>128</td>
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</tr>
<tr>
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<td>0.35</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>128</td>
<td>728</td>
<td>300</td>
<td>1.7</td>
</tr>
<tr>
<td>VII 50C+50FA+20HL</td>
<td>(0.25)</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>257</td>
<td>852</td>
<td>214</td>
<td>1.7</td>
</tr>
<tr>
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<td>257</td>
<td>599</td>
<td>257</td>
<td>740</td>
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<tr>
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<td>257</td>
<td>599</td>
<td>257</td>
<td>628</td>
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<td>1.7</td>
</tr>
<tr>
<td>VIII 50C+50FA+30HL</td>
<td>(0.25)</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>257</td>
<td>852</td>
<td>214</td>
<td>1.7</td>
</tr>
<tr>
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<td>856</td>
<td>257</td>
<td>599</td>
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<td>740</td>
<td>257</td>
<td>1.7</td>
</tr>
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<td>856</td>
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<td>599</td>
<td>257</td>
<td>628</td>
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</tr>
<tr>
<td>IX 30C+70FA</td>
<td>0.25</td>
<td>856</td>
<td>257</td>
<td>599</td>
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<td>1111</td>
<td>214</td>
<td>1.7</td>
</tr>
<tr>
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<td>257</td>
<td>599</td>
<td>0</td>
<td>999</td>
<td>257</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
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<td>856</td>
<td>257</td>
<td>599</td>
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<td>887</td>
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<td>1.7</td>
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<td>X 30C+70FA+15HL</td>
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<td>856</td>
<td>257</td>
<td>599</td>
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<td>961</td>
<td>214</td>
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<tr>
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<td>856</td>
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<td>599</td>
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<td>257</td>
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<td>738</td>
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<td>XI 30C+70FA+30HL</td>
<td>(0.25)</td>
<td>856</td>
<td>257</td>
<td>599</td>
<td>257</td>
<td>811</td>
<td>214</td>
<td>1.7</td>
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<td>599</td>
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<td>700</td>
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<td>599</td>
<td>257</td>
<td>588</td>
<td>300</td>
<td>1.7</td>
</tr>
</tbody>
</table>

(W/B) was not possible to produce, due to its low workability.

After mixing and prior to casting, was evaluated the workability by flow table test [13]. They were produced four samples with 40x40x160 mm³ for each mixture. 24 hours after mixing, the samples were demoulded [14], and subjected to two kind of curing until 38 days of age: two samples in wet curing immersed in water (H); two samples in water saturated with calcium hydroxide curing (CH). After, these samples were subjected at 40 °C for 2 days in an oven. Then, all the samples were prepared for the carbonation test, and four opposing surfaces were sealed with paraffin and stored in a carbonation chamber with 4 ± 0.5% CO₂, 55 ± 5% RH and 20 ± 2 °C [15]. After 100 days of testing, they were measured the carbonation depth in two samples, one for each type of curing: a cross-sections were taken, to measure the carbonation depth, using thymolphthalein acid-base indicator. The faces broken were divided...
into equidistant parts, so, the carbonation depth of each mixture, was the arithmetic average of 10 measured points.

**Table 2: materials characteristics**

<table>
<thead>
<tr>
<th>Chemical properties [%]</th>
<th>C 42.5 R</th>
<th>FA Pego</th>
<th>HL Calcidrata</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>18.27</td>
<td>49.12</td>
<td>56.83</td>
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<tr>
<td>Al₂O₃</td>
<td>4.75</td>
<td>27.30</td>
<td>28.59</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.23</td>
<td>8.19</td>
<td>6.86</td>
</tr>
<tr>
<td>CaO</td>
<td>63.94</td>
<td>2.36</td>
<td>1.80</td>
</tr>
<tr>
<td>CaO free</td>
<td>1.39</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MgO</td>
<td>1.31</td>
<td>1.42</td>
<td></td>
</tr>
<tr>
<td>SO₃</td>
<td>3.05</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td>K₂O</td>
<td>0.57</td>
<td>3.34</td>
<td>1.97</td>
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<tr>
<td>Na₂O</td>
<td>0.99</td>
<td>1.78</td>
<td></td>
</tr>
<tr>
<td>TiO₂</td>
<td></td>
<td>2.32</td>
<td></td>
</tr>
<tr>
<td>Cl⁻</td>
<td>0.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td></td>
<td>15.4 vitr.*</td>
</tr>
<tr>
<td>Carbon</td>
<td></td>
<td></td>
<td>3.65</td>
</tr>
</tbody>
</table>

**Physical properties**

| D10 (laser an.) [%] | 1.33 | 1.53 | 1.36 |
| D50 (laser an.) [%] | 9.30 | 8.55 | 3.80 |
| D90 (laser an.) [%] | 24.16| 20.19| 7.73 |
| Sp. gravity [kg/m³] | 3100 | 2420 | 2240 |
| Blaine [cm²/g]      | 4315 |      |      |

**Mineralogical properties [%]**

| Quartz | 18.4 |
| Mullite | 7.2 |
| Hematite | 3.4 |
| Total crystalline phase * | 29.0 |
| Total glassy phase | 71.0 |
| Portlandite | 75.8 |
| Calcite | 22.6 |

* vitreous phase by Rietveld method

The materials used were: cement type I 42.5R from the Outão production center of Secil; fly ash from Portuguese Pego thermoelectric power plant, hydrated lime type class 1, grade A, type 1, from Calcidrata [16]; superplasticizer based on modified polycarboxylic, reference BASF Glenium Sky 617; river rolled sand 0/4 mm. The particles of materials were analysed by laser analysis which, whose results are reflected in Table 1 (physical properties). It was
observed that the cement and fly ash had a similar particle size and hydrated lime was the thinner material. Fly ash were analysed with X-ray diffraction (XRD) and X-ray fluorescence spectroscopy (XRF). Besides, the identification and quantification of reactive phase was carried out by Rietveld method. Simultaneous thermal analysis were also carried out for the hydrated lime, to confirm the lime purity. A scanning electron microscope Hitachi SU1510 (SEM) was used for evaluation fly ash and hydrated lime. It was found that: fly ash were cenospheres particles, some trapped within in carbon clusters. The hydrated lime consists mainly small particles, slightly clustered, without visible impurities (Figure 1). These trials were analysed according a previous study [17] and are reflected in the Table 2.

3 RESULTS

3.1 Workability

The flow test results varied between 134 to 345 mm, although some blends presented no valid results: segregation or no workability (Figure 2). It was found that the introduction of fly ash increases the workability, as it increases the volume of incorporation (II and IX). With 70% FA, this increase was very high, which led to the segregation of the mixture com w/b=0.35 (IX-c). This is consistent with as expected, since the fly ash are finer than the cement, spherical and without gaps. Otherwise, the introduction of hydrated lime, continually reduced the workability and prevented the execution of some mixtures (VI-a, VII-a-VIII-a, XI-a-b).

3.2 Carbonation depth

The 100 days carbonation depth, for water curing samples is shown in Figure 3.
Carbonation depth was very high when the fly ash was used and it is further with the incorporation of more volume of this addition (II and IX). This is because the fly ash consume the calcium hydroxide released during the cement. It was observed that the hydrated lime may be able correct this problem, mainly with lower w/b ratio. In fact, it was found that the best results were between 2.5 to 10% hydrated lime additions. First, it was noted that increasing alkalinity is not enough by itself to reduce carbonation depth. On the other hand, addition further hydrated lime, may not find enough fly ash vitreous phase to react with calcium hydroxide. The worst results with higher w/b ratio may be due to the increased porosity of the matrix or leaching.

To analyse the difference of the curing type, the relative carbonation depth ratio was calculated according Eqn. (2).

\[
\text{ratio} = \frac{\text{calcium hydroxide depth} - \text{water depth}}{\text{water depth}}
\]

The Figure 4 shows the result of this ratio. It was noted that in general, the calcium hydroxide curing improve carbonation resistance. Most of the samples, had lower depths of carbonation. However, it was also observes that in mixtures with just fly ash, the results were modest (II-a,b and IX-a,b) or worse (II-c). This could mean that the calcium hydroxide curing does not offer great advantages for this type of mixtures. Moreover, the best results for mixing with hydrated lime, can be justified by the reduction in the alkali gradient between the samples and the curing solution, avoiding leaching compounds.
3.3 Compressive strength

The 100 days carbonation compressive strength is shown in Figure 5. As expected, the binary mixtures with cement and fly ash, had less strength (II and IX). This is mainly due to the lower amount of available cement. The addition of hydrated lime had better results than the binary mixtures, mainly from 2.5 to 10% and low w/b ratio (III, IV and V). In fact, a consistent result with the carbonation depth. This may indicate that the best results in both resistance and carbonation was mainly due to an improvement in the matrix porosity. So, the alkalinity provided by lime, may not have had such a significant role.

It also analysed the influence of calcium hydroxide curing according Eqn. (2). Through the Figure 6, it was suggested that the results were generally worse for calcium hydroxide curing samples. This result was different from the carbonation test. One hypothesis is that, the cure into calcium hydroxide may slow the rate of binder hydration. Other explanation, it occurred an increased porosity (reduces strength) at the same time as the alkalinity increases (decreases carbonation depth).

4 CONCLUSIONS

The results suggest that the use of high volumes of ash greatly reduces the strength and carbonation durability. The use of hydrated lime within the concrete composition may be a tool to mitigate these problems. The results suggest that for better results, the hydrated lime should be added until 10% binder. Also, such mixtures must be designed for low w/b ratio concretes or mortars. The curing results showed that the use of hydrated lime in the solution of curing, can increase carbonation durability, but the strength further worsens.

Additional studies will be needed to analyse the concrete microstructure changes, study the reaction products and his different characteristics.
5 ACKNOWLEDGEMENTS

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REFERENCES


CARBONATION-RESISTANT EVALUATION OF THE FLY-ASH CONCRETE IN CONSIDERATION OF THE POZZOLANIC REACTION

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Key words: Carbonation; Fly ash; Pozzolanic reaction; Accelerated test

Abstract. There are opinions that concretes which used fly-ash for as mixture materials are superior in durability compared with port-land cement plain concrete by characteristics such as the formation of elaborate structure and long-term strength development by the pozzolanic reaction. In contrast, when executes durable evaluations of fly-ash concretes by accelerated tests, there is particularly difficult to evaluate potential ability in carbonation resistance appropriately, because it is exposed to extremely severe from true environment before the progress of the pozzolanic reaction.

We thought that it was necessary to activate the pozzolanic reaction before accelerated carbonation tests so that the formation of elaborate structure by pozzolanic reaction if targeted the tests. Firstly, sealed the specimen side of test pieces, and took treatment of them in environment temperature 40 degrees Celsius. Thereafter, examined accelerated carbonation tests, compressive strength tests, and quantitative analysis of the Calcium hydroxide by the differential thermal analysis method.

As a result, it was revealed that the progress of the pozzolanic reaction depends on the temperature from compressive strength tests and quantitative analysis of the Calcium hydroxide, and incorporated the formation of elaborate structure by the pozzolanic reaction in the carbonation resistant, it became clear that approximately 20% evaluation level improved on the average. From this, it was not only expansion of the utilization of the industrial by-product to use fly-ash concrete, and the possibility that an aspect the improvement durability of the concrete could contribute to sustainability was suggested.

1 INTRODUCTION

Carbonation of concrete is a chemical reaction between calcium hydroxide and CO₂ in atmosphere. The decrease pH by carbonation does not give harm to concrete itself, in spite that causes a fatal defect in reinforced concrete structure such as crack and exfoliation of concrete cover caused by corrosion of reinforcing steel. From this perspective, it is said that
the number of years to reach reinforcing steel of carbonation depth determines the service life of a concrete structure.

Meanwhile, it is known that concretes which used fly-ash (FA) for as mixture materials have characteristics such as the formation of elaborate structure and long-term strength development by the pozzolanic reaction. However, when executes durable evaluations of FA concretes by accelerated tests, there is particularly difficult to evaluate potential ability in carbonation resistance appropriately, because it is exposed to extremely severe from true environment before the progress of the pozzolanic reaction. Therefore, to activate the pozzolanic reaction before accelerated carbonation tests so that the formation of elaborate structure

2 EXPERIMENTAL PROGRAM

2.1 Materials

Properties of cementitious materials are shown in Table 1 and Table 2. Table 1 shows the properties of ordinary Portland cement, and Table 2 shows that of FA. Ordinary Portland cement is complied with the requirement of JIS R 5210, and FA is complied with that of JIS A 6201. Class 2 FA is generally used as cement replacement materials in Japan.

Properties of aggregates are shown in Table 3. Fine aggregate was made by crush sandstone. Coarse aggregate of crushed hard sandstone with a maximum size of 20 mm was used.

2.2 Concrete mixture composition

The mix proportion of concretes for this experiments are shown in table 4. For each mixture, unit water content was 180kg/m^3. Replacement methods were 2 types. One of the types was cement replacement by mass, the other was fine aggregate replacement by volume.

<table>
<thead>
<tr>
<th>Table 1 : Properties of cements</th>
<th>Table 2 : Properties of FA</th>
<th>Table 3 : Properties of aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Item</td>
<td>JIS R5210 standard value</td>
<td>test value</td>
</tr>
<tr>
<td>density (g/cm^3)</td>
<td></td>
<td>3.16</td>
</tr>
<tr>
<td>Blaine fineness (cm³/g)</td>
<td></td>
<td>2500 or higher</td>
</tr>
<tr>
<td>compressive strength (N/m²)</td>
<td></td>
<td>7-day: 125 or higher</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7-day: 22.5 or higher</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28-day: 42.5 or higher</td>
</tr>
<tr>
<td>chemical composition (%)</td>
<td></td>
<td>5.0 or lower</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3CaO · SiO₂</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2CaO · SiO₂</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3CaO · Al₂O₃</td>
</tr>
<tr>
<td>4CaO · Al₂O₃ · Fe₂O₅</td>
<td></td>
<td>5.0 or lower</td>
</tr>
<tr>
<td>MgO</td>
<td></td>
<td>3.5 or lower</td>
</tr>
<tr>
<td>Total alkalinity</td>
<td></td>
<td>0.75 or lower</td>
</tr>
<tr>
<td>chloride ion</td>
<td></td>
<td>0.035 or lower</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>a kind of aggregate</th>
<th>absolute dry density (g/cm³)</th>
<th>surface dry density (g/cm³)</th>
<th>water absorption (%)</th>
<th>absolute volume (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>fine aggregate</td>
<td>crushed sandstone</td>
<td>2.55</td>
<td>2.62</td>
<td>2.71</td>
</tr>
<tr>
<td>coarse aggregate</td>
<td>crushed hard sandstone</td>
<td>2.68</td>
<td>2.70</td>
<td>0.58</td>
</tr>
</tbody>
</table>
2.2 Curing method

In this work, two types of curing methods were applied in reference in [1-3]. Figure 1 presents the details. On the occasion of curing, sealed the test specimen side with aluminum tape, assuming the use of FA concrete as a wall or a slab, and to suppose that there was not water emission from the test specimen side. The outline is given in Figure 2.

2.3 Measurement of carbonation

The carbonation depths were measured by split of specimens. The depth of carbonation were determined by spraying on split surface with the phenolphthalein solution, and average uncolored portions. The state is given in Figure 3.

Table 4 : Mix proportion of concretes

<table>
<thead>
<tr>
<th>classification</th>
<th>symbol</th>
<th>W/C (%)</th>
<th>W/B (%)</th>
<th>FA replacement ratio (%)</th>
<th>fine aggregate ratio (%)</th>
<th>unit water content</th>
<th>unit volume (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>C by mass</td>
<td>S by volume</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>plain N60</td>
<td>N60-C10</td>
<td>67</td>
<td>60</td>
<td>10</td>
<td>50.7</td>
<td>270</td>
<td>885</td>
</tr>
<tr>
<td>cement and sand</td>
<td></td>
<td>67</td>
<td>53</td>
<td>10</td>
<td>50.4</td>
<td>240</td>
<td>875</td>
</tr>
<tr>
<td>replacement N60-C10S5</td>
<td>N60-C20S5</td>
<td>75</td>
<td>60</td>
<td>20</td>
<td>5</td>
<td>270</td>
<td>841</td>
</tr>
<tr>
<td>symbol</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>plain N50</td>
<td>N50-C10</td>
<td>56</td>
<td>50</td>
<td>10</td>
<td>49.2</td>
<td>324</td>
<td>833</td>
</tr>
<tr>
<td>cement and sand</td>
<td></td>
<td>56</td>
<td>44</td>
<td>10</td>
<td>48.8</td>
<td>288</td>
<td>820</td>
</tr>
<tr>
<td>replacement N50-C10S6</td>
<td>N50-C20S6</td>
<td>63</td>
<td>44</td>
<td>20</td>
<td>47.6</td>
<td>324</td>
<td>782</td>
</tr>
</tbody>
</table>

Fig. 1 : Curing method

Fig. 2 : Outline of curing

Fig. 3 : State of Measuring carbonation
3 Results and Discussion

3.1 Compressive strength

Fig. 4 and Fig. 5 present the compressive strength of each mix proportion cured in air 20 degrees Celsius by time course. Development of compressive strength depends on a water cement ratio and the water-binder ratio. In addition, these figures shows that compressive strength become lower according to increase of FA replacement rate for OPC, without depending at time of demold. FA concrete decreases strength with cement replacement, but can control strength drop by combination of cement replacement and sand replacement. In the case of cement replacement ratio less than 10% with sand replacement, strength development characteristics at the same level as OPC concrete.

Fig. 4: compressive strength by time course (W/C=60%)

Fig. 5: compressive strength by time course (W/C=50%)
Fig. 6 and Fig. 7 present the compressive strength of each mix proportion cured in air 40 degrees Celsius by time course. Development of compressive strength depends on a water cement ratio and the water-binder ratio in higher temperature environment. These figures show that strength development in 40 degrees Celsius environment are higher than that of in 20 degrees Celsius environment. Furthermore, strength development of FA concrete is higher than that of OPC concrete about the FA concrete from the early age. Therefore, it is guessed in particular by curing at higher temperature that reactivity of FA improved. Comparing cement replacement group and cement and sand replacement group, by cement replacement mixture, with much quantity of FA, by cement-sand replacement mixture, with little quantity of FA develop higher compressive strength. This suggests that the influence by the mixed method of FA under higher temperature environment excels than that by quantity of cement.
3.2 Amounts of Ca(OH)$_2$

Amounts of Ca(OH)$_2$ was calculated by differential thermal analysis, based on mass decrease of samples in the temperature zones that thermolysis happens of Ca(OH)$_2$ or CaCO$_3$. The quantity of CaCO$_3$ converted it into quantity of Ca(OH)$_2$ which caused the reaction by mol conversion.

Fig. 8 presents the quantity of Ca(OH)$_2$ cured in 20 degrees Celsius environment in materials age 56 days. The correlation was low, but the FA concrete was a tendency than OPC concrete with a little quantity of Ca(OH)$_2$ macroscopically.

Fig. 9 presents the quantity of Ca(OH)$_2$ cured in 40 degrees Celsius environment in materials age 56 days. As for the Ca(OH)$_2$ quantity, in case of quantity of cement has the equality between cement replacement and cement-sand replacement, the latter contain fewer Ca(OH)$_2$ quantity. This proves that the pozzolanic reaction was activated and the reaction consumed Ca(OH)$_2$ in 40 degrees Celsius environment.
3.3 Carbonation coefficients

Fig.10 and Fig.11 show carbonation coefficients of every mixture for each curing conditions. Carbonation coefficients was calculated by equation the square-root-t-law,

\[ c = a \sqrt{t} \]  

where \( c \) is measured carbonation depth (mm), \( t \) is the period of exposure in accelerated environment (week), \( a \) is the carbonation coefficient.
Fig. 10 presents carbonation coefficients curing in 20 degrees Celsius. With increasing the water-cement ratio and the FA replacement ratio, carbonation coefficients is rising. By contrast, carbonation coefficients curing in 40 degrees Celsius in Fig. 11, the defined difference by the water-cement ratio and the FA replacement ratio was not confirmed. This causes include by curing in high temperature, micro structure densification in particular in FA concrete.

3.4 Carbonation coefficients – Compressive strength

Fig. 12 and Fig. 13 show the relations between the carbonation coefficients and the inverse of compressive strength in materials age 56 days (the point of accelerated carbonation test start). Fig. 12 presents the relations curing in 20 degrees Celsius, and Fig. 13 presents the relations curing in 40 degrees Celsius.

These figures show straight line relations between the carbonation coefficients and the inverse of compressive strength and there is a highly correlation, which is similar to the trend of concrete observed by various researchers [4-6].

On the other hand, about sand replacement mixtures, it was located rather high from the approximate line of all mixtures, and the carbonation-resistant improvement with the strength increase by sand replacement was not confirmed in Fig. 12.

Similarly in Fig. 13, the angle of approximately line of sand replacement is incline compared with that of all mixtures. Therefore, curing in 40 degrees Celsius activated the pozzolanic reaction, and Ca(OH)$_2$ consumption of the pozzolanic reaction exceeds the refinement of pore structures due to the strength development.
3.5  carbonation coefficient ratio

At this point, to define the thing which divided a carbonation coefficient of FA concrete by that of OPC as a carbonation coefficient ratio. The list of carbonation coefficient ratio in each curing temperature in table.5.

Curing in 20 degrees Celsius before accelerated carbonation tests, FA concrete are inferior to OPC concrete in the evaluation of carbonation resistant. On the other hand, curing in 40 degrees Celsius, FA concrete are superior to OPC concrete.

These are due to activation of the pozzolanic reaction before accelerated carbonation tests. Namely, to activate of the pozzolanic reaction before accelerated carbonation tests improves the evaluation of carbonation resistant of FA concrete, and the degree of improvement evaluation level was approximately 20% on the average.

Table 5 : Carbonation coefficient ratio

| W/C | replacement | carbonation coefficient | carbonation coefficient ratio | average | W/C | replacement | carbonation coefficient | carbonation coefficient ratio | average
|-----|-------------|------------------------|-------------------------------|---------|-----|-------------|------------------------|-------------------------------|---------
| 60  | none        | 7.82                   | 100%                          |         | 60  | none        | 9.06                   | 100%                          |         |
|     | c 10%       | 8.5                    | 109%                          | 117%    |     | c 10%       | 8.63                   | 95%                           |         |
|     | c 20%       | 9.45                   | 121%                          |         |     | c 20%       | 8.38                   | 92%                           |         |
|     | c 10%, s 5% | 8.94                   | 114%                          |         |     | c 10%, s 5% | 8.33                   | 92%                           |         |
|     | c 20%, s 5% | 9.78                   | 125%                          |         |     | c 20%, s 5% | 8.7                    | 96%                           |         |
| 50  | none        | 6.33                   | 100%                          |         | 50  | none        | 7.28                   | 100%                          |         |
|     | c 10%       | 7                      | 111%                          | 116%    |     | c 10%       | 7.4                    | 102%                          |         |
|     | c 20%       | 7.54                   | 119%                          |         |     | c 20%       | 7.06                   | 97%                           |         |
|     | c 10%, s 6% | 6.92                   | 106%                          |         |     | c 10%, s 6% | 7.13                   | 98%                           |         |
|     | c 20%, s 6% | 8.11                   | 123%                          |         |     | c 20%, s 6% | 7.21                   | 99%                           |         |

4  CONCLUSIONS

From this work, the following conclusions are made;

- The pozzolanic reaction has temperature dependence, and the reaction was activated in 40 degrees Celsius.
- As a result that the pozzolanic reaction was activated, pozzolanic reaction products contributed to compressive strength, and the strength development that was larger than OPC concrete.
- Carbonation coefficients and inverse of compressive strength show straight line relations and those correlation are high.
- Relatively high volume of FA mixtures, the carbonation-resistant improvement with the strength increase by sand replacement was not confirmed.
- In the sight of carbonation coefficient ratio, the improvement of the carbonation – resistant evaluation was confirmed by raising curing temperature.
- The degree of improvement evaluation level was approximately 20% on the average.
REFERENCES


CHANGES IN CHLORIDE PENETRATION PROPERTIES CAUSED BY REACTION BETWEEN SULFATE IONS AND CEMENT HYDRATES

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Key words: Chloride binding, Sulfate ion, Friedel’s salt, Kuzel’s salt

Abstract. Many studies have focused only on the penetration of chloride ions. However, seawater contains various other ions such as sulfate and magnesium. Coexisting ions in seawater cause an electric effect and deterioration of hardened cement paste, which means that sulfate and magnesium ions react with CH or C-S-H, and generate gypsum dihydrate and brucite. Based on our previous research, chloride penetration was suppressed in all solutions compared with that of the NaCl solution. This is believed to be due to the chemical reaction between sulfate ions and cement hydrates. It is well known that when chloride ions from environmental solutions penetrate into concrete, some of them are captured by the cement hydrates; this is called chloride binding. It is considered that the chloride binding affects the chloride penetration, but there have been few studies on the influence of coexisting ions on chloride binding.

The present study examined the effect of sulfate ions on chloride ion penetration properties by conducting an immersion test. A reaction test was also conducted to understand the influence of sulfate ions on chloride binding. From the immersion test results, chloride penetration into the mortar was suppressed until the immersion period of 476 days, but was accelerated during the immersion period of 476 to 792 days. It is considered that the suppressive effect was due to the influence of the pores filled with gypsum dihydrate produced by the reaction between cement hydrates and sulfate ions, and the accelerative effect was due to the increment of free chloride ions in the pore solution, which can be presumed by the reduction of bound chloride amount caused by sulfate ions.
1 INTRODUCTION

Japan’s extensive public infrastructure that was built during the high economic growth period will be 50 years old by 2020. Public infrastructure has always been actively maintained and operated, with additional maintenance measures implemented after the ceiling of the Sasago Tunnel collapsed in December 2012. A major part of this public infrastructure is concrete structures built using cement composites.

As an island nation surrounded by the sea, many of Japan’s concrete structures are exposed to the risk of salt damage caused by the penetration of chloride ions from seawater. It is necessary to consider the durability of these concrete structures against such damage. This requires a clear understanding of the penetration of chloride ions into concrete.

Many studies have focused only on the penetration of chloride ions. However, seawater contains various other ions such as sulfate and magnesium, so it is necessary to consider the effects of these ions on the penetration of chloride ions.

Coexisting ions in seawater cause an electric effect and deterioration of hardened cement paste, which means that sulfate and magnesium ions react with CH or C-S-H, and generate gypsum dihydrate and brucite [1]. Moreover, in natural diffusion, since chloride ions move together with positive ions, if the diffusion velocity of a cation is large, the diffusion velocity of chloride ions will also become large [2].

To examine the effect of coexisting ions on chloride ion penetration, an immersion test was conducted using solutions prepared by adding Na⁺, Mg⁺, SO₄²⁻ and Cl⁻ [3]. The test results showed that chloride penetration was suppressed in all solutions compared with that of the NaCl solution. This is believed to be due to the chemical reaction between sulfate ions and cement hydrates. It is well known that when chloride ions from environmental solutions penetrate into concrete, some of them are captured by the cement hydrates; this is called chloride binding. It is considered that the chloride binding affects the chloride penetration, but there have been few studies on the influence of coexisting ions on chloride binding. Zibara [4] reported that the decrease in binding capacity as a result of an increase in sulfate concentration might be due to the competition between sulfate and chloride ions for chemical reaction with unhydrated C₃A and calcium aluminate hydrates. However, its influence is not yet clearly understood.

The present study examined the effect of sulfate ions on chloride ion penetration properties by conducting an immersion test. A reaction test was also conducted to understand the influence of sulfate ions on chloride binding.

2 IMMERSION TEST

2.1 Experimental outline of immersion test

The experiment was conducted using ordinary Portland cement (“C”, density 3.15 g/cm³) and river sand from Fujikawa, Yamanashi (“S”, density 2.60 g/cm³). The mix design is shown in Table 1. Specimens were φ10×12.5 cm columns (see Figure 1).

Demolded specimens were cured in water for 28 days and then coated with epoxy resin except for the surface to be exposed to the immersion solution. The specimens were then immersed in NaCl + Na₂SO₄ or NaCl + MgSO₄ solution for 91, 476 or 792 days, and in NaCl
solution for 91 days. Some studies have examined the influence of seawater ions on hardened cement and the diffusion of chloride ions. Cations such as Na⁺ and Mg²⁺ affect the diffusion coefficient of chloride ions [5]. The depth to which Mg permeated was defined as the “deterioration depth,” and it has been suggested that this depth can be used as an index of the deterioration of concrete [6]. Therefore, in order to grasp the influence of seawater ions on the chloride permeability of concrete, Na⁺, Mg²⁺, Cl⁻ and SO₄²⁻ were investigated. The type of solution and concentration of ions are shown in Table 2.

The amount of total chloride ions was measured by potentiometric titration according to JIS A1 1154. Samples were collected from the mortar at different distances from the exposed surface: 0–7 mm, 7–14 mm, 14–21 mm, 21–28 mm and 28–35 mm. An image of a cut specimen is shown in Figure 1.

Table 2: Each ion concentration in the mixed solution

<table>
<thead>
<tr>
<th>Solution</th>
<th>Cl⁻</th>
<th>Na⁺</th>
<th>Mg²⁺</th>
<th>SO₄²⁻</th>
</tr>
</thead>
<tbody>
<tr>
<td>NaCl</td>
<td>1.90</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>NaCl + Na₂SO₄</td>
<td>1.90</td>
<td>2.09</td>
<td>-</td>
<td>0.10</td>
</tr>
<tr>
<td>NaCl + MgSO₄</td>
<td>1.90</td>
<td>0.10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2 Results of immersion test

The results of measuring total chloride ion concentration after immersion for 91 days in each solution are shown in Figure 2. The chloride ion concentration is higher in the order of NaCl, NaCl + Na₂SO₄, NaCl + MgSO₄.

The measurement results after the immersion test (476 and 792 days) for NaCl + Na₂SO₄ and NaCl + MgSO₄ and the result for NaCl, which was calculated by using the apparent diffusion coefficient (Dₐ) and the surface chloride ion concentration (C₀) obtained from a theoretical solution of the diffusion equation based on Fick’s law under the condition of constant surface concentration applied to the measurement data of 91 days, are shown in Figure 3. Chloride penetration of mortar in the case of NaCl + Na₂SO₄ is apparently slower than that for NaCl although in the case of NaCl + MgSO₄, it is almost the same as for NaCl. In order to better understand the ease of chloride penetration into mortar, the number of years (T_corr) up to the limit concentration (Cₚₑ₅) that causes steel corrosion was estimated. Cₚₑ₅ in OPC50 of this experiment was 1.9 kg/m³, which complies with the Standard Specification for Concrete Structures of 2012 “Design” in Japan. The apparent diffusion coefficient (Dₐ) and the surface chloride ion concentration (C₀), which was obtained from the regression analysis of each immersion test result, are shown in Table 3. The value of T_corr taken to reach Cₚₑ₅ in 5 cm cover was calculated by using a theoretical solution of the diffusion equation based on...
Fick’s law, and the results are shown in Figure 4. The $T_{corr}$ ratio in the figure means the $T_{corr}$ of each result normalized by the result for NaCl for the immersion period of 91 days. From these calculation results, it can be seen that chloride penetration into the mortar was suppressed until the immersion period of 476 days. However, chloride penetration was accelerated during the immersion period of 476 to 792 days, because the $T_{corr}$ taken to reach $C_{lim}$ in 5 cm cover of both NaCl + Na$_2$SO$_4$ and NaCl + MgSO$_4$ was almost the same as that for NaCl.

![Figure 2: Chloride ion concentration distribution (immersion age = 91 days)](image)

![Figure 3: Chloride ion concentration distribution (Left: 476 days, Right: 792 days)](image)

<table>
<thead>
<tr>
<th></th>
<th>$C_0$ (kg/m$^3$)</th>
<th>$D_s$ (cm$^2$/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NaCl</td>
<td>19.2</td>
<td>1.48</td>
</tr>
<tr>
<td>NaCl + Na$_2$SO$_4$</td>
<td>9.0</td>
<td>2.30</td>
</tr>
<tr>
<td>NaCl + MgSO$_4$</td>
<td>7.3</td>
<td>1.65</td>
</tr>
</tbody>
</table>

Table 3: Each ion concentration in the mixed solution

<table>
<thead>
<tr>
<th></th>
<th>91 days</th>
<th>476 days</th>
<th>792 days</th>
<th>91 days</th>
<th>476 days</th>
<th>792 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>NaCl</td>
<td>19.2</td>
<td>-</td>
<td>-</td>
<td>1.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>NaCl + Na$_2$SO$_4$</td>
<td>9.0</td>
<td>17.6</td>
<td>11.6</td>
<td>2.30</td>
<td>0.46</td>
<td>2.10</td>
</tr>
<tr>
<td>NaCl + MgSO$_4$</td>
<td>7.3</td>
<td>30.2</td>
<td>20.0</td>
<td>1.65</td>
<td>0.60</td>
<td>1.72</td>
</tr>
</tbody>
</table>
3 REACTION TEST

From the immersion test results, chloride penetration into the mortar is suppressed during the early immersion period, a phenomenon that was also observed in our previous research results [3]. This time, a reaction test was conducted to understand the influence of sulfate ions on the chloride binding.

3.1 Chloride binding properties

Chemical binding generally results from the reaction between chlorides and C₃A to form Friedel’s salt or the reaction with C₄AF to form an analogue of Friedel’s salt [7, 8]. The formation of Friedel’s salt may be due to a direct reaction between C₃A and CaCl₂, according to the following reaction schemes when NaCl is admixed [4, 9].

\[
\begin{align*}
\text{Ca(OH)}_2 + 2\text{NaCl} & \leftrightarrow \text{CaCl}_2 + 2\text{Na}^+ + 2\text{OH}^- \\
\text{C}_3\text{A} + \text{CaCl}_2 + 10\text{H}_2\text{O} & \rightarrow \text{C}_3\text{A} \cdot \text{CaCl}_2 \cdot 10\text{H}_2\text{O}
\end{align*}
\]

Yonezawa [10] suggested the ion exchange mechanism to explain the formation of Friedel’s salt and its formation as expressed in the following reaction.

\[
\text{R-OH}^- + \text{Na}^+ + \text{Cl}^- \rightarrow \text{R-Cl}^- + \text{Na}^+ + \text{OH}^- 
\]

where, \( \text{R} \): Ion exchanger

Suryavanshi [11] suggested two mechanisms for the formation of Friedel’s salt and its analogue: adsorption and anion exchange, the same as Yonezawa’s suggestion. In the adsorption mechanism, Friedel’s salt is formed when bulk chloride ions present in the pore solution are adsorbed into the interlayers of the principal layers, \([\text{Ca}_2\text{Al(OH)}_6\cdot 2\text{H}_2\text{O}]^+\), of the AFm structure to balance the charge. An amount of Na⁺ ions equivalent to the absorbed chloride ions (in moles) are removed from the pore solution, and bind with the calcium silicate hydrate (C-S-H) gel lattice to balance the charge arising due to the replacement of Si⁴⁺ ions by Al³⁺ and Fe³⁺ ions.
When chloride ions penetrate into concrete from external environments, as the chloride ion concentration increases, AFm transforms into Kuzel’s salt first, and then into Friedel’s salt at higher concentrations [4]. The AFm phase binds chloride ions as shown in the following equations.

\[
\begin{align*}
C_3A\cdot CaSO_4\cdot 12H_2O + NaCl & \rightarrow C_3A\cdot 0.5CaSO_4\cdot 0.5CaCl_2\cdot 10H_2O + Na^+ + 0.5SO_4^{2-} + 2H_2O \\
C_3A\cdot CaSO_4\cdot 12H_2O + 2NaCl & \rightarrow C_3A\cdot CaCl_2\cdot 10H_2O + 2Na^+ + SO_4^{2-} + 2H_2O
\end{align*}
\] (4)

3.2 Experimental outline of reaction test

Cement paste was used as the test specimen and W/C was 50%. Demolded specimens were cured in water for 91 days. A cured specimen was crushed and sieved into 1.70‒2.36 mm. Cement paste (20 g) and solution (200 ml) were placed in a cup, which was then covered and stored at 20°C. In order to understand the influence of sulfate on the chloride binding of cement hydrates, two solutions saturated with Ca(OH)₂ were prepared. One was NaCl (3.09%) solution and the other was NaCl (3.09%) + Na₂SO₄ (0.41%) solution. The concentrations of each ion were determined according to the seawater composites.

After the reaction test, the total chloride ions (C_{tot}) and soluble chloride ions (C_{sol}) contained in the sample were measured by potentiometric titration according to JIS A1 1154. Friedel’s salt and Kuzel’s salt were measured by using XRD. The experimental conditions of the XRD analysis were as follows: measurement range (2\theta) 5‒60°, scan interval 0.02°, voltage 40 kV, and current density 15 mA.

3.3 Result of reaction test

The measurement results for chloride ion concentration are shown in Figure 5 and those for Friedel’s salt and Kuzel’s salt by XRD are shown in Figure 6. Both C_{tot} and C_{sol} in the case of NaCl + Na₂SO₄ solution were smaller than that for NaCl solution. In the case of NaCl solution, the transformation from Kuzel’s salt to Friedel’s salt was observed as described in Section 3.1 and Friedel’s salt gradually increased. On the other hand, in the case of NaCl + Na₂SO₄ solution, Friedel’s salt gradually decreased and some amount of Kuzel’s salt still remained, possibly due to the influence of sulfate ions.

![Figure 5: Chloride ion concentration (Left: total Chloride, Right: soluble Chloride)](attachment:image.png)
4 CONCLUSIONS

From the experimental results, the following facts can be founded.
- Chloride penetration into the mortar was suppressed until the immersion period of 476 days, but was accelerated during the immersion period of 476 to 792 days.
- Both the total and soluble chloride ion concentration in the case of NaCl + Na₂SO₄ solution were smaller than that for the NaCl solution.
- Friedel’s salt gradually decreased and some amount of Kzel’s salt still remained in the case of NaCl + Na₂SO₄ solution.

ACKNOWLEDGMENTS

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REFERENCES


CHANGES IN MICROSTRUCTURE AND PORE STRUCTURE OF LOW-CLINKER CEMENTITIOUS MATERIALS DURING EARLY STAGES OF CARBONATION

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Key words: Carbonation, drying, cement paste, slag, early age, pore structure

Abstract. Carbonation is one of the main causes of reinforced concrete damage. It leads to decrease the pH of the concrete pore solution and to CaCO3 formation from the reaction between CO2 and cement hydrates. The mechanisms of carbonation are complex in low-clinker cementitious materials. At a given age, the hydration degree in low-clinker cementitious materials is reduced compared to CEM I materials which is due to the slower reaction of mineral additives and leads to the underdeveloped of microstructure. Therefore, the coupling between hydration, drying and carbonation at early age needs to be analysed, in order to understand and predict the durability of low-clinker cementitious materials. In this work, the coupled process leads to a faster drying of GGBS (Ground Granulated Blastfurnace Slag) cement paste at early age compared to the CEM I case. However, this trend changes when the GGBS reactions occur that are confirmed by the MIP results. Similar carbonation depths at early age are observed but a lower carbonation degree suggests a lower diffusivity and finer microstructure for GGBS paste.

1. INTRODUCTION

Rising energy cost and environmental considerations have shifted the cement manufacture industry to follow a low clinker cement approach. However the reactions of Supplementary Cementitious Materials (SCMs) are slower than clinker [1]. If they are cured like a CEM I system, the hydration does not occur completely [2] and consequently, the microstructure stays underdeveloped, more porous [3] and susceptible to the ingress of aggressive species, which primary starts with the process of carbonation as the cementitious structures are directly in contact with the atmospheric CO2 after demoulding. Therefore, these kinds of materials require a specific curing condition [4]. In theory, a longer curing time is better for the durability as the microstructure is more developed. Nevertheless, in practice, it is impossible to keep the frameworks until material properties are fully obtained. A curing time of 3 days seems to be a favourable compromise.

In the CEM I cementitious materials, the carbonation reaction is sum up by the following equation:

$$\text{CO}_2(g) + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 + \text{H}_2\text{O}$$ (1)

The use of supplementary cementitious materials (SCMs) causes a decrease in the Ca(OH)2 content due to the dilution of clinker and its consumption by the pozzolanic reaction. In the blended cements, as the portlandite content decreases, the carbonation of the C-S-H cannot be
ignored ([5-7]). Consequently, the mechanisms of carbonation become more complex in low-clinker cementitious materials. These mechanisms depend on the hydration and saturation degree.

The importance of the curing time is shown by Aruhan [8], the carbonation depth decreases with the curing time. Drying causes a decrease in the internal relative humidity that leads to slow down or to stop the hydration reaction as shown by Jensen [9]. Furthermore, there is less hydrates phases which may react with the carbonates and the microstructure is coarser. Therefore, the carbonation depth progress is delayed with the curing time.

In this paper, the effects of coupling between carbonation and drying at early age (after 3 days of water curing) on the pore structure, the saturation and the microstructure profiles of GGBS cement pastes are studied and compared to CEM I cement pastes.

2. MATERIALS AND METHODS

2.1. Materials

Two paste systems (CEM I (PCM) and CEM I with replacement by 60% GGBS (Ground Granulated Blastfurnace Slag) (P6S) with W/B ratio of 0.57 were investigated. The replacement in the manufacture of the two systems was by volume. The chemical composition of CEM I 52.5 R and GGBS were determined by X-ray fluorescence (XRF) (Table 1). The clinker phase contents are calculated according to Bogue’s approach and shown in Table 2. The Blaine fineness is 4740 cm²/g for the GGBS and 4900 cm²/g for the CEM I.

<table>
<thead>
<tr>
<th>LOI</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
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<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>K₂O</th>
<th>Na₂O</th>
<th>P₂O₅</th>
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<tr>
<td>CEM I 52.5R (%)</td>
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<td>4.60</td>
<td>2.40</td>
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<td>0.00</td>
<td>63.40</td>
<td>2.00</td>
<td>3.60</td>
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<td>0.13</td>
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<td>GGBS (%)</td>
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<td>36.06</td>
<td>11.22</td>
<td>0.38</td>
<td>0.74</td>
<td>0.22</td>
<td>41.75</td>
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<td>0.52</td>
<td>0.19</td>
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Table 2: Mineralogical composition of CEM I 52.5R

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<th>C₃S</th>
<th>C₃A</th>
<th>C₄AF</th>
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<tbody>
<tr>
<td>CEM I 52.5R (% / cement)</td>
<td>68</td>
<td>10</td>
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</tbody>
</table>

2.2 Method

Samples were cast in PVC moulds (Ø=5.5 cm, h=19 cm) and were unmoulded after 24h. Then, they were kept in water during 48h. After this curing time (t₀=3 days), some samples were sawn. 3 cm of each sample on the Top and the bottom were removed to limit the heterogeneity. Two kinds of samples were produced: cut and uncut. Fig. 1 shows the two kinds of samples. The uncut samples were used to validate results of the cut samples by comparison between the mass loss kinetics of the two kinds of samples. The samples cut in slices allow one to obtain the porosity and the saturation profiles.

After this sawing operation, the bottom and the lateral sides of each specimen were sealed with two layers of aluminium foil sheets to obtain a moisture transfer in one dimension. All samples were kept in a chamber with a relative humidity (RH) of 65% at T=20°C in an environment without CO₂ (use of soda lime to trap CO₂ in the chamber). After t₀+28 days, a
part of samples were put in an environment with T=20°C, RH=65% and [CO2]=3%. Throughout the rest of the article, the only drying condition is named condition D and the coupled drying-carbonation condition is named condition C. Characterisations using different techniques presented thereafter were performed on the samples at t₀=3 days, t₀+7, t₀+28 and t₀+56 days. In addition to these characterisations, weight monitoring was carried out for all samples.

Porosity accessible to water and saturation degree were obtained by hydrostatic weighting and drying at 80°C. This measurement is performed on 3 samples. For TGA, XRD and MIP analyses, samples were freeze dried for 24h at P=0.1 mbar. These measurements are performed on one sample. Thermogravimetric analysis (TGA) was collected using a NETZSCH STA 409E under nitrogen atmosphere for samples with GGBS and under air for the other samples, heating from 25°C to 1250°C at a heating speed of 10°C/min. For the samples with GGBS substitution, a nitrogen atmosphere is used to prevent slag compounds oxidation which leads to an increase of sample mass. This technique was used to quantify, the amount of Ca(OH)₂ and CaCO₃ and the chemically bound water content in the samples [10]. The temperature limit between the free water and the chemically bound water is difficult to estimate. Therefore, the following hypothesis was done: the water lost below 105°C is the free water [10]. The mass losses are measured by the derivative method because it is easy to apply and this method is appeared to be reproducible. The Portlandite content is calculated from Eq. 2 and the CaCO₃ one from Eq. 3.

\[
x_{\text{CH}} = \frac{\Delta m_{T_1 \rightarrow T_2}}{m_{s \_T=1100°C}} \cdot \frac{MW_{\text{CH}}}{MW_w} \cdot 100 \tag{2}
\]

where \(x_{\text{CH}}\) (%) is the Portlandite content per g of cement, \(\Delta m_{T_1 \rightarrow T_2}\) (g) is the mass loss between T₁=465°C and T₂=560°C in the CEM I case and between T₁=430°C and T₂=530°C for the GGBS case, \(m_{s \_T=1100°C}\) (g) is the sample mass at T=1100°C, \(MW_{\text{CH}}\) (g.mol⁻¹) is the Portlandite molecular weight and \(MW_w\) (g.mol⁻¹) is the water molecular weight.

\[
x_{\text{CC}} = \frac{\Delta m_{T=560°C \rightarrow T=1100°C}}{m_{s \_T=1100°C}} \cdot \frac{MW_{\text{CC}}}{MW_{\text{CO}_2}} \cdot 100 \tag{3}
\]
where \( x_{C}^C \) is the calcium carbonate content per g of cement in percent, 
\( \Delta m_{T=560°C \rightarrow T=1100°C} \) mass loss between \( T=560°C \) and \( T=1100°C \), 
\( m_{T=1100°C} \) sample mass at \( T=1100°C \), 
\( MW_{CC} \) calcium carbonate molecular weight and 
\( MW_{CO_2} \) CO\(_2\) molecular weight. The average degree of hydration (DoH) is also evaluated with TGA results [10], [11] by using the equation 4.

\[
DoH = \frac{W_D}{W_{D(DoH=100%)}} \times 100
\]

where \( W_D \) is the chemically bound water which is evaluated by the equation 5 and 
\( W_{D(DoH=100%)} \) is the chemically bound water for a DoH=100%. This value is equal to 24.6% for the CEM I and 23.6% for the CEM I +60% GGBS [12], [13].

\[
W_D = W_{D,CH} + W_{D,C-S-H} + W_{D,Other\_hydrates} + W_{D,CH→C\bar{C}} + W_{D,C-S-H→C\bar{C}}
\]

where \( W_{D,CH} \) is the chemically bound water for the portlandite, \( W_{D,C-S-H} \) is the chemically bound water from the C-S-H, \( W_{D,Other\_hydrates} \) is the chemically bound water from the other hydrates, 
\( W_{D,CH→C\bar{C}} \) is the chemically bound water from the portlandite change in CaCO\(_3\) and 
\( W_{D,C-S-H→C\bar{C}} \) is the chemically bound water from the C-S-H change in CaCO\(_3\). In this study, we do the assumption that the quantity of the chemically bound water the C-S-H change in CaCO\(_3\) is negligible.

X-ray diffraction (XRD) patterns were collected with Philips PW3830 equipped with CoK\(_\alpha\) radiation source between 4-76°20, a step size of 0.02 and a dwell time of 2.0 s were used. This technique was used to determine the presence of crystalline phases (C\(_3\)S, C\(_2\)S, C\(_3\)A, C\(_4\)AF, Portlandite, Calcite and Ettringite).

Finally, pore size distribution was obtained by Mercury Intrusion Porosimetry (MIP) with a commercial porosimeter (Micromeritics' AutoPore IV 9500 Series) using a maximum intrusion pressure of 400 MPa. This technique is used to assess the pore size distribution in a comparable manner and to obtain a qualitative evolution of this pore size distribution. The porosity obtained by MIP is calculated as the mercury intrusion volume.

The carbonation depth is measured by a colorimetric method based on phenolphthalein spraying tests. The obtained depth correspond to a pH value roughly 9. This implies a sharp carbonation front while this one is normally gradual (profile). Indeed, the main disadvantage of this method is that it can differentiate only between fully carbonated sample and other zones which might vary between being completely unaltered and being almost fully carbonated. This is why, the calcite profile measurements were proposed in parallel to determine the gradual front of carbonation.

### 3 RESULTS AND DISCUSSION

#### 3.1 MICROSTRUCTURE

Fig. 2a shows the evolution of portlandite content with time and depth for CEM I cement paste. At \( t_0 \), the portlandite content per g of cement is the same in the entire sample and around 24%. For each depth, the portlandite content increases with the time between \( t_0 \) and 28 days. This is due to the hydration of clinker. Moreover, in the condition D, the Portlandite content is
constant between 28 and 56 days (blue and purple lines). This finding is the same in the
condition D where the Portlandite content is also the same at the depths of 1-1.5 cm and 6.5-
10.0 cm. The Portlandite content in the GGBS cement paste is lower than in the CEM I cement
paste. This is due to the clinker dilution. Nevertheless, at the depth of 0-0.5 cm, the Portlandite
contents decrease due to carbonation which is confirmed by the Fig. 3a&amp;b where the CaCO₃
contents are constant at all the time except for the surface at t₀+56 days with the condition C for
both binders. In the GGBS cement paste, the CaCO₃ contents stays lower (4.0%) than in the
case of CEM I (7.8%). This result is confirmed by carbonation depth measurement at t₀+56
days with phenolphthalein spraying; a carbonation depth of 0.81±0.24 mm is obtained for the
CEM I and 0.55±0.40 mm for the GGBS cement paste. These results need to be confirmed for
longer time scales. For both binders at all the time in the condition D, the CaCO₃ contents near
to the surface are higher than the other depths. This is due to the carbonation of a thin layer at
the surface.

Figure 2: Evolution of Portlandite content with time for PCM (a) and for P6S (b)
D: drying condition – C: coupled drying and carbonation conditions

Figure 3: Evolution of CaCO₃ content with time for PCM (a) and for P6S (b), D: drying condition – C: coupled
drying and carbonation conditions (dashed line = carbonation depth according to phenolphthaleine)
Furthermore, the degree of hydration (DoH) is calculated by the method described by Bhatty and Pane [10], [11]. For the CEM I cement paste (see Fig. 4a), results can be summarized as following:

- At the depth of 0.0-0.5 cm, the DoH increases until t₀+28 days, it is unchanged for t₀+56 days in the condition D. Therefore, the saturation degree threshold, below which the hydration cannot be possible, is deeper than 0.5 cm between t₀+7 days and t₀+28 days. At t₀+56 days in the condition C, the DoH is higher than at t₀+28 days, which is probably due to the water produced during the carbonation reaction (see eq 1).
- At the depth of 1.0-1.5 cm, the DoH increases until t₀+28 days, it is unchanged for t₀+56 days in both conditions. Therefore the saturation degree threshold is deeper than 1.5 cm between t₀+7 days and t₀+28 days.
- At the depth of 6.5-10.0 cm, the DoH increases until t₀+56 days. Therefore at this depth the drying does not occur.

For the slag cement paste (see Fig. 4b), results can be summarized as following:

- At the depth of 0.0-0.5 cm, the trend is the same than the CEM I but the DoH is lower because the slag reaction rate is slower than the clinker one.
- At the depth of 1.0-1.5 cm, the degree of hydration increases until t₀+56 days in the drying condition. Therefore the saturation degree threshold is between 0.5 and 1.0 cm for a time t₀+28 days and t₀+56 days.
- At the depth of 6.5-10.0 cm, the DOH increases until t₀+56 days.

At t₀, the DoH is higher at the surface than in the bulk in both cases. The surface of GGBS cement paste dries faster before t₀+7 days than the CEM I one. After t₀+28 days, drying has an impact at the depth of 1.0-1.5 cm in the CEM I case while it is not the case for the GGBS cement paste. Therefore, after a faster drying at the surface at early age, the drying seems to slow down for the slag cement after t₀+28 days (see Fig 6). This can be due to a finer porosity of GGBS cement paste as see in Fig. 8b.

According to XRD results, the following phases are present in CEM I and GGBS cement pastes: Portlandite, Calcite, C-S-H, Ettringite, monosulfoaluminate, C₂S, C₃S, C₃A, C₄AF. Gehlenite and Wolastonite are only present in GGBS cement pastes. For the CEM I cement paste at a depth of 0.0-0.5 cm (Fig. 5a):

![Figure 4: Evolution of degree of hydration with time for PCM (a) and for P6S (b) [11]
- The calcite peak seems unchanged for condition D except for condition C. It increases compared to that at \( t_0 + 56 \) days in condition D. This result is confirmed by the TGA.
- The C\(_2\)S and C\(_3\)S peak intensities seem reduced between \( t_0 \) and \( t_0 + 28 \) days. Between \( t_0 + 28 \) days and \( t_0 + 56 \) days, the intensities are quite similar.

For the GGBS cement paste (Fig. 5b):
- The calcite peak seems unchanged except for the condition C. It increases compared to that at \( t_0 + 56 \) days in condition D. This result is confirmed by the TGA. Vaterite and Aragonite are not found in the sample in opposition to other studies [14], [15]. Therefore, the carbonation of C-S-H does not occur during the experiment, only the carbonation of Portlandite [14].

The calcite peak is higher in the CEM I paste than in the slag cement paste which is confirmed by TGA. This result can be explained by higher Portlandite and other hydrates contents in the CEM I paste compared to the slag cement paste.

![Figure 5: XRD pattern at the depth of 0.0-0.5 cm for PCM (a) and for P6S (b) at several times](image)

3.2 Mass loss kinetics

Evolutions of mass loss per drying surface versus the square root of time are shown in Fig. 6a for the CEM I cement paste and Fig. 6b for the GGBS cement paste. In the CEM I cement paste, the evolution is linear in the square-root of time scale with a slope of \(-510 \text{ g/(m}^2\text{.d}^{0.5}\text{)}\) and the repeatability seems correct. Therefore, there is no significant change in the microstructure during drying which started after 3 days of curing. In the case of CEM I +60\% GGBS, the evolution is non-linear (Fig. 6b). The slope changes around 1 week. This is due to the drastic microstructure change when the GGBS reactions take place [16]. Indeed, the result is confirmed by the attenuation of the slope in the GGBS case, before 1 week the slope is \(-1200 \text{ g/(m}^2\text{.d}^{0.5}\text{)}\) and after the slope value is \(-210 \text{ g/(m}^2\text{.d}^{0.5}\text{)}\). Therefore, the GGBS cement paste drying is slower than the CEM I one after 1 week. This can be due to a finer porosity and therefore a lower apparent diffusivity. In the Fig. 5a, the ratio mass variation to surface is smaller in the condition C than the condition D. This is due to the carbonation which leads to a mass increase by CO\(_2\) binding. The difference between the conditions D and C is less important for the GGBS cement paste. This difference can be explained by lower CO\(_2\) binding as shown with the TGA results.
Figure 6: Evolution of ratio mass variation to drying surface with the square root of time for (PCM) (a) and (P6S) (b)

D: drying condition – C: coupled drying and carbonation conditions

3.3 Pore structure

Fig. 7a shows the pore size distribution at the surface for the CEM I cement paste. Between $t_0$ and $t_0+56$ days, in condition D, the pore structure becomes a little coarser while the porosity decreases during the hydration process. The pore diameters are smaller at $t_0+56$ days in the condition C than in only the condition D.

Fig. 7b shows the pore size distribution at the surface for a GGBS cement paste. Between $t_0$ and $t_0+56$ days in both conditions, the pore radius decreases due to the hydration. Concerning the case with or without carbonation, the pore size distribution stays quite the same in both conditions at $t_0+56$ days because this phenomenon is not enough developed in the slag cement paste, as shown by the carbonation depth measurements.

Figure 7: Evolution of the pore size distribution for PCM (a) and for P6S (b) at a depth of 0.0-0.5 cm
D: drying condition – C: coupled drying and carbonation conditions

In Fig. 8a, the pore diameter decreases between $t_0$ and $t_0+56$ days from 220 nm to 20 nm due to hydration process. The same trend is highlighted in the Fig. 8b for the blended slag paste where the pore diameter decreases from 630 nm to 11 nm. At the depth of 6.5-10.0 cm, samples
are subjected to neither drying nor carbonation. At the surface, the GGBS cement pastes have a coarser porosity than the CEM I cement paste one due to the drying. The GGBS cement pastes have a finer pore structure than the CEM I cement paste one at a depth of 6.5-10.0 cm without drying. These results can explain the kinetics of the mass loss observed in Fig 6.

![Figure 8: Evolution of the pore size distribution for PCM (a) and for P6S (b) at a depth of 6.5-10.0 cm.](image)

D: drying condition – C: coupled drying and carbonation conditions

The comparison between the porosity obtained by MIP measurement and the porosity accessible to water is shown in Fig. 9 and table 3. The MIP porosity values are smaller than water porosity values because the mercury does not have access to all the porosity. The difference between the 2 methods of measurement increases with the time. For both binders, in the condition D, the porosity decreases due to the hydration process which contributes to fill the porosity by hydrates formation. For the CEM I cement paste in the condition C, the porosity decreases for the depth 0.0-0.5 cm due to the carbonation treatment. Moreover, the MIP results show a decrease of pore size distribution. Therefore, the carbonation leads to filing of the porosity by formation of calcium carbonate, this observation is in agreement with the literature [17]. For the GGBS cement paste, the decrease of porosity for the depth 0.0-0.5 cm is smaller. The porosity increases for the GGBS cement paste in the condition C by MIP. This result may be due to cracking of sample. The GGBS cement pastes have a higher porosity around 10% (measured by MIP) more than the CEM I cement pastes. This is due to slower hydration kinetics as shown in Fig. 4.

Fig. 10 shows the evolution of saturation degree profiles at several times in the conditions C or D for a CEM I cement paste (Fig. 10a) and for the GGBS cement paste (Fig. 10b). For both binders the saturation degree decreases when the depth goes to the surface due to drying. For the CEM I cement paste (Fig. 10a), the surface seems to be close to equilibrium with the atmosphere from t₀+28 days. The depth affected by the drying at t₀+7 days is 3 cm and 5 cm from t₀+28 days. For the slag cement paste (Fig. 10b), the surface seems to be close to equilibrium with the atmosphere from t₀+7 days. The depth impact by the drying at t₀+7 days is 4 cm and 6 cm from t₀+28 days. After 56 days profiles for both pastes are very similar. The difference of saturation evolution can be explained by an initial faster drying (t < 7 days) in the
slag cement paste due to a higher porosity. In both cement pastes, the saturation profiles are similar in the conditions C and D.

![Graph showing porosity evolution with time for PCM and P6S](image)

Figure 9: Evolution of porosity with time for PCM (a) and for P6S (b)

Table 3: Results of porosity obtained by MIP or accessible to water measurement at several times (Standard deviation (S.D))

<table>
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<tr>
<th>Depth (cm)</th>
<th>0.0-0.5 Hg</th>
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<th>0.0-0.5 water S.D</th>
<th>1.0-1.5 Hg</th>
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<th>6.5-10.0 Hg</th>
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</thead>
<tbody>
<tr>
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</tbody>
</table>

The above described experimental results show evidence difference of the two cement pastes behaviour when they are submitted to the coupled drying-carbonation condition at early age. Indeed, the Portlandite content is lower for the GGBS cement paste due to the dilution. The lower CaCO3 shown by the TGA and XRD results for the GGBS cement paste can be explained by lower hydrates can be carbonated contents related to dilution of OPC. This result is confirmed by the carbonation depth measurement. For both binders at all the time in the condition D, the CaCO3 contents near to the surface are higher than for the other depths. This is due to the carbonation of a thin layer at the surface. This cannot be avoided unless these experiments are performed under air void. The XRD results are only qualitative, to complete this work using of quantitative method is necessary. The drying of GGBS cement paste at early age compared to the CEM I case is faster. However, this trend changes when the GGBS
reactions occur, that are confirmed by the MIP results and by the saturation profiles. Indeed, the pore radii of GGBS cement paste are bigger at early age and consequently, the moisture transfer is easier. While after the GGBS reactions occur, the pore structure is thinner then moisture transfer is slowed down.

![Graph showing saturation degree profiles](image)

Figure 10: Evolution of saturation degree profiles with the time for PCM (a) and for P6S (b)

4 CONCLUSION

- In this work, the coupling process leads to a faster drying of GGBS cement paste at early age compared to the CEM I case. However, this trend changes when the GGBS reactions occur that are confirmed by the MIP results.
- After 3 days of hydration in water, the carbonation depths are similar at early age but a lower carbonation degree suggests a lower diffusivity and finer microstructure for GGBS paste.
- These results of drying and carbonation need to be confirmed for a longer time scale.

5 ACKNOWLEDGEMENTS

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6 REFERENCES


CHLORIDE DIFFUSION IN ALKALI ACTIVATED CONCRETE

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Key words: Durability, Alkali activated cementitious binder, corrosion initiators, Influence of chloride.

Abstract

The corrosion resistance of steel reinforcement embedded in an alkali activated cementitious material (AACM) developed by Sheffield Hallam University (SHU) is being investigated. Two main initiators of corrosion, chloride diffusion and carbonation are being considered. This paper will present the results of chloride diffusion in the AACM concrete compositions under long term exposure (up to 180 days) by immersion in a 5% sodium chloride solution. Different mix compositions of the AACM concrete (four grades of mixes) suitable for structural application have been selected, representing different concentrations of the alkali activator and different binder and aggregate ratios. Parallel investigations on control samples of normal concrete of similar strength have been conducted. The chloride diffusion profiles with depth have been obtained. The results show a good correlation of chloride diffusion with Fick's second law of diffusion. A general reduction of chloride ion content within the AACM concrete matrix was observed compared to the control samples of normal concrete. Low liquid/binder ratio has a positive impact on strength, shrinkage and chloride diffusion of AACM concrete.

1. INTRODUCTION

Chloride induced corrosion in reinforced concrete commences when a critical (threshold) concentration of chloride reaches at the surface of the embedded steel in the concrete (Nilsson et al., 1996) [1]. A single value for chloride threshold level is not true for different types of concrete, steels and environment. However, the maximum acceptable chloride concentrations expressed as the percentage of chloride by mass of cement have been recommended for concrete (BS EN 206, 2013) [2]. The error function solution of Fick's second law, given in eqn. 1 (Crank, 1975) [3], applies to the unidirectional chloride diffusion process in concrete:

\[
C_{(x,t)} = C_0 \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_\tau}} \right) \right]
\]

(1)

Where

\(x\) = distance from concrete surface exposed to chloride (m);

\(C_{(x,t)}\) = chloride concentration at depth (x) and time (t);

\(C_0\) = initial chloride concentration at surface;

\(D_\tau\) = effective chloride diffusion coefficient.
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\[ t = \text{Time (seconds)}; \]
\[ D_C = \text{diffusion coefficient (m}^2/\text{s}); \]
\[ C_0 = \text{equilibrium chloride concentration on concrete surface}; \]
\[ C_{(x,t)} = \text{chloride concentration at position } x \text{ and time } t. \]

Non-linear regression analysis of experimental data between \( C_{(X,t)} \) and \( x \) to solve eqn. 1 determines the diffusion parameters \( C_0 \) and \( D_C \).

The relationships between diffusion coefficients, \( D_C \) and exposure period, \( t \), for marine concrete and concrete repair materials were determined by Mangat and Limbachiya, 1999 \[4\] and Laurence, 1989 \[5\]. Mangat and Limbachiya, 1999 \[4\] gave diffusion coefficient values ranging between \( 3.6 \times 10^{-7} \) and \( 123 \times 10^{-7} \text{ cm}^2/\text{s} \) under long-term marine exposure. In addition, values ranging between \( 0.2 \times 10^{-7} \) and \( 2.2 \times 10^{-7} \text{ cm}^2/\text{s} \) were reported by Laurence, 1989 \[5\] under long-term tidal zone and splash zone marine exposure conditions. This research also established that the diffusion coefficient, \( D_C \), is time-dependent. The following relationship between \( D_C \) and time was proposed by Mangat and Molloy, 1994 \[6\]

\[ D_C = D_l t^{-m} \tag{2} \]

Where \( D_l \) is the effective diffusion coefficient at time \( t \) equal to 1s and \( m \) is an empirical constant. Similarly, the relationship between surface chloride concentration, \( C_0 \) and exposure period, \( t \), was determined. The maximum value for \( C_0 \), at 180 days exposure was 0.85% by weight of concrete compared with 0.27% to 0.6% at 28 and 90 days respectively. Using the semi-empirical relationship between \( D_C \) and time from Eqn. 2, a prediction model for chloride concentrations after 10 years of exposure to a chloride environment was established (Mangat and Molloy, 1994) \[6\].

The transportation mechanism by which chloride ions penetrate hardened concrete is governed by the mix composition, degree of saturation within the pore structure, temperature and type of chloride exposure. Chloride ingress in a fully saturated hardened concrete matrix is by the diffusion process which occurs when at least one surface of the fully saturated matrix is exposed to a chloride solution (Puertas et al., 2003) \[7\]. In the process, diffusion occurs as the solution seeks to attain equilibrium causing the chloride ions from high concentrations to move to low concentrations within the concrete matrix.

The chlorides in concrete can be present in two forms namely chemically/physically bound chloride (acid soluble chloride) and free chloride (water soluble chloride) (Nilsson et al., 1996) \[11\], (Neville, 2002) \[8\] and (Lambert and Page, 1991) \[9\]. The sum of the acid and water soluble chloride is termed the total chloride. The free chloride is related to the concentration of the chemically bound chloride. Since acid soluble chlorides are chemically bound (immobile) in the concrete matrix, only the water soluble chloride can penetrate to the ferrous oxide protective layer of the steel reinforcement to initiate corrosion in reinforced concrete (Tang and Nilsson, 1993) \[10\]. Therefore, the threshold level for corrosion initiation is evaluated by the amount of water soluble chloride present in the concrete matrix expressed in percentage of the total amount of binder in the mix (Arya et al., 1987) \[11\].

A review of chloride diffusion test methods is given by Stanish et al. (1997) \[12\]. The research reported in this paper uses saturated AACM concrete specimens, completely immersed in an aqueous sodium chloride solution at a regulated temperature \((20 \pm 2^\circ\text{C})\). The chloride ions in the aqueous solution diffuse through the pore water of concrete to reach an equilibrium
concentration. The non-steady state diffusion process is enhanced by the gradient of the free chloride ions in the pore solution (Cement concrete and aggregate Australia, 2009)\textsuperscript{[13]}. 

2 EXPERIMENTAL PROCEDURE

2.1 Materials and Mixes

The AACM concrete comprises of a cementitious binder, fine aggregate, coarse aggregate and the alkali activator. A proprietary hybrid alkali activated cementitious binder comprising of low and high calcium constituents was used. The cementitious binder was developed at Sheffield Hallam University\textsuperscript{[14]} and versions of this are currently manufactured under licence. A medium grade sand and 10mm nominal size coarse aggregate was used for the AACM concrete mixes. Trial mix S\textsubscript{1} was performed prior to the selection of the mix composition used for the chloride ingress, carbonation and corrosion investigation in this research work. The trial mixes were made with the proprietary AACM binder and activator incorporating a range of 438 kg/m\textsuperscript{3} to 585 kg/m\textsuperscript{3} of fine aggregate and 988 kg/m\textsuperscript{3} to 1171 kg/m\textsuperscript{3} of coarse aggregate. The trial mix S\textsubscript{1} was tested for strength, shrinkage and workability to achieve a practicable field mix of AACM concrete with high strength. The experimental mixes S\textsubscript{2} to S\textsubscript{6} used in this research were produced based on the trial mix S\textsubscript{1}. The AACM binder content and fine aggregate content of the mixes S\textsubscript{2} to S\textsubscript{5} was reduced relative to mix S\textsubscript{1} while the liquid/binder and coarse aggregate content was increased to adjust the workability and setting time required for practical mixes. The optimum mix composition of the five series of mixes S\textsubscript{2} to S\textsubscript{6} is given in Table 1. S\textsubscript{6} is the control mix produced with 100\% OPC binder of C40 grade concrete. The reference alkali activator liquid\textsuperscript{[14]} was diluted with tap water by 2.15\%, 4.24\%, 8.12\% and 12\% in mixes S\textsubscript{2} to S\textsubscript{5} respectively (Table 1).

<table>
<thead>
<tr>
<th>Constituent</th>
<th>S2</th>
<th>S3</th>
<th>S4</th>
<th>S5</th>
<th>S6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid(Kg/m\textsuperscript{3})</td>
<td>285(12%)</td>
<td>295(12%)</td>
<td>293(12%)</td>
<td>291(12%)</td>
<td>140(6%)</td>
</tr>
<tr>
<td>Binder(Kg/m\textsuperscript{3})</td>
<td>688(29%)</td>
<td>619(26%)</td>
<td>619(26%)</td>
<td>619(26%)</td>
<td>350(16%)</td>
</tr>
<tr>
<td>Liquid/Binder</td>
<td>0.41</td>
<td>0.48</td>
<td>0.47</td>
<td>0.47</td>
<td>0.48</td>
</tr>
<tr>
<td>Activator Dilution (%)</td>
<td>2.15</td>
<td>4.24</td>
<td>8.12</td>
<td>12.00</td>
<td>-</td>
</tr>
<tr>
<td>Fine Aggregate(Kg/m\textsuperscript{3})</td>
<td>438(18%)</td>
<td>423(17%)</td>
<td>423(17%)</td>
<td>423(17%)</td>
<td>585(26%)</td>
</tr>
<tr>
<td>Coarse Aggregate(Kg/m\textsuperscript{3})</td>
<td>988(41%)</td>
<td>1091(45%)</td>
<td>1091(45%)</td>
<td>1091(45%)</td>
<td>1170(52%)</td>
</tr>
</tbody>
</table>

Note: Percentage (%) given in brackets are by weight of total mix.

2.2 Specimen Preparation and Curing

All the aggregates were in saturated surface dry state before mixing. Half of the aggregate content was first poured inside the cretangle concrete mixer of 150 kg capacity followed by the binder composition. The remaining half of the aggregate content was then added to cover the binder. The binder and the aggregates were mixed for one minute. Approximately half of
the liquid activator was added to the mixer and mixed for two minutes. To ensure homogeneity, the mix was briefly mixed by hand, particularly concentrating on the material sticking around the edge and corner of the mixer. The retarder reagent was mixed with the second half of the liquid activator and stirred until fully dispersed. The mixture was then added to the mixer and mixed for a further two minutes. A shrinkage admixture was then added followed by an additional one minute of mixing.

75 mm x 75 mm x 75 mm dimension steel moulds were used for casting specimens for compressive strength. Three gang 160 mm x 40 mm x 40 mm prism steel moulds were used for casting specimens for shrinkage test. 250 mm x 250 mm x 75 mm dimension polystyrene moulds were used for casting chloride diffusion specimens. The internal surfaces of the moulds were oiled to prevent the hardened AACM from sticking to the surface. The mould was filled in three layers. Each layer was compacted on a vibrating table for up to 60 seconds to attain homogeneity and minimise the presence of voids. The AACM concrete surface was gently trowelled to obtain a smooth and level finish. The cast specimens were placed on a flat table surface and covered with polythene sheets to prevent moisture loss.

The mixes were prepared under ambient conditions in the laboratory (approximately 20 deg C, 60% RH). The specimens were demoulded after 24 hours. The chloride diffusion test samples of series S2 to S6 were cured in water at 20 deg C. for 28 days. Subsequently after the 28 days, the chloride diffusion test samples of series S2 to S6 were fully immersed in a 5% sodium chloride solution until each chloride test age up to 180 days of exposure in chloride solution. A total of 42 cubes were cast for each series. Series S2 to S5 were cured in air at 20 deg C. and 60% R.H. for 28 days. Series S6 were cured in water at 20 deg C. for 28 days. The air cured specimens were covered with polythene sheets to prevent rapid water loss. Curing of the shrinkage samples was performed in accordance with BS ISO 1920-8 (2009) [15]

### 2.2.1 Chloride Exposure Solution

Bulk diffusion tests (NordTest, 1995) [16] and (DD CEN/TS 12390-11, 2010) [17] were adopted for measuring the long-term chloride diffusion in the AACM and OPC concrete of Series S2 to S6 mixes. The two standard test methods have similar procedures with the exception of the concentration of the chloride exposure solution. DD CEN/TS 12390-11, 2010 [17] specifies 3% NaCl solution (by weight) whereas NordTest, 1995 [16] specifies 165g ± 1g NaCl per dm³ solution (16.5% NaCl solution by weight). A 5% NaCl solution in tap water was prepared for chloride exposure of the test specimens. The 5% NaCl solution provides an accelerated chloride diffusion test which gives higher chloride concentrations at given depths than under normal marine exposure. Therefore, the permissible chloride concentrations allowed in (DD CEN/TS 12390-11, 2010) [17] for different types of chloride exposure cannot be directly related to the data presented in this paper. The transportation mode of chloride ion was by diffusion only. This was achieved by curing the test specimens in water for 28 days after casting to attain full saturation of the pores thereby eliminating the initial sorption effect upon chloride exposure. All the faces of the chloride diffusion test specimens were coated with 1mm thick layer of bituminous paint except the 250 mm x 250 mm face to be exposed to the 5% NaCl solution, as shown in Fig 1a.
2.3 Testing

2.3.1 Sampling for Chloride Analysis

At each test age of 55, 90, 120 and 180 days chloride exposure, two specimens for each mix S2 to S6 were removed from the NaCl solution (Fig 1a). Each 250 mm x 250 mm x 75 mm slab specimen was cut into two equal halves along the longitudinal plane perpendicular to the uncoated face as shown in Fig 1b. The direction of the cut by the masonry saw was from the bottom bitumen coated face of the sample to its top uncoated face. After cutting, the specimens were left to dry in the laboratory (20 deg. 60% RH) for 24hrs. Each freshly sawed face (perpendicular to the chloride-exposed face) was drilled at 8, 15, 20, 25, 35, 50 and 65 mm depths from the uncoated surface of the specimen which was exposed to the chloride solution as shown in Fig 1c. Six holes were drilled per depth making a total of 42 holes for each specimen. The location of each drilled hole was kept at least 20 mm away from the bitumen coated end faces of the specimens to avoid any edge effect errors on the chloride measurements. Each line on the cut face of the sample (Fig. 1c) represents a constant depth from the uncoated face which was directly exposed to chloride diffusion. The first 5 mm depth of the drilled powder were discarded from each hole. A further 15 mm deep hole was then drilled to provide a powder sample for chemical analysis. The concrete powder sample was carefully collected and stored in a self-sealing polythene bag to avoid contamination from the bituminous coating.

2.3.3 Strength and Shrinkage

Compressive Strength - 75 mm x 75 mm x 75 mm cubes of mixes S2 to S6 were cast in steel moulds and tested for compressive strength at ages 1, 2, 3, 7, 14, 21 and 28 days for each series of mixes given in Table 1. The compression test was performed on three cubes at each test age and the average of the three readings was reported. The compression test was performed in accordance with BS EN 12390-3 (2009)\textsuperscript{118}.

Shrinkage - Three gang steel moulds of 160 mm x 40 mm x 40 mm were used to cast prism specimens for the shrinkage test. The fresh mix was placed in the prism moulds in three layers and compacted on a vibrating table. After casting, the moulds were covered with polythene sheets to minimise moisture loss. The specimens were stored in the laboratory for 16-24 hrs at 60% RH and 20°C. After de-moulding, stainless steel demec points were fixed along two parallel longitudinal faces of each prism specimen at a gauge length of 100 mm. The datum reading was taken at 24 hours age (after casting). The specimens were immersed in water for 7 days and expansion readings were taken at regular intervals with the demec extensometer. After 7 days the specimens were removed from the water and surface dried with a damp cloth. A demec extensometer reading was then taken for each specimen. The drying shrinkage readings were subsequently taken for 50 days period under 20 deg C, 60% RH exposure. The average value for three specimens per mix was recorded. The test was performed in accordance with BS ISO 1920-8 (2009)\textsuperscript{115}.
Pal. S. Mangat* and Olalekan.O. Ojedokun†,

Fig. 1: Concrete Powder collection for chloride analysis (a) Specimens immersed in NaCl Solution (b) Slab specimen halves showing the exposed surface cut by Masonry Sawing (c) Location of Drilled holes.

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Compressive Strength and Shrinkage

The compressive strength and shrinkage of mixes S2 to S6 are given in Table 2. A liquid/binder ratio of 0.414, 0.477, 0.473, 0.470 and 0.486 was used for the series of mixes S2, S3, S4, S5 and S6 respectively. The dilution levels of the alkali activator were 2.15%, 4.15%, 8.12%, 12.0% for AACM concrete series S2 to S5 mixes respectively. Series S6 mixes were produced with 100% OPC binder, with w/c of 0.486.
Pal. S. Mangat * and Olalekan.O. Ojedokun †,

Table 2: Mechanical Properties of AAC & OPC Concrete Mixes

<table>
<thead>
<tr>
<th>Series(Liquid/binder, Dilution)</th>
<th>Compressive Strength</th>
<th>Shrinkage (Microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7days (MPa)</td>
<td>28days (MPa)</td>
</tr>
<tr>
<td>S2(0.414;2.15%)</td>
<td>55.71</td>
<td>60.11</td>
</tr>
<tr>
<td>S3(0.477;4.15%)</td>
<td>51.05</td>
<td>62.03</td>
</tr>
<tr>
<td>S4(0.473;8.12%)</td>
<td>47.15</td>
<td>53.09</td>
</tr>
<tr>
<td>S5(0.470;12.0%)</td>
<td>41.15</td>
<td>50.76</td>
</tr>
<tr>
<td>S6(0.486;OPC)</td>
<td>32.56</td>
<td>45.67</td>
</tr>
</tbody>
</table>

3.1.1 Influence of Liquid/Binder ratio and Coarse Aggregate content

Rapid early strength development of the AACM concrete (mixes S2 to S5 mixes) was recorded, relative to the OPC concrete reference mix S6, for the first 7 days. An average strength of 92, 86, 81 and 85% of the 28 day strength was achieved within 7 days by mixes S2, S3, S4 and S5 respectively. The control mix of OPC concrete (series S6) gained 78% of the 28 day strength within 7 days. These results indicate that the AACM concrete develops strength more rapidly than OPC concrete. This early strength gain phenomenon is in agreement with other researchers (Susan et al., 2014) [19], (Vaidya et al., 2011) [20] and (Puertas et al., 2003) [21].

3.1.2 Influence of Activator Dilution

The liquid/binder ratios of the AACM concrete mixes S3, S4 and S5 were similar (0.47 to 0.477). However, their 28 day compressive strength decreased from 62 to 50.8 MPa with increasing dilution of the activator with water from 4.15% to 12%. The activator dilution effect overshadows the slight differences in the liquid/binder ratios of these mixes. A similar effect of activator dilution is observed on the 50 day shrinkage values which increase from 706 microstrain (mix S3) to 984 microstrain (mix S5). The coarse aggregate contents of mixes S3, S4 and S5 are the same at 1091 kg/m³. The increased volume of water in the micropores due to activator dilution is the likely cause of increased shrinkage.

3.1.3 Influence of coarse aggregate content

The AACM concrete mix S2 has a liquid/binder ratio of 0.41 compared to 0.48 for mix S3 while their coarse aggregate contents are 988 and 1091 kg/m³ respectively (Table 1). Their activator dilution is comparable at 2.15% and 4.15%. The coarse aggregate content of mix S3 (1091 kg/m³) is the dominant factor in reducing its shrinkage (706 microstrain) compared with mix S2 (728 microstrain) and overshadows the effect of both the lower liquid/binder ratio (0.41) and activator dilution (2.15%) of mix S2 compared with the corresponding values of 0.48 and 4.15% for mix S3. The shrinkage restraining effect of coarse aggregates in concrete, due to their higher elastic modulus than the surrounding matrix, is well recognised (Neville, 2002) [8].

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The shrinkage of the AACM concrete mixes is significantly greater than the reference concrete mix S6 (*Table 2*). This agrees with the results of other researchers on AACM binders who attribute this primarily to chemical differences in hydration (Neville, 2002)[8], (Provis and Jannie, 2014)[22] and (Mangat and Tu, 1993)[23]. However, mix design parameters also have very significant impact on the shrinkage of AACM concrete. The binder (OPC) content of the reference mix S6 is 350 kg/m3 compared with 619-688 kg/m3 for mixes S2 to S5. Further, the fine and coarse aggregate contents of the OPC concrete mix S6 are much higher at 585 kg/m3 and 1170 kg/m3 respectively compared with corresponding values of 423 and 1091 kg/m3 for AACM concrete mixes. Consequently the shrinkage restraining effect of the aggregates is much more dominant in the OPC concrete.

### 3.2 Chloride Ingress

#### 3.2.1 Chloride Concentration Profiles

The chloride concentration profiles with depth of the AACM concrete mixes S2 to S5 and the OPC concrete mix S6, whose details are given in *Table 1*, were determined by complete immersion in a chloride solution. The results are plotted in *Fig 2*. They show good correlation with the best fit profiles derived from Fick’s 2nd law of diffusion which is given in *Eqn 1*. The values of correlation between the chloride concentration profiles and the best fit profiles are within the range 0.88 to 0.99.
Fig. 2: Chloride Concentration by Weight of AAMC Concrete from (a) Series S2 (b) Series S3 (c) Series S4 (d) Series S5 and (e) Series S6 in 5% NaCl Solution

The relationships between the chloride content of AAMC concrete (series S2 to S5 mixes) and depth of penetration are plotted in Fig. 2(a) - (d). Fig. 2(e) represents the control 100% OPC concrete mix S6. The period of exposure to the sodium chloride solution was 55, 90, 120 and 180 days for each mix. The best fit curves represent the chloride diffusion profiles for each of these exposure periods showing the chloride concentration at the concrete surface, \( C_0 \), and up to a depth of 70mm. The surface chloride concentration, \( C_0 \), has been determined by regression analysis of the test data to fit equation 1 (Fick's second law of diffusion). An investigation into the microstructure of both AAC and OPC concrete using mercury intrusion.
Porosimetry is being conducted by the authors and its relationship with chloride diffusion will be published in future.

A comparison of Figure 2e (for OPC concrete) with Figures 2a - 2d (for AACM concrete) generally shows that the long term chloride concentrations and depths of chloride penetration are lower in the AACM concrete. The AACM mix S2 with the lowest activator/binder ratio (0.41) and the lowest activator dilution (2.15%) shows the maximum resistance to chloride diffusion. The surface chloride concentration, \( C_0 \), for the OPC concrete mix S6 (Fig 2e) at 180 days is about 0.92% by weight of concrete. The comparative values for the AACM mixes (Figs 2a - 2d) range between approximately 0.57% and 0.8% by weight of AACM concrete. These values are in the range given in literature for marine concrete (Mangat and Limbachiya, 1999\(^4\) and Laurence, 1989\(^5\)).

**4 CONCLUSIONS**

The following conclusions are drawn from the experimental results:

- The activator/binder ratio has significant impact on the strength, shrinkage and chloride diffusion in AACM concrete, with lower ratios resulting in superior properties. Dilution of the activator with water generally has the opposite effect.
- Higher coarse aggregate content reduces the shrinkage of well designed AACM concrete mixes.
- Chloride diffusion experimental data of AACM concrete shows good correlation with Fick's second law of diffusion.
- The chloride diffusion in AACM mixes, under long term chloride exposure, is lower (in concentration and penetration depth) than the control OPC concrete mix. Lower activator/binder ratios and lower degrees of activator dilution provide maximum chloride resistance.

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COAL BOTTOM ASH RESEARCH PROGRAM FOCUSED TO EVALUATE A POTENTIAL PORTLAND CEMENT CONSTITUENT
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Key words: Coal bottom ash, Portland cement, Compressive Strength, Durability.

Abstract. Coal bottom ash is a by-product of the coal combustion produced in electrical power stations. It is formed together with the coal fly ash in the same boiler. Then, it would be possible to find several similarities with regard to their chemical composition and mineralogical phases. However, the main characteristic recognized in these ashes is their quite different grain size. This fact could have an important effect on the chemical composition as well as on the mineralogical phases of both ashes. Therefore, the first step within this research program was to get a similar grain size, which was reached by bottom ash grinding. The second step was to compare both ashes from a mechanical, chemical, mineralogical and durable point of view. This comparison helped to evaluate the potential of the coal bottom ash as a Portland cement constituent. Finally, the third step was to check the viability of the ground coal bottom ash as a Portland cement constituent from a performance point of view.

The considered mechanical tests were 28-days compressive strength and flexure strength in order to classify the blended cement with regard to the strength. But also, 3, 7, 14 and 90 days compressive and flexure strength were performed. Mechanical test results in ground coal bottom ash showed a good performance, which was similar to that of fly ash or even better.

Durability tests were planned taken into account those properties in which coal ashes do not improve significantly the blended cement performance: Natural carbonation, and frost-thaw resistance. Also, alkali-silica reaction and sulfate resistance were also studied.

Natural carbonation tests were developed according to the European standard EN 112011:2007 “Products and systems for the protection and repair of concrete structures -Test methods- Determination of carbonation depth in hardened concrete by the phenolphthalein method”. Ground coal bottom ash mortars showed a worse carbonation resistance than the reference mortars, but similar to that of fly ash mortars.
Frost-thaw resistance tests after 28 and 42 cycles showed that ash mortars were lesser resistant than the reference mortars without any addition. Nevertheless, coal bottom ash mortars showed a better performance than coal fly ash mortars.

Finally, it can be highlighted that alkali-silica reaction and sulfate resistance were the only two durable properties considered in this research program in which a clear improvement was evidenced.

1 INTRODUCTION

In 2013, the total worldwide cement world cement production was about 4.0 billion tonnes [1]. Due to a world-wide increase in common Portland cement demand, cement production currently represent more than 5% of total global warming CO₂ emissions [2]. The increased usage of some by-products such as coal fly ash, silica fume and granulated blast furnace slag as cement constituents in the production of common Portland cement is motivated by a number of considerations related to the decrease of CO₂ emissions, but also to avoid the disposal of industrial residues in landfill.

The use of blended cements is one of the main trends within a sustainable development strategy and also is an important element in the development of concrete technology. It enables the properties of the concrete to be improved, particularly with regard to the resistance to the aggressive agents present in the environment. Some by-products have been used successfully [3]. Use of such additions makes waste management more efficient, reduces energy consumption during cement production and lowers CO₂ emissions. Therefore, these good properties will cause an increased demand for blended cements, and then, in order to fulfill their future responsibilities cement manufacturers need to have a high degree of flexibility in the selection and use of mineral cement constituents [1]. In this way, new possible constituents as ground coal bottom ash could be considered for its use in mentioned blended cements.
Bottom ash and fly ash are two industrial by-products materials presenting pozzolanic characteristics occurring due to burning the pulverized coal in the thermal power plants. The main effects of the use of fly ash in mortars and concretes among other possible applications have been already extensively presented elsewhere [4]. Currently, fly ash is used as a constituent in the production of cement, in concrete production, replacing fine aggregates, as a filling material in dams or in retaining walls, as a base and sub-base material in highway construction, and for production of light construction materials, among other applications in the concrete sector [4]. Then, it can be said that fly ash is a well-known construction material.

Fly ash pozzolanic activity is due to its content of a surplus content of silica, alumina and iron oxide, but also, as well as other pozzolanic materials, it presents a filler effect because it is composed of very fine particles. It has been reported that filler effect in some supplementary cementitious materials (SCM) could be more effective than the pozzolanic characteristics when affecting the properties of concrete [5].

The effect of fly ash fineness on compressive strength has been reported in the literature [6, 7]. In general, mortars made of ground fly ash provided higher early strength than mortars produced with coarse ashes due to the improving result of a high fineness. Therefore, a finer fly ash or similarly, ground bottom ash provides a more compact and denser made hardened cement paste in mortars and concretes.

Then, it is expected an increase of the fly ash pozzolanic activity index as fly ash fineness is increased. According to Kiattikomol et al., the most important factor affecting the fly ash pozzolanic activity index was its fineness, but not its chemical composition [7]. Chindapasirt et al. [6] found a higher pozzolanic activity and compressive strength when the fly ash fineness is increased from 1800 cm²/g to 9300 cm²/g in cements with 40% of fly ash.

The Spanish Institute of Cement and its Applications (IECA) in co-operation with the Spanish Institute of Construction Sciences “Eduardo Torroja” (IETcc) a research project has been carried out to contribute to a better understanding of the coal bottom ash (CBA) mortar performance in comparison to coal fly ash (CFA) mortar performance in order to evaluate it viability to be used as a new Portland cement constituent. Thus, this paper presents a systematic approach for assessing the use of CBA mortars and concretes as a reliable measure to reduce the CO₂ emissions associated with the cement industry.

Striving for improved CO₂ performance of cement should thus be a high priority. Then, this paper is focused on the relevance of the use coal bottom ash as a new cement constituent.

2 THEORETICAL APPROACH

Cement is a key construction material which production requires use of considerable natural raw materials and energy (fossil fuels among others) and results in many different types of emissions causing environmental impacts. Particularly, the cement industry is under pressure to reduce its CO₂ emissions. Cement demand will increase but CO₂ emissions must decrease. Recently, several studies have dealt with a range of different measures about existing and emerging technologies to improve energy efficiency and CO₂ reduction performance [8, 2].

Some alternatives exist to effectively reduce or avoiding CO₂ emissions associated with cement production. They could be subdivided into three main lines: clinkerization process, fuel combustion and clinker/cement ratio (Fig. 2).
For assessing the CO₂ emission reduction potential, the following main sources of CO₂ emissions can be considered:

- Clinkerization (calcination process: CO₂)
- Fuel combustion
- Energy demand
- Indirect measures
- Clinker/cement factor

Calcination process during clinker production in the kiln system is the de-carbonation of limestone into lime releasing about 500 kg CO₂ for 1 tonne of clinker produced, depending on the production system.

Fuels combustion is another major source of CO₂ emissions (40%) during clinker production. Measures for CO₂ emission reduction are related to use less non-biogenic CO₂, i.e. increase the share of renewable fuels.

Reducing the energy demand of clinker/cement production per tonne of product by means of energy efficiency measures, CO₂ emissions can be reduced since most of the heat and electricity used in cement production has fossil origin.

Indirect measures cause less combustion of fossil fuels. For instance, the heat of the cement plant could be used in another industrial process (heat recovery, raw materials drying, co-generation, and so on).

Finally, the clinker/cement factor has a direct impact on CO₂ emissions. The lower clinker/cement ratio, the lesser CO₂ emissions. Clinker substitution is a mature technology with high CO₂ improvement potential. But also, one of the most viable approaches that are gradually emerging is the study of the technical and viability of new cement constituents.
Then, new cement constituents will help in this scope and also improved properties of blended cements are expected.

Measures having both a high applicability and a high technological maturity were selected as possible options to prioritize (Fig. 3). Clinker substitution has both characteristics: a high applicability and a high technological maturity. Therefore, it is feasible from a technological and economic perspective. The clinker substitution rate constantly increased during the last decade. However, there is still a considerable potential for improvement. In Europe, 719 kg CO₂ per ton of cement was emitted in 1990, 695 kg CO₂ per ton of cement in 2000 and, currently, 627 kg CO₂ per ton of cement (2013). On the other hand, in the United States this value remains in 737 kg CO₂ per ton of cement.

Regarding CBA, its potential use as cement constituent will depend on market conditions and whether there is additional CFA capacity. In Europe, blending is carried out to a large extent at the cement production plants, rather than by concrete producers. It is possible that the American market will develop in that direction in the future due to the higher homogeneity and better quality control of the binder.

![Figure 3: Results of the assessment for CO₂ emissions reduction potential - implementation maturity and technical applicability.](image)

An overview of the assessment framework for CBA-cement standardization is shown in Fig. 3. The framework consists of five main parts and ten steps. The first part (steps 1, 2 and 3) concerns the coal fly ash and coal bottom ash assessment. The main source of information for the coal fly ash and coal bottom ash assessment of the assessment is literature. The second part of the framework is the mortar experimental program (steps 4 and 5). The third part of the framework is the concrete experimental program (steps 6 and 7). The fourth part is the onsite study (steps 8 and 9). Finally, the last part is the standardization application.
2 EXPERIMENTAL PROGRAM

The research was divided into two main parts as follows:

- The influence of coal bottom ash (CBA) on the properties of fresh and hardened mortars in comparison to coal fly ash (CFA).
- Viability of coal bottom ash (CBA) as main Portland cement constituent.

The research examined the influence of the following factors on the properties of CBA:

- Physical properties of coal bottom ash (Fineness – CBA grinding).
- Chemical properties of coal bottom ash (LOI, particularly).
- Mechanical properties of coal bottom ash and mixtures of coal bottom ash-fly ash.
- Durability of coal bottom ash-made cementitious materials.
2.1 Materials

The materials and mixture proportioning of the tested cements in the research were chosen according to EN 197-1:2011 for fly ash cements.

In the present study, a cement type CEM I 42.5 N (EN 197-1:2011), coal bottom ash and coal fly ash collected from Carboneras thermal power plant were used as constituents in the new cement mixtures. Properties of the used coal bottom ash, coal fly ash and cement were given in Table 1. Blaine fineness of the collected coal fly ash was 3976 m²/kg. The coal bottom ash was grinded in a ball mill and they were obtained with Blaine fineness similar to that of the Portland cement (5-8% residue on 45 µm sieve, 3463 m²/kg). The specific surface Blaine of the cement used in the present study is 4050 m²/kg. Cement was partially replaced by fly ash and/or bottom ash in order to prepare laboratory blended cements with similar compositions to the CEM II/A-V, CEM II/B-V and CEM IV/A (V) standardized cements. The different mixes were given elsewhere [9].

Table 1: Chemical composition of raw materials: coal bottom ash (CBA), coal fly ash (CFA) and cement, %.

<table>
<thead>
<tr>
<th>Oxides</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>K₂O</th>
<th>TiO₂</th>
<th>P₂O₅</th>
<th>LOI</th>
<th>IR*</th>
<th>Cl⁻</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>20.9</td>
<td>4.3</td>
<td>3.5</td>
<td>62.7</td>
<td>1.9</td>
<td>3.4</td>
<td>0.9</td>
<td>0.25</td>
<td>0.10</td>
<td>3.7</td>
<td>1.04</td>
<td>0.023</td>
</tr>
<tr>
<td>FA</td>
<td>50.5</td>
<td>28.9</td>
<td>4.7</td>
<td>5.0</td>
<td>1.8</td>
<td>0.21</td>
<td>0.8</td>
<td>1.56</td>
<td>0.76</td>
<td>3.6</td>
<td>71.3</td>
<td>0.000</td>
</tr>
<tr>
<td>BA</td>
<td>52.2</td>
<td>27.5</td>
<td>6.0</td>
<td>5.9</td>
<td>1.7</td>
<td>0.13</td>
<td>0.6</td>
<td>1.53</td>
<td>0.74</td>
<td>1.8</td>
<td>75.7</td>
<td>0.001</td>
</tr>
</tbody>
</table>

*Insoluble residue determined by the Na₂CO₃ method (European standard EN 196-2:2013).

2.2 Testing

Mechanical strength tests were performed according to the European standard EN 196-1:2005. Setting times and soundness were determined according to the European standard EN 196-3:2005+A1:2008. Evaluation of the pozzolanicity was done following the method described in the European standard EN 196-5:2011 and natural carbonation was tested following the technical specification CEN/TS 12390-10 [10]. Alkali-silica reaction and sulfate resistance tests were based on the American standards ASTM C-1260 [11] and ASTM C-1012 [12], respectively. Frost-thaw resistance was evaluated by means of a procedure reported elsewhere [13].

3 RESULTS AND DISCUSSION

Results of physical properties, chemical properties, mechanical properties and durability of bottom ash cementitious materials are discussed below.

3.1 Characteristics of coal bottom ash (CBA) and coal fly ash (CFA)

As shown in Table 1, it is evident that the main chemical compositions in the coal bottom ash belong to CaO-SiO₂-Al₂O₃-Fe₂O₃ system, which is similar to that of the common pozzolanic mineral supplementary cementitious materials (SCM) used in cement-based materials. Compared with the coal fly ash (CFA), coal bottom ash (CBA) is also combusted in the same incinerator, but it is denser, contributing to the low loss on ignition (LOI) in coal bottom ash, even to 1.8% (Table 1). Moreover, the LOI of the coal fly ash is usually higher (above 3.5%), which is mainly due to the unburned coal particles.
Fig. 6 illustrates SEM photographs of the ground coal bottom ash and coal fly ash. The surface of coal bottom ash has irregular particles; whereas coal fly ash is characterized by a more uniform distribution of spherical grains with a fairly smooth surface.

The spherical shape is probably produced because the particles of fly ash usually experience higher temperatures and such spherical shape is formed due to the shrinkage of the liquid phase.

3.2 Results of mechanical strength

The effects of the content of coal bottom ash or coal fly ash or mixes of both of them on compressive strength of mortars at 1, 3, 7, 14, 28 and 90 days are presented in Figure 7.

As shown in Figure 7, the compressive strength of the mortar specimens at 28 days decrease gradually with increasing of coal bottom ash or fly bottom ash or mixes of them. Moreover, early strength of the mortars decreases when the coal bottom or fly ash content is over 10%. This shows that the coal bottom ash or coal fly ash cement activity is lower at early ages and, then, both ashes delay the cement hydration as the content is increased.

The 35% replacement of cement by coal bottom ash in mortars, CEM IV/A (V), showed a worse in strength evolution (compressive strength of about 30% of the reference mortars at 28 days), while the 10% and 25% replacement showed much better compressive strength performance. It is noticeable, that the 28-days compressive strength of 35% replacement cement, CEM IV/A (V), was slightly over 32.5 MPa, which is the limit of the strength class 32.5 according to the European standard EN 197-1:2011. Therefore, the coal bottom and fly
ashes have some pozzolanic activity, but the reactivity is relatively lower. Then, they will lead to retardation of cement hydration. In this research, the limit of coal bottom ash replacement of cement could be set at 35% to ensure the compressive strength performance, CEM IV/A (V). The 28-days compressive strength of CEM IV/A (V)-Ω, with coal bottom ash is higher than CEM IV/A (V)-β, with coal fly ash.

The compressive strength activity index of coal bottom ash and coal fly ash at 28-days was 85% and 80%, respectively. Oppositely, the compressive strength activity index at 90-days was 98% and 109%, respectively. Consequently, when coal bottom ash or coal fly ash are added into cement with the same substitution, coal bottom ash cements show a higher compressive strength than those of coal fly ash cements at 28-days, but lower at 90-days.

Flexural strength results of coal bottom ash and coal fly ash cement mortars showed a similar trend [9].

### 3.3 Water demand for normal consistency and setting time

The water demand for normal consistency and setting times of the matrices are presented in Figures 8 and 9, respectively.

The water demand for normal consistency increases from CEM II/A-V to CEM IV/A (V) cements. The higher amount of ash in the cement, the higher water demand for normal consistency. Nevertheless, no differences in the performance of fly ash and bottom ash were found in CEM II/A-V and CEM II/B-V cements (Figure 8). At the same time, the initial and final of setting times are slightly reduced than the control specimen when bottom ash (Ω) is incorporated in CEM II/A-V and CEM II/B-V cements (Figure 9).

On the other hand, the incorporation of coal bottom ash slightly increases the water demand for normal consistency in CEM IV/A (V), which is increased with increasing of the content of coal bottom ash from γ to Ω.

![Figure 8: Water demand for normal consistency of CEM II/A-V, CEM II/B-V and CEM IV/A (V), from left to right, in comparison to the CEM I 42.5 N (α).](image)

Setting times of CEM II/A-V and CEM I 42.5 N are quite similar but, in general, the more ash in the cement, fly ash or bottom ash, the longer setting times were recorded. Therefore, CEM IV/A (V) showed the highest setting times. However, bottom ash cements (Ω) evidenced shorter setting times than fly ash cements (β) as shown in Figure 9.
The initial and final setting times were retarded when fly ash (β) were used instead of bottom ash (Ω). This fact could be partially explained because coal fly ash has a higher amount of cement hydration retarders such as Cu$_2$O, among others (ZnO, Pb$_2$O, Cd$_2$O and so on) [14]. Coal fly ash has 0.013% Cu$_2$O, while coal bottom ash has 0.009% Cu$_2$O and the reference cement has 0.014% Cu$_2$O. However, the effect of the ZnO present in both types of ashes is diluted by the higher amount of Zn$^{2+}$ cation in the reference cement (0.033% ZnO) than the present in coal bottom ash and coal fly ash, 0.08% ZnO and 0.016% ZnO, respectively.

### 3.4 Soundness

Cement soundness increases from CEM II/A-V to CEM IV/A (V) cements. Thus, the more ash is used, the more soundness was found. Even more, the higher amount of bottom ash in the cement (from β to Ω), the more soundness was also found (Figure 10). Nevertheless, all the values were lower than 1.75 mm.
3.5 Durability

Since CEM II/A-V can be considered the typical cement produced in Spain, it has been chosen to show qualitatively the durability performance of bottom ash (Ω) cements and fly ash (β) cements in comparison to the reference cement, CEM I 42.5 N (α) (Figure 11).

Ground coal bottom ash mortars showed a worse carbonation resistance than the reference mortars, but similar to that of fly ash mortars [15].

Frost-thaw resistance tests after 28 and 42 cycles showed that coal ash mortars were less resistant than the reference mortars without any addition. Nevertheless, coal bottom ash mortars showed a better performance than coal fly ash mortars [16].

Finally, it can be highlighted that alkali-silica reaction and sulfate resistance were the only two durable properties considered in this research program in which a clear improvement was evidenced [16].

In general, it can be said that coal bottom ash mortars showed a similar, and sometimes better, durable performance than coal fly ash mortars.

![Figure 11: Durability performance of bottom ash (Ω) cements and fly ash (β) cements in comparison to the reference cement, CEM I 42.5 N (α).](image)

4 CONCLUSION

It has been stated that coal bottom ash (CBA) chemical composition is quite similar to that of the coal fly ash (CFA) supplied by the same electrical power plant.

Also, coal bottom ash cementitious materials tested in the research program showed mechanical strength and durability performance (alkali-silica reaction, sulfate resistance carbonation resistance, natural carbonation, and frost-thaw resistance) similar to that of coal fly ash mortars and concretes. Therefore, coal bottom ash alone or mixed with coal fly ash is suitable to be used in cement production. In consequence, the incorporation of coal bottom ash as a new main constituent in the Portland cement standards is proposed.
ACKNOWLEDGEMENTS

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CONCRETE AS A RADON BARRIER AND ITS CHARACTERIZATION

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Key words: Concrete, diffusion, radon, oxygen, permeability.

ABSTRACT

Radon gas is a broadly known cause for lung cancer. Radon mainly enters buildings by convection though cracks and by diffusion though the buildings envelope material mass.

There are several ways of preventing this, such as sealing cracks or sub slab ventilation, among which we can outline using concrete as a radon barrier.

Concrete is a porous material which due to its diffusion coefficient generally allows an amount of gas to be diffused though its pores. Radon diffusion coefficient can vary in concretes commonly used in buildings from $10^{-6}$ to $10^{-11}$ m$^2$/s. Because concrete is used in relatively quite thick layers to build building envelopes, around 30 cm, diffusion through its mass can be controlled.

However determination of radon diffusion coefficient in concrete is not a simple task. There is not a general standard which determines the test procedure, radon sources are not that inexpensive nor easy to acquire alpha radiation sensors to be able to detect radon gas.

Other tests or alternative parameters could be used, such as non-radioactive gases diffusion coefficient tests or even permeability coefficients. All these parameters are dependant of materials porosity and moisture content. There are already proposed generic correlations in scientific literature for both relationships.

This paper shows the preliminary results of a comprehensive research that has been carried out to analyse these generic correlations and to propose specific ones. A diffusion chamber has been designed. Two types of concrete mixes have been studied, with two different curing processes and three humidity contents. Each case has been tested to get oxygen diffusion and permeability coefficients. The obtained oxygen diffusion coefficients with the designed diffusion chamber are similar to those presented in literature.

1.- INTRODUCTION

Radon gas (Rn) is a natural radioactive gas present in some soils and able to penetrate buildings through the building envelope in contact with the soil. Radon can accumulate within buildings and consequently be inhaled by their occupants. Because it is a radioactive gas, its disintegration process produces alpha particles that, in contact with the lung epithelia, can produce alterations potentially giving rise to cancer. Many international organizations related to health protection, such as WHO$^1$ confirm this causality.

One way to avoid the accumulation of radon in buildings is to use the building envelope as a radon barrier. The extent to which concrete provides such a barrier is described by its radon diffusion coefficient ($D_{Ra}$).
The measurement of the radon diffusion coefficient presents challenges, due to the absence of standard procedures, the requirement to establish adequate air tightness in testing apparatus (referred to here as the diffusion cell), and due to the fact that measurement has to be carried out in an environment certified for use of radon calibrated sources. In addition to this calibrated radon sources are costly.

The measurement of the diffusion coefficient for non-radioactive gas is less complex, but nevertheless retains a degree of difficulty due to the need to provide reliably airtight apparatus for all tests.

Other parameters that can characterize and describe the process of gas transport through concrete include the permeability coefficient (K), which can be measured relatively easily with standardized procedure. The use of this parameter would simplify the characterization of concrete behaviour as a radon barrier. Although earlier studies exist, describing correlation among these parameters, there is, as has been observed in the literature, little common ground between the various research efforts. For precisely this reason, prior to any attempt to measure radon diffusion, it was deemed necessary to carry out further research in this area, as a foundation to the current work, to explore potential relationships among the following parameters: oxygen diffusion coefficient and permeability coefficient.

2.- THEORETICAL BASE

Theoretical base is provided firstly on the relationship between radon diffusion coefficient and that of another gas (oxygen) and secondly on the correlation between diffusion and permeability coefficients.

2.1.-Relation between radon diffusion coefficient and other gases

Diffusion coefficient of a gas D can be expressed in the following way:

\[ D = D_0 \cdot \varepsilon^\tau \]  

Where:

- \( D_0 \): diffusion coefficient in air \([\text{m}^2/\text{s}]\),
- \( \varepsilon \): porosity of the media,
- \( \tau \): tortuosity factor.

This is an expression that can be derived from Archie’s law and one of Einstein’s laws \(^2\), which are respectively the following ones:

\[ \rho = \rho_0 \cdot \varepsilon^\tau \] 

Where:

- \( \rho \): electrical resistivity of a porous material \([\Omega \cdot \text{m}]\),
- \( \rho_0 \): electrical resistivity of water \([\Omega \cdot \text{m}]\), approximately 0.5.

\[ D_\varepsilon = \frac{k}{\varepsilon} \]  

Where:

- \( \rho \): electrical resistivity of a water saturated porous material \([\Omega \cdot \text{cm}]\),
- \( D \): diffusion coefficient \([\text{cm}^2/\text{s}]\),
- \( k \): constant.
This means that a gas diffusion coefficient depends on media properties, such as tortuous porosity $\varepsilon^T$, as well as on properties of the studied gas, such as its diffusion coefficient in air.

This means that theoretically from the experimental data of a gas diffusion coefficient it should be possible to derive the tortuous porosity and therefore another gas (such as radon) diffusion coefficient.

2.2.- Relation between diffusion and permeability coefficients

As it has been mentioned above other parameter that can characterize and describe the process of gas transport through concrete is the permeability coefficient (K), which can be measured relatively easily with standardized procedure.

Some authors identify a relationship between diffusion and permeability coefficients:

$$k = a \cdot D^n$$

(4)

Where:

$a$: $A/(8\pi D_{020})$,

$A$: sample cross-section,

$D_{020}$: diffusion coefficient in air (m$^2$/s).

Other studies experimentally relate both coefficients of different types of concrete confirming that this relationship exists, depending on microstructure of pores, as represented by the simplified expression:

$$k \approx D^n$$

(5)

In each particular study a different value for $n$ was established, varying from 1.3 to 2.5. These proposed models do not allow estimating diffusion coefficient directly from permeability coefficient because they require diffusion coefficient to be measured to be able to establish $n$. Therefore a new model is required that allows the determination of a value for $n$ in a more general way.

3.- METHODS AND MATERIALS

The proposed method is based on experimentally obtaining permeability and diffusion coefficient of oxygen in several concrete samples with different composition, humidity contents, w/c ratios and curing processes; and finding the relationships between them that can be used for the analytic estimation of oxygen diffusion coefficient from permeability coefficient. Porosity has been obtained as well, being perceived as the most important inherent parameter related to gas transport.

Parameters have been measured using the standardized procedures or purpose-built in the laboratory for the study of equations 4 and 5. Oxygen permeability coefficient has been experimentally obtained following the method described UNE 83981, porosity according to the Mercury Injection Porosimetry (MIP) and diffusion coefficients using the purpose-made diffusion cell and methodology. MIP test provides information on the global porosity $\varepsilon$.

3.1.- Diffusion coefficient

The experiments to obtain diffusion coefficients have been carried out using an airtight cell in which the concrete sample is placed at the open end of the chamber sealing the joint between sample and cell. The chamber is ventilated with nitrogen until a null oxygen concentration is achieved and then the evolution of oxygen concentration within the chamber is monitored, which increases because of oxygen diffusion from the air through the sample.
Several airtightness tests have been carried out. Firstly the airtightness of the whole cell is tested. In order to do this the chamber is closed, ventilated with nitrogen until a null oxygen concentration is achieved and then the evolution of oxygen concentration within the chamber is monitored. Results of this monitoring show that oxygen concentration within the chamber does not increase. Therefore it can be derived that the cell is airtight. Secondly the airtightness between the sample and the cell is tested and assessed. A diffusion test is performed with a capped sample (with double layer of aluminium tape). Results of the monitoring show that a small amount of oxygen is able to enter the cell. The air ventilation rate is calculated and compared to the limit value 3,78\times10^{-3}\text{ h}^{-1} established in ISO 11665-10 draft. The calculated ventilation rate is smaller than the limit value, so the airtightness is considered adequate and the leak dismissed.

### 3.2. Study cases

Two concrete types are studied varying composition and w/c ratio, aspects which have determinant influence on concrete porosity. These two concrete types have been chosen to have a range of the concrete types regularly used in Spanish construction and civil works. From each concrete type two kinds of curing process are studied, because this is another aspect that has a strong influence on porosity: curing in humid chamber (98% RH) during 2 days and the rest of time until 28 days at laboratory environment, and curing in humid chamber during 28 days. The concrete samples are cylinders 75 mm diameter (0.0044 m² area) and 50 mm high.

![Figure 1. Measurement equipment](image)

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Cement [kg/m³]</th>
<th>Cement type</th>
<th>Water [kg/m³]</th>
<th>W/C ratio</th>
<th>Gravel (4/11) [kg/m³]</th>
<th>Sand (0/4) [kg/m³]</th>
<th>Additive* [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>300</td>
<td>CEMI 42,5 R</td>
<td>165</td>
<td>0,55</td>
<td>928</td>
<td>994</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>400</td>
<td>CEMI 42,5 R</td>
<td>166</td>
<td>0,39</td>
<td>886</td>
<td>957</td>
<td>2</td>
</tr>
</tbody>
</table>

*Rebouilt 1000

In addition to composition, w/c ratio and curing process, water content in pores has as well a big influence on gas transport. Therefore three relative humidity are studied: approximately 50% (laboratory environment), 75% and 83% RH. The samples have been conditioned following the
natural method specified in UNE 83966 for both 75% and 83%, replacing NaCl for KCl in this last case.

4.- RESULTS

Results include experimental values of oxygen diffusion and permeability coefficients, as well as porosity of studied types of concrete and water contents.

4.1.- Porosity and related parameters

The obtained global porosity values are represented in the next figure for the four studied types of concrete.

![Figure 2. Global porosity (%)](image)

As expected, porosity is bigger for concrete with less cement, bigger w/c ratio and a shorter curing in the humid chamber. A better curing process (longer time in the humid chamber) reduces porosity, as well as a lower w/c ratio and a bigger amount of cement.

4.2.- Permeability coefficients

Oxygen permeability coefficients are included in the next table.

<table>
<thead>
<tr>
<th>Concrete type (kg/m³-W/C-curing)</th>
<th>Relative humidity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>300-0,55-2 days</td>
<td>6.73E-19</td>
</tr>
<tr>
<td>300-0,55-28 days</td>
<td>2.35E-19</td>
</tr>
<tr>
<td>400-0,39-2 days</td>
<td>3.43E-19</td>
</tr>
<tr>
<td>400-0,39-28 days</td>
<td>2.19E-19</td>
</tr>
</tbody>
</table>
4.3.- Diffusion coefficients

The proposed methodology for measuring gas diffusion coefficient provides values for the oxygen diffusion coefficients that are within the range of those proposed by the literature (10^{-7} to 10^{-8} m^2/s) and are consistent with the other studied parameters.

Table 3. Oxygen diffusion coefficients (m^2/s).

<table>
<thead>
<tr>
<th>Concrete type (kg/m^3-W/C-curing)</th>
<th>Relative humidity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>300-0.55-2 days</td>
<td>1.95E-07</td>
</tr>
<tr>
<td>300-0.55-28 days</td>
<td>4.85E-08</td>
</tr>
<tr>
<td>400-0.39-2 days</td>
<td>1.02E-07</td>
</tr>
<tr>
<td>400-0.39-28 days</td>
<td>4.05E-08</td>
</tr>
</tbody>
</table>

5.- DISCUSSION

The objective of the study is to provide a model for the calculation of the oxygen diffusion coefficient (D_{O2}) from other concrete parameters of easier experimental determination. Those parameters can be the permeability coefficient (k) and the concrete porosity (\( \varepsilon \)).

One way to obtain the D_{O2} from present results from the k value is through the calculation of the \( n \) exponent in the equations (4) and (5). Once \( n \) is obtained, through inverse analysis D_{O2} can be obtained for Ordinary Portland Cement concrete in any condition.

As it has been said above \( a \) is a constant (see eq. 4) which in our case study (with A=0.0044 m^2 and D_{O20}=2E-05 m^2/s) takes the value of 439453.12.

From the experimental values of k and D_{O2}, a particular value of \( n \) has been obtained for each type of concrete with its specific global porosity. With these \( n \) values that correspond to each defined porosity, regression analysis has been carried out to provide an expression relating \( n \) to the porosity.

Finally, with this expression of \( n \), it has been proposed a general expression that allows obtaining analytically D_{O2} (via k=D^0 or k=a·D^b) from experimental values of \( \varepsilon \) and k.

5.1.- Values of \( n \) obtained from experimental values of diffusion and permeability coefficients

From the relations (4) and (5) and the experimental values of permeability and diffusion coefficients the following \( n \) values have been obtained for the different types of concrete (cement amount, w/c ratio and curing process) and humidity contents.
In both cases $n$ values are quite consistent for every relative humidity and type of concrete.

5.2.- Relation between $n$ and porosity

With both equations (4) and (5) several regression analyses have been carried out to obtain an expression that allows obtaining $n$ from the porosity.

Taking into account that 50% RH conditioning is the easiest to achieve because is done at laboratory conditions the analyses have been carried out only for this water content.

The results are given in expressions 6 and 7 and are shown in figures 5 and 6:

with (4): $n=0.33L^ɛ+4.22$ \hspace{1cm} R$^2=0.94$ (6)

with (5): $n=0.23L^ɛ+3.18$ \hspace{1cm} R$^2=0.93$ (7)
5.3.- Expressions to estimate oxygen diffusion coefficient

Relations (4) and (5) can be expressed as:

from (4): \[ D = e^{\frac{LK}{n}} \] \hspace{1cm} (8)
from (5): \[ D = e^{\frac{L(K/a)}{n}} \] \hspace{1cm} (9)

Therefore diffusion coefficient can be obtained from relations (8) and (9) with experimentally obtained permeability coefficient and using the estimated \( n \) from the expressions (6) y (7).

Expressions (8) and (9), replacing \( n \) with the general expressions (6) and (7), turn respectively into:

\[ D = e^{0.338 \frac{L(K/c)}{LD}} \] \hspace{1cm} (10)
\[ D = e^{0.2373 \frac{LK}{LD}} \] \hspace{1cm} (11)
They enable the diffusion coefficients to be directly estimated from the experimental permeability coefficients and porosity.

Figures 7 and 8 show how close estimated diffusion coefficients are to experimental values.

![Figure 7. Estimation of $D_{O_2}$ from global porosity using $K=a \cdot D^n$.](image)

![Figure 8. Estimation of $D_{O_2}$ from global porosity using $K=D^n$.](image)

5.- CONCLUSIONS

At a practical level many questions are still open in relation to experimental measurement of gas diffusion coefficient. Among them it can be outlined the way of providing air tightness to the diffusion cell and to the joint between the cell and the studied sample.

The conclusions of the study are:

1. An arrangement for measuring gas diffusion coefficient in concrete has been proposed that provides values for the oxygen diffusion coefficients that are within the range of those proposed in the literature ($10^{-7}$ a $10^{-8}$ m$^2$/s).
2. A gas diffusion cell has been designed and manufactured that, after several adjusts and tests, has proofed to be air tight.

3. Two expressions are proposed to estimate oxygen diffusion coefficients in Ordinary Portland Cement concrete directly from permeability coefficients and porosity:

\[
D = \frac{\text{LNK}}{0.0896 \cdot \text{LNK} + 4.229} \quad \text{or} \quad D = \frac{\text{LNK}}{0.2973 \cdot \text{LNK} + 8.187}
\]

6.- REFERENCES


CORROSION PROTECTION EVALUATION OF GALVANIZED STEEL REINFORCED CONCRETE FOR SERVICE LIFE EXTENSION IN CHLORIDE AGGRESSIVE ENVIRONMENTS

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Key words: galvanized steel, concrete, corrosion, service life

Abstract. In the present research, concrete with 100% ordinary Portland cement (OPC) and concrete with blended cement containing blast furnace slags and limestone filler (SL) have been used to fabricate specimens embedding galvanized rebars. After the chloride exposure of the samples, the galvanized coating thickness loss was theoretically estimated through corrosion rate measurements and the real coating loss was identified by microscopic characterization. The theoretical galvanized coating thickness loss was higher for the SL concrete and the microscopic characterization confirmed this prevision. Local attacks generally happened at the rebars surface but the study of the initial and residual thickness in several zones of the rebars showed that coating thickness loss is not significantly affected by the rebar zone. The study of the galvanized coating destruction performed is intended to be a useful contribution for the service life prediction of galvanized reinforced concrete structures.

1 INTRODUCTION

Durability loss of concrete structures is mainly questioned due to the risk of the corrosion of steel reinforcement in aggressive environments with further influence on the service life performance of the structures. Hot-dip galvanizing of the steel reinforcement is a competitive solution to enlarge the service life if the risk of concrete carbonation exists or the chlorides can reach the rebar level during service life [1, 2]. Galvanized steel rebars are used as a protection method to prevent corrosion in reinforced concrete (RC) structures exposed to aggressive environments such as marine. The galvanized coating of the rebars acts as a sacrificial coating of the bare steel delaying the corrosion onset of the reinforcement and increasing in this way the service life of the structure.

The interaction between galvanized steel and concrete has been studied along the last decades by different authors and all of them agree that when zinc is passivated in the alkaline environment of the concrete a protective layer is produced on the surface of the galvanized...
coating. This passive layer mainly based on calcium hydroxyzincate (Ca\,[Zn(OH)\textsubscript{3}]\textsubscript{2}\cdot2H\textsubscript{2}O) separates the galvanized coating from the alkaline concrete and also provides protection against exterior agents like chlorides [2-4].

The effect of chloride ions on the corrosion performance of the galvanized rebars has been studied both in chloride-containing alkaline solutions [3-5] and in mortars or concrete matrix [1, 2, 6-8]. Researchers agree that, even though chloride ions promote a localized attack [3, 4] the galvanized coating resists a higher amount of chlorides than bare steel [2]. As a result of a compilation of the investigations on the chloride corrosion of galvanized steel reinforcements until 2004, it was reported [2] that the chloride threshold varied between 5 and 6 times higher than for bare steel in alkaline solutions and between 2 and 10 times in the actual concrete environment. Bellezze [6] carried out a study with 3 different types of galvanized steel and found a range of chloride thresholds between 1.4 to 4% by weight of cement, between 3 to 10 times higher than those reported for bare steel rebars. Due to this fact, the time for the onset of corrosion of the galvanized steel is 3-5 times longer with respect to the bare steel [2], and therefore hot-dip galvanizing of the steel reinforcement becomes a competitive solution to enlarge the service life of RC structures when there is risk that chlorides reach the rebar level.

As mentioned above the galvanized coating corrodes in presence of chlorides and a local attack of the galvanized coating is produced [3,4], so that the extension of the life time extends since the moment that a pit corrosion of external Zinc layer of the galvanized coating is initiated until the bare steel corrosion takes place. Although this model described in [2] for the service life description of the galvanized coating is accurate, there is a lack of information on the quantification of this protection period, what limits the beneficial contribution of galvanized reinforcement to deal with more sustainable concrete structures.

Besides of the known galvanized coating protection capacity, the quantification of the coating loss in aggressive environments has not been paid special attention. The knowledge of the galvanized coating loss process could be useful to better understand the behavior against corrosion of the galvanized steel in concrete and to predict a more precise service life for the RC structures containing this corrosion protection method.

In the present paper, a quantification of the galvanized coating thickness loss due to the penetration of chlorides through ordinary concrete and concrete containing high amount of mineral additions has been carried out.

2 EXPERIMENTAL PROCEDURE

The present study was performed with 12 mm diameter galvanized BS 500 SD steel rebars. The galvanized coating was reached through the hot-dip galvanizing method, i.e. the immersion of the deformed bars in a hot bath of Zn (99% purity) at 450ºC during 1 minute approximately and posterior cooling at atmosphere temperature. The elastic limit and yield strength of the steel rebars were not modified by the galvanizing process as shown in [9].

For the elaboration of the concrete, 2 types of binders were employed: a conventional OPC, CEM I 42.5 R/SR, and a ternary mix prepared with 64% CEM I 42.5 R/SR, 30% blast furnace slags and 6% limestone filler (SL). The concrete dosages are shown in the Table 1. Both types of concrete presented a soft consistency in the Abrams cone test. The Table 2 shows the chemical composition of CEM I 42.5 R/SR, filler and BFS.
Table 1: Concrete dosages

<table>
<thead>
<tr>
<th>Component</th>
<th>OPC concrete</th>
<th>SL concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I 42.5R/SR (kg/m³)</td>
<td>387</td>
<td>247</td>
</tr>
<tr>
<td>Blast furnace slags (kg/m³)</td>
<td>-</td>
<td>117</td>
</tr>
<tr>
<td>Limestone filler (kg/m³)</td>
<td>-</td>
<td>23</td>
</tr>
<tr>
<td>6-12 mm aggregates (kg/m³)</td>
<td>883</td>
<td>883</td>
</tr>
<tr>
<td>0-6 mm aggregates (kg/m³)</td>
<td>850</td>
<td>850</td>
</tr>
<tr>
<td>w/c</td>
<td>0.56</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Table 2: CEM I 42.5 R/SR, limestone filler and blast furnace slags chemical composition (%)

<table>
<thead>
<tr>
<th>Component</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>Al₂O₃</th>
<th>CaO</th>
<th>Fe₂O₃</th>
<th>SiO₂</th>
<th>MgO</th>
<th>SO₃</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I 42.5 R/SR</td>
<td>0.18</td>
<td>0.34</td>
<td>4.68</td>
<td>60.30</td>
<td>5.08</td>
<td>17.40</td>
<td>1.78</td>
<td>3.17</td>
</tr>
<tr>
<td>Limestone filler</td>
<td>0.55</td>
<td>2.65</td>
<td>7.75</td>
<td>26.09</td>
<td>2.57</td>
<td>21.39</td>
<td>5.80</td>
<td>0.05</td>
</tr>
<tr>
<td>Blast furnace slags</td>
<td>0.42</td>
<td>0.49</td>
<td>10.39</td>
<td>45.50</td>
<td>0.29</td>
<td>36.57</td>
<td>7.46</td>
<td>0.05</td>
</tr>
</tbody>
</table>

For the corrosion tests, 10 cm side concrete cubes embedding 9 cm of a 12 mm diameter galvanized rebar in the center were fabricated. Then, the samples were cured in a wet chamber at 20±2 °C temperature and 95±5 % relative humidity during 1 month. After the curing, the cubes were cut by the surface through which the chloride ions would be forced to enter, so that the final concrete cover was 1.5 cm. The specimens were subjected to preconditioning by a drying process for 10 days in a climatic chamber at 20 ± 2 °C and 30 % RH in order to reduce the time for the arrival of Cl⁻ to the rebar level and allow an amount of approximately 1% Cl⁻ at 2-3 few mm from the rebar level. All the surfaces except that of 1.5 cm cover were painted with epoxy resin to avoid the chlorides entrance from these directions.

The specimens were immersed in a solution whose concentration was 1M NaCl up to a level of 8 cm (allowing 2 cm at the atmosphere). Throughout the test, the concentration of chlorides solution was periodically increased, 1M NaCl to 340 days, 2M to 530 days and 3M till the end of the test at 650 days. Figure 1 shows the arrangement of the samples under testing. At the end of the immersion time the concrete samples were broken, the galvanized rebars removed and samples of concrete near the rebars were taken to analyse the chlorides content after acid attack of the powdered concrete sample. A potentiometric titration method was employed with a selective chlorides electrode. The total chlorides content in the concrete at the rebar level was around 11.0±0.1% by weight of binder in OPC and 6.6±0.3% in SL concrete.

During the test the corrosion potential of the reinforcement was continuously measured and the corrosion rate ($i_{corr}$) was periodically determined through the measurement of the polarization resistance. Furthermore the electrochemical coating loss of galvanized steel has been estimated through the Faraday’s law.
The real damage of the galvanized coating was also evaluated. First of all, the rebars were observed with the stereoscope in order to localize zones that show the highest signs of local corrosion attack, as shown in Figure 2a, or eventually the presence of rust indicating the corrosion onset due to a complete galvanized coating loss. Then, a transversal cut was performed in the selected zone and the sample was embedded in resin, polished up to reach a mirror surface appearance as shown in Figure 2b and the main regions of rebar geometry, transversal rib top (Transv Rib), valley and longitudinal rib top (Long Rib), were examined under the metallographic microscope. In each zone, a minimum of 20 measurements were taken in order to carry out a reliable statistic treatment on the residual galvanized coating thickness consequence of the corrosive action of chloride ion, as shown in Figure 2c. In the cases in which a total loss of the galvanized coating was detected, the scanning electron microscopy (SEM) was also employed in order to observe the attack on the microstructure of the different Zn alloyed layers.

The initial coating thickness variation of the galvanized steel is shown in Figure 3 for the different critical zones of the ribbed reinforcement: longitudinal rib top, transversal rib top
and valley, identified in Figure 2b, presenting a normal distribution of average and standard deviation of \( N(221,17) \), \( N(207,23) \) and \( N(208,27) \) respectively.

![Figure 3: Normal probability distribution of the initial galvanized coating thickness in the different critical zones of the ribbed reinforcement: lateral rib top, transversal rib top and valley](image)

### 3 RESULTS AND DISCUSSION

The electrochemical corrosion response of galvanized steel reinforcement is analyzed in the Figure 4a that shows the \( \text{icorr} \) evolution of the 2 specimens for each concrete studied during the 650 days of exposure to the chlorides solution. The time of exposure is subdivided in 3 stages corresponding to the different concentrations of the NaCl solution. The over limit of passive state considered was associated to \( \text{icorr} \) values > 0.2 \( \mu \text{A/cm}^2 \). Values of \( \text{icorr} \) above this limit were not detected before 20 days in the SL concrete and delayed till 40 days in OPC concrete. Furthermore the \( \text{icorr} \) progressively increases in both concrete indicating that the corrosion of the galvanized steel was progressing since the Cl\(^-\) continued penetrating. It can also be observed in the figures clear differences of performance in function of the type of concrete surrounding the galvanized rebar. In the SL case, higher corrosion rates were measured all along the test once corrosion is initiated, suggesting that the interfacial zones between the two types of concrete and galvanized steel is different. Stand by periods of \( \text{icorr} \) are observed that could be due to different reasons, one is the chloride binding that would retard the penetration of chlorides stabilizing in this way the corrosion of the galvanized coating, but also the progressive corrosion of the galvanized layers, as could be deduced from the theoretical calculation of the galvanized coating thickness loss from \( \text{icorr} \) measurements and the Faraday’s law, as shown in the Figure 4b.
Figure 4: a) $i_{\text{corr}}$ evolution of the galvanized steel rebars embedded in the OPC and SL concrete during the chlorides solution exposure time, b) Electrochemical galvanized coating thickness loss of the galvanized rebars embedded in the OPC and SL concrete during the chlorides solution exposure time

The Figure 5 presents a comparison between the theoretical and real galvanized coating thickness loss. The real galvanized coating thickness loss when the rebars were embedded in OPC is slightly lower than the predicted value. A smaller thickness loss also was measured for the SL concrete but in this case a higher difference respect to the predicted value was detected. In general, it could be said that the theoretical estimation of the galvanized coating thickness loss could allow a reliable prediction of the service life of the structure.

Figure 5: Theoretical and real galvanized coating thickness loss
The real corrosion attack of the galvanized coating is given in Figure 6. This figure shows images of the different parts of the rebars surface geometry damaged as consequence of the chloride ions entrance through the concrete. For all zones considered, it can be observed a local attack of the galvanized coating resulting in a heterogeneous thickness of the residual coating. These images also confirm a higher galvanized coating loss for the reinforcement embedded in the SL concrete respect to OPC. There are zones where the coating is practically intact, zones where the external zinc layer has been consumed and even some few zones, for the SL case, where the galvanized coating has totally disappeared.

The residual galvanized coating thickness has been summarized and compiled in the Figure 7. A higher attack and as consequence a higher galvanized coating thickness loss is produced when the galvanized rebars are embedded in the SL concrete. It also can be observed in the Figure 7 that the galvanized coating thickness was smaller when the rebars were embedded in SL.

The Figure 8 shows an image taken with the SEM from the valley zone of a galvanized rebar embedded in SL concrete. It can be observed that the Zn-Fe alloyed sublayers still remain but the external zinc layer has been consumed in some zones what explain a faster coating loss for the zones without external Zn layer.

The Figure shows the variability of the residual coating thickness in the different parts of the rebar surface considered for both concrete and allows better appreciate if the galvanized coating thickness loss depends on the rebar zone. The study of the initial and residual thickness in several zones of the rebars showed that galvanized coating thickness loss is not significantly affected by the rebar zone.
Figure 7: Residual galvanized coating thickness

Figure 8: Detail of the residual galvanized coating in a valley zone of a galvanized rebar extracted from SL concrete: a) metallographic microscope (x10) and b) SEM (x150)

The durability of RC structures is affected by the use of mineral additions for the elaboration of more sustainable concrete not just when galvanized but also when naked rebars are embedded. For the case of non-galvanized rebars, the passivation might even not happen, as when low-pH cementitious materials are used [10].

It is important to notice the higher corrosion protection capacity of the galvanized rebars in relation to the naked rebars. A compilation of the literature existing on the chloride threshold values for black rebars made by Alonso et al [11] established this parameter as 1.35% and 0.5% by weight of binder based only on OPC and made also with mineral addition respectively. In this research, the total chlorides content in the concrete at the rebar level was
around 11.0±0.1% by weight of binder in OPC and 6.6±0.3% in SL concrete and no attack to steel based was still produced. This determines the benefits of using galvanized steel in aggressive environments.

It should be also noticeable the lower contain of Cl⁻ found in the SL case. This concrete delays the chlorides penetration [12] and it should compensate the more active corrosion produced in this concrete with respect to OPC concrete. Therefore, it is clear the contribution to the sustainability of the type of concrete containing mineral additions and embedding galvanized rebars studied in the present research.

5 CONCLUSIONS

The present research has allowed reaching the followings main conclusions:
- The theoretical prediction of the galvanized coating loss from the corrosion measurements might lead to determine an early end of the service life of a RC structure.
- Galvanized steel rebars suffer a higher coating thickness loss when embedded in the concrete with mineral additions but this surrounding delays the arrival of the chlorides to the rebar level.
- The diminution of galvanized coating thickness is constant for any zone studied along the rebar surface.

6 ACKNOWLEDGEMENTS

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ECO-MECHANICAL ANALYSIS OF TYRE-FIBER-REINFORCED CEMENT-BASED COMPOSITES


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Key words: Tyre-Fiber-Reinforced Concrete (TFRC), Fracture Toughness, Mechanical performances, Ecological Performances, Carbon Footprint, Sustainable Structures

Abstract. A comparison among different tyre-fiber-reinforced-concrete (TFRC) mixtures is performed in this paper. The results of an extensive experimental campaign carried out as part of the European funded Anagennisi Project are examined and discussed. The TFRC mix with the best mechanical and environmental performances was identified through the implementation of the so-called eco-mechanical analysis developed at the University of Turin. The main parameters taken into account to determine the Eco-mechanical index (EMI) of the examined TFRC mixes were: the crack width of ideal RC ties; the embodied energy; and the carbon footprint released by the manufacturing of the TFRC mixes. As a result, it can be concluded that recycled tyre-fibers can reduce the crack width of concrete structures during their service life, and therefore can successfully be used as alternative to manufactured fibers without penalizing structural performance. The use of a blend of tyres-fibers and industrial fibers seems to yield the best overall performance.

1 INTRODUCTION

In the case of concrete structures, 90% of total life-energy is necessary to create the material (i.e., mostly clinker), whereas only 10% is related to the product manufacturing, transportation and use1. Thus, an eco-design approach should be based on the selection of concrete components, with low embodied energy and low carbon footprint, and on the minimization of the concrete mass. The following strategies can be applied in order to achieve such targets:

- Material performance strategy, which is based on the reduction of clinker by reducing
the total amount of concrete, and thus the volume of structures. Obviously, the mechanical performances, such as the compressive strength $f_c$, need to be increased with respect to conventional concrete under the same loading demand.

- Material substitution strategy, which consists in substituting clinker with cementitious and/or pozzolanic mineral admixtures (e.g., fly ashes, silica fumes, etc.).
- The use of LCA (life cycle assessment), in order to select concrete components having the lower carbon footprint.

From a structural point of view, however, the application of the above-mentioned strategies can affect negatively in the mechanical performances of concrete. Indeed, high strength concrete can show very brittle behaviour, especially under compression. Conversely, if recycled aggregates with a more favourable carbon footprint are used in place of virgin aggregates, a dramatic reduction of compressive strength (even larger than 50%) can be observed.2

For these reasons, the environmental aspects must be considered within a set of other design aspects, such as strength and ductility of concrete. Accordingly, the best concrete mixture must show the highest Eco-Mechanical Index ($EMI$), in which both ecological and mechanical aspects are included:

$$EMI = M I / E I$$

where $MI =$ Mechanical Index; and $EI =$ Ecological Index.

For instance, Damineli et al.4 consider the total carbon dioxide emitted and the amount of binder necessary to deliver 1 MPa of compressive strength, as possible values of $EI$. However, in this case, and also in the paper by Flower and Sanjayan5, $MI$ comprises only the compressive strength of concrete, as the post-peak energy absorption (i.e., ductility) is neglected. In this way, the beneficial effects of fibers are not adequately evaluated. If the fiber volume fraction is lower than 1%, strength (and therefore $MI$) does not vary, whereas the ecological impact (i.e., $EI$) increases. As a result, the eco-mechanical performances of fiber-reinforced concrete (FRC) appears to be lower than that of plain concrete.

In a more comprehensive definition of $EMI$, material properties should be combined with structural performances at serviceability and ultimate limit states. In other words, the definition of $MI$ should be based not only on concrete strength, but also on the results of structural tests. However, such tests are much more complicated to be carried in standard laboratories, especially when full scale concrete members need to be investigated. Thus, it is desirable that not only strength but also other mechanical properties of concrete are taken into account. As serviceability and durability of concrete structures are associated with the fracture mechanism of concrete in tension, the work of fracture can be effectively used to evaluate $MI$. This is particularly true for reinforced concrete (RC) beams in bending, whose code rules require narrow crack widths.

2 EVALUATION OF ECOLOGICAL AND MECHANICAL INDEXES

According to fib - Guidelines for Green Concrete Structures, the most relevant ecological parameters are carbon footprint, embodied energy and water consumption used to produce a cubic meter of concrete. Consequently, the following equation can be used to compute $EI$: 

$$E I = \text{carbon footprint} + \text{embodied energy} + \text{water consumption}$$
EI = (α \cdot wc_α) \cdot (β \cdot wc_β) \cdot (γ \cdot wc_γ) \tag{2}

where \( \alpha \) = quantity of carbon dioxide (CO₂); \( \beta \) = quantity of embodied energy; and \( \gamma \) = volume of water. As the ecological performances are related to the local condition in the place of use\(^6\), three weighting coefficients \((wc_α, wc_β, wc_γ)\), which can be properly adjusted depending on water shortage, transportation, grabbing of raw materials, etc., are also introduced within Eq.(2). For instance, the longer the distance between concrete plant and building site, the higher the value of \( wc_γ \) due to the impact of transportation.

Moreover, Eq. (2) can be enriched by other available parameters, such as biodiversity, toxic substances, and resource depletion. However, in the evaluation of ecological performances only the carbon footprint and the embodied energy of cement and steel fibers are assumed to be relevant\(^6\). Such values, taken from fib\(^6\) and from the EcoInvent database\(^8\), are reported in Table 1. It must be noted that the procedure here introduced is suitable only for comparison among similar types of concrete, cast in the same place and having the same type and amount of aggregates, and not for detailed calculation.

Arguably, although shredding tyres and separating rubber and steel result in processing costs, it is acceptable to consider that recycled steel fibers have zero carbon and energy impacts. On the other hand it is important to bear in mind the impacts associated with the treatment of the product at the end of its life. In other words, assuming no environmental impact for the recycled steel fibers gives credit for the net CO₂ that is saved when a product is reused or recycled.

<table>
<thead>
<tr>
<th>Carbon footprint Embodied energy</th>
<th>Normal Portland Cement 0.83 4.73</th>
<th>Industrial Steel fibers 1.5 20.6</th>
<th>Recycled fibers 0 0</th>
</tr>
</thead>
<tbody>
<tr>
<td>kgCO₂/kg MJ/kg</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In accordance with fib\(^6\), both the material and the structural performances must be incorporated into the mechanical index. Thus \( MI \) can be estimated by means of the following equations:

\[
MI = m_{m} \cdot m_{p} \tag{3}
\]

where \( m_{m} \) = mechanical parameter of concrete material; and \( m_{p} \) = mechanical parameter of concrete structure. If the durability needs to be taken into account, \( m_{p} \) is in inverse proportion to the crack width measured at a pre-established external load. Indeed, in a reinforced concrete structure, permeability is mainly influenced by the presence of cracks in the tensile zones, rather than the concrete porosity. Such cracks are the results of the so-called “tension stiffening” interaction between concrete and steel. Both numerical and experimental results\(^9, 10\) show that the higher the fracture energy of concrete, the lower the crack width produced by tension-stiffening. In other words, the work of fracture in tension inevitably characterizes the durability performance of RC structures, and \( m_{m} \) can be considered equal to the work of fracture. Consequently, not only material tests (tensile and/or bending tests) are...
required to compute $M_I$, but also the measure of crack width in full-scale structures (e.g., RC ties or beams) needs to be examined.

In what follows, an application of the above-mentioned eco-mechanical procedures is performed. In particular, 12 SFRC mixtures tested within the Anagennisi project are examined in terms of their ecological and mechanical performance.

3 TESTS ON THREE-POINT BENDING NOTCHED FRC PRISMS

3.1 Anagennisi project

Anagennisi project is an EU-funded collaborative project led by experts at the University of Sheffield, in association with the European Tyre Recyclers Association. The aim of the project is to develop innovative solutions to reuse all tyre components in high value innovative concrete applications with reduced environmental impact.

Specimens manufactured using twelve different Steel Fiber Reinforced Concrete (SFRC) mixes (see Tab.2) were tested under three point bending, in a Shimadzu 300 kN AG-X plus series testing machine at the Heavy Structures laboratory of the University of Sheffield (UK).

The SFRC mixes were made with a normal strength concrete C25/30, reinforced with manufactured and/or reused steel fibers at different blends and dosages ranging from 0.38% to 0.57%. The performance of a typical concrete mix was evaluated using a 30 kg/m$^3$ fiber dosage (commonly used in practice for slabs on grade) and 45 kg/m$^3$ (used for suspended slabs), in various mixes.

Table 2: Composition and properties of the SFRC mixtures investigated

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>$R_{cm}$ [N/mm$^2$]</th>
<th>Fiber [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix A</td>
<td>43.9</td>
<td>0.38</td>
</tr>
<tr>
<td>Mix B</td>
<td>42.6</td>
<td>0.38</td>
</tr>
<tr>
<td>Mix C</td>
<td>44.3</td>
<td>0.38</td>
</tr>
<tr>
<td>Mix D</td>
<td>44.6</td>
<td>0.38</td>
</tr>
<tr>
<td>Mix E</td>
<td>41.8</td>
<td>0.38</td>
</tr>
<tr>
<td>Mix F</td>
<td>42.9</td>
<td>0.45</td>
</tr>
<tr>
<td>Mix G</td>
<td>42.9</td>
<td>0.57</td>
</tr>
<tr>
<td>Mix H</td>
<td>42.8</td>
<td>0.57</td>
</tr>
<tr>
<td>Mix I</td>
<td>50.3</td>
<td>0.57</td>
</tr>
<tr>
<td>Mix J</td>
<td>44.5</td>
<td>0.57</td>
</tr>
<tr>
<td>Mix CSI1</td>
<td>38.8</td>
<td>0.57</td>
</tr>
<tr>
<td>Mix CSI2</td>
<td>42.8</td>
<td>0.57</td>
</tr>
</tbody>
</table>

The tensile behaviour of the steel fiber-reinforced concrete was evaluated in terms of residual flexural tensile strength values determined from the load-crack mouth opening displacement curve or load-deflection curve obtained by applying a centre-point load on a simply supported notched prism.
3.2 Experimental results

The results of the three point bending tests\textsuperscript{12} for all specimens are shown in Fig. 1 in terms of bending stress and average mid-span deflection.

The behaviour of the stress-deflection curves is characterized by:
1) a linear elastic phase up to the first crack,
2) a concrete cracking phase with fiber debonding which results in the stress drop,
3) a phase during which the concrete main crack is developed, dominated by fiber pull-out\textsuperscript{13}.

Mix A (made with only industrial fibers) and Mix B (reinforced with both industrial and recycled fibers) exhibited the best flexural performance amongst the 30kg/m\textsuperscript{3} mixtures. Their flexural performance is very similar, therefore reused fibers can substitute part of the manufactured fibers without penalising the flexural behaviour.

When only reused fibers are used (Mix E), the concrete flexural behaviour deteriorates dramatically after first cracking. Whereas, for Mix I, Mix H and Mix CSI2, the synergetic effect of manufactured fibers and reused fibers appeared to enhance the flexural concrete behaviour.

4 MECHANICAL PERFORMANCES

The mechanical characterization of SFRC should be based on nominal properties in the post-cracking stage. The $\sigma$-$w$ response can be obtained through a uniaxial tensile test, or by an indirect method using three or four point bending tests on prism beam specimens, in conjunction with an inverse analysis that assumes some predefined deterministic relationship. Although a direct tensile test is the most reliable method for determining the residual properties of SFRC, such a test is very complex and requires specialized testing equipment.

As already mentioned above, three point bending tests were carried out on concrete prisms
and the method proposed by Rilem TC 162-TDF\textsuperscript{14}, and depicted in Fig.2, was adopted to estimate the $\sigma$-$\varepsilon$ relationship.

The mechanical response of the SFRC specimens was derived from the stress-strain diagram shown in Fig. 2, where:

\begin{align*}
\sigma_1 &= 0.7f_{\text{fcum,fl}}(1.6-d) \\
\sigma_2 &= 0.45f_{R,1}K_h \\
\sigma_3 &= 0.37f_{R,4}K_h \\
\varepsilon_1 &= \sigma_1/E_c \\
\varepsilon_2 &= \varepsilon_1 + 0.1\% \\
\varepsilon_3 &= 25\%
\end{align*}

where $K_h = \text{size factor assumed (equal to 1)}$; $f_{\text{fcum,fl}}$ is the mean concrete flexural tensile strength (equal to 4.33 N/mm\textsuperscript{2} for C25/30); $f_{R,1}$ and $f_{R,4}$ are the residual flexural tensile strength respectively at CMOD1 and at CMOD4; $\varepsilon_1 = 0.00010$; $\varepsilon_2 = 0.00110$; and $\varepsilon_3 = 0.025$.

Table 3 shows the values of the main parameters of the idealized bi-linear stress-strain diagrams depicted in Fig. 2.

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>$\sigma_1$</th>
<th>$\sigma_2$</th>
<th>$\sigma_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N/mm\textsuperscript{2}</td>
<td>N/mm\textsuperscript{2}</td>
<td>N/mm\textsuperscript{2}</td>
</tr>
<tr>
<td>Mix A</td>
<td>3.01</td>
<td>1.62</td>
<td>0.871</td>
</tr>
<tr>
<td>Mix B</td>
<td>3.01</td>
<td>1.67</td>
<td>0.933</td>
</tr>
<tr>
<td>Mix C</td>
<td>3.01</td>
<td>1.53</td>
<td>0.861</td>
</tr>
<tr>
<td>Mix D</td>
<td>3.01</td>
<td>1.41</td>
<td>0.792</td>
</tr>
<tr>
<td>Mix E</td>
<td>3.01</td>
<td>1.43</td>
<td>0.661</td>
</tr>
<tr>
<td>Mix F</td>
<td>3.01</td>
<td>1.69</td>
<td>1.217</td>
</tr>
<tr>
<td>Mix G</td>
<td>3.01</td>
<td>1.87</td>
<td>1.400</td>
</tr>
<tr>
<td>Mix H</td>
<td>3.01</td>
<td>1.91</td>
<td>1.447</td>
</tr>
<tr>
<td>Mix I</td>
<td>3.01</td>
<td>2.17</td>
<td>1.275</td>
</tr>
<tr>
<td>Mix J</td>
<td>3.01</td>
<td>2.05</td>
<td>1.073</td>
</tr>
<tr>
<td>Mix CSI1</td>
<td>3.01</td>
<td>1.68</td>
<td>1.186</td>
</tr>
<tr>
<td>Mix CSI2</td>
<td>3.01</td>
<td>2.10</td>
<td>1.483</td>
</tr>
</tbody>
</table>
As the cracking phenomenon of FRC in tension is ruled by the pullout mechanism of a single fiber from the cement-based matrix, the value for $L_C$ (length of the softening region) can be considered equal to the maximum length of the adopted fibers. Starting from the constitutive relationships recommended by Rilem TC 162-TDF, and multiplying the strain by the maximum length of the fiber, the post-peak stress ($\sigma$) – crack opening displacement ($w$) curves are derived for each concrete investigated herein. The $\sigma$–$w$ curves, are then normalized with respect to the tensile strength $f_{ct}$ and limited to the maximum crack width of 0.3 mm. As an example, Fig.3 shows the curve $\sigma_{/f_{ct}}$ – $w$ for the specimens manufactured with concrete Mix B.

![Figure 3: Normalized stress – crack opening displacement curve of Mix B](image)

The area $A_{Ft}$, delimited by the relationship up to 0.3 mm, is a function of the fracture process. If this area is multiplied by the strength $f_{ct}$, the work of fracture in tension $G_{Ft}$ can be obtained. The values of $G_{Ft}$, for the concrete mixes investigated in this paper are summarized in Table 4, along with the $A_{Ft}$ and the tensile strength $f_{ct}$. All mixes show a very ductile response, which can be attributed to the presence of steel fibers. Mix E, containing only tyre fibers, has the lowest work of fracture $G_{Ft}$.

The crack pattern of RC structures is related to the fracture mechanics of concrete in tension and to the bond-slip between steel and concrete. Both of these two mechanisms contribute to the so called tension-stiffening phenomenon, which is the structural contribution provided by the uncracked concrete between two consecutive cracks. To investigate the tension-stiffening, the crack pattern and hence the structural performances, the maximum crack width of idealised FRC ties is theoretically predicted by means of the block model introduced by Fantilli and Chiaia. It is applied to ideal RC ties depicted in Fig.4, and made with one of the 12 mixtures investigated in the Anagennisi Project (see Fig.5).

As Fig. 5 shows, fibers can reduce the crack width, under a given external load $N$. This aspect can be observed for all ties, even those manufactured from Mix E, which show a wider crack opening because of the lower strength of the concrete mix. The last column of Table 4 summarises the average maximum values of crack width estimated at incipient yielding of steel reinforcement for all specimens [i.e., $N = 80$ kN].
Table 4: Stress-strain values and residual tensile strengths

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>$f_{ct}$ (MPa)</th>
<th>$A_f$ (mm$^2$)</th>
<th>$G_{ft}$ (N/mm)</th>
<th>Maximum crack width at $N = 80$kN (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix A</td>
<td>2.79</td>
<td>0.177</td>
<td>0.49</td>
<td>0.102</td>
</tr>
<tr>
<td>Mix B</td>
<td>2.72</td>
<td>0.185</td>
<td>0.50</td>
<td>0.100</td>
</tr>
<tr>
<td>Mix C</td>
<td>2.82</td>
<td>0.167</td>
<td>0.47</td>
<td>0.110</td>
</tr>
<tr>
<td>Mix D</td>
<td>2.83</td>
<td>0.156</td>
<td>0.44</td>
<td>0.118</td>
</tr>
<tr>
<td>Mix E</td>
<td>2.68</td>
<td>0.146</td>
<td>0.39</td>
<td>0.118</td>
</tr>
<tr>
<td>Mix F</td>
<td>2.74</td>
<td>0.191</td>
<td>0.52</td>
<td>0.094</td>
</tr>
<tr>
<td>Mix G</td>
<td>0.207</td>
<td>0.207</td>
<td>0.57</td>
<td>0.080</td>
</tr>
<tr>
<td>Mix H</td>
<td>2.73</td>
<td>0.212</td>
<td>0.58</td>
<td>0.077</td>
</tr>
<tr>
<td>Mix I</td>
<td>3.13</td>
<td>0.207</td>
<td>0.65</td>
<td>0.079</td>
</tr>
<tr>
<td>Mix J</td>
<td>2.83</td>
<td>0.212</td>
<td>0.60</td>
<td>0.072</td>
</tr>
<tr>
<td>Mix CSI1</td>
<td>2.51</td>
<td>0.204</td>
<td>0.51</td>
<td>0.082</td>
</tr>
<tr>
<td>Mix CSI2</td>
<td>2.73</td>
<td>0.230</td>
<td>0.63</td>
<td>0.049</td>
</tr>
</tbody>
</table>

Figure 4: The geometrical properties of the reinforced concrete ties

Figure 5: Theoretical evaluation of the maximum crack width in FRC ties
5 PRACTICAL ASSESSMENT OF THE ECO-MECHANICAL PERFORMANCES

The environmental impact and the mechanical behaviour of the different FRCs are assessed through the evaluation of their Eco-Mechanical index (EMI), implementing the technique developed at Politecnico di Torino.

Based on the results of the three point bending tests performed as part of the Anagennisi project, the values of $EI$ and $MI$ for the related concrete mixes can be found (see Table 4). Eq.(2) (with $\gamma = 0$ and $wc_\alpha = wc_\beta = wc_\gamma = 1$) is used to evaluate $EI$, whereas $MI$ is computed through Eq.(3), in which $m_{ps}$ is the inverse of the maximum crack width of the FRC ties and $m_{pm} = GFt$.

When the ecological and mechanical indexes are known, Eq.(1) can be used to determine the value of $EMI$ for each of the mixtures (Fig.6). The eco-mechanical performances increase when tyres fibers are combined with a smaller amount of industrial fibers. Indeed, industrial fibers enhance the ecological impact but, on the other hand, if added to normal concrete they increase the mechanical performances.

Table 4: The ecological and mechanical indexes estimated for the FRC ties

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>$EI = \alpha \cdot \beta$</th>
<th>$MI = m_{ps} \cdot m_{pm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix A</td>
<td>294</td>
<td>2037</td>
<td>598878</td>
<td>9.774 $\cdot$ 0.494 = 4.827</td>
</tr>
<tr>
<td>Mix B</td>
<td>279</td>
<td>1831</td>
<td>510849</td>
<td>10.038 $\cdot$ 0.503 = 5.053</td>
</tr>
<tr>
<td>Mix C</td>
<td>272</td>
<td>1728</td>
<td>469152</td>
<td>9.090 $\cdot$ 0.470 = 4.273</td>
</tr>
<tr>
<td>Mix D</td>
<td>264</td>
<td>1625</td>
<td>429000</td>
<td>8.475 $\cdot$ 0.443 = 3.755</td>
</tr>
<tr>
<td>Mix E</td>
<td>249</td>
<td>1419</td>
<td>353331</td>
<td>8.497 $\cdot$ 0.392 = 3.331</td>
</tr>
<tr>
<td>Mix F</td>
<td>302</td>
<td>2140</td>
<td>645210</td>
<td>10.638 $\cdot$ 0.523 = 5.562</td>
</tr>
<tr>
<td>Mix G</td>
<td>317</td>
<td>2346</td>
<td>742509</td>
<td>12.425 $\cdot$ 0.566 = 7.038</td>
</tr>
<tr>
<td>Mix H</td>
<td>302</td>
<td>2140</td>
<td>645210</td>
<td>13.069 $\cdot$ 0.580 = 7.577</td>
</tr>
<tr>
<td>Mix I</td>
<td>283</td>
<td>1883</td>
<td>532277</td>
<td>12.615 $\cdot$ 0.648 = 8.169</td>
</tr>
<tr>
<td>Mix J</td>
<td>264</td>
<td>1625</td>
<td>429000</td>
<td>13.888 $\cdot$ 0.599 = 8.321</td>
</tr>
<tr>
<td>Mix CSI1</td>
<td>283</td>
<td>1883</td>
<td>532277</td>
<td>12.189 $\cdot$ 0.511 = 6.233</td>
</tr>
<tr>
<td>Mix CSI2</td>
<td>302</td>
<td>2140</td>
<td>645210</td>
<td>20.407 $\cdot$ 0.628 = 12.808</td>
</tr>
</tbody>
</table>

Figure 6: Eco-mechanical performances of FRC ties through the evaluation of $EMI$
The best eco-mechanical performances are shown by Mix J and Mix CSI2. In fact, the EMI of these mixes are remarkably higher than those of the other FRC mixes. The highest value of EMI exhibited by mix CSI2 is due to the highest strength (12.81 MPa), which is obtained with the same amount of manufactured fibers of Mix F, but with the addition of reused tyres fibers. By contrast, the considerably high EMI index of Mix J is primarily due to the very low environmental impact, as well as the high tensile strength (8.49 MPa). The low value of embodied energy is a result of the low content of manufactured fibers used in Mix J, which amounts to only 10 kg/m³.

In addition to the evaluation of EMI through Eqs. (1)-(3), it can be useful to report the values of MI and EI within the diagram depicted in Fig. 7. In this diagram, MI_{inf} is the lower bound value of mechanical performances, whereas the upper bound value of environmental impact is represented by EI_{sup}. Both of these bounds can take as those suggested by code rules, or imposed by tender requirements. Accordingly, four different zones can be detected within the non-dimensional diagram:

Zone 1: Low mechanical performances– Low ecological performances,
Zone 2: High mechanical performances– Low ecological performances,
Zone 3: High mechanical performances– High ecological performances,
Zone 4: Low mechanical performances– High ecological performances.

In the present work, the performance related to the Mix F are taken as reference (EI_{sup} = 6.5 \times 10^5 kgCO_2-MJ/m³, MI_{inf} = 5.56 MPa). By means of this mixture, the maximum crack width at yielding is lower than 0.1 mm (as prescribed by code rules) with the lowest amount of fibers. Accordingly, the performances of the FRC mixtures can be described by Fig. 8.

The worst results are obtained in the case of Mix A, Mix B, Mix C, Mix D and Mix E. They are located in the Zone 4, and do not satisfy the minimum mechanical requirements. If industrial steel fibers are added, mechanical performances can be improved, but the environmental impact increases. As a result, Mix G cannot be viable as it is characterised by a high EI. Only three mixtures (Mix J, Mix I, and Mix CSI1) clearly fall within the Zone 3 and, therefore, fulfilling both mechanical and ecological requirements. Mix H and Mix CSI2 are on the borderline between Zone 3 and Zone 2 because EI is more or less similar to the reference
value. Among the mixtures which fall within the Zone 3, the best performances can be measured by the distance from the point where $MI/MI_{inf} = EI_{sup}/EI = 1$. The larger this distance, the higher the performance of the concrete. Thus, Mix J is certainly the best concrete tailored within the Anagennisi project. It is also the cheapest concrete, because of the low content of industrial steel fibers and the large use of recycled fibers.

![Non-dimensional diagram for the eco-mechanical comparative analysis](image)

**Figure 8.** Non-dimensional diagram for the eco-mechanical comparative analysis ($EI_{sup} = 6.5 \cdot 10^5$ kgCO₂ MJ/m³, $MI_{inf} = 5.56$ MPa)

6 CONCLUSIONS

From the results of experimental and numerical analyses previously described, the following conclusions can be drawn:

- The non-dimensional diagram depicted in Fig. 7 can be effectively used to tailor concrete mixtures with the best ecological and mechanical performances;
- If manufactured and reused fibers are properly combined, a new class of cement-based composites, having the mechanical performances of a traditional concrete, but a reduced environmental impact and a better durability, can be developed;
- Recycled tyre-fibers reduce the crack width during the serviceability stage of concrete structures, and therefore can substitute the manufactured fibers without penalising the structural performances.
- A small volume of tyre-fibers (0.5% in volume) and 0.1% in volume of industrial fibers seem to be the best reinforcement. In fact, the synergetic effect of manufactured and recycled fibers enhances the concrete performances and, at the same time, ensures a low environmental impact.

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REFERENCES

Effect of incorporating Sugarcane Bagasse Ash (SCBA) in mortar to examine durability of sulfate attack

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Keywords: Sugarcane Bagasse Ash; Mortar; Concrete; Sulfate Attack; Durability; Strength

Abstract. Utilization of various supplementary cementitious materials significantly influences fresh and hardened properties of mortar. Sugarcane bagasse ash is a by-product from sugar industries and can be used as supplementary cementitious material in concrete. In the present work an experimental research was carried out to investigate the influence of residual sugarcane bagasse ashes in the mechanical properties of mortar and durability. The aim of this study is to optimize the quality of sugarcane bagasse ash (BA) and to evaluate sulfate resistance of mortars containing bagasse ash (BA). In this study, the optimum temperature and duration of burning were determined. Then various experiments were carried out to get sulfate resistance of mortars containing 0, 10, 15, 20, 25 and 30% bagasse ash to examine durability. Tests included measuring concrete samples weight loss and compressive strength loss for 6 months, and length change of mortar prisms in sulfate solution. Moreover, since exposure conditions significantly affect the resistance of cements to sulfate attack, plain and blended cement mortar specimens were stored in three different conditions of exposure to Na2So4 and MgSo4, separately and simultaneously. Results show that mortar containing BA had higher compressive strength at various ages compared to control samples. In addition, results show that BA as an artificial pozzolanic material enhanced the durability of BA mortar, and reduced the detrimental effects of sulfate attack.

1 INTRODUCTION

The dramatic increase in worldwide demand on cement is due to the unprecedented ratio of growth of world population in the last 100 years and the accompanied needs of
infrastructure and buildings. The worldwide cement production is estimated at 4 billion tons per year [1].

Recent concrete construction environment focuses more on using environmentally friendly concrete. It has been established that mineral admixtures can be successfully used as complementary materials in Portland cement based mortars. The concrete needs to achieve both high early strength and ultimate strength capacity to improve the construction process. Other than the mechanical properties of the concrete, the durability properties of the concrete have to be of high standards in order to sustain a longer life span. These mineral additions delay the penetration of ionic species through the concrete, thus preventing degradation of concrete due to sulphate ions [2].

Sustainability has been a critical issue over a decade. One of the most crucial global problems that have potential to endanger the sustainability of environment, is global warming. Greenhouse gases play important role in global climate changing. Among these gases, CO2 contributes about 65% of global climate changes. Additionally, cement production, and resulting emissions are expected to increase by 100% from the current level by 2020 [3]. In other words, CO2 emission has been a dangerous problem in the world due to the greenhouse effect. Major CO2 producing sectors, such as power generation, transportation, oil refining and manufacturing of steel and concrete are under pressure to adopt measures that would drastically reduce the global CO2 emission rate by 2030. Within the concrete industry, cement manufacturing is the main culprit [1]. Cement plants produces a large amount of greenhouse gas emissions, mostly CO2 and is one of the major consumers of natural resources. Each year, the concrete industry produces approximately 12 billion tons of concrete and uses about 1.6 billion tons of PC worldwide [4]. The production of cement is increasing about 3% annually [5].

The manufacturing process of cement results in the depletion of limited natural resources, consumption of excess energy and also leads to significant levels of carbon dioxide emission to the atmosphere. It is thus very important to effectively utilize alternative supplementary cementitious materials in concrete.

Sugarcane bagasse ash, is a byproduct of the sugar factories, after burning sugarcane bagasse which itself is produced after the extraction of all economical sugar from sugarcane. Recently Sugarcane bagasse ash has been tested in some parts of the world for its pozzolanic property and has been found to improve some of the properties of the paste, mortar and concrete, such as compressive strength and permeability in certain replacement percentages and fineness. Consequently, the concrete industry can be the ideal place for consumption of the large amounts of bagasse; by replacing cement with bagasse ash in concrete.

Growing development of agricultural and industrial productions has led to the problem of increasing of agriculture wastes. Every years, millions of tons of wastes are produced in the country which leads to increase of environmental pollutions and therefore causes damages to residents of that region. One of these substances is bagasse which is transported from factories as a waste after extracting canes of sugarcane and burned around sugarcane farms which leads to environmental pollutions. Thus, we can meet the cement demand in the future without increasing the production [6]. The pozzolanic property of sugarcane bagasse ash come from the silicate content of the ash. This silicate goes under a pozzolanic reaction with the hydration products of the cement and results in reduction of the free lime in the concrete [7].
Steel corrosion, and alkali silica reactions. High resistance against chlorides and sulphates is because of concrete permeability reduction due to the Pozzolanic reactions. The observations also showed that diffusion coefficient of chloride and sulfate ions are lower in concretes with pozzolan compared to normal concretes. In addition, Bagasse Ash, similar to other agricultural wastes, contains siliceous ash, which has some advantages. There is a large amount of silica in ash which reacts with lime and forms hydrated calcium silicate with adhesion properties. The obtained ash includes a maximum of 10% alkalis, a small amount of iron oxide and aluminium, calcium, magnesium, and traces the oxides of elements.

Bagasse ash has been a problem for the environment regarding its disposal. The most significant pollutant emitted from the boilers. Sometimes some auxiliary fuels, typical fuel, or natural gas may be used during startup of the boiler or when the moisture content of the bagasse is too high to support combustion, in such cases the emissions of SO2 and NOX will increase. Since preparing high quality Bagasse Ash with pozzolanic activity depends on burning condition and the possibility of controlling the temperature and time of burning, the results of performed researches on this material vary based on the burning type and used furnaces, and researchers try to control burning condition, in better way by designing modern furnaces [8]. Therefore, according to the above mentioned, bagasse ash should also be examined in Iran.

Concerning all the discussions mentioned above, the aim of this paper is to study the mechanical properties and durability of mortar containing bagasse ash against sulfate attack. The results are compared with the control mortar, i.e., mortar made using Portland cement as the sole cementitious material. This knowledge could be beneficial for using this type of waste product in concrete, especially regarding the durability and heat reduction in mass concrete.

## 2 LITERATURE REVIEW

Sugar Cane Bagasse (SCB) wastes are chosen as an ideal raw material in manufacturing new products because of its low fabricating costs and high quality green end material. It is ideal due to the fact that it is easily obtainable given the extensive sugar cane cultivation making its supply constant and stable. The associated costs of extraction, chemical modifications and/or other pre- treatments of SCB in the transformation process to ready-to-be used materials are potentially reduced as the complex processes are simplified by the mere usage of SCB. When appropriate modifications and manufacturing procedures are applied, SCB displays improved mechanical properties such as tensile strength, flexural strength, flexural modulus, hardness, and impact strength. SCB is also found to be easily treated and modified with chemicals besides blending well with other materials to form new types of composite materials. It also satisfies the greening requirements by being biodegradable, recyclable and reusable [8].

In the application of SCBA for cement replacement, researchers indicated that compressive strength value for cements containing 5%, 10%, 15%, and 20% SCBA increased and were...
higher compared to ordinary Portland cement (OPC) which may have been caused by the filler effect and the pozzolanic reaction between Ca(OH)$_2$ from cement hydration and reactive SiO2 from the SCBA. Reduction in compressive strength caused by dilution effect was observed in 15% and 20% SCBA. A diffraction peaks of ettringite, Ca(OH)$_2$ and C–S–H phases noticed in all samples obtained during hydration for (X-ray diffraction) XRD pattern of OPC and 10% SCBA admixtured cement at 7 days and 28 days. Samples of SCBA, indicated that when C–S–H increased (strength improved), peaks of Ca(OH)$_2$ diminished, showing that pozzolanic reaction occurred between Ca(OH)$_2$ and amorphous silica present in SCBA [9].

Pozzolanic reactivity and cementitious behavior of SCBA was reported through tests on blended cement mortars and pastes. It was found that a complete combustion of SCBA occurred at 800°C (8 h) while crystallization of minerals occurred at above 650°C. The most suitable burning condition was identified as 600°C (5 h). SCBA has low density and higher surface area which favors the pozzolanic reactivity of amorphous silica and other minerals as compared to ordinary Portland cement (OPC), with ash particles size of 28.9 µm. Flow values decreased as SCBA has higher specific surface area which required more water to wet the surface. All blended mortars (at 7, 28, and 91 days) had strength activity index (AI) higher than minimum requirement. Beginning from CS test (3 and 7 days) all blended mortar specimens have higher strength than control due to pozzolanic activity and relative fineness of SCBA. Therefore, SCBA blended mortar as 10–15% substitution of OPC with SCBA should be utilized to make stronger material than the control one. Hence, high quality masonry and concrete blocks with elevated strength can be manufactured [10].

Nuntachai Chusilp et al [11] investigated physical properties of concrete containing ground bagasse ash (BA) including compressive strength, water permeability, and heat evolution. The results showed that, at the age of 28 days, the concrete samples containing 10–30% ground bagasse ash by weight of binder had greater compressive strengths than the control concrete (concrete without ground bagasse ash), while the water permeability was lower than the control concrete. Concrete containing 20% ground bagasse ash had the highest compressive strength at 113% of the control concrete.

A few researches has been done on bagasse sulfate resistance. Nuntachai Chusilp, Chai Jaturapitakkul, and Kraiwood Kiattikomol [11] considered the effects of LOI of ground bagasse ash on the compressive strength and sulfate resistance of mortars. In this study, raw bagasse ash collected from the Thai sugar industry has a high loss on ignition (LOI) of ~20%. When ground and ignited at 550 °C for 45 min, the LOI was reduced to ~5%. These high and low LOI of ground bagasse ashes were blended in the ratios of 1:2 and 2:1 by weight to give ground bagasse ashes with LOIs of 10% and 15%, respectively. Each of these ground bagasse ashes was used to replace type I Portland cement at 10%, 20%, 30%, and 40% by weight of binder to cast mortar. The results showed that the development of compressive strengths of mortars containing ground bagasse ash with high LOI was slower than that of mortar containing ground bagasse ash with low LOI. However, at the later age, both types of ground ash mortars displayed similar compressive strengths. Mortars containing high LOI (~20%) of ground bagasse ash at 20% and 30% by weight of binder could produce higher compressive strengths than a control mortar after 28 and 90 days, respectively. Mortar bars containing ground bagasse ash at 10% showed a greater potential sulfate resistance and displayed a reduce expansion compared to a control mortar. However, mortar bars containing high LOI
(larger than 10%) of ground bagasse ashes showed greater deterioration from sulfate attack than the mortar bars containing low LOI (less than 10%) of ground bagasse ashes, especially at high replacement levels (30–40%).

The deterioration of concrete under sulfate attack occurs as a result of the reaction between sulfates and hydrated cement paste forming ettringite and gypsum [12]. The sources of sulfate, deterioration mechanism, classification of the attack, and methods of assessment are discussed in the following sections. Temperature can be one of the factors affecting sulfate attack by enhancing delayed ettringite formation in internal sulfate attack. The effect of temperature of sulfate solution on the resistance of mortars was investigated by Akoz et al. [13], by exposing mortars to sodium and magnesium sulfate solutions of temperatures 20 and 40°C. The results showed that raising temperature of sulfate solution did not accelerate the deterioration of mortars [13]. The pH of sulfate solutions is another major parameter that influences the severity of sulfate attack. It was found that the expansion of specimens exposed to sulfate solution is particularly sensitive to the pH of the solution; the aggressiveness of sulfate attack decreases with the increase of the pH [14].

3 EXPERIMENTAL PROGRAM

3.1 Materials

Type I ASTM Portland cement was used in experiments. Physical properties and chemical composition of cement bagasse ash used are given in Tables 1 and 2. According to the literature review of the optimization, Bagasse ash is generated at the temperature of 800 °C for half an hour.

<table>
<thead>
<tr>
<th>Table 1: Chemical analysis of cement and bagasse ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>composition</td>
</tr>
<tr>
<td>SiO$_2$</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
</tr>
<tr>
<td>CaO</td>
</tr>
<tr>
<td>SO$_3$</td>
</tr>
<tr>
<td>MgO</td>
</tr>
<tr>
<td>Na$_2$O</td>
</tr>
<tr>
<td>K$_2$O</td>
</tr>
<tr>
<td>P$_2$O$_5$</td>
</tr>
</tbody>
</table>
Table 2: Physical properties of cement and bagasse ash

<table>
<thead>
<tr>
<th>Physical tests</th>
<th>Portland cement (Type I ASTM), %</th>
<th>Bagasse ash, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (gr/cm³)</td>
<td>3.2</td>
<td>2.2</td>
</tr>
</tbody>
</table>

As mentioned in the previous chapter, the bagasse was burned in the furnace at temperature of 800°C, situated in Amirkabir University of Technology. The chemical analysis of the bagasse ash were identified by XRF as shown in Table 1. In addition, the results of XRD analysis presented that the silica of ash is completely amorphous and non-crystalline and there is no crystalline particles in the ash in Figure 1.
In this study fine aggregates were obtained from a local source in Tehran. Crushed gravel with a maximum size of 19 mm and sand were used as coarse and fine aggregates, respectively. The grading of the coarse and fine aggregates are presented in Figure 2. Physical properties of aggregates are shown in Table 3.

![Grading Curve](image)

**Figure 2:** Grading curve based on considerations of ISIRI Standard

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Density (gr/cm³)</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>2.58</td>
<td>1.8</td>
</tr>
<tr>
<td>Sand</td>
<td>2.55</td>
<td>1.6</td>
</tr>
</tbody>
</table>

The superplasticizer is an admixture. This material is a new type of water-reducing admixture which have more effects than aforementioned materials. These substances do not change the surface tension of water considerably and their prime function is in the distribution of cement particles (often after suspension of the cement particles in water). The effects of superplasticizer will be effective only for a short time (maybe 10 minutes) and after 30 to 90 minutes, its efficiency reduced as usual. The superplasticizer used in this study has the base of polycarboxylate ether and the containing aqueous solution salt of polycarboxylate polymer. This additive is opaque in cloud color and its density at 20 °C is 1.1 kg/m³ and contains less than 0.1% chloride ion in it. Furthermore, no function than reducing water. Domain of usage is about 0.2% to 1.2% by weight of cement consumption.

### 3.2 Mixture design

It should be mentioned that since the objective of this study was to evaluate long-term performance and durability of mortar and it is required to get reliable results in this period, the mixtures designs organized in order to assess durability of mortar in shorter period. Therefore,
the ratio of water to cementitious materials and cement content selected 0.45 and 420 kg/m³ in all designs, respectively. For this study, following parameters were chosen:

1. The cement content: 420 kg/m³ of concrete
2. The amount of sand: 930 kg/m³ of concrete
3. The amount of gravel: 772 kg/m³ of concrete
4. The water-cement ratio: 0.45

3.3 Samples casting

Mortar specimens with dimensions of 5 cm were made according to standard. All samples prepared by using superplasticizer mortar similar to Type I Portland cement mortar control specimen. The workability was kept between 100 and 120 mm by using the flow rate see (Figure 3). A total of 6 series of mortars were made. For each series, cubic specimens with 5 cm dimension was cast. Mortar samples prepared by using a vibrating table and after casting and numbering, put in the humidity room. The control specimen was removed from the mold after 24 hours, but this action happened slower for the control specimens which contain pozzolan to achieve enough strength and to prevent damage to the samples during the removal of the mold they were demold after 36 to 48 hours. All samples after demolding kept in saturated lime water solution at laboratory usually in temperature of 21 to 25 °C until the age of compressive strength test.

![Figure 3: Use table flow to determine and control the workability of mortar specimens](image)

In another series of experiments mortar specimens are made for evaluating expansion due to sodium sulphate and magnesium sulphate attacks, with replacement percentages of 10, 15, 20, 25 and 30 bagasse ash and cast into 2.5×2.5×28.5 prisms. The process of making and opening mold and curing mortar were similar to specimens which were made for compressive strength. In Table 4, dimensions of the specimens for each test, curing process and the standard tests are presented.

<table>
<thead>
<tr>
<th>Age of experiments</th>
<th>Specimens dimensions</th>
<th>Curing</th>
<th>standards</th>
<th>Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-7-28</td>
<td>5 cm cubic mold</td>
<td>Cured in saturated lime</td>
<td>ISIRI 393</td>
<td>Compressive Strength</td>
</tr>
</tbody>
</table>
### 3.4 Preparation and testing procedure

The compressive strength of the mortar can represent quality of hydration and pozzolanic reactions. It could be possible to establish a straight relationship between the compressive strength of mortar specimens and its quality. That is why this test was conducted on specimens of mortar. Since wet and dry specimens have some differences in compressive strength, all the tested tried to be conducted right after removing specimens from lime water saturated solution. In each age, three cubic specimens were tested from each mixture design. Since the presence of sulfate in concrete is a hazardous factor for concrete structures and sulfates from gypsum and ettringite by attacking to various hydration products. It causes many damages in concrete, therefore it is required to compare the performance of different designs of concrete and mortar against destructive reactions of sulfate in the durability analysis. One of the evaluating method and comparing the performance, is the ASTM C1012 standard. It should be noted that the results of this method is evident after 100 days.

<table>
<thead>
<tr>
<th>1-2-4-8-16 week</th>
<th>5 cm cubic mold</th>
<th>Cured in saturated lime water for 7 days</th>
<th>ASTM C1012</th>
<th>expansion of mortar prism in sulfate solution</th>
</tr>
</thead>
</table>

2.5 × 2.5 × 28.5 cm prism mold put in humidity container of laboratory after casting mortar and covered them with wet sacking. Specimens demolded after 36 to 48 hours, and were placed in a solution of saturated lime water. Specimens which contained pozzolan were removed after 48 hours and specimens were cured in water container till the test time. Since mortar specimens were kept in lime-saturated water solution until achieving 20 ± 1 MPa compressive strength, also some 5 cm cube specimens of mortar were made. After getting 20 ± 1 MPa compressive strength, specimens were placed in 5% sodium sulfate solution. The samples were kept in a closed container at room temperature. Mortar expansion was measured by using strain gauges weekly. To evaluate well, specimens were put in 10% sodium sulfate, 10% and magnesium sulfate and 5% mixed of two solutions.

Developed Standard has not been designed to evaluate mortar sulfate attack yet, researchers have proposed various methods to investigate the attack. Another difficulty in assessing the durability of mortar against sulfate attack is the duration of the experiment. Thus by applying some conditions it could be possible to see sulfate attack in the samples rapidly. It should be noted that mortar durability against sulfate attack cycle assessment under both wet and dry conditions could be different from the real environment.

5 cm cube specimens for mortar were cured in lime-saturated water solution until the age of 7 days. Then were placed in solution of 5% sodium sulfate, 5% magnesium sulfate and 5% combination of solution of sodium and magnesium sulfate. The weight of specimens before placing in solutions were measured. Also 7-day compressive strength of specimens was determined. Regimes which used in this study were selected as follows:
Regime 1: 6 days placing the specimens in 5% sodium sulfate solution and then dried in an oven at 80°C for 1 day.
Regime 2: 6 days of placing the specimens in a 5% magnesium sulfate solution and then dried in an oven at 80°C for 1 day.
Regime 3: 6 days of placing the specimens in a solution of combined magnesium-sodium sulfates at 5% and then dried in an oven at 80°C for 1 day.
This regimen was continued for 16 weeks and considering the number of specimens, it is possible to continue this test at longer ages.

4 RESULTS AND DISCUSSION

4.1 Compressive strength

Compressive strength test of mortar specimens was conducted at ages of 3, 7, 28, 56 and 91 days. The test results for various mixture designs at different ages are shown in Table 5 and Figure 4.
According to the results, for bagasse ash replacement up to 25%, compressive strength increased due to the pozzolanic reaction and reduction of capillary pores, and maximum increase in compressive strength at different ages was for mortar with 15% replacement of bagasse ash, which reaches the strength of 50.6 MPa after 91 days, and has greater compressive strength about 11% than that in the control specimen.

<table>
<thead>
<tr>
<th>Table 5: The results of the compressive strength of mortar specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compressive strength of mortar specimens at different ages (MPa)</strong></td>
</tr>
<tr>
<td>Mixture designs</td>
</tr>
<tr>
<td>CTRL</td>
</tr>
<tr>
<td>10 % SCBA</td>
</tr>
<tr>
<td>15 % SCBA</td>
</tr>
<tr>
<td>20 % SCBA</td>
</tr>
<tr>
<td>25 % SCBA</td>
</tr>
<tr>
<td>30 % SCBA</td>
</tr>
</tbody>
</table>
4.2 Expansion of mortar specimens

Tests conducted on mortar specimens in solutions of sodium sulfate and magnesium, 5% and 10% respectively, and 5% solution of combination of sodium sulfate and magnesium. The results shown in Figures 5 to 9 in 26 weeks based on the percentage change in length. In this experiment pH of sulfate solution maintained between 6 and 8, and the samples were kept at 23 ± 2 °C temperature. One of the factors contributing to sulfate attack and its intensity, is the concentration of sulfate ions. Based on the charts and tables, in any of the sulfate solution by increasing sulfate concentration from 5% to 10%, the change in length is increased due to the adding concentration of sulfate.

Between 5% solution of magnesium sulfate and sodium sulfate, magnesium had more destructive effects on the expansion of the specimens as expected. For example, after 26 weeks, specimens containing 10, 15, 20, 25 and 30% bagasse ash in a 5% solution of magnesium sulfate, have been change in length 0.106, 0.069, 0.035, 0.046, 0.086 and 0.120 percent, respectively. By considering diagrams carefully it have been found out that the damage and length expansion is more severe along with the increase in the amount of sodium sulfate. After the 26 weeks specimens containing 10, 15, 20, 25 and 30% bagasse ash in 10% sodium sulfate, compared to the 5% sodium sulfate solution, have been changed in their length 1.05, 1.05, 1.24, 1.09, 1.06, and 1.04 times, respectively. However, in similar situation in 10% magnesium sulfate, compared to the 5% sodium magnesium solution, have been changed in their length 1.04, 1.03, 1.06, 1.08, 1.03, and 1.02, respectively. The lowest expansion percentage in all solutions is for the replacement of 15% bagasse ash and then 20, 10, and 25 percent have the lowest percentage of change. And the expansion with replacement of 30% is
higher than the control specimen. Also, specimens which kept in dissolved mixture of 5% sodium sulfate and magnesium sulfate, have had the least change due to the low concentration of sodium sulfate.

Figure 5: Expansion of prismatic specimens in 5% sodium sulfate solution

Figure 6: Expansion of prismatic specimens in 5% magnesium sulfate solution
Figure 7: Expansion of prismatic specimens in 10% sodium sulfate solution

Figure 8: Expansion of prismatic specimens in 10% magnesium sulfate solution
4.3 Mortar durability in cycles of sulfate attack

Due to the long-term durability tests for sulfate attack, especially weight loss and compressive strength, it is necessary to develop an accelerated test. So, the mortar specimens were made and applied by three regimens which were described in previous sections, and the results are given in Tables 6 to 9 and Figures 10 to 12.

Reduction in the strength of mortar specimens applied by regime 1 and 3 for 16 weeks, are less than by regime 2 for mortar specimens. Also as results of the expansion of the mortar specimens, 5% sodium sulfate and magnesium sulfate had minimal damage effects, since sodium sulfate concentration is low and it reduces degradation.

In the presence of magnesium sulfate, calcium hydroxide turn into calcium sulfate and other production of this reaction such as calcium hydroxide, are not effective at increasing pH. For this reason, the formation of large amounts of calcium sulfate and magnesium hydroxide resulted in destabilizing the CSH and in the presence of sodium sulfate, calcium hydroxide turn into calcium sulfate and sodium hydroxide and this reaction increase pH.

In high sulfate concentrations and pH conditions, the possibility of transforming gypsum to Ettringite is existed and Ettringite structure is stable. Therefore it can be concluded that less changes in pH can be the reason of more degradation in magnesium sulfate solution.

Table 8 presents that the strength of control samples reached 19.2 MPa after 16 weeks, but specimens with 15% bagasse ash have 22.5 MPa compressive strength. This is due to pozzolanic reactions and reduce the permeability of mortar which reduces penetration of sulfate ions into the capillary pores of the mortar.

Test results in Tables 10 to 12 are shown the mortar weight experiments. It obtained that weight of control specimens have been reduced only after 1 month in a solution of sodium sulfate.
sulfate. Whereas for specimens which containing 10, 15 and 20% bagasse ash, weight reduction start after 2 months of in sulfate solution. This acceleration is caused by applied wet and dry cycles. The samples have been dried up in the oven then dissolved sulfate ions can enter the capillary specimens by placing them in a solution of sulfate and it expedites the process of degrading.

According to the above table, the mortar specimens with 10%, 15% and 20% replacement, had less weight than the control specimens. Minimum weight loss is for 15% replacement bagasse ash and maximum weight loss in all mix designs are for control specimens and samples which produced with 25% and 20% replacement bagasse ash. It should be noted that, the replacement of bagasse ash reduce C₃A and calcium hydroxide and produced less gypsum to Ettringite as a factor of sulfate attack.

Table 6: compressive strength of mortar specimens in lime water

<table>
<thead>
<tr>
<th>Mixture designs</th>
<th>1st week</th>
<th>2nd week</th>
<th>4th week</th>
<th>8th week</th>
<th>16th week</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>26.7</td>
<td>33</td>
<td>41.2</td>
<td>44.8</td>
<td>45.6</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>27.5</td>
<td>34.1</td>
<td>43.3</td>
<td>47.5</td>
<td>47.7</td>
</tr>
<tr>
<td>15% SCBA</td>
<td>28.4</td>
<td>35.2</td>
<td>44.2</td>
<td>49</td>
<td>51</td>
</tr>
<tr>
<td>20% SCBA</td>
<td>27.8</td>
<td>34.5</td>
<td>43.5</td>
<td>48</td>
<td>48.1</td>
</tr>
<tr>
<td>25% SCBA</td>
<td>25.6</td>
<td>21.9</td>
<td>40.1</td>
<td>43.7</td>
<td>44.5</td>
</tr>
<tr>
<td>30% SCBA</td>
<td>23.5</td>
<td>19.8</td>
<td>38</td>
<td>41.6</td>
<td>42.4</td>
</tr>
</tbody>
</table>

Table 7: Compressive strength of mortar specimens under regime 1
**Table 8:** Compressive strength of mortar specimens under regime 2

<table>
<thead>
<tr>
<th>Mixture designs</th>
<th>1st week</th>
<th>2nd week</th>
<th>4th week</th>
<th>8th week</th>
<th>16th week</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>26.7</td>
<td>32.8</td>
<td>30.5</td>
<td>29.2</td>
<td>23.8</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>27.5</td>
<td>32.3</td>
<td>34.1</td>
<td>30.2</td>
<td>25</td>
</tr>
<tr>
<td>15% SCBA</td>
<td>28.4</td>
<td>37.3</td>
<td>36.5</td>
<td>31</td>
<td>29</td>
</tr>
<tr>
<td>20% SCBA</td>
<td>27.8</td>
<td>35.1</td>
<td>34.9</td>
<td>31.2</td>
<td>27.2</td>
</tr>
<tr>
<td>25% SCBA</td>
<td>25.6</td>
<td>31.7</td>
<td>29.4</td>
<td>28.1</td>
<td>22.7</td>
</tr>
<tr>
<td>30% SCBA</td>
<td>23.5</td>
<td>29.6</td>
<td>27.3</td>
<td>26</td>
<td>20.6</td>
</tr>
</tbody>
</table>

**Table 9:** Compressive strength of mortar specimens under regime 3

<table>
<thead>
<tr>
<th>Mixture designs</th>
<th>1st week</th>
<th>2nd week</th>
<th>4th week</th>
<th>8th week</th>
<th>16th week</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>26.7</td>
<td>34.3</td>
<td>35</td>
<td>38.3</td>
<td>26.2</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>27.5</td>
<td>35.1</td>
<td>36.3</td>
<td>39</td>
<td>31.6</td>
</tr>
<tr>
<td>Replacement Level (%)</td>
<td>15% SCBA</td>
<td>20% SCBA</td>
<td>25% SCBA</td>
<td>30% SCBA</td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
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<td>----------</td>
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<td>----------</td>
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<td></td>
<td>28.4</td>
<td>27.8</td>
<td>25.6</td>
<td>23.5</td>
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<td></td>
<td>36.7</td>
<td>35.3</td>
<td>33.2</td>
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</tr>
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<td></td>
<td>39.1</td>
<td>37.8</td>
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<td>37.1</td>
<td>37.2</td>
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<td>37.4</td>
<td>33.5</td>
<td>25.1</td>
<td>23.0</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 10:** Compressive strength of mortar specimens under regime 1

**Figure 11:** Compressive strength of mortar specimens under regime 2
Figure 12: Compressive strength of mortar specimens under regime 3

Table 10: weight mortar specimens under regime 1

<table>
<thead>
<tr>
<th>Mix design</th>
<th>Initial</th>
<th>1st month</th>
<th>2nd month</th>
<th>3rd month</th>
<th>4th month</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>299.5</td>
<td>294</td>
<td>282</td>
<td>274</td>
<td>266</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>294</td>
<td>294</td>
<td>282</td>
<td>272.5</td>
<td>266.5</td>
</tr>
<tr>
<td>15% SCBA</td>
<td>275.5</td>
<td>276</td>
<td>272.5</td>
<td>264</td>
<td>258.5</td>
</tr>
<tr>
<td>20% SCBA</td>
<td>290</td>
<td>290</td>
<td>287</td>
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<tr>
<td>25% SCBA</td>
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<td>30% SCBA</td>
<td>269.5</td>
<td>264</td>
<td>252</td>
<td>244</td>
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</tbody>
</table>

Table 11: weight mortar specimens under regime 2

<table>
<thead>
<tr>
<th>Mix design</th>
<th>Initial</th>
<th>1st month</th>
<th>2nd month</th>
<th>3rd month</th>
<th>4th month</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>299.5</td>
<td>294</td>
<td>282</td>
<td>274</td>
<td>266</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>294</td>
<td>294</td>
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<td>272.5</td>
<td>266.5</td>
</tr>
<tr>
<td>15% SCBA</td>
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<td>272.5</td>
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<td>258.5</td>
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<tr>
<td>20% SCBA</td>
<td>290</td>
<td>290</td>
<td>287</td>
<td>272</td>
<td>269</td>
</tr>
<tr>
<td>25% SCBA</td>
<td>289.5</td>
<td>284</td>
<td>272</td>
<td>264</td>
<td>256</td>
</tr>
<tr>
<td>30% SCBA</td>
<td>269.5</td>
<td>264</td>
<td>252</td>
<td>244</td>
<td>236</td>
</tr>
</tbody>
</table>
Table 12: weight mortar specimens under regime 3

weight mortar specimens under regime 3 (gr)

<table>
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<tr>
<th>Mix design</th>
<th>Initial</th>
<th>1st month</th>
<th>2nd month</th>
<th>3rd month</th>
<th>4th month</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTRL</td>
<td>296</td>
<td>287</td>
<td>280.5</td>
<td>271.5</td>
<td>268.5</td>
</tr>
<tr>
<td>10% SCBA</td>
<td>287.5</td>
<td>284.5</td>
<td>281.5</td>
<td>275.5</td>
<td>269.5</td>
</tr>
<tr>
<td>15% SCBA</td>
<td>297.5</td>
<td>291.5</td>
<td>288.5</td>
<td>288.5</td>
<td>282</td>
</tr>
<tr>
<td>20% SCBA</td>
<td>291.5</td>
<td>285</td>
<td>282.5</td>
<td>279.5</td>
<td>273.5</td>
</tr>
<tr>
<td>25% SCBA</td>
<td>286</td>
<td>277</td>
<td>270.5</td>
<td>261.5</td>
<td>258.5</td>
</tr>
<tr>
<td>30% SCBA</td>
<td>266</td>
<td>257</td>
<td>250.5</td>
<td>241.5</td>
<td>238</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

The experimental analysis of basic physical characteristics, mechanical properties and durability characteristics of mortar containing bagasse ash as partial replacement of Portland cement. The main results can be summarized as follows:

1. In mortar specimens, for bagasse ash replacement level up to 25% the compressive strength increased, and maximum increase in compressive strength at different ages was for mortar with 15% replacement of bagasse ash.
2. In all specimens containing bagasse ash placed in 5% sulfate, compared to the control specimens, weight loss was reduced and bagasse ash specimens with 20%, had least weight loss than the control specimens.
3. Compressive strength reduction of mortar specimens are more than mortar specimens containing bagasse ash. Minimal reduction in compressive strength is for specimens
containing 20% bagasse ash and maximal reduction in compressive strength of control specimens.

4. Mortar specimens with bagasse replacement of 10, 15, 20 and 25 percent indicate less weight change than the control specimen and the lowest weight loss is belong to replacement of 15%. Highest weight loss are for the replacement of 30% bagasse ash and control specimens.

5. The lowest expansion percentage in all solutions is for the replacement of 15% bagasse ash and then 20, 10, and 25 percent have the lowest percentage of change. And the expansion with replacement of 30% was higher than the control specimen.

6. Based on the results of the expansion tests and ASTM C1157 standard:

   * Specimens containing 10% and 25% bagasse ash have less than 0.05 percent expansion and categorized as cement with high strength in sulfate attack.
   * Specimens containing 10% and 25% bagasse ash have less than 0.1 percent expansion and categorized as cement with medium strength in sulfate attack.
   * Specimens containing 30% bagasse ash were vulnerable to sulfate attack as control specimens.

7. In the mortar specimens, Specimens containing 15% bagasse ash have acceptable resistance to sulfate attack, and 30% bagasse ash are vulnerable to sulfate attack. While, in the concrete specimens, 20% bagasse ash have acceptable resistance to sulfate attack, and control specimens are vulnerable to sulfate attack.

### 6 SUGGESTIONS

1. To study the effects of bagasse ash incorporation into blended cements thoroughly, conducting other tests such as alkali-aggregate reaction, dry and wet cycles, freeze and thaw cycles, and microstructure test are recommended.

2. Besides mortar specimens, cement paste specimens can also be used to evaluate length expansion in the sulfate solutions. Since in this project both mortar and concrete specimens were placed in sulfate and according to the observations, it is expected that the harmful effects of sulfates in a shorter period of time can be seen on the mortar and cement paste.

3. It is recommended that in another project, the effect of temperature and pH of sulfate solutions in which specimens with different bagasse replacement percentage are placed there, should be evaluated.

### 7 ACKNOWLEDGEMENTS

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REFERENCES


EFFICIENCY OF CHLORIDE EXTRACTION FROM REINFORCED CONCRETE WITH INTERMITTENT APPLICATIONS
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2email: yokota@eng.hokudai.ac.jp, http://www.eng.hokudai.ac.jp/labo/lifetime

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Key words: Chloride removal, electrochemical chloride extraction, desalinization, intermittent application

Summary: Electrochemical chloride extraction has been used as a rehabilitation method to prevent the degradation of chloride-contaminated reinforced concrete structures for many years. Although many studies have been conducted and many existing reinforced concrete structures have been rehabilitated, the extraction technique has generally used as a continuous application. Very few studies can be found regarding the electrochemical chloride extraction with intermittent applications. Therefore, its efficiency and advantage are still unclear. The purpose of this research is to compare the efficiencies of chloride extraction between continuous and various intermittent applications. The results indicate that although the total applied charge number is much lower in intermittent applications compared to continuous application. There is no significant difference in the final efficiency of chloride extraction in these cases, especially in the zone near the concrete-reinforcement interface. However, the much lower of total charge number had been spent to remove the approximate quantity of chloride ions in the intermittent applications compared to the continuous application may offer a great advantage. Consequently, the intermittent applications may cause less severe impacts on concrete and the bond strength between concrete and reinforcement, which have been the main sources of arguments about this method. It is concluded that the intermittent application has adequate to rehabilitate the performance and prolong the lifetime of structure.

1 INTRODUCTION

It has been more than a half century since the severe effect of chloride ions on the corrosion progress of steel in reinforced concrete structures was disclosed [1-2]. That disclosure has been the main root for many topics relating to structural performance including durability of reinforced concrete structures since they exposed to chloride environments. Along with the durability issues, repair and strengthening methods for reinforced structures of which performance degraded have been invented thanks to the aging of the structures. Electrochemical chloride extraction (ECE) has been studied and widely applied as one of the options to rehabilitate deteriorated reinforced concrete structures which are contaminated.
chloride ions for approximately 20 years [3-14]. In despite the controversies [5-6], many studies have proved that ECE can mitigate corrosion progress of steel bars after the electrical current was stopped for some weeks [8-9]. Moreover, the re-passive condition may last for several years [8, 12] although the mechanism of re-passivation progress is still uncertain. However, the effects of ECE on concrete are gloomy. Several studies have figured out that the longer time of the electrical current is applied, which increases proportionally to the total charge number applied, the more drawbacks ECE can cause to the structure of concrete [3, 5-7]. It puts down a big question for the later degradation process of reinforced concrete structures after the rehabilitation by using ECE is stopped.

Consequently, intermittent application has been expected as a technique which can not only remove chloride ions, but also can suppress the drawbacks of ECE. However, while some studies shown very potential results, the others indicated the insignificant differences [15, 16]. Moreover, until now there are very few studies on the intermittent application. Thus, the advantages of intermittent application are still uncertain.

In addition, it should be noted that most of the previous studies were conducted in structures which chloride ions was added during mixing of concrete. That method of supplying chloride may not fully be able to demonstrate the interaction between concrete and chloride ions as in the reinforced concrete structure which directly exposes to chloride environments.

Therefore, this study aims to compare the efficiency of chloride removal on various applications on reinforced concrete blocks in which chloride ions were supplied by immersing the reinforced concrete blocks in 3.5% NaCl solution with wet and dry cycles. Three manners of current application were employed which includes one continuous application and two intermittent applications.

2 EXPERIMENTAL PROCEDURE

Thirty three reinforced concrete blocks having a rectangular cross section as shown in Figure 1 were cast. The mix proportion of concrete is presented in Table 1. Ordinary Portland cement was used. One day after casting, the reinforced concrete blocks were taken out of the molds and covered with wet burlap at approximately 20 °C for 1 month. After that, their surfaces were dried in atmospheric condition within a week. Then, five surfaces of the blocks were coated with epoxy, except the bottom surface. One week later, since the epoxy was dried these blocks were immersed in 3.5% NaCl solution where the free-epoxy surface was turned over and directly exposed to the solution with wet and dry cycles. Each wet and dry cycle included 6 hours of wetting followed by 66 hours of drying at the condition of approximately 70% RH and 20 °C, approximately. After 60 cycles of immersing, since the chloride content in the concrete region beside the steel bars reached approximate 1.4-1.6kg/m³, the extraction was applied for 8 weeks with current density of 1A/m² with respect to concrete surface. Saturated calcium hydroxide was used as electrolyte. Titanium square weave mesh with the opening width of approximate 6.9 mm (3 mesh/inch) was cut into small segment with the size of 250 x 250 mm to utilize as temporary anode. Three modes of application were carried out which included one continuous mode, referred as mode 1, and two intermittent modes, referred as mode 2 and mode 3, respectively. Ten blocks were used in each mode. The extraction system on each block was set up as shown in Figure 2. The timetable of these
modes is presented in Table 2. After an each interval of two weeks, the extraction was stopped on two blocks of each mode. Table 3 presents the charge numbers (A.hour/m²) which were applied to the blocks before they were cut off the electrical current.

The powder samples of concrete at different depths were collected by drilling the concrete blocks. These powder samples were then ground by using laboratory ball mill pulverizer with the speed of 450 min⁻¹ within 6 minutes. Acetone was used as lubricant during pulverizing. Then, these ground samples were dried in desiccator a week with aspirator at 6 kPa before preserving in a vacuum chamber at 80 kPa.

Acid-soluble chloride content and water-soluble chloride content in concrete at different depths were analyzed in those powder samples according to ASTM C1152 / C1152M and ASTM C1218 / C1218M, respectively.

![Figure 1: Cross section of reinforced concrete block](image1.png)

**Figure 1:** Cross section of reinforced concrete block

<table>
<thead>
<tr>
<th>W/C</th>
<th>Water</th>
<th>Cement</th>
<th>Sand</th>
<th>Coarse Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>165</td>
<td>412</td>
<td>790</td>
<td>1020</td>
</tr>
</tbody>
</table>

**Table 1: Concrete mix proportions (kg/m³)**

![Figure 2: Chloride extraction set-up for reinforced concrete block](image2.png)

**Figure 2:** Chloride extraction set-up for reinforced concrete block
Table 2: Timetable of current application

<table>
<thead>
<tr>
<th>Time (weeks)</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The current was applied continuously throughout 8 weeks</td>
<td>The current was turned on 5 days then followed by 2 days off</td>
<td>The current was turned on 5 days then followed by 2 days off</td>
</tr>
<tr>
<td>2</td>
<td>The current was turned on 5 days then followed by 2 days off</td>
<td>The current was turned on 5 days then followed by 2 days off</td>
<td>The current was turned on 5 days then followed by 2 days off</td>
</tr>
<tr>
<td>3</td>
<td>For the rest 6 weeks: the current was turned for 4 days and then turned off 3 days within each week</td>
<td>The current was turned on 4 days then followed by 3 days off</td>
<td>The current was turned on 4 days then followed by 3 days off</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3: Applied charge number (A.Hour/m²) at certain period of different applications

<table>
<thead>
<tr>
<th>Time of application (weeks)</th>
<th>2</th>
<th>4</th>
<th>6</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1</td>
<td>336</td>
<td>672</td>
<td>1008</td>
<td>1344</td>
</tr>
<tr>
<td>Mode 2</td>
<td>204</td>
<td>432</td>
<td>624</td>
<td>816</td>
</tr>
<tr>
<td>Mode 3</td>
<td>204</td>
<td>432</td>
<td>624</td>
<td>768</td>
</tr>
</tbody>
</table>

3 RESULTS AND DISCUSSIONS

3.1 Removal of acid-soluble chloride ion

Based on the remaining acid-soluble chloride ions, which were analyzed at different depths of concrete, the effect of current application on the efficiencies of ECE on concrete cover can be categorized into three main regions. One is the region which is near the anodic site. This region includes the concrete zones at the depths of 0-1 cm and 1-2 cm from the surface of the block. The other is the region near the cathodic site, which includes the concrete zones at the depths of 3-4 cm and 4-5 cm from the surface of the block. The last region is the middle zone which is at the depth of 2-3 cm.

Figure 3a presents the remaining acid-soluble chloride ions at the depth of 0-1 cm. It figured out clearly that within the first two weeks of the extraction process, approximate 50 to 60% of acid-soluble chloride ions in this region were removed, and there was no significant difference since the different modes were applied. However, after that first period, the efficiencies of all three modes of application dropped quickly. Mode 1 showed a slightly higher efficiency in removing chloride ions than that in modes 2 and 3 from the beginning to 6 weeks of the extraction process. Nevertheless, in the last two weeks of extraction process, from 6 to 8 weeks, mode 1 removed a slightly lower content of acid-soluble chloride ions. At the end of the 8-week extraction process, 75% of acid-soluble chloride ions were removed in mode 1, while approximately 70% of them were removed in modes 2 and 3.

The efficiencies of chloride removal in concrete zone beside the previous mentioned region, which is at the depth of 1-2 cm from the surface, were quite diverse and strongly dependent on the modes of application in the first two weeks of extraction process, as presented in
Figure 3b. In mode 1, approximately 50% of acid-soluble chloride ions were removed; after that, it dropped quickly. For each two-week periods from the 2 to 8 weeks of extraction process, it was approximate 12%, 8%, and 5%, respectively. The efficiencies in mode 2 and mode 3 were significantly lower compared to mode 1 in the first two weeks of extraction process, they were only approximately 25%; nevertheless, they did not reduce substantially in the following two-week period, from 2 to 4 weeks of extraction process, as in mode 1; approximate 25% of acid-soluble chloride ions were removed in that period. At the later time, there was no significant difference in these modes. Therefore, at the end of the extraction process mode 1 resulted higher total efficiencies in removing chloride than that in modes 2 and 3; total 80% of acid-soluble chloride ions were removed in mode 1, while it was 74%, and 70% in modes 2 and 3, respectively.

Considering the remaining of acid-soluble chloride ions in concrete regions beside cathode, which are presented in Figure 4, it can be seen easily that in the first two weeks of extraction process the efficiencies of acid-chloride removal in these regions are resembled to which gained in concrete regions near the anodic site. As presented in Figure 4a, approximately 55% of acid-soluble chloride ions in the depth of 4-5 cm were removed within the first two weeks of extraction process by applying mode 1; in modes 2 and 3, they were about 50%. After that, similar to the concrete region at the depth of 0-1 cm, the efficiencies of both three modes dropped substantially at the later time of the extraction process. However, unlike to the concrete near anode, where mode 1 showed higher efficiencies, in this region modes 2 and 3 showed a higher capacity for removing acid-soluble chloride ions at the later periods of the extraction process. In mode 1 while about 7% of acid-soluble chloride ions were removed in the period from 2 to 4 weeks and only a half of that was removed in the each two-week period from 4 to 8 weeks, approximately 3%. In modes 2 and 3, approximately 10% of acid-soluble chloride ions were removed in the period from 2 to 4 weeks; in the later periods, approximately 5-7% of acid-soluble chloride ions were removed in each two-week period. Therefore, after the 8 weeks of extraction process modes 2 and 3 removed slightly higher total efficiencies than that in mode 1, although the higher amounts are insignificant. They were 70%, 72% and 74% in modes 1, 2, and 3, respectively.

Figure 3: Remaining acid-soluble chloride ions in concrete regions near anodic site at the depth of a) 0-1 cm and b) 1-2 cm from the surface
Figure 4b presents the remaining acid-soluble chloride ions in concrete at the depth of 3-4 cm from the surface. It can be seen that after first two weeks of extraction process, only 30-35% of acid-soluble chloride ions in this region were removed, significantly lower compared to the efficiencies in concrete at the depth of 4-5 cm. At the following periods, while mode 1 showed a consistently dropping of efficiencies, in modes 2 and 3 they figured out a different tendency. About 10% of chloride ions were removed within the period from 2 to 4 weeks in modes 2 and 3, however, their efficiencies were higher in the period from 4 to 6 weeks of extraction process, approximately 16% and 19%, respectively; after that, 12% and 8% of chloride ions were removed during the last extraction period, from 6 to 8 weeks, respectively. At the end of the extraction process, there was no significant difference in total efficiencies of chloride removal in these three modes, approximately 66-68%.

The remaining acid-soluble chloride ion in the middle concrete region, which is at the depth of 2-3 cm from the surface, is presented in Figure 5. Apparently, it shows very different in the tendency and efficiencies of the chloride removal process. During first two weeks of extraction process, only 12% of acid-chloride ions in the concrete at the depth of 2-3 cm were removed by using mode 1, it was even lower than that in modes 2 and 3, only approximately 4%. Obviously, they are substantially lower compared to the efficiencies in the other regions. The period from 2 to 4 weeks of extraction progress did not show significant improvement, only 6-8% of acid-soluble chloride was removed. However, from 4 to 8 weeks they were higher than that in the previous periods. Approximately 16% of chloride ions were removed in the period from 4 to 6 weeks by applying mode 1; in modes 2 and 3, they were about 21%. The last two-week period, from 6 to 8 weeks, the reduction of efficiencies was exhibited in modes 2 and 3, about 13% and 10% were extracted, respectively, while in mode 1, it did not show much difference to the immediately previous period, 16% of acid-soluble chloride ions were removed. It is remarkable that the total efficiencies of ECE in this concrete region were considerably lower than that in the other regions, especially in modes 2 and 3; after 8 weeks of extraction process, only 47% and 42% of acid-soluble chloride ions were removed in mode 2 and 3, respectively, while mode 1 resulted a higher, about 54%.
3.2 Removal of water-soluble chloride ion

The remaining water-soluble chloride ions in concrete at different depths, which are categorized as previously mentioned, are presented in Figures 6 to 8. As corresponding to the acid-soluble chloride ions, the extraction tendency of water-soluble chloride ions happened similarly. In the concrete zones which were immediately beside the cathodic and anodic sites, at the depths of 0-1 cm and 4-5 cm, approximately 53-60% of water-soluble chloride ions were removed within first two weeks of extraction process, as presented in Figures 6a and 7a; there were higher efficiencies in mode 1 than that in modes 2 and 3 in this period. After that, the efficiencies in these regions decreased quickly at the periods from 2 to 8 weeks. At the end of the extraction process, approximately 77% of water-soluble chloride ions in concrete at the depth of 0-1 cm were removed by using mode 1; in modes 2 and 3, the total efficiencies were slightly lower, about 75% and 73%, respectively. Nevertheless, at the depth of 4-5 cm from the concrete surface, modes 2 and 3 presented a slightly higher efficiencies in each two-week periods than that in mode 1; the rate of declining during extraction progress were also slower compared to mode 1. Therefore, at the end of extraction process modes 2 and 3 resulted slightly higher efficiencies than that in mode 1. They were 72% in mode 1 and 75% in modes 2 and 3.

In the zones next to these regions which are at the depths of 1-2 cm and 3-4 cm, the efficiencies of water-soluble chloride removal was lower and strongly dependent on the mode applied as shown in Figures 6b and 7b, respectively. In mode 1, about 57% and 37% of water-chloride ions in concrete at the depths of 1-2 cm and 3-4 were removed within the first two weeks, respectively; in modes 2 and 3 they were significantly lower, only approximately 30% at these depths. As the results, mode 1 showed a significantly declining of efficiency at the later extraction periods at the depth of 1-2 cm, while the others showed higher efficiencies. After the 8 weeks extraction, 75% of water-soluble chloride ions in concrete at the depth of 1-2 cm were removed by applying mode 1; it was slightly higher total efficiencies than that in modes 2 and 3, which their efficiencies were 71%. In contrast, in concrete at the depth of 3-4 modes 2 and 3 resulted an insignificantly higher in total
efficiencies compared to mode 1; they were 63% in mode 1 and about 69 in modes 2 and 3.

Figure 6: Remaining water-soluble chloride ions in concrete regions near anodic site at the depth of a) 0-1 cm and b) 1-2 cm from the surface

Figure 7: Remaining acid-soluble chloride content in concrete regions near cathodic site at the depth of a) 4-5 cm and b) 3-4 cm from the surface

Figure 8 displays the remaining water-chloride ions in the middle zone of concrete, where its depth is 2-3 cm from the surface of the block, versus the time of extraction. There is no significant removal of chloride ions in this zone within first four weeks of the extraction process. In mode 1, only 8% of water-soluble chloride was removed in the first two weeks of extraction progress; and only 3% were removed in the period from 2 to 4 weeks. In modes 2 and 3, water-soluble chloride ions in this zone were even higher than that before applying the extraction. However, their efficiencies became substantially increasing in the periods from 4 to 8 weeks of extraction process. About 24% to 34% of water-soluble chloride ions were extracted during the period from 4 to 6 weeks. Although they were followed by the reduction in the last two-week period, from 6 to 8 weeks of extraction process, the total efficiencies of extraction process were 56% in mode 1 and 47-50% in modes 2 and 3. It is easy to realize...
that, the efficiencies of chloride removal in concrete at the middle zone are relatively low compared to the other zones.

![Graph](image)

**Figure 8**: Remaining acid-soluble chloride content in the middle region of concrete, at the depth of 2-3 cm from the surface

Figure 9 presents the relationship between the remaining of chloride ions and the number of charge numbers applied in concrete at the depth of 4-5 cm, which surrounds the reinforcements. The properties and ionic concentrations in pore solution of this concrete region affect strongly to the state of steel bars, and of course, they also influence directly on the repassivation of steel reinforcement after stopping ECE. From Figure 9, it is obvious that the tendency of the removal progress of chloride ions in this concrete region is strongly depending on the mode applied although the final efficiencies seemed approximately.

Firstly, it is important to emphasize here that while the numbers of charge which applied in mode 1 were much greater than that in modes 2 or 3, the remaining of chloride ions in concrete after 8 weeks of extraction, or in the other way to mention, the total efficiencies in these modes were insignificantly different.

Secondly, the dropping level in mode 1 is the most intense in these modes. In mode 1, the removal of chloride ions reduced extremely after the applied charge numbers reached to 336 A.hour/m², which corresponds to 2 weeks of continuous application. Furthermore, the quantity of applied charge number seemed to approach a threshold value since it was approximately 700 A.hour/m². After that, although 700 A.hour/m² of charge numbers were further applied continuously the amount of chloride ions extracted were negligible. Nevertheless, in modes 2 and 3 although the efficiencies of ECE also reduced quickly after the applied charge numbers reached about 204 A.hour/m², the dropping rate was not as substantial as in mode 1. Consequently, the amount of chloride extracted was gradually increased since the applied charge numbers increased from about 200 to 800 A.hour/m².

Therefore, it is reasonable to assume that the interruption during applying ECE was greatly important to improve the electrical applied charge-effectiveness on removing chloride ions.
As well known, the generation of hydroxide ions takes place as the result of cathodic reactions during the current application as presented in Equations (1) and (2). The more charge numbers are applied, the more concentration of hydroxide ions is generated in pore solution of concrete. Therefore, it can be understood that at first two weeks of extraction, when the concentration of chloride ions in pore solution beside the steel bars was still high, chloride ions are one of the main ions which carried the charge of the current beside hydroxide ions. However, since the generation of hydroxide took place, increased and dominated in the ionic fluid in pore solution, chloride ions which concentration continuously reduced showed less effect on the current transference. Consequently, the efficiency of chloride removal dropped quickly.

\[
\begin{align*}
O_2 + 2H_2O + 4e^- & \rightarrow 4OH^- \\
2H_2O + 2e^- & \rightarrow 2OH^- + H_2
\end{align*}
\] (1) (2)

The influence of the electrical field to the ions in pore solution at the middle zone is certainly less than that in the near-electrode zones. Furthermore, the repulsion of chloride and hydroxide ions from cathode can cause the stuck of it in the middle zone at the first four weeks of the extraction process. After that period, since the concentration of chloride in ionic fluid was significant lower, that stuck could be resolved and chloride ions accumulated at that zone easily removed.

In concrete at the depth of 4-5 cm, the generation of hydroxide ions and the ionic concentrations in pore solution are impacted strongly and directly by the charge number applied. The more hydroxide ions generated, the lower relative concentration of a certain amount of chloride ions in pore solution. Therefore, the contribution of chloride ions in the ionic fluid for transferring the current reduced. Obviously, the applied charge numbers in mode 1 were much higher than that in modes 2 and 3; it certainly generated higher hydroxide ions compared to modes 2 and 3. Consequently, hydroxide ions dominated the ionic fluid quickly and the extraction of chloride ions was easy to reach the threshold value.

Nevertheless, the applied charge numbers in modes 2 or 3 were much lower than that in mode 1. Then, it could generate a lower content of hydroxide ions. Moreover, the interruption
time could be the time for the phases in concrete rebalanced. In this interruption time, free chloride ions which can contribute in ionic fluid to transfer the current can be replenished by bound chloride ions. Consequently, the dropping level of the chloride removal efficiencies in modes 2 and 3 were not as quick as that in mode 1.

4 CONCLUSIONS

The following conclusions were drawn in this experimental study:

- Approximate 50% to 60% of acid-soluble chloride ions and 55-60% of water-soluble chloride ions in concrete beside electrodes were removed in the first two weeks of extraction. After that, the efficiencies of extraction progress dropped quickly. Nevertheless, the efficiencies at the middle zone were insignificant within first four weeks of the extraction.
- At the beginning, the continuous application, mode 1 showed a slightly greater efficiency in removing chloride ions compared to intermittent applications, modes 2 and 3. However, since the hydroxide ions were generated at cathode more and more thanks to the increasing of applied charge numbers, the role of it in pore solution became dominated and consequently the chloride removal efficiencies of mode 1 reduced and turn out to be lower than that in modes 2 and 3.
- After 8 weeks of extraction process, the final efficiencies of three modes were insignificantly different. In the concrete regions immediately beside electrodes, approximately 70% of acid-chloride ions were removed while about 70-80% of water-soluble chloride ions were extracted. They were slightly lower in concrete regions beside these regions, at the depths of 3-4 cm and 1-2 cm. The efficiencies at the middle zone were lowest, approximately 42-54% of acid-soluble chloride ions were removed, while they were about 47-56% of water-soluble chloride ions were extracted.
- The electrical applied charge-effectiveness of modes 2 and 3 were much higher than that in mode 1.
- The interruption during applying ECE was greatly important to improve the electrical applied charge-effectiveness on removing chloride ions.

These results suggest that the intermittent application is a promising method to conduct ECE in order to improve the charge-effectiveness on removing chloride ions. This method may suppress some drawbacks causing by the utilization of the large applied charge numbers in continuous application of ECE and accordingly, mitigate the later deterioration of reinforced concrete structures. Therefore, intermittent application of ECE can be considered as a beneficial option which can maintain the service life of reinforced concrete structures exposing to chloride environments and contribute a method to solve the sustainability issues of reinforced concrete structure.

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EVALUATION OF MECHANICAL PROPERTIES AND ACCELERATED CHLORIDE ION PENETRATION (RCMT) IN ALKALI ACTIVATED SLAG CONCRETE

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Key words: Alkali activated slag (AAS) concrete, Blast furnace slag, Concrete permeability, Chloride ion penetration, Rapid chloride migration test (RCMT), Electrical resistivity.

Abstract. Cement is one of the main constituent in concrete that its production causes about 7% of CO₂ world emission. Alkali activated compounds refer to inorganic materials that cement is not the main constituent in their mixes. In fact, alkali activated binder structure consists of two parts: source material and alkaline activator liquid. The alkaline activator used in this study was sodium silicate solution (wt. ratio: SiO₂/Na₂O = 2.33) and 6 M potassium hydroxide and also, blast furnace slag is selected as a source material. For discovering the effect of each participant, 9 alkali activated concrete mix designs are utilized. In these mixes the influence of the amount of source material, solid part of alkaline activator, solid part of sodium silicate solution and also the ratio of water to binder is evaluated. In addition, one mix design for ordinary Portland cement (OPC) concrete is performed to compare with alkali activated slag (AAS) concrete characteristics.

In this research, the fresh concrete properties investigated by the means of slump loss test and compressive strengths were measured at 1, 7, 28 and 90 days. Permeability of the concrete was determined by capillary water absorption test and the amount of chloride penetration in concrete was measured by rapid chloride migration test (RCMT) at the ages of 7, 28 and 90 days. Besides, electrical resistivity was measured to determine the effect of parameters on durability of AAS concretes. Results indicate that mechanical properties and durability of AAS concretes are mainly influenced by the amount of alkali activators in a way that incorporating higher quantity of activators could bring about rapid hardening reactions at early ages and decelerate mechanical properties and durability development process.
1 INTRODUCTION

Cement industry is a significant source of carbon dioxide. In addition, in cement manufacturing process, a large amount of energy is consumed. Since, concrete is one of the most important construction materials in the world, discovering a suitable solution for reducing the problems caused by cement as the main constituent of concrete could be one of the topics which attract the attention of researchers. Besides these problems, today investigations consider the durability factors of concrete and efforts base on finding out durable concretes.

Many studies focused on using supplementary cementitious materials in concrete structure as cementing replacement materials. Due to these investigations, the concrete properties especially durability characteristics improve by utilizing these materials [1,2,3] One of the supplementary materials is blast furnace slag which is a by-product of steel production. While employing slag in concrete results in low early age compressive strengths, a novel approaches called alkali activated slag concretes introduced by researchers. Alkali activated slag (AAS) concretes consists of two components: source material and alkaline activator solution. AAS concretes usually possess better early age compressive strength than OPC concretes besides superior durability, suitable mechanical properties, lower hydration heat, enhanced performance against freeze-thaw, and lesser permeability [4,5,6,7]. Due to the researches AAS concretes have appropriate resistance against aggressive environment and factors in them that caused different deterioration [8,9]. One of these threatening factors is chloride ions penetration which influences the concrete durability by steel reinforcement corrosion.

In fact, ordinary iron and steel products are normally protected by a passive layer which covers them and makes steel impermeable. This protective film is stable until the pH of the concrete decreases. Chloride ions are one of the agents which activate the passive layer and threaten the steel embedded in concrete by initiating corrosion [1]. As chloride ions content increase than the threshold content in the presence of oxygen and humidity, the situation for corrosion is prepared [10]. Different tests methods are employed to model the chloride penetration in concrete by various mechanisms.

Since the studies about the durability against chloride ingress in AAS concretes are limited, in this investigation the main purpose was based on evaluating the chloride penetration in AAS concretes through rapid chloride migration test according to NT Build 492. Further properties such as slump loss, compressive strength, permeability and the effect of various parameters on these characteristics have been evaluated as well.

2 EXPERIMENTAL PROGRAM

2.1 Materials

Source material in the structure of alkali activated concretes was blast furnace slag which fulfilled the limitation of ASTM 989 with physical and chemical properties shown in Tables 1 and 2. Additionally, in OPC mix design type I Portland cement was employed with properties shown in Tables 1 and 2.
Table 1: Physical properties of blast furnace slag and cement

<table>
<thead>
<tr>
<th>Specification</th>
<th>Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slag</td>
</tr>
<tr>
<td>Density (g/cm$^3$)</td>
<td>2.79</td>
</tr>
<tr>
<td>Fineness (cm$^2$/g)</td>
<td>3383</td>
</tr>
<tr>
<td>Amount retained on 45 μm (No.325 sieve (%))</td>
<td>16.61</td>
</tr>
</tbody>
</table>

Table 2: Chemical composition of blast furnace slag and cement

<table>
<thead>
<tr>
<th>Material</th>
<th>CaO</th>
<th>SiO$_2$</th>
<th>Al$_2$O$_3$</th>
<th>Fe$_2$O$_3$</th>
<th>SO$_3$</th>
<th>MgO</th>
<th>Na$_2$O</th>
<th>K$_2$O</th>
<th>TiO$_2$</th>
<th>MnO</th>
<th>L.O.I.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slag</td>
<td>36.75</td>
<td>37.21</td>
<td>11.56</td>
<td>1.01</td>
<td>0.97</td>
<td>8.52</td>
<td>0.61</td>
<td>0.7</td>
<td>1.23</td>
<td>0.99</td>
<td>0.02</td>
</tr>
<tr>
<td>Cement</td>
<td>63.78</td>
<td>21.55</td>
<td>5.89</td>
<td>3.72</td>
<td>1.89</td>
<td>1.35</td>
<td>0.53</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
<td>0.94</td>
</tr>
</tbody>
</table>

In this study, the combination of two types of alkaline activator solution was utilized to make AAS concretes. First activator was potassium hydroxide (KOH) solution with 6 molar concentrations that was mixed 24 hours before the concrete mixes. The second one was sodium silicate solution (Na$_2$SiO$_3$) SiO$_2$/Na$_2$O ratio of 2.33 and solid particle ratio of 48%.

The aggregates categorized in three classes of different sizes. Fine aggregates (natural sand) had maximum size of 6 mm, with specific gravity of 2560 kg/m$^3$ and water absorption of 2.89% which incorporated with mass ratio of 0.6. Fine gravel with maximum size of 12.5 mm, contained specific gravity of 2550 kg/m$^3$ and water absorption of 2.39% and employed with ratio of 0.24. Finally, coarse gravel with maximum size of 19 mm possessed 2590 kg/m$^3$ specific gravity and also water absorption of 2.15% and used with ratio of 0.16. It should be mentioned that all of the specific gravity values are stated in saturated surface dry (SSD) condition.

Due to fast setting of AAS mixture in the first mixes, it was decided to use a superplasticizer for better workability of AASs. Hence, a copolymer polycarboxylate-ether type High-Range Water Reducer Admixture (HRWRA) with specific gravity of 1.1 incorporated with amount of 1% of binder materials in main AAS concretes mix designs which complies with ASTM C 949-Type G admixtures.

2.2 Mixing proportions

While various interactions of different factors affect the AAS concretes properties, distinctive parameters have been selected to control their influence on fresh and hardened concretes characteristics [6]. To evaluate the effect of source material quantity, three mixes were considered (S=300, S=350 and S=400). Through these three mix designs, S=400 selected as control mix. The results of utilization different ratio of water to binder were obtained during three proportions for water to binder (Mixes: w/b=0.35, S=400 and w/b=0.45). It should be stated that in AAS mixing proportions, the solid part of activator has been considered as a part of binder and the existing water in alkaline activator solutions has been taken as a part of total water in water to binder ratio. Moreover, the effect of the ratio of
sodium silicate to potassium hydroxide and also the amounts of alkaline activator solutions were controlled by the means of WGs and ACs mix designs. An OPC mix design was employed for comparing the AAS concrete properties with OPC concrete. The detailed information about mixing proportions is depicted in the Table 3.

Table 3: Mixing proportions

<table>
<thead>
<tr>
<th>Mix design ID</th>
<th>Cement (kg/m³)</th>
<th>Slag (kg/m³)</th>
<th>Water to binder ratio</th>
<th>solid part of alkaline activator to slag ratio</th>
<th>solid part of Na₂SiO₃ to KOH ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>S=300</td>
<td>-</td>
<td>300</td>
<td>0.4</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>S=350</td>
<td>-</td>
<td>350</td>
<td>0.4</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>S=400</td>
<td>-</td>
<td>400</td>
<td>0.4</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>w/b=0.35</td>
<td>-</td>
<td>400</td>
<td>0.35</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>w/b=0.45</td>
<td>-</td>
<td>400</td>
<td>0.45</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>WG=0.4</td>
<td>-</td>
<td>400</td>
<td>0.4</td>
<td>0.15</td>
<td>0.4</td>
</tr>
<tr>
<td>WG=0.8</td>
<td>-</td>
<td>400</td>
<td>0.4</td>
<td>0.15</td>
<td>0.8</td>
</tr>
<tr>
<td>AC=0.1</td>
<td>-</td>
<td>400</td>
<td>0.4</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>AC=0.2</td>
<td>-</td>
<td>400</td>
<td>0.4</td>
<td>0.2</td>
<td>0.6</td>
</tr>
<tr>
<td>OPC</td>
<td>426</td>
<td>-</td>
<td>0.4</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

2.3 Concrete casting

The AAS mixture which had been prepared in the 60 liters pan mixer, were casted in 10×10×10 cm, 15×15×15 cm cubic molds and 10×20 cm cylinder moulds, immediately after the end of mixing. The following mixing procedure was selected after different trials. At first, the aggregates and slag were dry-mixed. After perfect integration, KOH solution was added and mixed for 30 seconds. Continuously, following the hand mixing and digging the bottom of the pan, water glass solution, water and superplasticizer were added and mixing process were continued for 2 minutes and 30 seconds. Ultimately, after 24 hours of concrete casting, the specimens were demolded and cured in water until the age of tests. The method of curing was selected after initial AAS concrete mixture by measuring compressive strength and also through the observed surface of them. Furthermore, previous researches obtained similar conclusion and verified these curing procedure [11,12].

2.4 Test program

2.4.1 Slump loss test

The properties of fresh AAS concretes evaluated through the slump loss test which carried out based on ASTM C 143 [13]. In this test method the slump of fresh mixture is measured until reaching zero slump.

2.4.2 Compressive strength

One of the most important hardened concrete properties is compressive strength. In this investigation, compressive strength was measured at the ages of 1, 7, 28 and 90 days. This test procedure conducted on three 10×10×10 cm cubic molds at each ages.
2.4.3 Capillary water absorption

Determination of capillary water absorption of concrete specimens was carried out according to BS EN-480-5:1997 suggested test method [14]. After 7, 28 and 90 days, two 10×10×10 cm cubic concrete specimens were dried for 14 days in 50 °C in oven. At the end of this period of time, the specimens sealed on lateral faces and after weighting, put on rods in water in order to allow free access of water to specimen bottom surface. It should be considered that water should be above 5 mm of this surface constantly. The weight of the specimens should be reported at 3, 6, 24, 72 hours after commencing the experiment. By the means of this test, absorbed water per specimen surface area can be calculated.

2.4.4 Rapid Chloride migration test (RCMT)

Rapid chloride migration test (RCMT) is an accelerated test method that indicates the resistance of concrete against the chloride ions ingress. NT Build 492 states the test method details for determination of the chloride migration coefficient in concrete from non-steady-state migration experiments (Dnssm) [15].

Additionally, in order to perform this experiment, 10×20 cylinder AAS concrete specimens were cured in water until the ages of 7, 28 and 90 days. Besides, OPC mixture was tested after 28 and 90 days of curing in lime solution.

2.4.5 Electrical resistivity

One of the non destructive tests that employed for evaluating the permeability and concrete durability is electrical resistivity by means of Wenner method. In this study, the measurements were carried out at four quaternary longitudinal locations of the three specimens at ages of 28, 56 and 90 days and the average value was taken as electrical resistivity. Results obtained from this test method indicate the resistivity against passed charge in concrete. Therefore, this property is affected by electrical conductivity of binder matrix [16].

3 RESULTS AND DISCUSSION

3.1 Fresh concrete properties

It could be mainly concluded through the experiments and observations that the AAS compounds are viscous and possess low workability. According to the Figure 1, the following results can be concluded:

- Increase in slag content results in the better workability due to the increase of binder to aggregate ratio.
- Variation of water to binder ratio has low impact on workability of AAS mixtures.
- Among three mix designs which evaluate the influence of Na$_2$SiO$_3$/KOH (WG=0.4, S=400 and WG=0.8), S=400 had the best workability results.
- Increase the solid part of activators to slag ratio has improved the slump loss test conclusions because of larger inter-particle distances [17].
3.2 Compressive strength

In the Figure 2 the compressive strength results of all mixes can be observed. It is seen that at 90 days the highest compressive strength is for mixture with w/b=0.35.

The detailed information about the compressive strength improvement at each ages are shown in Figure 3. Comparison of different amounts of slag indicates that differences in this parameter quantity did not have considerable impact on compressive strength. It just can be stated that among these three mix designs, S=400 have the best performance through the maximum amount of this property. Due to different ratio of water to binder and similar to the OPC concretes, the mix design with lower w/b have higher compressive strength at all ages. The compressive strength of S=400, WG=0.4 and WG=0.8 are not varied considerably. Thus, it can be concluded that changes in the Na₂SiO₃/KOH ratio does not have significant impact on mechanical properties. On the other hand, the activator to slag ratio was one of the most
determinative AAS concrete characteristics as shown that the increase in this ratio results in reduction of long term compressive strength. Indeed, in 1 day the compressive strength of AC=0.2 was more than other two mixes due to the high amount of activator and rapid reactions. Nevertheless, by concrete age increasing, decrease in activator quantity brings about higher compressive strength. Ultimately, comparing control AAS mix and OPC mix indicates that the AAS concrete performs better compressive strength at each age.

3.3 Capillary water absorption

By capillary water absorption test the permeability of concrete can be estimated. In the Figure 4 the effect of source material amounts on this test can be observed. Results show that the differences between the absorbed water are little and there is not an accurate procedure for this parameter. In fact, it should be because of the micro cracks created in structure of the AAS concrete during the 50°C heating which are reasons for errors in this test method. The influence of water to binder ratio is completely clear. As determined in Figure 5, reduction in this ratio decreases the water absorption and permeability. Figure 6 demonstrates the results of water absorption by evaluating the water glass to KOH ratio. It can be stated that at 28 and 90 days, S=400 have had lower permeability due to this test results. Also in 7 days, WG=0.4 behaved better than two other mixes. By comparing three mix designs which evaluated the impact of alkaline activator amounts, lowest water absorption belongs to AC=0.1 (see Figure 7). Finally, in the Figure 8 results of capillary water absorption are available. Due to this Figure, in 28 days the amount of absorbed water for OPC concrete is slightly lower than the control AAS mix. This procedure changes at 90 days in a way that the results indicate the permeability of the AAS mix is lower than the OPC mix.
Figure 4: The effect of slag amount on capillary water absorption

Figure 5: The effect of water to binder ratio on capillary water absorption

Figure 6: The effect of Na$_2$SiO$_3$/KOH ratio on capillary water absorption
3.4 Rapid chloride migration test (RCMT)

To evaluate the chloride ions penetration in concrete in this study, it is essential to compare the migration coefficient. They can be observed in the Figure 9. Best performance is related to the S=400 among three mixes which compared the influence of slag amount. Through these test the w/b=0.35 have had the lowest migration coefficient as expected. In addition, changes in water glass to KOH ratio did not affect the migration coefficient in long term. Furthermore, increase in alkali activated amounts reduced the concrete durability against chloride ingress. Ultimately, AAS concrete contain lower migration coefficient and better durability against chloride ions penetration than OPC concrete.
3.5 Electrical resistivity

In the Figure 10 the electrical resistivity of all mixes can be observed. As it was stated, this test method is affected by the electrical conductivity of the material. Therefore, because of conductivity of AAS material through the usage of alkaline solutions, this test procedure did not indicate the permeability and durability of these concretes. In fact, increase in conductivity parameters causes reduction in electrical resistivity. These results are clear in the Figure 10. For instance, the highest amount of electrical resistivity belongs to AC=0.1 with lowest alkaline activator as one of the most important parameters for conductivity. Hence, implementing the electrical resistivity test method is not suggested for AAS concretes.

4 CONCLUSION

The following conclusion can be drawn from the results obtained in this investigation:
- Alkali activated slag (AAS) fresh concretes are viscous compounds with low workability. Thus, it is suggested to find out suitable admixture for AAS concretes.
- Increase in slag content as source material causes better fresh and hardened concretes properties.
- It is recommended that for enhanced recognition of the effects of two alkaline activator ratios (Na₂SiO₃/KOH), more than three proportions selected and tested.
- The most effective parameter in this study was the ratio of solid parts of alkaline activators to slag which indicated the influence of alkaline activators amounts. Slump loss test results indicated that AC=0.2 with the highest amounts of activator was more workable than two others. In contrast, the best hardened concrete performance was reported for AC=0.1 with the lowest activators. Therefore, it is necessary to employ lesser amounts of activators and try to improve fresh concrete properties by incorporating appropriate admixtures.
- Water to binder ratio behaved similar to OPC concretes in AAS hardened concretes and also it can be stated that it has no impact on fresh properties.
- Since the errors in capillary water absorption test, it is suggested to use other test methods for evaluating the permeability of AAS concretes.
- Electrical resistivity is not a suitable test for AAS concretes because of their electrical conductivities.
- Based on the results in this investigation, AAS concrete shows higher compressive strength and superior durability against chloride ions penetration than the OPC concrete.
- Considering main factors in sustainable development issues, durability of concrete structures in aggressive environment should be fulfilled. Thus, studied AAS mixtures in this research, demonstrated a greater durability in comparison with OPC, it could be concluded that incorporating AAS mixtures in concrete structures brings about more sustainable civil and infrastructure constructions.

REFERENCES


EXPERIMENTAL STUDY OF CONCRETE DETERIORATION DUE TO FROST ACTION

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Key words: Freeze-thaw cycles, damage, permeability

Abstract. The relationships between transport phenomena and durability of concrete is a main focus of research. The permeability is an important factor influencing deterioration of concrete durability. The coefficient can be used to compare the ability of concrete to resist the ingress of destructive substances. This experimental study concerns the frost durability of an ordinary concrete without air-entraining admixtures, with two water to cement ratio (w/c equal to 0.5 and 0.4). The evolutions of some physical properties during repeating freeze-thaw cycles have been analyzed. The damage is characterized by the reduction of elastic modulus, as well as the permeability evolution due to frost action. The changes of permeability, as well as compressive strength and the elastic modulus of concretes were investigated after 0, 50, 100 and 150 freeze-thaw cycles. Finally the interaction of frost action with permeability and mechanical properties is discussed. It was established that a variation in the mechanical properties induces an increase of water transport parameters such as intrinsic permeability coefficient.

1 INTRODUCTION

Concrete is the most commonly used construction material and its durability is a key factor from an economical point of view and depends mainly on its environment and its composition. Freeze-thaw damage remains one of the most significant problems concerning durability of the cement-based materials [1]. The destruction of material by frost is caused by freezing of liquid in pores which leads to a stress resulting from bigger volume of ice comparing with water [2]. Internal structure, mechanical and transport properties are affected by water swelling due to its solidification. Therefore, as well as microstructure, transport and mechanical properties of cement-based materials are of particular importance when the ice-induced deterioration is considered, since they determine the rate at which moisture can penetrate into the material and cause disruption.

The knowledge about physical and mechanical properties and their evolution due to the environmental loading is necessary for a more profound understanding of damage phenomena. Understanding how the frost action influences transport and mechanical properties is vital for development of more durable materials and accurate prediction models. It will help to create materials that can withstand many years and will serve future generations. Moreover, it will be possible to reduce the amount of used material because the buildings will be able to serve for more years.
This paper is devoted to analyze how the frost-induced damage influences physical and mechanical properties of concrete without air-entraining admixtures. Two kind concrete of different water to cement ratio were exposed to increasing number of freeze-thaw cycles: 50, 100 and 150, respectively. The damage of the material was characterized by the reduction of elastic modulus, compressive strength, as well as the permeability evolution. The results of experimental investigation were analyzed and discussed in this contribution.

2 EXPERIMENTAL PROGRAM

Experiments has been performed on concrete specimens with water to cement ratio equals to 0.5 and 0.4. Concrete mixes were prepared according to the mix proportions shown in Table 1. The cement was a Portland cement CEM I 42.5N-NA.

Concrete specimens, after casting, were kept 1 day in plastic foil covering for, demolded and placed in water at room temperature for 6 months. Afterwards, the reference samples were preserved in water, while other samples were subjected to freeze-thaw cycles (50, 100 and 150 respectively) and later placed in water as well. As soon as all the freeze-thaw cycles were finished, all the samples were removed from water.

Cube specimens were prepared in order to follow the compressive strength evolution during the freezing-thawing test. Cylinders of 150mm diameter were prepared so as to study elastic modulus and permeability evolution. After measurement of elastic modulus, the 300mm high cylinders were cut into smaller cylinders. Afterwards, the initial gas permeability of the specimens was measured.

Table 1: Concrete compositions

<table>
<thead>
<tr>
<th>Mix ingredients (kg/m³)</th>
<th>C-1</th>
<th>C-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM I 42.5N-NA</td>
<td>425</td>
<td>425</td>
</tr>
<tr>
<td>Water</td>
<td>212</td>
<td>170</td>
</tr>
<tr>
<td>Coarse aggregate, 8-16mm</td>
<td>524</td>
<td>556</td>
</tr>
<tr>
<td>Medium aggregate, 2-8mm</td>
<td>648</td>
<td>686</td>
</tr>
<tr>
<td>Sand, 0-2mm</td>
<td>576</td>
<td>612</td>
</tr>
<tr>
<td>Plasticizer</td>
<td>2.12</td>
<td></td>
</tr>
<tr>
<td>w/c</td>
<td>0.50</td>
<td>0.40</td>
</tr>
</tbody>
</table>

3 METHODS

3.1 Freeze-thaw cycles

The temperature changes were imposed by the freeze-thaw climatic chamber according to PN-88/B-06250 [3]. The temperature varied between -18°C and +18°C. One freezing-thawing cycle lasted 8h.

3.2 Mechanical properties

The data required for the assessment of the mechanical properties consists of axial stress and longitudinal strain. Tested samples were loaded under axial compression and the stresses and strains were recorded. The mean value of compressive strength was used to define the stress levels. The elastic modulus was deducted from stress-strain experimental curves.
obtained at the end of three loading-unloading cycles for a stress varying between 0.5MPa and 30% of the compressive strength [4].

A damage value \( d \), is regarded as the relative decrease in the modulus of elasticity:

\[
d = \frac{(E_{REF} - E_d)}{E_{REF}}
\]

where \( E_{REF} \) is the modulus of elasticity for references samples, and \( E_d \) is the modulus of elasticity obtained for concrete after succeeding freezing-thawing cycles [5].

Evaluation of frost resistance of the tested concretes was also conducted based on the compressive strength investigation. The test was performed on three specimens of each series and each number of freeze-thaw cycles. Additionally, the error, defined as the maximum difference between results obtained for particular specimens and their average value, is presented.

3.2 Permeability

Gas permeability tests were performed using Cembureau method, which is recommended by RILEM [6]. Due to a gas pressure gradient, which was induced between two surfaces of a sample located in the measuring chamber, a one-dimensional gas flow was imposed. Apparent gas permeability, \( k_a \), was calculated at a given pressure according to the Hagen-Poiseuille equation for laminar flow under steady-state equation,

\[
k_a = \frac{2 \cdot \rho_0 \cdot Q_i \cdot L \cdot \mu}{A \cdot (p_i^2 - p_o^2)}
\]

where \( p_i \) and \( p_o \) are the inlet and outlet pressure, \( Q_i \) is the volumetric flow rate, \( L \) the thickness of the sample in the direction of the gas flow, \( A \) the area of the sample cross-section, and \( \mu \) the viscosity of oxygen. The apparent gas permeability was obtained for three inlet pressures: 0.2, 0.25 and 0.3MPa. The establishing steady state flow before measurements requires significant time. This condition is verified by taking two measurements separated by 10 minutes time interval. If two values differ by less than 2% the steady state flow condition is assumed to be achieved.

To consider Knudsen flow an intrinsic coefficient of permeability, \( k_v \), was calculated using Klinkenberg relationship [7]:

\[
k_v = k_v \cdot \left[1 + \left(\frac{b}{p_m}\right)\right]
\]

where \( b \) is the Klinkenberg coefficient and \( p_m = (p_i + p_o)/2 \) is the mean gas pressure. The intrinsic permeability of concrete subjected to a laminar gas flow can be given by a linear regression of the apparent permeabilities to an infinite average pressure.

After the measurements of the elastic modulus, three concrete discs of 48mm in thickness were cut off from the central part of the cylindrical specimen with a diamond saw. The lateral sides of the disc were sealed with silicon in order to ensure one-dimensional gas flow through the discs. All specimens were oven-dried at 60°C, then they were cooled for 48h in a desiccators at 20°C before being tested. After drying process, oxygen permeability test was driving on damaged and undamaged (references) concrete. In order to obtain representative values of permeability, the test was conducted on a six samples.
4 RESULTS AND DISCUSSION

4.1 Mechanical properties

The effect of the repeated freezing thawing cycles on the mechanical resistance was investigated. The investigation was performed for the reference samples and those exposed to 50, 100 and 150 freeze-thaw cycles. The values of compression strengths for the tested concretes after different number of cycles are presented in Table 2. Each of the presented data is an average value measured for three samples. Additionally, the error, defined as the maximum difference between results obtained for particular specimens and their average value, is presented. After 50th freeze-thaw cycle the internal frost damage to C2 concrete remained insignificant, while that to C1 concrete much more extensive. At the end of 50, the concrete C2 loses 20% of its initial strength then the loss almost remains without major changes until 150 cycles. As a consequence, this was as expected, because concrete with higher w/c ratio has a matrix with lower stiffness and contents more freezable water.

Table 2: Results of the compressive strength of specimens after the freeze-thaw cycles and reference samples

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Number of freeze-thaw cycles</th>
<th>Control samples</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Compressive strength (MPa)</td>
<td></td>
</tr>
<tr>
<td>C-1 [w/c=0.5]</td>
<td>54.0 ± 5.9</td>
<td>52.4 ± 3.7</td>
</tr>
<tr>
<td>C-2 [w/c=0.4]</td>
<td>75.8 ± 1.8</td>
<td>65.6 ± 2.6</td>
</tr>
</tbody>
</table>

Finally the impact of frost action on the variation of elastic modulus is discussed. Table 3 represents the elastic modulus evolution during the freezing-thawing cycles. The test was conducted on three specimens of each series and each number of freeze-thaw cycles. The presented results are the average of three measurements.

Table 3: Results of the elastic modulus of specimens after the freeze-thaw cycles and reference samples

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Number of freeze-thaw cycles</th>
<th>Control samples</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td></td>
</tr>
<tr>
<td>C-1 [w/c=0.5]</td>
<td>31.3 ± 0.3</td>
<td>19.1 ± 4.8</td>
</tr>
<tr>
<td>C-2 [w/c=0.4]</td>
<td>32.5 ± 0.2</td>
<td>32.3 ± 1.0</td>
</tr>
</tbody>
</table>

4.2 Oxygen permeability

Values of obtained intrinsic coefficient of permeability are presented in Table 4. The average permeability of samples which were stored in water, $k_{v,REF}$, is used as the references value for the expression of permeability changes. $k_{v,x}$ is assumed as the permeability of the samples which were subjected to 50, 100 and 150 freezing-thawing cycles, respectively.

The evolution of gas permeability for C1 was followed during 100 cycles then was stopped because of a material problem. Their permeability was too high to measure using Cembureau permeability apparatus due to appearance of micro-cracks. For concrete C-1, prepared with the highest w/c ratio, significant increase of intrinsic coefficient of permeability
can be observed between 50 and 100 freeze-thaw cycles. The permeability of analyzed specimens after the 50th cycle is 1.5 times higher than the references permeability and at the end of the 100th cycle is approximately 30 times higher than the permeability of undamaged specimens.

The relative permeability of damaged concrete \( k_{v,x}/k_{v,\text{REF}} \) is defined as the ratio between the permeability coefficient of damaged concrete, \( k_{v,x} \), and the permeability coefficient of references concrete, \( k_{v,\text{REF}} \), see Table 5. It can be related to the damage parameter \( d \). The increase in relative permeability \( k_{v,x}/k_{v,\text{REF}} \) with the damage coefficient evaluated by dynamic method is illustrated in Figure 1.

**Figure 1**: Relation between increase in permeability and damage value for concrete with water to cement ratio equals to 0.5(A) and 0.4 (B)
Table 4: Intrinsic coefficient of permeability with coefficient of variation in brackets, defined as a ratio of the standard deviation divided by the mean value of permeability

<table>
<thead>
<tr>
<th></th>
<th>$k_{v_x}$ [$10^{-17}$ m$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>x</td>
</tr>
<tr>
<td>C-1;</td>
<td>3.18</td>
</tr>
<tr>
<td>w/c=0.50</td>
<td>(21%)</td>
</tr>
<tr>
<td>C-2;</td>
<td>0.85</td>
</tr>
<tr>
<td>w/c=0.40</td>
<td>(20%)</td>
</tr>
</tbody>
</table>

Table 5: The mean value of relative change of intrinsic coefficient of permeability

<table>
<thead>
<tr>
<th></th>
<th>$k_{v_x}/k_{v_{REF}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>x</td>
</tr>
<tr>
<td>C-1;</td>
<td>1.47</td>
</tr>
<tr>
<td>w/c=0.50</td>
<td>1.07</td>
</tr>
<tr>
<td>C-2;</td>
<td>1.07</td>
</tr>
<tr>
<td>w/c=0.40</td>
<td></td>
</tr>
</tbody>
</table>

8 CONCLUSIONS

The paper contains preliminary results concerning the influence of frost damage of concrete on the mechanical and transport properties. The performed tests enabled to estimate damage of the material structure. The evolution of compressive strength, the elastic modulus and the permeability during freezing-thawing cycles were followed. The experimental results show that the reduction in the mechanical resistance and elastic modulus are accompanied by an increase in the intrinsic permeability of cement-based materials.

Due to frost action, the damage can be characterized by a parameter $d$. Results shows that there is a relation between the increase permeability, $k_{v_x}/k_{v_{REF}}$, and damage value, $d$, for nonaerated concretes. The intrinsic permeability coefficient increased with the increase of freeze-thawing cycles and the concrete with higher w/c ratio shows higher values. This can be explained by the gas flow through the pores and additionally through cracks in the case of frost induced damage.

REFERENCES


FIRST APPROACH TO THERMOCHROMIC MORTARS: COMPATIBILITY BETWEEN THERMOCHROMIC PIGMENTS AND CEMENT

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Key words: thermochromic; cement-based materials; optical properties

Abstract. Construction materials with reversible thermochromic behaviour are of significant interest to improve energetic efficiency in buildings and to reduce the environmental problem of the heat island effect associated to urban development. Within this context, this work studies the compatibility between thermochromic pigments and cement, as a first approach to the development of thermochromic mortars. Three reversible thermochromic commercial products have been considered, both in powder form and in aqueous solution. Microencapsulated pigments have been studied to increase their resistance to the highly alkaline environment and to the mechanically demanding mixing and casting procedures usual in cement-based materials. Ordinary white Portland cement paste samples with addition of a 20% by weight of each pigment have been prepared and the loss of their thermochromic property has been confirmed. The highly alkaline environment of the cement matrix has been demonstrated to be the leading factor for pigments degradation.

1 INTRODUCTION

Thermochromic materials are characterized by a change of their colour for a critical temperature value (Tc). These materials are especially interesting for building envelopes when the change is from a light colour at temperatures beyond Tc, to a dark colour at temperatures below Tc. With this behaviour it is expected that the solar reflectance would be high when the ambient temperature is high, thus reducing over-heating of the building during summer. On the contrary, when the weather is cold, the material would show a low solar reflectance and absorption of solar radiation would help warming the building in winter. Consequently, using reversible thermochromic materials in the building envelope would help to meet the demand of a thermally comfortable indoor environment, with the subsequent improvement in energy efficiency, and also would reduce environmental impact of urban construction.

This type of solutions have already been considered for thermochromic coatings in smart windows [1,2] cold roofs [3] and building coatings [4-6]. In order to improve the results of
this strategy applied to facades, the development of a reversible thermochromic mortar would be necessary. As a first step to achieve this target, the compatibility between pigments and cement based materials must be addressed. The cement considered for these applications must be light coloured, while the pigments must be uncoloured for high temperatures and dark coloured for low temperatures to assess a suitable behaviour of the mortar [7].

Our group is involved in the development of thermochromic mortars for facades coating based in two types of cements: ordinary white Portland cement (WC) and an eco-efficient belite cement synthesized from fly ashes through a hydrothermal process, which shows a light cream color [8]. As a first approach to this development, this work studies the compatibility between WC and three commercial encapsulated reversible thermochromic pigments.

2 MATERIALS AND METHODS

An ordinary white Portland cement has been considered in this work of the type BL II/A-L 42.5R according to the Spanish standard UNE 80305. Three commercial pigments have been added to the cement in order to assess their compatibility. They are all reversible thermochromic pigments with blue color in the colored state and with a nominal value of 15°C for the changing temperature. The pigment is in the three products enclosed in polymeric microcapsules and they degrade upon long time exposure to high temperatures. Other important properties of the pigments are collected in table 1. It is important to note that pigment BT15 is presented in powder form, while pigments 15RBR and CZ15 are in aqueous solution.

<table>
<thead>
<tr>
<th>Code</th>
<th>Supplier</th>
<th>Product</th>
<th>Form</th>
<th>%Pigment</th>
<th>Capsule Size (µm)</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>BT15</td>
<td>Kelly Chem Corp.</td>
<td>BT15</td>
<td>Powder</td>
<td>&gt;99</td>
<td>10</td>
<td>5-7</td>
</tr>
<tr>
<td>15RBR</td>
<td>Gem’innov</td>
<td>SLTH-15-BR ReflexBlue</td>
<td>Aqueous dispersion</td>
<td>50</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>CZ15</td>
<td>TMC Hallcrest</td>
<td>Chromazone Blue15</td>
<td>Aqueous dispersion</td>
<td>48</td>
<td>&lt;6</td>
<td>5.0-5.5</td>
</tr>
</tbody>
</table>

Four cement paste samples have been prepared, one of pure white cement as a reference and three prepared by adding to the white cement a 20% by weight of each pigment and keeping the same water/solid ratio of 0.5. The weight of each component in the preparation of the samples is shown in table 2, in which the percentage of solid pigment and water in pigments 15RBR and CZ15 have been taken into account. Small amounts of cement pastes have been considered in the study as it is intended to be a preliminary analysis of the compatibility between cement and pigments.

For the preparation of the samples the corresponding pigment has been mixed with water and stirred for 5 minutes to achieve a homogeneous dispersion that is added to the cement, mixed by hand and compacted in a plastic box. After 24 hours, the samples have been removed from the box for analysis.
Table 2: Composition of the cement paste samples

<table>
<thead>
<tr>
<th>Sample</th>
<th>WC (g)</th>
<th>Pigment Code</th>
<th>Pigment (g)</th>
<th>Water (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WC</td>
<td>5</td>
<td>-</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>WC-A</td>
<td>4</td>
<td>BT15</td>
<td>1</td>
<td>2.5</td>
</tr>
<tr>
<td>WC-B</td>
<td>4</td>
<td>15RBR</td>
<td>2</td>
<td>1.5</td>
</tr>
<tr>
<td>WC-C</td>
<td>4</td>
<td>CZ15</td>
<td>2.1</td>
<td>1.4</td>
</tr>
</tbody>
</table>

3 RESULTS AND DISCUSSION

3.1 Reversible thermochromic behavior of the pigments

As a first step in the study, the reversible thermochromic behavior of the pigments has been assessed. Figure 1 shows the aspect of the three pigments at 27ºC (ambient temperature), after cooling them down to 4ºC, well below their nominal changing temperature (15ºC), and after subsequent stabilization at ambient temperature. It is clearly seen that the three pigments turn darker after cooling and recover their initial lighter color after heating.

![Figure 1](image1.png)

Figure 1 – Aspect of the pigments beyond and below their nominal value for color change

3.2 Cement pastes with reversible thermochromic pigments

The aspect of the four cement paste samples defined in table 2 immediately after mixing and compacting is shown in figure 2. It is interesting to highlight the green color of the cement paste WC-B, incorporating pigment 15RBR, which is clearly different from the blue color of the raw aqueous dispersion at ambient temperature (figure 1). Figure 3 shows the final aspect of the samples after curing for 24 hours and demoulding. The main feature observed in the case of sample WM-A is the color inhomogeneity. In fact, the sample is blue in the outside but when a piece is separated from the bulk it is clearly white in the inside. Upon contact with air for some minutes, the new surface of the separated piece turns blue. Regarding WM-B, it shows a homogeneous green color, lighter than immediately after mixing. Finally, WM-C keeps the same color during the 24 hours of curing, very similar to the color of the raw pigment CZ15.
In order to confirm the behavior of the cement pastes for low temperatures, pieces of the four samples have been cooled down to 4ºC for more than 12 hours, together with a sample of the corresponding pigments. While the pigments show a clearly dark blue color upon cooling, no significant change in the cement pastes color has been observed, as confirmed by figure 4.

The results indicate that the cement pastes prepared with incorporation of thermochromic pigments do not show thermochromic behavior, suggesting that the pigments are degraded.
either during the mixing procedure or during the cement hydration reaction. This degradation may be related to damage of the microcapsules that protect the pigments or to a certain parameter that inhibits the chemical reaction responsible for the color change.

### 3.3 Analysis of the pigments

Different factors have been considered as possible causes of the non-thermochromic behavior of the cement pastes. Degradation of the microcapsules or the pigments might be due to the temperature increase in the early stages of hydration. Nevertheless, it has been visually confirmed that the three pigments keep their thermochromic behavior even after being heated up to 80°C. This temperature is clearly higher than that expected in cement paste during hydration, especially in the case of samples with such a low mass as those considered in this work.

The main factor that may be considered as responsible for the degradation of microcapsules or pigments is the highly alkaline environment within the cement matrix. In fact, the pH value of a white cement paste has been experimentally determined to be equal to 13.0. Taking this into account, the three pigments have been added to a 0.1M solution of KOH with experimental value of pH equal to 12.9. Figure 5 shows the aspect of the solutions. In the case of pigment BT15 the mix is initially blue, but immediately turns to light grey. This change in color could explain the light color inside the paste WC-A (figure 3) as being due to the high pH value in the matrix. Similarly, the solution changes its color quickly after adding pigment 15RBR, in this case turning from blue to green, as observed in the cement paste WC-B (figure 2). Finally, after the addition of pigment CZ15, the solution keeps a constant blue color, also in agreement with the aspect of WC-C (figure 2).

![Figure 5.- Aspect of the three pigments in a 0.1M KOH solution (25°C)](image)

The three solutions have been cooled down to 8°C, but no change in color has been perceived in none of them, thus suggesting that the thermochromic property of the three pigments is degraded in the highly alkaline 0.1 M solution of KOH, with pH similar to that of the white cement paste.

A similar experiment has been performed with a CaCO\textsubscript{3} saturated solution of experimental pH equal to 7.9. As can be seen in figure 6 the solutions after adding the pigments keep in this
case the blue color corresponding to the pigments. Moreover, it has been observed that the solutions turn to a clearly darker color after cooling down to 4°C (see figure 7), thus confirming that in this pH value the thermochromic property of the pigments is not degraded.

![Figure 6.- Aspect of the three pigments in a CaCO₃ saturated solution (25°C)](image)

![Figure 7.- Aspect of the three pigments in a CaCO₃ saturated solution (4°C)](image)

4 CONCLUSIONS

The following conclusions have been obtained from the results of the present work:

- The three commercial encapsulated reversible thermochromic pigments analyzed lose their thermochromic property when added to an ordinary white Portland cement matrix
- The highly alkaline medium of the cement paste (pH=13.0) has been identified as the factor causing degradation of the pigments
- In order to develop a reversible thermochromic mortar it is mandatory to define the pH limit for proper behavior of the pigments and to design a cementitious matrix with this pH value.

ACKNOWLEDGEMENTS

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REFERENCES


FORMATION OF AIR PORES IN CONCRETE DUE TO THE ADDITION OF TIRE CRUMB RUBBER

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Key words: crumb rubber, air pores, freeze-thaw-resistance, durability, sustainability.

Abstract: As a particular advantage of using tire crumb rubber in concrete an increase of the freeze-thaw resistance has been identified. In this context the present work focuses on the formation of air pores in concrete due to crumb rubber addition. Initially theoretical models are established and discussed. The subsequent experimental program includes an extensive microscopic analysis of the pore structure, air content and pore size distribution. Specimens were made with five different particle sizes of crumb rubber up to 600 µm and in each case with five different crumb rubber contents up to 32 kg/m³. Main result is that air voids adhere – due to the hydrophobicity of tire rubber – at the surface of rubber particles during the mixing process. Some of them separate from rubber particles and form spherical air voids in the cement paste. This model provides a basis to predict the air content and air pore distribution based on the particle size and content of crumb rubber.

1 INTRODUCTION

In many places in the world, used tires are still considered as waste and subsequently landfilled (Fig. 1). Thus, the valuable secondary raw material tire rubber remains unused. In addition, such landfills are highly vulnerable to fire as the past has already shown [13]. Main reason for landfilling used tires is a lack of recycling options. One potential recycling option can be seen in the use of crumb rubber as an additive in concrete. Pilot studies of Topçu & Özçelikörs [14] and Vincent et al. [17] dating back to 1992 show that crumb rubber increases the freeze-thaw resistance of concrete. This result has been confirmed in numerous later studies (Section 2). Therefore, the question arises whether crumb rubber could be a useful alternative to air-entraining agents. Possibly it enables a more precise control of the air content and air pore distribution. Besides this, it also could improve the environmental footprint and reduce the economic costs. All these issues should be studied in future. In order to provide a basis for this potential reuse of crumb rubber, it is necessary to understand the chemical and mechanical actions of it in concrete and specifically the formation of air voids. The present study aims to prepare this basis.
In the present study, following methodological structure was used: First, the current state of research was studied and addressed (Section 2). Subsequently a working model for the formation of air voids in fresh concrete was developed and theoretical predictions were derived (Section 3). On this basis, an experimental program was designed (Section 4). Finally, the theoretical predictions were validated experimentally (Section 5).

2 CURRENT STATE OF RESEARCH

The air content is positively correlated with the crumb rubber content [1, 3, 4, 16]. The measured values of Turki et al. [16] suggest – as determined by the authors of present paper – a linear relationship between air content and crumb rubber content. Furthermore the air content increases with fineness of crumb rubber [4, 19]. Exception: After Durham & Kardos [3] the air content decreases at high crumb rubber contents of 45 to 60 kg/m³.

Presumably, the formation of air voids in fresh concrete is based on the rough, hydrophobic surface of rubber particles [2, 5, 8, 11]. It should be noted that this presumption was casually expressed and not further considered. Pelisser et al. [7] and Turki et al. [16] state that air pores formed by crumb rubber can be differentiated into two types: those that are located in the ITZ between rubber particles and cementitious matrix and those that are located in a cementitious matrix. A chemical influence of tire rubber on the air entrainment has not been established. X-ray diffraction analysis has shown that the silicon content in cementitious matrix is increased by crumb rubber [7, 16]. This suggests that the silicon content in tire rubber may react pozzolanically and form CSH phases.

Crumb rubber has no effect on the capillary porosity of cement matrix [12]. This finding suggests that the previously outlined formation of CSH phases is not very explicit. Nevertheless, the water absorption coefficient and hydraulic diffusivity are both negative correlated with the crumb rubber content [1, 18]. It is believed that a reduction of the water absorption coefficient is due to an interruption of capillary pore connectivity by air pores [1]. Therefore it can be assumed that the water absorption coefficient also correlates negative with the fineness of the crumb rubber. Experimental studies on this are still pending.
The external damage to the microstructure during cyclic freeze-thaw loads correlates negatively with the fineness and the content of crumb rubber [4, 6, 8, 9]. Exceptions: If particle sizes are less than 150 μm, damage to the external structural increases until the crumb rubber content has reached 40 kg/m$^3$ [6]. After Durham & Kardos [3] the external damage is positively correlated with the crumb rubber content. A distinct characteristic to other experiments is that Durham & Kardos set a very low water-cement ratio of 0.35. Possibly the use of a superplasticizer is the cause for this deviation.

The internal damage to the microstructure during cyclic freeze-thaw loads is negatively correlated with the fineness and content of crumb rubber, provided that the crumb rubber content does not exceed approximately 14 kg/m$^3$ [3, 6, 8, 9, 15, 19]. If the crumb rubber content exceeds this limit, damage to the inner structure increases [3, 4, 6, 19]. Exceptions: After Gadkar & Rangaraju [4] damage to the inner structure continues decreasing when exceeding a crumb rubber content of 18 kg/m$^3$ for grain sizes of 300–600 μm. After Topçu & Demir [15] this is also true for crumb rubber with a fineness modulus of 1.5. All other studies include either finer or coarser crumb rubber, so that this exceptional effect could be related to the grain class.

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>TREND</th>
<th>SOURCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>air content</td>
<td>increasing</td>
<td>[1, 3, 4, 16]</td>
</tr>
<tr>
<td>capillary porosity</td>
<td>constant</td>
<td>[12]</td>
</tr>
<tr>
<td>water absorption</td>
<td>decreasing</td>
<td>[1, 18]</td>
</tr>
<tr>
<td>hydraulic diffusivity</td>
<td>decreasing</td>
<td>[1]</td>
</tr>
<tr>
<td>frost resistance of the external structure</td>
<td>increasing</td>
<td>[4, 6, 8, 9]</td>
</tr>
<tr>
<td>frost resistance of the internal structure</td>
<td>cr &lt; 14 kg/m$^3$</td>
<td>increasing</td>
</tr>
<tr>
<td></td>
<td>cr &gt; 14 kg/m$^3$</td>
<td>decreasing</td>
</tr>
</tbody>
</table>

3 THEORETICAL PREDICTIONS

First step is to design a working model that describes the formation of air voids in fresh concrete. Based on the results of Pelisser et al. [7] and Turki et al. [16] gas-forming chemical reactions between tire rubber and cement paste are unlikely. Tire rubber consists primarily of styrene butadiene, carbon black, aromatic oil, natural resins, zinc oxide and silica. Reaction of these substances with eg. calcium hydroxide by forming gas is also unlikely. In order to exclude gas-forming reactions completely, further experiments such as electron microscopy, calorimetry and thermogravimetry have to be carried out. In present work it is assumed that the air formation is not based on a chemical but on a physical mechanism.

It already has been suggested that the air entrainment is caused by the rough, hydrophobic surface of rubber particles (Section 2). According to this, external air should adhere to rubber particles during the mixing process. In the further course some voids separate from the rubber particles and shift into cement paste. During this process, they receive a spherical shape, while their volumes remain constant (Fig. 2). From this basic work model, the following predictions can be derived.
3.1 Prediction of air pore structure

It is expected that two different types of air pores exist: those that adhere to rubber particles and those that are located in the cementitious matrix. The former depend on the morphology of the rubber particles while the latter are exposed as spheres (Fig. 2).

![Figure 2: Predicted air pore structure with two types of pores — those that adhere to rubber particles and those that are located in the cement stone; V1 to V4 = volumes of the air pores](image)

3.2 Prediction of air pore inner angles

Furthermore, the inner angle $\theta_p$ of those air pores that adhere to rubber particles can be predicted (Fig. 5). This angle depends on the ratio of surface tensions between tire rubber, water contained in the cement paste and air. Therefore, it can be calculated from the contact angle $\theta_w$ between water and tire rubber in air (Fig. 3, left side). The contact angle $\theta_w$ corresponds to the outer angle of air pores that adhere to the rubber particles (Fig. 3, right side). This results in the following equation.

$$\theta_p = 180^\circ - \theta_w$$

Segre et al. [10] have determined the contact angles between different types of tire rubber and pure water as $(120 \pm 20)^\circ$. Assuming that the surface tension of water in fresh cement paste is approximately equal with that of pure water, air pore inner angles should be approximately $(60 \pm 20)^\circ$.

![Figure 3: Relation between the contact angle $\theta_w$ and the air pore inner angle $\theta_p$ with the interfacial tensions $\gamma_{w,a}$ between water and air, $\gamma_{s,w}$ between solid and water and $\gamma_{s,a}$ between solid and air](image)
3.3 Prediction of air content

If the hydrophobicity of rubber particles is the main cause for the formation of air pores, a proportional relationship between surface area of all rubber particles per unit volume of concrete and air content should be expected. From this context, two predictions arise:

(i) Since the surface area of all rubber particles per unit volume of concrete is proportional to the content of crumb rubber, a proportional relationship between air content and crumb rubber content is predicted.

(ii) The surface of all rubber particles per unit volume of concrete correlates positively with the fineness of crumb rubber. In addition, height of the air pores that adhere to rubber particles, depends on the size of the lateral surfaces of rubber particles. Smaller rubber particles generally have a larger specific surface and at the same time smaller lateral surfaces. Thus, for smaller rubber particles, the relative number of air pores increases while their single volumes decrease. Overall, it is assumed that the air content is positively correlated with the fineness of crumb rubber.

3.4 Prediction of pore size distribution

According to the designations in Figure 4, the volume of adhering air pore is equal to

\[ V_p = \frac{\pi}{3} \cdot h^2 \cdot (3r - h). \]  

With \( h = r \cdot (1 - \cos \theta_0), r = a/\sin \theta_0 \) and \( \theta_0 = \theta_p \), it follows

\[ V_p = \frac{\pi}{3} \cdot (2 - \cos \theta_p) (1 - \cos \theta_p) \left( a/\sin \theta_p \right)^3. \]

Assuming that the angle \( \theta_p \) is constant, this equation can be simplified to

\[ V_p = \kappa \cdot a^3. \]

Therein \( \kappa \) is a constant that depends on the hydrophobicity of tire rubber and the surface tension of water. For the predicted air pore inner angle \( \theta_p = 60^\circ \) (Section 3.2) it applies \( \kappa = 0.6 \). Equation 4 shows that the air pore volume largely depends on the radii \( a \), representing the bases of the hemispherical shaped air pores. These radii in turn are derived from the sizes of facets of the rubber particles. If \( d = 2a \) is the most frequent diameter of the facets of all rubber particles, the most frequent air pore volume can be predicted to

\[ V_p = \kappa \cdot d^{3/8}. \]

**Figure 4:** Designations of geometrical parameters of a hemispherical shaped air pore

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Adrian Zimmermann, Frank Röser and Eduardus A. B. Koenders
It is assumed that the particle size distribution of crumb rubber can be approximated as log-normal, after the grinding process. From this assumption it follows that the probability density function of the air pore volumes can also be approximated by a log-normal distribution. Usually crumb rubber becomes divided into different size fractions after the grinding process. Thus, the lognormal particle size distribution of rubber particles becomes divided into intervals. This leads to the following predictions:

(i) If the maximum particle size of a grain fraction falls below the average diameter of unfractionated particles, the proportion of larger air pores decreases.
(ii) If the maximum particle size of a grain fraction exceeds the average diameter of unfractionated particles, the proportion of larger air pores increases.
(iii) If a fine grain group is removed from unfractionated particles, the proportion of smaller air pores decreases.

An increase or decrease in larger or smaller air pores manifests itself by a vertical displacement of the distribution density function of air pore volumes for a specific interval. The boundaries of this interval can be calculated with equation 5.

4 EXPERIMENTAL PROGRAM

The subsequently described experimental program was designed to validate the work predictions that are postulated in Section 3.

4.1 Source materials

Crumb rubber was provided by Genan Business & Development A/S (Dorsten, Germany). It was fractionated in grain groups 0–125 μm, 125–180 μm, 180–250 μm, 250–400 μm and 400–600 μm. Its density was approximately 1.15 kg/m³ and its maximum water absorption was ≤ 1 % by weight.

As aggregate quartz rock, grain classes 0.08–0.2 mm, 0.2–0.5 mm, 0.5–1 mm, 1–2 mm and 2–8 mm was used. All aggregates met the requirements of DIN EN 12620:2003-4. Its density was approximately 2.6 kg/m³. The particle shapes could be described as sub-angular.

The binder was Portland cement CEM I 42.5 R according to DIN EN 197-1:2011-11 from Heidelberg Cement (Mainz, Germany). Its density was supposed to be 3.1 kg/m³.

A correction of the fresh concrete consistency was achieved by the addition of the PCE superplasticizer Sika® Viscocrete®-20 Gold from Sika (Stuttgart, Germany). Target value of the mortar slump according to DIN EN 12350-5:2009-08 was (19 ± 1) cm.

4.2 Mix designs

Crumb rubber with grain sizes 0–125 μm, 125–180 μm, 180–250 μm, 250–400 μm and 400–600 μm were substituted for aggregates of comparative particle classes. The crumb rubber content was in each case 0, 2, 4, 6 and 8 % by cement weight, respectively. The notation for the reference mixture without crumb rubber was ‘REF’. All mixtures with crumb rubber were notated in the format ‘P[maximum grain size of crumb rubber in μm]-[crumb rubber content in % by cement weight]’. A detailed overview of mix designs and their notations is given in Table 2.
4.3 Preparation of samples

Mixing of the source materials was carried out with a mortar mixer according to DIN EN 196-1:2005-5. The mixing sequence and mixing rate corresponded to the instructions in DIN EN 480-1:2015-01. The steel formworks had a cubic shape with an edge length of 15 cm. For compaction of the fresh concrete a vibrating Table with a frequency of 60 Hz was used. Curing of the specimens corresponded to the procedure described in Corrigendum 1 to DIN EN 12390-2:2009-08, Section NA.2. For the microscopic analysis, the cut surfaces were polished with abrasive paper up to grit P2000.

4.4 Microscopic analysis

Measurement of the air pore inner angles and air pore chord distances was carried out with a digital microscope. For determination of the air pore inner angles a 400 fold magnification was applied. Determination of the air pore chordal distances was done according to the manuel method described in DIN EN 480-11:2005-12. To obtain a sufficient measurement accuracy of about 5 μm, a 200 fold magnification was chosen instead of the normative recommended 100 fold. The air content and air pore distribution were calculated from the air pore chordal distances.
5 VALIDATION OF PREDICTIONS

Hereafter, experimental results are presented, interpreted and compared with the predictions from Section 3.

5.1 Validation of predicted air pore structure

As mentioned before, two types of air pores were identified: those that adhere to rubber particles and those that are located in the bulk cementitious matrix (see Section 3.1). As the micrographs in Figure 5 show, both predicted types of air pores are clearly visible. This observation is consistent with the descriptions of Pelisser et al. [7] and Turki et al. [16]. The upper right micrograph shows how air voids sequentially separated themselves from a rubber particle during the mixing process.

5.2 Validation of predicted air pore inner angles

Based on the interfacial tensions between tire rubber, water and air, an air pore inner angle of $(60 \pm 20)^\circ$ was predicted (Section 3.2). To validate this prediction, 50 of these angles were measured at 400 fold magnification (Fig. 5). The average measured air pore inner angle was $(60.5 \pm 7.8)^\circ$, which is in line with the predicted range.

It should be noted that superplasticizer may have an impact on the surface tension of water. According to the above theory, a lower surface tension of water causes an increase of the air pore inner angles. The dosage of superplasticizer was always less than 1 % of the cement content and thus very low. Nevertheless, this influence should be considered in future.
5.3 Validation of predicted air content

Furthermore, a proportional relationship between the crumb rubber content and air content was predicted (Section 3.3). Measured values of the present work can be interpolated linearly with a coefficient of nearly 1 (Fig. 6). Comparable data from Turki et al. [16] can be interpolated linearly with a coefficient of determination of 0.9974. It should be noted that the measurement data in Figure 6 are only based on the spherical air voids that are located in the bulk cementitious matrix. Therefore, the total air content is larger than the measured one.

![Figure 6: Relationship between the crumb rubber content and the air content](image)

It was predicted that the air content is positively correlated with the fineness of the crumb rubber (Section 3.3). In contradiction to this, measured air contents show a maximum for the grain group 180–250 μm. A possible reason for this could be statistical deviations due to the small number of specimens. To validate this results, further experiments have to be done.

![Figure 7: Relationship between the grain size of crumb rubber and the air content](image)

Figure 7: Relationship between the grain size of crumb rubber and the air content for a normalized crumb rubber content of 16 kg/m³
5.4 Validation of predicted pore size distribution

It was predicted that the air pore distribution is influenced by the maximum size of rubber particles (Section 3.4). The blue highlighted intervals in the graphs below (Fig. 8) show the predicted air pore volumes for each grain group of crumb rubber. For calculation of these intervals, equation 5 was used. Therein for parameter $d$ the maximum grain size of each grain group was set.

If the predicted maximum air pore volume is located on the left side of the functional maximum, the proportion of larger air voids decreases. Consequently, the function moves downwards on the right side of the blue highlighted interval. If the predicted maximum air pore volume is located on the right side of the functional maximum, the proportion of larger air voids increases. Consequently, the function moves upwards on the right side of the blue highlighted interval. A superposition of the probability density functions of air pore volumes for all grain groups provides the function shown at the bottom right of Figure 8. As predicted (Section 3.4), this function can be approximated by a log-normal distribution.

Smallest deviation from a log-normal distribution was found for grain group 180–250 $\mu$m (Fig. 8, middle left). This finding suggests that most frequent particle size of the unfractionated crumb rubber is approximately 250 $\mu$m. The manufacturer's grading curve is consistent with this result.

Figure 8: Distribution density functions of the air pore volumes for different grain groups of crumb rubber
6 CONCLUSION

Based on the model idea that the air entrainment in concrete is caused by hydrophobic surfaces of the rubber particles, the following conclusions were drawn:

- There are two types of air pores: hemispherical shaped, that adhere to the rubber particles and spherical shaped, that are located in the cementitious matrix (Section 3.1).
- Angles between the surfaces of rubber particles and surfaces of the air pores are approximately $(60 \pm 20)^\circ$ (Section 3.2).
- Crumb rubber content and air content are proportional (Section 3.3).
- Fineness of crumb rubber and air content correlate positive (Section 3.3).
- If $d$ is the most frequent diameter of facets of all the rubber particles, most frequent air pore volume is approximately $V_p = 0.6 \cdot d^{3/8}$ (Section 3.4).
- If unfractionated crumb rubber is used, the probability density function of air pore volumes can be approximated by a log-normal distribution (Section 3.4).
- If the maximum particle size of a grain fraction falls below the average diameter of unfractionated particles, the proportion of larger air pores decreases (Section 3.4).
- If the maximum particle size of a grain fraction exceeds the average diameter of unfractionated particles, the proportion of larger air pores increases (Section 3.4).
- If a fine grain group is removed from the unfractionated particles, the proportion of smaller air pores decreases (Section 3.4).

Most of these statements were confirmed experimentally (Section 5). Only the positive correlation between the fineness of crumb rubber and air content could not be verified (Section 5.3). Overall, no substantial contradiction to the original model idea was revealed. To confirm the model idea of air entrainment by crumb rubber more thoroughly, further studies on chemical reactions between tire rubber and cement paste are required. Only after the influences of crumb rubber on the formation of air pores are fully understood, it is appropriate to focus on considerations regarding the frost resistance.

From a practical point of view it seems advantageous that the air pore distribution can be controlled. Thus, it is possible to increase the proportion of micro air pores and consequently to improve the frost resistance. At the same time, concrete strength can probably be increased by the reduction in the proportion of macro air pores. Consequently, concrete components can be made thinner and in this way, improving the sustainability of concrete structures.

REFERENCES


FUNDAMENTAL STUDY ON SORPTION CHARACTERISTIC OF RADIONUCLIDE ION IN CEMENT AND BLAST FURNACE SLAG BASED SAMPLES

ICCS16

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Key words: Immobilization of Nuclear Waste, Cementitious Materials, Sorption, C-S-H

Abstract. Structures made of cement based materials deteriorate by various factors. Calcium (Ca) is a main component of cement matrices which is hardened with hydration process of cement. When water percolates into the cement hardened body under saturated conditions, the hydration products dissolve into pore solution and Ca ion leaches out from the hardened body, which makes its permeability become low. Since this phenomenon proceeds very slowly, it is not regarded as important deterioration in normal structures. In the context of radioactive waste management, however, long-term durability is one of the crucial aspects. The lifetime should be over thousands of years for the cement products used for the nuclear waste encapsulation and disposal facilities [1]. In radioactive waste processing structures, however, long-term durability is required, since the radioactive isotopes are hazardous for over thousands of years in extreme cases due to their half-life period if released into the environment. In the above-mentioned case, it is necessary to consider deterioration due to Ca leaching. The long-term durability of the cement products for nuclear waste applications is currently studied using the experimental leaching techniques [2][3]. In this study, simulated nuclear waste was tested in dissolution alteration and the behavior of radioactive nuclides was investigated. The results showed the improvement of dissolution deterioration resistance by replacement of Strontium (Sr) ion with Ca ion under NO₃ ion coexistence, but revealed that the phenomenon described above does not occur under Cl ion coexistence. Using the test results with simulated nuclear waste forms containing Cesium (Cs) and Sr, the solid-liquid equilibrium model focusing on Ca amount in solid phase and Ca, Sr and Cs ion in liquid phase were proposed.
1 INTRODUCTION

Structures made of cement based materials degrade by various factors. Leaching of calcium is one of such factors. Calcium is one of the main constituents forming hydration products of cementitious materials. When water percolates into the hardened cement body under saturated conditions, the hydration products dissolve into pore solution and calcium ions leach out from the hardened body. Since this phenomenon proceeds slowly, it is not regarded as serious deterioration in normal concrete structures. In the context of radioactive waste management, however, long-term durability is one of the crucial aspects. The lifetime should be over thousands of years for the cement products used for the nuclear waste encapsulation and disposal facilities [1]. It is essential to understand the long-term behaviour of materials to assure the sufficient properties and performance, including the degradation associated with calcium leaching.

The long-term durability of the cement products for nuclear waste applications is currently studied using the experimental leaching techniques [2][3][4]. As for the evaluation method for dissolution characteristics of cementitious materials, solid-liquid equilibrium model, which indicates dissolution rate of calcium hydrates at any solid-liquid ratios, has been proposed [5]. Using the similar experiments, sorption mechanism of nuclides on C-S-H, main component of cementitious material, and leaching behavior based on Ca/Si ratio of C-S-H has been also explained [2]. Additionally considering co-existing ions in ground water or seawater environment, the dissolution test with liquid-solid ratio (L/S) can show long term dissolution and leaching behavior chemically from cement hardened materials with nuclides in real environment.

In this study, Cesium (Cs) and Strontium (Sr) have been targeted. Although the behaviors of Cs and Sr in Ordinary portland cement (OPC, specific surface area: 3520cm²/g) and Blast furnace slag powder (BFS, specific surface area: 2860cm²/g) hardened materials have been reported in previous studies [6], in this study, the above mentioned dissolution tests have been performed on OPC/BFS with Cs/Sr loaded sample and OPC sample to investigate the leaching and dissolution behavior with a consideration on co-existing ions in the leachate.

2 EXPERIMENTAL OUTLINES

The OPC sample with W/C of 0.4 was prepared and cured for 2 years (N sample). The blended cement sample was BFS:OPC at 9:1 ratio with W/B of 0.37 cured for 42 days (B sample). To incorporate into the blended cement, caesium nitrate CsNO₃ and strontium nitrate Sr(NO₃)₂ were dissolved in water before mixing with binders (CS sample), since it has been reported that Low-Level Waste (LLW) slurry contains Nitrates [7]. Once cured, the cement samples were cut into square discs with dimensions of 5x40x40 mm, and each specimen was immersed into a large volume of acetone at 20°C for 30 minutes under vacuum condition in order to arrest the hydration reaction. Chemical components of OPC and BFS is shown in Table 1 and Mixing formula of the blended cement sample is shown in Table 2.

After curing, 5x5x40mm samples were dipped into a large volume of acetone at 20°C for 30 minutes under reduced pressure (20°C, $2.33 \times 10^3$ Pa), in order to terminate the hydration reaction. Afterwards, reduced pressure drying or D-drying ($6.666 \times 10^{-2}$ Pa) were carried out for 24 hours to be ready the dissolution test. The thin discs were respectively ground into powders<105µm.
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Liquid-solid (L/S) ratio, weight ratio of solution to powder sample, was controlled to be set at 10, 50, 100, 500, 1000 with respectively using deionized water (D) or leachate solution (L), which obtained in static leaching test [4]. Each container was sealed and statically left for 56 days in R.H. 100% and 20°C environment. This test with variety of L/S can simulate different equilibrium conditions on dissolution of hydration products and ion exchange behavior. Outline of dissolution test is described in Figure 1.

**Table 1. Chemical component of OPC and BFS**

<table>
<thead>
<tr>
<th></th>
<th>CaO</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>MgO</th>
<th>SO₃</th>
<th>K₂O</th>
<th>Na₂O</th>
<th>Chloride</th>
<th>Insolubles</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>64.58</td>
<td>20.96</td>
<td>5.24</td>
<td>2.61</td>
<td>2.09</td>
<td>2.46</td>
<td>0.59</td>
<td>0.28</td>
<td>0.048</td>
<td>0.27</td>
</tr>
<tr>
<td>BFS</td>
<td>42.10</td>
<td>34.50</td>
<td>13.74</td>
<td>0.97</td>
<td>7.29</td>
<td>-</td>
<td>0.49</td>
<td>0.22</td>
<td>0.022</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table 2. Mixing formula of cement samples**

<table>
<thead>
<tr>
<th>Sample (Label)</th>
<th>Cement Matrix (g)</th>
<th>Waste Simulant (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OPC</td>
<td>BFS</td>
</tr>
<tr>
<td>Blank (B)</td>
<td>80</td>
<td>720</td>
</tr>
<tr>
<td>3wt% Cs and 3wt% Sr (CS)</td>
<td>80</td>
<td>720</td>
</tr>
</tbody>
</table>

**Figure 1. Outline of dissolution test**

In this study, two types of waste simulated samples described above were tested in three levels (test 1-1, 1-2 and 2) with different solutions are explained below. It is be noted that the dissolution deterioration test with NO₃ ion (test 1-1) in order to understand the sorption characteristics of the radionuclide ion with respect to the sample in Test 1-1 and 1-2 or Cl ion (test 1-2). Test 2 the dissolution deterioration test in order to understand the characteristics using sample N, B and CS respectively.

In test 1-1, the sample particle size <105μm was prepared by ball milling. The solid-liquid ratio, which is mass ratio of outer solution to the mass of the powder sample in weight, is 10, 50, 100, 500, 1000 and 2000, with respect to 0.01mol/L of Sr(NO₃)₂ (solution 1), 0.01mol/L of CsNO₃ (solution 2), and both mixed 0.01mol/L each (solution 3). The conditions were statically kept for 56 days in 20°C. They were sealed with stirring every 7 days to reach the equilibrium condition.

In test 1-2, SrCl₂ solution of 0.01mol/L as the solid-liquid ratio with respect to the same powder sample is 10, 100, 1000 (solution 4), CsCl solution of 0.01mol/L (solution 5), SrCl₂ and CsCl respectively 0.01mol/L mixed solution (solution 6) were used, similarly with test 1-1, sealed for 56 days at 20°C environment. Moreover, stirring of the powder sample and the
solution was carried out with the same frequency as test 1-1. In test 2, the samples prepared (N, B and CS) were pulverized by ball milling into particle size 250µm or less of a powder sample. The weight ratio of outer solution (Cs ion concentration 0.111mol/L and Sr ion concentration is 0.076mol/L (solution 7)) to the powder samples was set at 10, 50, 100, 500, 1000 and 2000. Also, the containers were sealed for 56 days in 20°C, where the solution was stirred at 1, 7, 14, 21, 28, 56 days elapsed before measuring pH.

X-ray diffraction (XRD) is an analytical technique where crystalline phases of a material diffract a beam of X-rays at specific angles. The intensity of the diffracted X-rays are measured at different angles between a range defined by the user of the XRD machine. As for the data included in this paper, the powdered samples are placed on an Aluminum sample holder with range being 5-60° with a step size of 0.05°. The XRD machine is set to a speed of 2°/minute. Also, thermogravimetry and differential thermal analysis (TG-DTA) is used to examine how the weight of a sample changes with respect to the temperature of the sample. This data can then be used to calculate the weight of the different phases present in the sample for a quantitative analysis using when the weight of the sample changes with respect to temperature.

3 RESULTS AND DISCUSSION

3.1 TG-DTA peaks of hydration products

In test 1-2, when the solid-liquid ratio is less than or equal to 100, the peak that indicates the crystalline CSH-phase was increased from the initial state, as different results from test 1-1, although the C-S-H peak has decreased with all of the solid-liquid ratio in test 1-1 results of L/S 1000.

As shown in Figure 2, the increment of C-S-H phase, in the case the solid-liquid ratio is less than or equal to 100, was probably led by hydration of the unreacted cement. On the other hand, in test 1-1 when NO₃ ions coexisting, the peak of the C-S-H phase was increased, in test 1-2 when Cl ion coexisting, the peak of C-S-H phase was reduced. That is, rather than under NO₃ ions coexisting environment, exchange of Ca ions and Sr ions that results in high C-S-H stability was likely to have performed under Cl ion coexisting.

3.2 XRD peaks of hydration products

Hydration phase, especially calcium aluminates containing Cs and NO₃ could not be confirmed. In this result, Sr ions were precipitated as a solid (SrCO₃) by immobilization or carbonation not in C-S-H phase, when Cs ions were supposed to be hardly taken into the cement hydration production phase. Figure 3 shows XRD peak patterns.

3.3 Ion sorption characteristics on Ca hydrates

Figure 4 shows an example of solid-liquid equilibrium model for Ca hydrates system under Cs and Sr ion coexistence. A comparison between the sample N and B, sample B has a larger decomposition rate of Ca, despite smaller amount of cement hydration, from the solid phase than sample N. That is, as for the solubility resistance of Ca ions, it was confirmed
sample B is lower than sample N. On the other hand, although the amount of cement used in sample CS the same as sample B, the equivalent slope shows the similar trend with sample N for the solid-liquid equilibrium related to Ca ions.

**Figure 2. TG-DTA results with NO₃ and Cl coexistence**

Based on Ca and Sr ion concentrations, the distribution has been shifted to the left (lower ion concentration) in the order of Cs, Sr and Ca. Consistent results with an alkali metal sorption experiments for C-S-H describe the possibility of electrostatic sorption on the negatively charged surface. In addition, according to the experimental results in this study, the sorption amount of the alkali metal on C-S-H was increased when the hydration ionic radius becomes small, where Ca/Si ratio of C-S-H causes the electrical surface potential. In other words, Sr ion is immobilized and replaced with Ca ions of C-S-H, and Sr ions can be released when Ca/Si ratio of C-S-H is reduced due to decomposition of Ca hydration system. However, in consideration of Cs ion sorption behavior on C-S-H surface, the above results explain that Cs ion concentration in the liquid phase compared to Sr ion is able to be kept low based on the description of electrically negative charge.

**Figure 3. XRD patterns of hydration products**
3.4 Leaching behavior from simulated waste samples

Since Ca concentration of CS samples was raised up at high solid-liquid ratio, it is considered that the Ca hydrate lost the solubility resistance as shown in Figure 5. As alkali metal dissolution, such as Na and K, is theoretically the dominant diffusion phenomena, a certain amount is leached out from the powder sample regardless of the L/S (driving force of dissolution and diffusion). Otherwise, the total leaching amount was perfectly not depending on the experimental condition set in this study. The reason why Ca concentrations were not constant according to L/S is the leaching from Portlandite, AFm and AFt which were less dominantly soluble than Na and K. Sr and Cs ions did not show much change in leaching amount from the original situation almost in the same way as Na and K. It can be said that Sr bonded in C-S-H or other Ca-Al phase, which must be difficult to dissolve in static leaching test, was probably ready to be soluble from the powder samples. There is a high possibility of Cs leaching amount was controlled by electrostatic sorption to the C-S-H of Na and K ion.
3.5 Solid-liquid equilibrium of hydration phase with co-existing ions

Evaluating above mentioned behavior of Ca ion based on the consideration of coexisting ions including nuclides such as Cs and Sr that are mainly targeted to disposal strategy of radioactive wastes, solid-liquid equilibrium models, shown in Figure 6, can be described and proposed to numerically understand the long term behavior of cementitious hardened waste form.

![Figure 6. Examples for solid-liquid equilibrium models](image)

4 CONCLUSIONS

- When NO₃ ion coexisting, there is a possibility that the stability of C-S-H phase is improved due to the exchange of Ca with Sr ions. On the other hand, since the ion exchange is not performed under Cl ion coexisting, the stability of C-S-H phase is not improved and less durable than original hydration products.

- NO₃ and Cl ion with any species of coexisting ions, Cs ions is hard to immobilized in hydrated production phase, but the sorption of Cs ions as Ca/Si ratio is lowered with decomposition of C-S-H phases can be expected.

- To obtain Ca ion dissolution and leaching behavior, this study proposed for the examples the solid-liquid equilibrium relationship based on the ion concentration of Cs and Sr in nuclear waste form.

ACKNOWLEDGEMENT

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GEOPOLYMERISATION ACTIVITY OF EIFEL TUFF

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KEY WORDS: volcanic tuff, microstructure, geopolymers, alkali activated binders, sustainability.

SUMMARY. Alkali activated binders could significantly reduce the CO₂ emissions associated with the manufacturing of conventional cement, as they are made solely from industrial by-products and/or natural alumino-silicates. Alkali activation of volcanic tuff from the Eifel volcanic area known as Rhenish Trass is investigated here. Apart from silicate and aluminate content that exhibits good pozzolanic reactivity, the use of the Tuff in alkali activated binders is further motivated by its alkali content. Geopolymer paste mixtures were mixed with different amounts of sodium silicate and sodium hydroxide activation solutions to investigate the effect in molar ratios of Si/Al (2.8, 3.1, 3.4, 3.6) and SiO₂/Na₂O (8.6, 6.9, 5.7, 5.2, 4.1, 3.6) on mechanical strength and rheology. Best results were obtained by using waterglass activator mixes with Si/Al=3.4, with an optimal SiO₂/Na₂O ratio of 5.2. Workability of the mixes with waterglass increased with Si/Al ratio. Geopolymerisation reaction indicated an increase in amount of non-evaporable water and an increase in amount of amorphous and zeolite (Analcime and Phillipsit) phases. The results indicate a potential on utilisation of natural Eifel Tuff as a novel raw material for alkali activated binders.

1 INTRODUCTION

Alkali activated binders (AAB) are made solely from industrial by-products and/or natural alumino-silicates, comprising no traditional cements. Thus they could significantly reduce the CO₂ emissions associated with the manufacturing of conventional cement which is responsible for about 5% of the anthropogenic global emissions. However, application of this new material also requires consideration of potential emissions, price and hazards (high pH) of the activator solution. Therefore, lower amounts of alkalies in activator solutions are desired. Alkali hydroxide solutions and soluble silicate (waterglass) solutions with SiO₂/Na₂O molar ratios smaller than 1.45 are considered hazardous, which limits the field application of geopolymer mixes using such solutions. High alkali contents in powder percursors, e.g. as found in the Eifel tuff (Table 1), could allow for reducing the concentration of alkali solutions, used as activator [1,2,3].

Rhenish Trass is the local name for a volcanic tuff located in the eastern Eifel (Germany). It was created by the eruption of the Laacher See and about 100 other volcanos in the eastern Eifel that were active until about 13,000 years ago. The origin of the term Trass comes probably from the Latin word terrenus = earthly. A more general, international name for it is...
Eifel tuff. It is grey to cream-colored and largely composed of pumiceous dust, which may be further regarded as a trachytic tuff. In weathering, trachytic tuffs often change to clay-stones with secondary quartz. The fine-grained matrix of Eifel tuff consists of glassy solidified ash particles and crystallites as leucite, nosean, feldspars, pyroxenes, hornblende and mica. The matrix embeds basalt fragments and non-magmatic xenoliths rock inclusions. The glass content is about 50-60 % [4,5]. After the eruptions, gradually with time the glass component of the initial volcanic tuff crystallizes to Zeolite minerals like Phillipsit and Chabazite, and in further reactions also to Analcime. Today, Rhenish Trass is offered and processed in blended binders by two companies: Tubag and Meurin. Mining is currently done exclusively in the Selbergittuff in Thürer forest belonging to Riedener volcano complex. In mortars, Trass is today mainly used because of its benefits as a fine-grain filler and less due to its pozzolanic properties. The pozzolanic reaction of Rhenish Trass, in a shaken calcium hydroxide solution, is significant only after 7 days [6], but is almost complete after 180 days [5].

Table 1: Chemical composition (mass %) of used materials

<table>
<thead>
<tr>
<th>Material</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>CaO</th>
<th>Fe₂O₃</th>
<th>MgO</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>H₂O</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trass</td>
<td>58.60</td>
<td>17.72</td>
<td>4.66</td>
<td>6.18</td>
<td>2.35</td>
<td>3.41</td>
<td>5.06</td>
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</tr>
<tr>
<td>NaOH 50%</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td>38.75</td>
<td>61.25</td>
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<td>Water glass</td>
<td>27.15</td>
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<td></td>
<td></td>
<td></td>
<td>8.25</td>
<td>64.60</td>
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</tr>
</tbody>
</table>

Geopolymers belong to a class of alkali activated binders and are sub-categorised by a criteria of low Ca content (below 5-10 mass %). They harden due to formation of AlO₄⁻-SiO₄ tetrahedral framework, where alkali or alkaline earth cations (Na⁺, K⁺, Ca²⁺), as well as H⁺ act as charge balancing species [2,7,8]. Their structure is considered to be an amorphous analogue to zeolites. Geopolymer chemical structure varies significantly depending on the prevailing factors of their structural frameworks, in analogy to great variety of zeolite structures [8,9]. Various geopolymers have different semi-crystalline structures arising from variations in composition, distribution and ordering of AlO₄⁻-SiO₄ tetrahedral units in linkages of their global structural (Al-O-Si-) framework [7,9]. Primarily, variability in Si/Al ratio results in differences in the location, amount and distribution of negative charge density in the structural frameworks, cages or pores of different diameters, nature or absence of hydration water or other ligands and presence and position of charge compensating cations. This paper investigates alkali activation of the Eifel tuff, and in particular the effects of Si/Al and SiO₂/Na₂O molar ratios on mechanical strength and rheology. Geopolymisation activity was investigated using XRD, thermogravimetry (TG) and Fourier transformed infra-red spectroscopy (FTIR).

2 EXPERIMENTAL

Materials used for geopolymer synthesis are listed with their chemical composition in Table 1. Trass was obtained as a commercial product “TM Trassmehl” (DIN 51043) produced by Tubag Trass Vertrieb GmbH & Co.KG (LOI = 5.40 %, Blaine 6000 cm²/g, 2.65 g/cm³, compressive strength of Trass/Ca(OH)₂ mortars (DIN 51043) was 7.8 MPa). Commercial water glass and NaOH pellets (≥ 99 % p.a.) are obtained from Carl Roth GmbH & Co.KG.
NaOH 50% solution was prepared by mixing the pellets with water at least one day before mixing and casting the specimens. In the experimental plan (Table 2) the Si/Al and Na₂O/SiO₂ ratios were varied by using different amounts of the same starting solutions (Table 1) and adjusting the addition of extra water. Geopolymer paste specimens (40×40×160 mm) were prepared according to EN-196-1 but without sand and with a water to reactive component ratio (w/g) of 0.375. Workability (fluidity) of the fresh state mortar was tested using the standard flow table test according to EN 1015-3. Specimens were sealed by plastic and cured in an oven at 80 C for 3 and 7 days. Compression tests at 3 and 7 days were done according to EN 1015-11. Original Trass and SA3.4-SN5.2 geopolymer samples were investigated by powder XRD, Fourier transformed infra-red spectroscopy (FTIR) and thermogravimetry (TG). Hardened paste was ground to reduce the grain size and both powdered paste samples and original Trass samples were dried for 3 days at 105 °C.

**Table 1:** Mix design of geopolymer paste samples: notation, molar ratios, w/g mass ratio and amount of precursors (kg/m³)

<table>
<thead>
<tr>
<th>Sample name</th>
<th>Si/Al</th>
<th>SiO₂/Na₂O</th>
<th>w/g</th>
<th>Sodium-silicate</th>
<th>Sodium-hydroxide</th>
<th>H₂O Trass</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA2.8-SN8.6</td>
<td>2.8</td>
<td>8.6</td>
<td>0.375</td>
<td>/</td>
<td>121</td>
<td>431</td>
</tr>
<tr>
<td>SA2.8-SN6.9</td>
<td>2.8</td>
<td>6.9</td>
<td>0.375</td>
<td>/</td>
<td>179</td>
<td>398</td>
</tr>
<tr>
<td>SA2.8-SN5.7</td>
<td>2.8</td>
<td>5.7</td>
<td>0.375</td>
<td>/</td>
<td>235</td>
<td>367</td>
</tr>
<tr>
<td>SA2.8-SN5.2</td>
<td>2.8</td>
<td>5.2</td>
<td>0.375</td>
<td>/</td>
<td>271</td>
<td>347</td>
</tr>
<tr>
<td>SA2.8-SN4.1</td>
<td>2.8</td>
<td>4.1</td>
<td>0.375</td>
<td>/</td>
<td>360</td>
<td>297</td>
</tr>
<tr>
<td>SA2.8-SN3.6</td>
<td>2.8</td>
<td>3.6</td>
<td>0.375</td>
<td>/</td>
<td>417</td>
<td>265</td>
</tr>
<tr>
<td>SA3.1-SN8.6</td>
<td>3.1</td>
<td>8.6</td>
<td>0.375</td>
<td>297</td>
<td>76</td>
<td>271</td>
</tr>
<tr>
<td>SA3.1-SN6.9</td>
<td>3.1</td>
<td>6.9</td>
<td>0.375</td>
<td>294</td>
<td>135</td>
<td>239</td>
</tr>
<tr>
<td>SA3.1-SN5.7</td>
<td>3.1</td>
<td>5.7</td>
<td>0.375</td>
<td>290</td>
<td>195</td>
<td>208</td>
</tr>
<tr>
<td>SA3.4-SN8.6</td>
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<td>8.6</td>
<td>0.375</td>
<td>501</td>
<td>45</td>
<td>161</td>
</tr>
<tr>
<td>SA3.4-SN6.9</td>
<td>3.4</td>
<td>6.9</td>
<td>0.375</td>
<td>495</td>
<td>108</td>
<td>129</td>
</tr>
<tr>
<td>SA3.4-SN5.7</td>
<td>3.4</td>
<td>5.7</td>
<td>0.375</td>
<td>489</td>
<td>168</td>
<td>99</td>
</tr>
<tr>
<td>SA3.4-SN5.2</td>
<td>3.4</td>
<td>5.2</td>
<td>0.375</td>
<td>486</td>
<td>208</td>
<td>79</td>
</tr>
<tr>
<td>SA3.4-SN4.1</td>
<td>3.4</td>
<td>4.1</td>
<td>0.375</td>
<td>477</td>
<td>305</td>
<td>30</td>
</tr>
<tr>
<td>SA3.4-SN3.6</td>
<td>3.4</td>
<td>3.6</td>
<td>0.375</td>
<td>477</td>
<td>366</td>
<td>/</td>
</tr>
<tr>
<td>SA3.6-SN8.6</td>
<td>3.6</td>
<td>8.6</td>
<td>0.375</td>
<td>687</td>
<td>16</td>
<td>60</td>
</tr>
<tr>
<td>SA3.6-SN6.9</td>
<td>3.6</td>
<td>6.9</td>
<td>0.375</td>
<td>679</td>
<td>80</td>
<td>30</td>
</tr>
<tr>
<td>SA3.6-SN5.7</td>
<td>3.6</td>
<td>5.7</td>
<td>0.375</td>
<td>671</td>
<td>142</td>
<td>0</td>
</tr>
</tbody>
</table>

XRD analysis of powdered samples was done using a D2 PHASER (Bruker) desktop diffractometer configured with the fast linear LYNXEYE detector and CuKα1,2 radiation (30 kV, 10 mA). The samples were spiked with 10 wt % corundum (SRM 674) and homogenized. American Mineralogist Crystal Structure Database was used for mineral structure referencing [10].

FTIR spectroscopy analysis of powdered samples was done on VERTEX 70 ATR (Bruker). TG analysis was performed on STA 449 F3 Jupiter® (Netzsch) at a heating rate of 10 °C/min by using 20 mL/min of nitrogen flow and alumina crucible.
3 RESULTS AND DISCUSSION

The mix design calculations were implemented in a spreadsheet model (Figure 1). To control the water content in the mix design, a parameter ‘Water-to-Geopolymer reactive solids mass ratio’ (w/g) was used. In w/g the w represents the mass of water from both the NaOH and waterglass solutions as well as extra added water. The g represents the total mass of materials in the solid phase (i.e. Trass) plus the reactive dissolved precursor part in the solution: i.e. amount of dissolved Na$_2$O and SiO$_2$.

![Figure 1: Excel spreadsheet mix design model.](image)

Compressive strength increases with time of curing for all mixes (comparison of Figure 2 for 3 days and Figure 3 for 7 days). Table 3 shows results for all mixtures, while for clarity reasons Figure 2 and Figure 3 depicts only results for Si/Al ratios 2.8 and 3.4. Best results were obtained by using waterglass activator mixes with Si/Al = 3.4. For this Si/Al case there is an optimal value for SiO$_2$/Na$_2$O of 5.2 (Figure 2), and from this point on the left side the strength gradually decreased with an increase of alkalinity (towards smaller SiO$_2$/Na$_2$O ratios), while on the right side the strength decreased rapidly with a decrease in alkalinity. This optimum could be explained by the fact that higher alkalinity makes Trass more dissolvable, but on the same time the formed geopolymer products are also more dissolved. During Trass dissolution, alkalies are released [6] that could catalyze further dissolution, but the overall reaction rate is more complicated, also by the effect of soluble alkalies from other geopolymerisation reaction steps, such as for example condensation of reaction products. There was no significant effect of Si/Al ratio on strength results for mixes using waterglass within the comparable range of 5.7 ≥ SiO$_2$/Na$_2$O ≥ 3.6. Activation of Trass by only NaOH (Si/Al = 2.8, no waterglass) exhibited the lowest strengths.
### Table 3: Compressive strength and spread of Geopolymer paste samples

| Sample name | Compressive strength | Spread | | | |
|-------------|----------------------|--------|---|---|
|             | 3d | 7d | [MPa] | [cm] | |
| SA2.8-SN8.6 | 5.3 | 6.3 | 14.1 | | |
| SA2.8-SN6.9 | 4.0 | 6.8 | 14.0 | | |
| SA2.8-SN5.7 | 3.6 | 4.1 | 14.7 | | |
| SA2.8-SN5.2 | 4.2 | 10.7 | / | | |
| SA2.8-SN4.1 | 2.8 | 8.5 | / | | |
| SA2.8-SN3.6 | 2.7 | 4.5 | / | | |
| SA3.1-SN8.6 | 4.9 | 5.8 | 13.2 | | |
| SA3.1-SN6.9 | 7.3 | 11.9 | 15.4 | | |
| SA3.1-SN5.7 | 5.2 | 11.4 | 14.6 | | |
| SA3.4-SN8.6 | 1.4 | 8.9 | 14.5 | | |
| SA3.4-SN6.9 | 7.4 | 9.1 | 16.4 | | |
| SA3.4-SN5.7 | 6.2 | 11.4 | 15.8 | | |
| SA3.4-SN5.2 | 9.1 | 22.7 | / | | |
| SA3.4-SN4.1 | 2.8 | 14.6 | / | | |
| SA3.4-SN3.6 | 1.8 | 5.2 | / | | |
| SA3.6-SN8.6 | 2.4 | 12.4 | 15.1 | | |
| SA3.6-SN6.9 | 8.1 | 12.5 | 17.7 | | |
| SA3.6-SN5.7 | 6.8 | 9.5 | 17.9 | | |

**Figure 2:** Compressive strength [MPa] after 3 days, as a function of molar SiO$_2$/Na$_2$O ratio
Effect of Si/Al and SiO$_2$/Na$_2$O molar ratios on rheology of the freshly prepared pastes is shown in Figure 4. One can observe that workability of the mixes with waterglass increases with Si/Al ratio. The effect of SiO$_2$/Na$_2$O is complex. For mixes activated only by NaOH (Si/Al = 2.8, no waterglass) there is no effect on workability for SiO$_2$/Na$_2$O = 8.6 and 6.9, but an improvement with the highest alkalinity (SiO$_2$/Na$_2$O = 5.7). For lowest alkalinity, SiO$_2$/Na$_2$O = 8.6, compared to mixes without waterglass, the workability decreased for low Si/Al ratios (3.1 and 3.4). For Si/Al ratios 3.1 and 3.4 there is an optimal workability for middle value of the alkalinity. On the other hand for Si/Al ratio of 3.6 workability continuously increases with alkalinity.

Figure 3: Compressive strength [MPa] after 7 days, as a function of molar SiO$_2$/Na$_2$O ratio

Figure 4: Spread of Geopolymer paste mixes with molar Si/Al ratio 2.8, 3.1, 3.4, 3.6 and molar SiO$_2$/Na$_2$O ratio 8.6, 6.9, 5.7
Figure 5 shows the mineralogical composition of Trass and SA3.4-SN5.2 geopolymer sample cured for 7 days. Generally, both diffractograms share same mineralogical composition. With reference to spiked corundum peaks, a semi-quantitative analysis was performed. The amount of phases decreases in following order: Amorphous phase, Quartz, Sanidine, Muscovite, Analcime, Pyroxene, Chabazite, Chlorite, Leucite, Albite, Phillipsite, Calcite. Low and broad diffraction lines can be observed, indicating existence of amorphous phases, 55 % [4,5]. For the geopolymer specimens, these amorphous phases could be a combination of the unreacted amorphous oxides in the original Trass and the newly generated amorphous geopolymer compounds. Semi-quantitative comparison of the two samples indicated following trends as a result of a geopolymerisation reaction: 1) increase in amount of amorphous phase, Analcime and Phillipsite; and 2) decrease in amount of Quartz, Sanidine, Pyroxene, Chabazite, Chlorite, Leucite, Albite and Calcite. The increase in amount of amorphous phase would mean that there is a net greater amount of produced amorphous Geopolymer products than the amount of dissolved amorphous phase in Trass. This can be attributed to additional amount of dissolved solids in activator solution that together with dissolved Trass produces amorphous geopolymer phase, which amounts to 19.6 % of the total reactive solids (remaining 80.4% is from Trass). Considering the dilution effect of the additional solids in the system from the activator solution, would result in even bigger increase in amorphous phase as a result of geopolymerisation reaction. Apart from formation of amorphous geopolymer, the reaction also forms significant amounts of zeolite crystalline products like Analcime and
Phillipsite. It is not possible to proof the reactivity of the Trass with semi-quantitative approach, because it is difficult to separate the dilution and reactivity effect which both decrease the peaks in measured diffractograms. Quantitative analysis employing Rietveld refinement would give better insight about the reactivity of the Trass and amounts of new geopolymer (amorphous and zeolite) phases formed.

Figure 6: Thermogravimetric (TG) analysis of Trass and geopolymer paste samples SA3.5-SN5.2 cured for 3 and 7 days.

TG analysis (Figure 6) shows the presence of water and carbonates, the total amount of which is increased with geopolymerisation reaction. Water is present in the loosely held H$_2$O molecules condensed in small pores. Secondly the water molecules are inside the structure in the form of solvated H$_2$O molecules, which form most probably octahedron coordination around the charge compensating alkalies: Na$^+$$(H_2O)_6$ or K$^+$$(H_2O)_6$. Thirdly the more strongly bounded hydroxyl (–OH$^–$) groups are present. When heating to about 400°C, the main part of water dehydrates. After about 600°C the mass loss is due to decarbonation, whose effect is decreased with geopolymerisation reaction.

Analysis of infra-red (FTIR) spectrum (Figure 7) shows a band shift of the broad absorption band at ~980 cm$^{-1}$ as a result of a geopolymerisation reaction. This band corresponds to Si-O-T (T=Al or Si) asymmetric stretching vibrations and in original Trass its maximum is located at 983 cm$^{-1}$, while it is shifted to lower value in geopolymer sample (970 cm$^{-1}$). This shift is due to Al incorporation into original Si-O-Si skeletal structure.
4 CONCLUSION

Geopolymer paste mixtures were obtained with different amounts of Trass, sodium silicate and sodium hydroxide activation solutions. The effect of molar ratios of Si/Al (2.8, 3.1, 3.4, 3.6) and SiO\textsubscript{2}/Na\textsubscript{2}O (8.6, 6.9, 5.7, 5.2, 4.1, 3.6) on mechanical strength and rheology was investigated.

Best results were obtained by using waterglass activator mixes with Si/Al=3.4. For this Si/Al case there is an optimal value for SiO\textsubscript{2}/Na\textsubscript{2}O of 5.2, and the strength gradually decreased with increase of alkalinity, while on the other side the strength decreased rapidly with decrease in alkalinity. Activation of Trass by only NaOH (Si/Al = 2.8, no waterglass) exhibited the lowest strengths.

Workability of the mixes with waterglass increases with Si/Al ratio. For Si/Al ratios 3.1 and 3.4 there is an optimal workability for middle alkalinity. On the other hand for Si/Al ratio of 3.6 workability continuously increases with alkalinity.

Semi-quantitative XRD analysis indicated an increase in the amount of amorphous phase, and Analcime and Phillipsite zeolite phases as a result of geopolymerisation reaction.

The amount of non-evaporable water, left after drying at 105°C for 3 days, increases with geopolymerisation reaction. When heating to about 400 °C, the main part of water dehydrates. After about 600°C the mass loss is due to decarbonation, whose effect is decreased with geopolymerisation reaction.

IR spectrum shows a band shift of the broad absorption band at ~980 cm\textsuperscript{-1} as a result of a geopolymerisation reaction.

Apart from the silicate and aluminate content that exhibits pozzolanic reactivity, the use of Trass in alkali activated binders is further motivated by its alkali content. The Rhenish Trass contains about 8 mass % of alkali metals. The high alkali content of powder precursors could allow for a reduction of the concentration of alkali solutions used as an activator in alkali activated binders.
REFERENCES


IMPROVEMENT OF FREEZING AND THAWING DURABILITY ON
SCALING OF ECO-CEMENT EXTREMELY DRY CONCRETE UNDER
DEICING AGENT CONDITION

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Key words: Eco-cement, Extremely Dry Concrete, Deicing Agent, Freezing and Thawing

Abstract. Eco-cement is developed for long-term use of landfill areas near large cities and
natural resource conservation during cement production in Japan. It mainly consists of
incinerated ash from combustible city wastes as raw material, which is burnt in a rotary kiln
to produce clinker similar to ordinary portland cement. Therefore, it is expected to enhance the
sustainability of concrete production compared to its ordinary portland cement counterpart.
This study addresses the mechanical properties of eco-cement-based extremely dry concrete
for roller compacted concrete pavement (RCCP) and its freezing and thawing durability.
Despite its adequate mechanical properties for pavement use, the material shows a relatively
high decrease in mass compared to ordinary portland cement mixture during freezing and
thawing tests in the presence of a deicing agent. Therefore, ground granulated blast-furnace
slag and limestone powders with higher specific surface areas than both cements are added to
improve the scaling behavior under deicing conditions. These mineral admixtures enhance the
scaling property of the eco-cement-based extremely dry concrete in the presence of a deicing
agent. Moreover, CO₂ emission and utilized waste mass are estimated in units of volume of
concrete mixtures to evaluate environmental performances.
1 INTRODUCTION

Eco-cement is produced using dried incinerated city waste ash (Fig. 1) as the main raw material. One ton of eco-cement is produced with more than 500 kg of waste-derived ash in its raw materials. Therefore, the use of eco-cement is recommended from the standpoint of conservation of natural resources, long-term use of landfill area, and sustainability of concrete construction in Japan.

Quality requirements for eco-cement were specified by the Japanese Industrial Standard JIS R 5214 in 2002[1]. These specifications are shown in Table 1 along with an example of chemical composition and specifications based on JIS R 5210 for ordinary portland cement. Eco-cement may exhibit slightly higher Na2Oeq. and Cl− content than the ordinary portland cement, because of the large amount of ash used as the main raw material.

![Fig. 1: Dried incinerated city waste ash as main raw material for eco-cement](image)

Table 1: Specified chemical compositions and an example of eco-cement

<table>
<thead>
<tr>
<th></th>
<th>MgO</th>
<th>SO3</th>
<th>ig-loss</th>
<th>Na2Oeq</th>
<th>Cl−</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC Spec.</td>
<td>&lt;5.0%</td>
<td>&lt;3.5%</td>
<td>&lt;5.0%</td>
<td>&lt;0.75%</td>
<td>&lt;0.035%</td>
</tr>
<tr>
<td>Eco-cement example</td>
<td>1.63%</td>
<td>3.27%</td>
<td>2.57%</td>
<td>0.49%</td>
<td>0.040%</td>
</tr>
</tbody>
</table>

The mitigation of alkali-silica reaction and corrosion of embedded steel relies on decreasing the unit cement content in concrete. Extremely dry concrete for roller compacted concrete pavement (RCCP) is one of the most suitable eco-cement compositions, because it consists of a low volume and high density cement paste. Moreover, the pavements do not usually require any structural reinforcement.

The fundamental mechanical properties of eco-cement-based extremely dry concrete, such as its compressive and bending strengths, elastic modulus, and drying shrinkage[2] are evaluated in addition to its freezing and thawing durability in fresh water and 3% sodium chloride (NaCl) solution[3]. Pore structure, air void system and moisture content of hardened concrete are also investigated to consider the scaling behavior of the material[3].

In spite of its satisfactory mechanical properties for pavement use, the extremely dry concrete made with eco-cement shows a relatively higher mass loss than its ordinary portland cement equivalent during freezing and thawing test in 3% NaCl solution. Ground granulated blast-furnace slag and limestone powders having higher specific surface areas than eco-cement and its ordinary portland cement counterpart are added to improve the scaling under these conditions. Finally, CO2 emission and utilized waste mass in units of volume of concrete mixture are estimated to evaluate the environmental performance of each mixture.

2 EXPERIMENTAL PROCEDURE

2.1 Materials
Ordinary portland cement (N), eco-cement (E), ground granulated blast-furnace slag powder (BFS) and limestone powder (L) are used as cementitious materials. Table 2 shows the density and specific surface area of each powder. BFS and L present higher specific surface areas than N and E. Table 3 displays fine and coarse aggregate properties. Terrace sand and crushed rock coarse aggregate are general aggregates used for concrete making in Japan. A water reducing agent is combined with an air entraining agent to introduce entrained air in the concrete.

Table 2: Properties of cementitious materials

<table>
<thead>
<tr>
<th>Type</th>
<th>Symbol</th>
<th>Density (g/cm³)</th>
<th>Blain specific surface area (cm²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>N</td>
<td>3.16</td>
<td>3210</td>
</tr>
<tr>
<td>Eco-cement</td>
<td>E</td>
<td>3.15</td>
<td>4020</td>
</tr>
<tr>
<td>GGBFS</td>
<td>BFS</td>
<td>2.91</td>
<td>6480</td>
</tr>
<tr>
<td>Limestone powder</td>
<td>L</td>
<td>2.71</td>
<td>7170</td>
</tr>
</tbody>
</table>

Table 3: Properties of aggregates

<table>
<thead>
<tr>
<th>Type</th>
<th>Density in S.S.D. (g/cm³)</th>
<th>Absorption (%)</th>
<th>Unit bulk weight (kg/L)</th>
<th>Solid content (%)</th>
<th>F.M.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine agg.</td>
<td>Terrace sand</td>
<td>2.59</td>
<td>1.50</td>
<td>1.70</td>
<td>66.7</td>
</tr>
<tr>
<td>Coarse agg.</td>
<td>Crushed rock 5-20mm</td>
<td>2.64</td>
<td>0.65</td>
<td>1.58</td>
<td>60.1</td>
</tr>
</tbody>
</table>

Table 4: Mixture proportions of extremely dry concretes

<table>
<thead>
<tr>
<th>Mixture symbol</th>
<th>W/B</th>
<th>Km</th>
<th>Kp</th>
<th>Air content (%)</th>
<th>Unit content (kg/m³)</th>
<th>W.R. (B x %)</th>
<th>AEA (B x %)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.35</td>
<td>1.60</td>
<td>2.5</td>
<td>110</td>
<td>W</td>
<td>N</td>
<td>E</td>
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<td>722</td>
<td>251</td>
</tr>
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<td>B20</td>
<td>1.70</td>
<td>251</td>
<td>63</td>
<td></td>
<td>126</td>
<td>710</td>
<td>190</td>
</tr>
<tr>
<td>B40</td>
<td>1.72</td>
<td>236</td>
<td>63</td>
<td></td>
<td>126</td>
<td>710</td>
<td>173</td>
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<tr>
<td>B60</td>
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<td></td>
<td>126</td>
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<td>LB40</td>
<td>1.74</td>
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<td>63</td>
<td>16</td>
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<td>LB60</td>
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<td>236</td>
<td>63</td>
<td></td>
<td>126</td>
<td>710</td>
<td>110</td>
</tr>
</tbody>
</table>

2.2 Mixture proportions of extremely dry concretes

Mixture proportions of extremely dry concretes are shown in Table 4. Km corresponds to the ratio of mortar volume to interparticle space of coarse aggregate, and Kp denotes ratio of paste volume to interparticle space of fine aggregate. The measured entrained air volume is included in the paste volume. All mixtures present a Km value, water binder ratio (W/B), unit water content, and designed air content of 1.60, 0.35, 110kg and 2.5%, respectively. The Km value depends on the extremely dry concrete consolidation and the air content is set according to a previous study to achieve sufficient freezing and thawing durability[4].

The eco-cement in the mixture “E” is replaced by BFS mass ratios of 20%, 40%, and 60% in mixtures B20, B40, and B60, respectively. The eco-cement in mixtures LB20, LB40, and LB60 is replaced by a limestone powder (L) mass ratio of 5% with respect to the total binder
mass. The control mixture N only comprises ordinary portland cement under the same conditions.

2.3 Air content measurement in extremely dry concrete

The air content of freshly mixed extremely dry concrete is relatively difficult to determine because the concrete cannot be compacted easily in the measuring vessel. This measurement is achieved by replacing uncompacted voids with water. Specifically, freshly mixed extremely dry concrete (10kg) is immersed in water (2L) in a Washington air meter and subjected to steady tapping using a wooden hammer. This treatment allows water to eventually fill the voids. Next, the observed air content is converted into actual air content using the unit weight in 0% air content state of the corresponding concrete obtained from mixture proportion.

2.4 Mechanical properties of concrete

Compressive strengths, bending strengths, and elastic moduli are measured according to the corresponding Japanese industrial standards. Cylindrical specimens measuring 100 mm in diameter and 200 mm in height are fabricated for compressive strength and elastic modulus measurements. Prism specimens (100mm×100mm×400mm) are used for bending strength tests. Because this evaluation is related to extremely dry concrete, all specimens need to be in a fully compacted state. Consequently, the freshly mixed concrete presenting a relative density of 100% with respect to the corresponding specimen volume is accurately weighed and compacted with a table vibrator and tamper to achieve a fully compacted state. All specimens are cured in water at 20 °C water for 28 days.

2.5 Freezing and thawing behavior of concrete

The freezing and thawing test is conducted according to the JIS A 1148 method A, in which specimens are frozen and thawed in water. Here a 3% NaCl solution is used as a surrounding
liquid phase to simulate winter deicing conditions. Before testing specimens are cured in water at 20 °C for 28 days. The dynamic elastic modulus and mass are measured during the freezing and thawing cycle.

3 RESULTS AND DISCUSSIONS

3.1 Mechanical properties

All compressive strengths, bending strengths, and elastic moduli are shown in Figs. 2-4, respectively. The replacement of eco-cement by BFS and limestone powders increases bending strength of concrete to a greater extent than its compressive strength and elastic modulus.

Figure 5 relates compressive and bending strengths. All bending and compressive strengths are satisfactory for pavement use. For the same compressive strength, mixtures B and LB exhibit a greater increase in bending strength than the original mixture E. These improved bending strengths may result from the hydration and filler effect of mineral admixtures, which are finer than eco-cement particles. Because bending strength is a direct index of tensile strength, its enhancement may clearly reflect a slight increase in matrix strength.

3.2 Freezing and thawing behavior in 3% NaCl solution

Figure 6 shows the change in relative dynamic elastic modulus for each mixture during a freezing and thawing cycle. All extremely dry concretes retain high relative dynamic elastic moduli during freezing and thawing cycles even in 3% NaCl solution indicating their sufficient durability under deicing conditions.

Figure 7 shows a decrease in mass during freezing and thawing cycle for mixtures N and E. This loss is more pronounced in mixture E than mixture N. The replacement of eco-cement by BFS and limestone powders limits this decrease for mixtures B and LB, especially, when the BFS replacement ratio exceeds 40% (Fig. 8). This improvement may result from the surface strength enhancement observed in Fig. 3. Figure 9 relates BFS replacement ratio and mass loss after 300 freezing and thawing cycles. Generally, the decrease in mass after the 300 cycles declines with increasing replacement ratio.

3.3 Evaluation of environmental performance

Table 5 lists the CO₂ emission of each cementitious material and waste mass utilized as raw
material in each binder.

For ordinary portland cement, these environmental performance indicators originate from the inventory data distributed by the Japan Cement Association [5]. For eco-cement, they are calculated using public data from the Regional Association for Waste Management and Resource Recycling [6], which deals with resources recycling and manages a landfill in the Tama area (Tokyo). The CO₂ emissions of BFS and limestone powder are given by a committee of the Japan Society of Civil Engineers [8]. Because BFS is a by-product from the iron industry, its totality is treated as utilized waste.

Table 6 shows estimated CO₂ emission and utilized waste mass for all concrete mixtures. Mixture E exhibits a slightly higher CO₂ emission and about 2.5 times greater utilized waste mass than its N counterpart (Fig. 10). Moreover, the CO₂ emission decreases and the utilized waste mass increases when the BFS replacement ratio increases.

Table 5: CO₂ emission and utilized waste mass of each binder

<table>
<thead>
<tr>
<th>Material</th>
<th>CO₂ emission (kg-CO₂/ton)</th>
<th>utilized waste (kg-waste/ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>764.3</td>
<td>250</td>
</tr>
<tr>
<td>Eco-cement</td>
<td>794.0</td>
<td>637</td>
</tr>
<tr>
<td>BFS</td>
<td>26.5</td>
<td>1000</td>
</tr>
<tr>
<td>Limestone powder</td>
<td>16.1</td>
<td>0</td>
</tr>
</tbody>
</table>

Fig. 7: Decrease in mass with freezing and thawing action (N and E-mixture)

Fig. 8: Decrease in mass with freezing and thawing action (all mixtures)

Fig. 9: Relationship between replacement ratio by GGBFS and decrease in mass after the 300-cycles
**Table 6: CO₂ emission and utilized waste mass of each concrete mixture**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CO₂ emission (kg-CO₂/m³)</th>
<th>Utilized waste (kg-waste/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>240.0</td>
<td>78.5</td>
</tr>
<tr>
<td>E</td>
<td>249.3</td>
<td>200.0</td>
</tr>
<tr>
<td>B20</td>
<td>201.0</td>
<td>222.9</td>
</tr>
<tr>
<td>B40</td>
<td>153.4</td>
<td>246.4</td>
</tr>
<tr>
<td>B60</td>
<td>105.1</td>
<td>269.3</td>
</tr>
<tr>
<td>LB20</td>
<td>189.3</td>
<td>213.3</td>
</tr>
<tr>
<td>LB40</td>
<td>141.0</td>
<td>236.2</td>
</tr>
<tr>
<td>LB60</td>
<td>92.6</td>
<td>259.1</td>
</tr>
</tbody>
</table>

**Fig. 10: CO₂ emission and utilized waste from/in each mixture**

4 CONCLUSIONS

A sustainable social system for a long-term final landfill area near Tokyo hinges on the use of eco-cement. Despite its similar chemical composition to ordinary portland cement, this material has a remarkable mass loss tendency during freezing and thawing cycles in the presence of a deicing agent. To mitigate this behavior, eco-cement is replaced by BFS and limestone powders in various ratios in concrete mixtures. Basic mechanical properties of extremely dry concretes containing these binders are also evaluated. Our findings suggest the following conclusions:

The replacement of eco-cement by BFS and limestone powders strongly increases the bending strength of concrete. For the same compressive strength, extremely dry concretes comprised of two or three ingredients as binders including eco-cement, BFS, and limestone powder, exhibit higher bending strengths than original eco-cement concrete. All extremely dry concretes retain their high relative dynamic elastic moduli during freezing and thawing cycles even in 3% NaCl solution. The decrease in mass during a freezing and thawing cycle in 3% NaCl solution is more pronounced for eco-cement mixture than for its ordinary portland cement equivalent. The replacement of eco-cement by BFS and limestone powders limits this
mass loss, especially for BFS replacement ratios exceeding 40%. The eco-cement extremely dry concrete displays a slightly higher CO₂ emission and approximately 2.5 times greater utilized waste mass than its ordinary portland cement counterpart. As the BFS replacement ratio increases, CO₂ emission decreases and utilized waste mass increases.

ACKNOWLEDGMENTS

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REFERENCES

INFLUENCE OF C₃A CONTENT ON CHLORIDE TRANSPORT IN CONCRETE

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Key words: C₃A, Chloride, Binding, Corrosion, Transport

Abstract. As the C₃A content in cement matrix has been intuitively thought as an inhibitive nature, which can remove mobile chlorides from the pore solution, no hard evidence on binding capacity with respect to the corrosion resistance has been clearly identified. The present study presents a detailed experimental study on the influence of C₃A content on chloride transport. The C₃A content of specimens ranged 6.0, 10.5, 13.7 and 16.9% by modifying oxide compositions of cement, in particular Al₂O₃ and CaO. The water soluble chloride concentration, regarded as free chloride which has corrosiveness in the pore solution, was measured from boiling the extracted dust concrete sample in water. Moreover, the pore structure of concrete and penetration of chloride ions were determined by Mercury Intrusion Porosimetry and profiling test respectively. As a result, the chloride binding capacity was not affected by C₃A content, except for a very high range of C₃A. For chloride transport, the apparent diffusion coefficient of chloride ions was not influenced by C₃A. However, it was observed that an increase in the C₃A content resulted in an increase in the surface chloride concentration, leading to increased chloride ingresses at a given duration of exposure to a salt solution. Substantially, the benefit of increased chloride binding capacity in the corrosion resistance may be offset by the increase in surface chloride concentration.

1 INTRODUCTION

It is well known that C₃A and C₄AF bind chlorides to form 3CaO·Al₂O₃·CaCl₂·10H₂O (Friedel's salt) and 3CaO·Fe₂O₃·CaCl₂·10H₂O respectively. Chloride binding can be defined as the interaction between the cement matrix and chloride ions, which results in the removal of chlorides from the pore solution. Binding capacity has been regarded as important in the corrosion of steel in concrete structures because of the conventional view that chemically bound chlorides do not participate in the corrosion process [1]. Hence, it has been thought that a higher binding capacity is associated with less corrosion risk.

Moreover, binding of chloride ions in the cement matrix may delay chloride transport, due
to immobilisation of chlorides at a certain level. Thus, increased binding capacity of chlorides in a C₃A high cement matrix would benefit in lowering the corrosion risk: lower chloride transport and increased corrosion resistance. For the resistance to chloride-induced corrosion, a number of previous studies already addressed; for example, Hussain et al [2] found that the corrosion reactivity increased with C₃A content; three plain cements with C₃A contents of 2.43, 7.59 and 14% were tested and chloride threshold value for those cements were 0.35, 0.62 and 1.00% respectively. However, the influence of C₃A content in terms of chloride binding has been less experimented to date; only the chloride ingresses at different C₃A contents was previously modelled. Simultaneously, as the chloride binding capacity is intuitively dependent on C₃A content in cement clinker, its relationship between them has been less experimentally determined.

In the present study, the influence of C₃A content on chloride transport was determined by the apparent diffusion coefficient of chloride ions in four cements in the range of 6.0-16.9% of C₃A. To determine the rate of chloride transport, mortar specimens were immersed in salt solution for 100 days then to measure the chloride ingresses. Simultaneously, the pore structure was examined to identify the role of C₃A in mobility of chloride ions. Finally, chloride binding capacity at different C₃A contents was measured by the water extraction method.

2 EXPERIMENTS

2.1 Concrete mix design

To investigate the influence of chloride binding in terms of C₃A content in cement on the rate of chloride transport, Portland cements containing four different levels of Al₂O₃ in oxides were manufactured as given in Table 1: the C₃A calculated by the Bogue equation ranges 6.0, 10.5, 13.7 and 16.9%. For testing of a development of the compressive strength, cylindrical concrete specimens (Ø100×200 mm) were produced; their ratio of cement, water, sand (Grade M) and 10 mm gravel was 1.00: 0.45: 2.52: 3.23. The concrete specimens produced were cured in a damp chamber at 20±2° and the compressive strength was measured at 7, 28 and 56 days. For chloride transport and pore structure examination, mortar specimens were manufactured with an identical mix design to concrete one except for gravel, and cement paste for the chloride binding test.

Table 1: Oxide composition of cement used (%)

<table>
<thead>
<tr>
<th>C₃A</th>
<th>CaO</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>SO₃</th>
<th>Fe₂O₃</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0%</td>
<td>70.3</td>
<td>17.4</td>
<td>4.0</td>
<td>4.2</td>
<td>4.1</td>
</tr>
<tr>
<td>10.5%</td>
<td>64.2</td>
<td>20.1</td>
<td>6.0</td>
<td>2.0</td>
<td>3.2</td>
</tr>
<tr>
<td>13.7%</td>
<td>63.5</td>
<td>19.7</td>
<td>7.1</td>
<td>2.0</td>
<td>3.1</td>
</tr>
<tr>
<td>16.9%</td>
<td>62.8</td>
<td>19.3</td>
<td>8.3</td>
<td>1.9</td>
<td>3.0</td>
</tr>
</tbody>
</table>

2.2 Rate of chloride ingress

After 56 days of curing of mortar disc (Ø100×50mm), the specimen was coated by polymer-based resin except for one surface to drive one-dimensional chloride transport. Then the mortar specimens were immersed in 4.0M NaCl solution for 100 days. Subsequently, the chloride
profile was obtained from samples collected by grinding the mortar surface in 5.0mm depth increments to 30.0 mm. The chloride content of each sample was determined by acid soluble extraction in a nitric acid solution followed by potentiometric titration against silver nitrite. The apparent chloride diffusion coefficient was determined for each specimen by fitting the error function solution to Fick's second law, for non-steady state diffusion in a semi-infinite medium. A curve is fitted to the chloride profile with the surface chloride and the diffusion coefficient as the independent variables.

The characteristics of the pore structure were examined using mortar sample. The mortar samples, after 56 days of curing, were obtained from the middle of specimens. The sample was dried out in an oven at 50°C for 24 h. The sample was initially evacuated to about 50 μm mercury (Hg) and the low pressure was generated up to 0.21 MPa by nitrogen gas, and then the pressure was gradually increased to 117.21×10³ MPa at the rate of 9.1×10³ kPa/s. The pressure was converted to the equivalent pore diameter using the Washburn equation as given in Equation 1. Then the pore volume distribution at a given pore diameter was achieved. The pore volume was adjusted to the percentage of the volume of sample.

\[
d = -\frac{4\gamma \cos \theta}{P}
\]

where,
- \(d\): pore diameter, m
- \(\gamma\): surface tension, N
- \(\theta\): contact angle, °
- \(P\): pressure, MPa.

2.3 Binding of chlorides

Cement paste was cast to determine the chloride binding capacity in cement at different C₃A content. Eight levels of chlorides were admixed in mixing water as NaCl: 0.0, 0.2, 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0% by weight of cement. After casting the paste, the specimen was rotated at 6.0 rpm for 24 hours to avoid segregation of chloride ions, and then cured by wrapping in a polythene film at 20±2°C for 56 days. Then, the specimen was dried in an oven at 104°C for 24 hours and then crushed/ground to obtain dust sample, which was subsequently sieved into a 300 μm sieve in the fineness. The dust sample was stirred for 5.0 minutes in 50°C distilled water to extract the water-soluble chloride. After a further 30 minutes standing of the sample, the concentration of water soluble chloride ions in the cement paste was measured by the potentiometric titration against silver nitrate, which is taken as free chlorides [3]. Non-soluble chlorides in this procedure were, in turn, considered as bound ones.

3 RESULTS

3.1 Development of compressive strength

As expected, the compressive strength was increased with time for all concrete mixes as given in Figure 1. It is evident that the cement containing high levels of C₃A in clinker produced the higher strength at all ages. Cement at 13.7 and 16.9% of C₃A achieved 35.5-36.8 MPa at 7
days and subsequently reached 42.2-43.5MPa at 56 days. However, cement at low or medium level of C₃A produced lower strengths, ranging 30.3-32.2MPa at 7 days and 3.49-37.8MPa at 56 days, respectively. The higher level of compressive strength for cement at high C₃A at an early may be attributed to rapid setting of the cement matrix, arising from C₃A, leading to a substantially high range of the strength even at a long term. It seems that the high range of C₃A could benefit in raising concrete properties, in particular, strength and thus presumably resistance to ionic transport from an external source.

![Figure 1: Development of compressive strength of concrete at different C₃A content in cement](image)

### 3.2 Chloride transport

Chloride ingress was much affected by C₃A in clinker; the higher C₃A in cement clinker produced the higher level of chloride ingresses, as seen in Figure 2. Chloride ingress for cement at high range of C₃A was significantly high in the vicinity of the concrete surface, ranging from 1.85 to 2.15% by weight of cement. Due to high level of the surface chloride, cement at high C₃A led to high ingresses of chloride at all depths. To quantitatively determine the rate of chloride transport, the apparent diffusion coefficient of chloride and surface chloride concentration were calculated, given in Table 2. It is evident that there was no clear relation between C₃A in cement clinker and diffusivity of chloride ions. However, an increase in the C₃A content resulted in an increase in the surface chloride concentration. A dramatic increase in the surface chloride may lead to the higher gradient of chloride concentration, which substantially may produce a higher range of chloride ingress in concrete at a given pore.
structure. However, the rate of chloride transport was not always high in cement concrete at a high C₃A, as the pore structure is simultaneously modified depending on C₃A content.

![Graph showing chloride ingress in mortar at different C₃A content in cement after 100 days of exposure to 4.0M NaCl solution.](image)

**Figure 2:** Chloride ingress in mortar at different C₃A content in cement after 100 days of exposure to 4.0M NaCl solution

**Table 2:** Rate of chloride transport in mortar at different C₃A content

<table>
<thead>
<tr>
<th>C₃A Content (%)</th>
<th>C₃A: 6.0%</th>
<th>C₃A: 10.5%</th>
<th>C₃A: 13.7%</th>
<th>C₃A: 16.9%</th>
</tr>
</thead>
<tbody>
<tr>
<td>D (×10⁻¹² m²/s)</td>
<td>5.79</td>
<td>6.65</td>
<td>4.13</td>
<td>4.56</td>
</tr>
<tr>
<td>CS (%)</td>
<td>1.17</td>
<td>1.20</td>
<td>2.24</td>
<td>2.72</td>
</tr>
</tbody>
</table>

*D:* Apparent diffusion coefficient  
**CS:** Concentration of chloride ions on the right surface of concrete (Mortar)

The pore structure was influenced by the C₃A content; but there was no clear relation to pore volume, as given in Figure 3. In fact, the pore volume was less crucial to ionic transport rather than pore distribution, as the ionic path in concrete was governed by the intensity of pore volume at the range of capillary pores ranging from 0.01-1.0 µm. For all cement pastes, the peak pore size was restricted to 0.1 and 1.0 µm, indicating that pores generated in all the cement matrix. It implies that Portland cement at a wide range of C₃A may provide appropriate paths for chloride ions to transport in concrete. To substantially modify the pore structure for blocking external ions from concrete body, a further addition of finely grained binder, a low free W/C or/and chemical admixture must be accompanied, rather than a modification of oxides in terms of C₃A.
Figure 3: (a) Pore distribution in mortar at different C₃A content after 56 days of curing and (b) pore intensity to a given pore diameter.
3.3 Chloride binding capacity

Chloride binding capacity was depicted by the relation of C₃A with concentration of bound chlorides at a given chloride in cast in cement paste, as given in Figure 4. It is evident that cement paste at the highest C₃A (i.e. 16.9%) produced the highest level of chloride binding at all concentrations of total chlorides. However, the higher content of C₃A in cement does not always produce the higher chloride binding capacity. This may be due to the complexity of chloride binding. Chlorides present in the cement matrix may be chemically bound by C₃A and C₄AF. Simultaneously, chloride ions may be adsorbed on the surface of hydration products such as precipitated Ca(OH)₂ and C-S-H gel. To vary C₃A content in cement clinker, Al₂O₃ in oxides must be different, in which the procedure other oxides such as SiO₂, CaO and Fe₂O₃ can be modified. Substantially, the C₄AF content can be influenced, leading to the content of Friedel’s salt. Simultaneously, a modification of hydration products may affect the chloride binding capacity. Thus, it can be said that the higher C₃A content in clinker may not result in the higher chloride binding capacity at all time.

![Figure 4: Chloride binding capacity in cement paste at different C₃A content after 56 days of curing with the variation in total chloride in cast](image)

4 DISCUSSION

The high level of C₃A in cement clinker has been thought as increased chloride binding capacity, followed by the higher resistance to chloride-induced corrosion of steel in concrete [1]. Chloride ions present in the cement matrix react with C₃A to form Friedels’ salt then to be removed from concrete pore solution. Thus, binding of chloride ions may be considered as an inhibitive nature of cement hydration products. Moreover, a removal of free chlorides from the
concrete pore solution can lower the rate of chloride transport. Recent studies have modelled the influence of binding on the penetration of chloride [4, 5]. Glass and Buenfeld [4] found that with increased chloride binding capacity, total chloride contents increase nearer the surface of the concrete, but decrease deeper in the concrete. An increase in the surface chloride content is induced by the binding effect which allows the progressive build-up of higher total chloride content at increasing distances from the concrete surface. However, in these modelling schemes, a given pore structure was assumed, irrespective of C3A content in cement.

In this study, the variation in the C3A content has no influence on the rate of chloride diffusivity, in terms of apparent diffusion coefficient. Despite a removal of chlorides, no reduction of chloride ingress may be attributed to modified pore structure, depending on the C3A content. In fact, the higher C3A content in cement clinker could have accelerated hardening process of fresh concrete, leading to shrinkage and thermal cracking. Thus, increased binding of chloride ions had no benefit in lowering the mobility of chloride ions. However, an increase in the C3A content resulted in increased surface chloride concentration, which could increase mobility of chloride ions, arising from increased gradient of chloride ion concentration. Increased binding capacity may enhance a build-up of bound chloride on the surface of concrete at a given free chloride concentration, then to increase the concentration of total chloride. Thus, increased C3A content in cement may adversely increase the risk of steel corrosion in concrete.

Moreover, it was shown in the present study that an increase in C3A content did not always result in an increase in the chloride binding capacity, in turn, no benefit in raising the corrosion resistance could be achieved. In the majority of previous studies [6], the higher C3A content was associated with the higher chloride binding in beaker test. Also, a previous study showed that the corrosion rate of steel in concrete was dependent on C3A content, once corrosion started [7]. As chloride binding capacity is influencing on the corrosion reactivity, increased chloride binding capacity can be an inhibitive measure. However, no increase in the chloride binding capacity resulting from increased C3A may be indicative of inhibition effect.

5 CONCLUSION

In the present study, the influence of C3A content on chloride transport in Portland cement concrete was determined by measuring the rate of chloride transport, in terms of apparent diffusion coefficient and surface chloride concentration. As a result, diffusivity of chloride ions was marginally influenced by C3A content. However, an increase in the C3A content resulted in an increase in surface chloride concentration, leading to increased mobility of chloride ions. Moreover, it was shown that the chloride binding capacity was not dependent on C3A; the binding of chloride ions in cement matrix is simultaneously affected by other hydration products and clinkers. Thus, unlike intuitive hypothesis, increased C3A may not have any benefit in lowering the risk of chloride-induced corrosion in concrete.

6 ACKNOWLEDGEMENT

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INFLUENCE OF CARBONATION ON THE CHLORIDE ION
DIFFUSION COEFFICIENT IN FLY ASH CONCRETE

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Key words: Chloride, Carbonation, Concrete, Fly Ash, Durability

Abstract. The incorporation of fly ash (FA) in cementitious matrices has been frequently
used in order to make the matrix more resistant to the action of chlorides. On the other hand,
it is known that Ca(OH)₂ existing in the matrix is consumed by the pozzolanic reactions,
which makes easier the carbonation front advance. Given that the chloride ingress and
carbonation are the two main causes of degradation in reinforced concrete, we speculate about
the behaviour of FA concrete when the structure is submitted simultaneously to chlorides and
carbonation.

This work studied the influence of carbonation on the chloride migration coefficient in FA
concrete. For this, specimens with 0% and 40% replacement of cement CEM I 42.5R by FA
were casted with water/binder 0.55 and 0.50 respectively. After 90 days of curing period, half
of samples were subjected to accelerated carbonation (20°C, 55%RH and 4%CO₂) for 1, 2 and
3 months. The other half was protected with plastic film during the same period.
Non-steady-state migration tests, according to LNEC E463 (Portuguese specification), were
performed with specimens subjected to both experimental conditions.

The results show that, for these conditions, the carbonation has a direct influence on chloride
diffusion coefficient, increasing it. For FA concrete samples this effect is more evident. In
these cases, the carbonated samples studied showed a diffusion coefficient up to two times
higher than noncarbonated ones.

The increase in large capillary pores, caused by carbonation, can be responsible for the
increase of chloride penetration into concrete subjected to combined action.
1 INTRODUCTION

To ensure the sustainability of the construction industry it is essential to guarantee the long term durability of concrete structures without unforeseen needs of maintenance. However, the premature deterioration of concrete structures has been observed worldwide, resulting in great financial losses, in the order of millions of euros [1] for repair and reconstruction of these structures, not including incalculable environmental and social impact. One option to address the sustainability and durability of concrete structures from the design phase has been to incorporate industrial wastes or byproducts into the mix as a partial replacement of cement.

In light of recent research internationally [2, 3, 4] more attention is now being drawn to the combination of degradation mechanisms and the effect on the concrete durability. Despite the advances in knowledge on concrete durability, little is known on how carbonation affects and interacts with chloride penetration.

The incorporation of fly ash (FA) in cementitious matrices has been used to make concrete more sustainable and also to make its matrix more resistant to the action of chlorides, as confirmed by published research [5, 6, 7]. However, taking into account carbonation, when replacing cement by FA there is a double effect: the reduction in matrix permeability, which reduces the penetration of carbon dioxide in the structure; the consumption of calcium hydroxide due to the pozzolanic reactions, which decreases the alkalinity of the concrete, resulting in a more rapid carbonation when compared to the concrete without pozzolanic addition [8, 9, 10].

The action of chloride ions and carbon dioxide on the concrete structures has been investigated as individual phenomena. However, the study on the combination of degradation mechanisms is relatively recent and achieves greater expressiveness from the year 2000 [11, 12, 13, 14, 15]. Thus, despite the combined action of these mechanisms being a reality, there is no consensus on the effect of combination of chlorides and carbonation on the durability of concrete with and without FA.

According to [11], for instance, for Ordinary Portland Cement (OPC) mortar, the exposure to carbon dioxide environment does not lower the chloride penetration resistance. They submitted the specimens to carbonation chamber (5% CO₂, 23°C and 50% RH) for 28 days and after that, the specimens were tested for rapid chloride penetration test, modified rapid migration test and chloride immersion test (3% NaCl solution). On the other hand, [13] say that, for OPC concrete, after carbonation, the chloride diffusion coefficient increases. They submitted the specimens to carbonation chamber (20% CO₂, 20°C and 70% RH) for 0,14 and 28 days. After that wet and dry cycle test was carried out. The specimens were immersion, for one week, in 5% NaCl solution and drying, for one week, in laboratory environment cycles to study the effect of carbonation on chloride transport. The cycles were repeated for twelve weeks.

In order to contribute to this important and ongoing scientific discussion, the main objective of this research is to study the effect of carbonation dioxide on chloride ion diffusion coefficient of concretes with incorporation of fly ash.
2 EXPERIMENTAL WORK

2.1 Materials and mixing proportion

In this work one has used Ordinary Portland Cement (OPC), CEM I 42.5R, and Fly Ash (FA) as binder materials, which chemical compositions are presented in Table 1. Fine aggregate with a fineness modulus of 5.04 and coarse aggregate with maximum size 12.70 mm were used. Tape water was used for all the produced mixtures.

<table>
<thead>
<tr>
<th>Composition</th>
<th>OPC</th>
<th>FA</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>20.33</td>
<td>55.10</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.59</td>
<td>26.60</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.06</td>
<td>5.70</td>
</tr>
<tr>
<td>CaO</td>
<td>62.30</td>
<td>2.58</td>
</tr>
<tr>
<td>MgO</td>
<td>2.12</td>
<td>1.30</td>
</tr>
<tr>
<td>SO₃</td>
<td>3.10</td>
<td>-</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.76</td>
<td>1.41</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.19</td>
<td>0.26</td>
</tr>
<tr>
<td>TiO₂</td>
<td>-</td>
<td>1.33</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>-</td>
<td>0.73</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>0.07</td>
<td>-</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>1.78</td>
<td>3.07</td>
</tr>
<tr>
<td>Insoluble residue</td>
<td>1.21</td>
<td>-</td>
</tr>
<tr>
<td>Remaining elements</td>
<td>0.49</td>
<td>1.92</td>
</tr>
</tbody>
</table>

Two mixtures were adopted in this study, which are presented in Table 2. Cylindrical concrete specimens with φ100x200 mm were casted with the mixtures presented. After casting, the specimens were covered with a plastic sheet and were stored in a humidity chamber (21 °C and 98% RH) for one day. Then, they were removed from the moulds and were cured immersed in water for 90 days. The choice of this curing period was to give enough time for pozzolanic reactions in specimens containing FA to occur.
Table 2: Mixtures and properties of the developed concrete

<table>
<thead>
<tr>
<th>Materials</th>
<th>Binder</th>
<th>100% OPC</th>
<th>40% FA</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Relative proportions</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>380</td>
<td>228</td>
<td></td>
</tr>
<tr>
<td>Fly ash (kg)</td>
<td>-</td>
<td>152</td>
<td></td>
</tr>
<tr>
<td>Water (l)</td>
<td>209</td>
<td>190</td>
<td></td>
</tr>
<tr>
<td>Sand 0/4 (kg)</td>
<td>911.16</td>
<td>926.18</td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate 4/8 (kg)</td>
<td>254.37</td>
<td>281.18</td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate 6/12 (kg)</td>
<td>461.84</td>
<td>483.10</td>
<td></td>
</tr>
<tr>
<td><strong>Property</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>100.5</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>Compressive strength (MPa):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28 days</td>
<td>33.66</td>
<td>24.62</td>
<td></td>
</tr>
<tr>
<td>90 days</td>
<td>45.16</td>
<td>34.48</td>
<td></td>
</tr>
<tr>
<td>365 days</td>
<td>47.44</td>
<td>51.59</td>
<td></td>
</tr>
<tr>
<td>Capillary absorption (kg/m²/min¹/²)</td>
<td>0.128</td>
<td>0.116</td>
<td></td>
</tr>
<tr>
<td>Total porosity (%)</td>
<td>10.5</td>
<td>9.99</td>
<td></td>
</tr>
</tbody>
</table>

2.2 Experimental program

After the curing period, all samples were cut in three slices. The top and bottom of the samples were discarded (Figure 1). The new samples (ϕ100x50mm) were divided in two groups, A and B. The group A was subjected to accelerated carbonation in carbonation chamber (4% CO₂, 20 °C and 55% RH), following the recommendations of the European Standard [16], for 1, 2 and 3 months while the group B (reference group) remained in laboratory environment for the same period.
Before accelerated carbonation the specimens of group A were subjected to preconditioning procedures in order to achieve the equilibrium with environmental humidity near 60% and accelerated the carbonation process. This preconditioning had three stages. In the first stage, the specimens remained in a chamber with controlled temperature and humidity (20 °C and 60% RH) until to reach constant mass. Second, they had the lateral and the bottom covered with paraffin in order to guarantee the one-dimensional attack of aggressive agents through the top of the specimen. Third, they were protected with several layers of plastic film and put in an oven (40 °C), for 20 days, to distribute the humidity inside the specimen. The specimens of group B were wrapped with plastic film and, after, they were stored in laboratory environment.

When finished the carbonation tests, Non-Steady-State Migration Tests (NMT) were performed [17] with specimens previously subjected to accelerated carbonation, group A, and with reference specimens, group B. For specimens of group A, the paraffin was removed and then all the specimens were subjected to NMT. These specimens were previously saturated with limewater under vacuum conditions and stored in the solution for 18 ± 2 h. Afterwards, the samples were placed in rubber sleeves, as it is shown in Figure 2. The catholyte was filled with a 10% NaCl solution and the anolyte with a 0.3M NaOH solution. The test duration and applied voltages were determined based on the initial current measured at 30 V.

![Figure 2: Non-Steady-State Migration Test scheme (a) and set-up (b)](

At the end of the test, the samples were split perpendicularly to the aggressive agents penetration direction. For group B, the halfs were only sprayed with 0.1 M AgNO₃ solution in order to determine the penetration depth of chlorides. For group A, the halfs were used in different ways: the first one was sprayed with a 1% phenolphthalein solution in 70% ethyl alcohol [18] in order to determine the advance of the carbonation front; and the other one was sprayed with 0.1 M AgNO₃.

The penetration depth of chloride was used to determine a chloride ion diffusion coefficient and following equation derived from the Nernst–Plank equation [19] (1) where \( D \) is chloride ion diffusion coefficient (cm²/s), \( R \) is gas content (8.314 J/K mol), \( T \) is temperature (K), \( L \) is the thickness of specimens (cm), \( z \) is the ion valence, \( F \) is Faraday content (9.65 \( \times 10^4 \) C/mol), \( E \) is values of potential (V), \( x_f \) is the inflection point of the chloride ion profiles that needs to be related to the depth given by the colorimetric technique and \( t \) is the time of test duration (s).

\[
D = \frac{R T z F}{8.314 J/K mol} \frac{9.65 \times 10^4 C/mol}{E} \frac{L}{x_f^2} \frac{1}{t}
\]
\[ D = \frac{RTL}{zFE} \times \frac{x}{t} \]  

The schematic diagram of experimental program is shown in Figure 3.

![Schematic diagram of experimental program](image)

**Figure 3**: Schematic diagram of experimental program

3 RESULTS

The next sections present results of concrete carbonation and chloride penetration. Each point represents an average of three specimens results.

3.1 Carbonation

Figure 4 shows the depth of carbonation front obtained for the specimens and exposure periods studied in group A. It is possible to see the results for OPC and FA concrete.

![Carbonation depth for OPC and FA concrete](image)

**Figure 4**: Carbonation depth for OPC and FA concrete in different test times

As expected, FA concrete presented deeper carbonation depth than OPC concrete. The difference between OPC and FA carbonation depth increases with the increase in testing time. In the third month this difference achieves 40.2%.
It can also be seen that there is an increase in carbonation depth with the increase of testing time. This increase is more pronounced in FA concrete where it is possible to see a 88% increase from first to third month.

### 3.2 Chlorides

Figures 5 and 6 show the chloride ion diffusion coefficients obtained from NMT, for OPC and FA concrete. Figure 5 shows the results for group B, reference group, only chlorides, and Figure 6 presents results for group A, carbonation and chlorides.

![Figure 5](image1.png)

**Figure 5**: Chloride diffusion coefficient for OPC and FA concrete subjected only chloride action

As expected, OPC concrete has higher chloride ion diffusion coefficients than FA concrete. In the first month, the OPC diffusion coefficient, $23.35 \times 10^{-12}$ m²/s, is almost 13 times higher than FA diffusion coefficient, $1.90 \times 10^{-12}$ m²/s. In OPC concretes there is a tendency to decrease diffusion coefficient with the test time increase. This tendency can be related with the development of cement hydration along time that decreases the concrete poros and, consequently, decreases the chloride diffusion coefficient. In FA concrete this tendency could not be observed.

![Figure 6](image2.png)

**Figure 6**: Chloride diffusion coefficient for OPC and FA concrete subjected combined action, CO₂ and Cl⁻
In Figure 6 it is possible to observe the influence of carbonation on the chloride ion diffusion coefficient. Again, the OPC concrete presented higher chloride ion diffusion coefficients than FA concrete but, in this case, the diffusion coefficient values are higher than in the situation without carbonation, regardless FA presence. Thus, the results suggest that carbonation increases the chloride ion diffusion coefficient.

In OPC concretes there is a tendency to decrease the diffusion coefficient with the test time increase. But in FA concretes there is the opposite tendency, that is, the diffusion coefficient increase with the test time increase.

4 INFLUENCE OF CARBONATION ON CHLORIDE DIFFUSION COEFFICIENT

The results presented in section 3 show that the carbonation has influence on chloride ion diffusion coefficient of studied concretes.

For OPC concrete, under the studied conditions, there is an increase in diffusion coefficient when the samples are subjected to combined action of carbonation and chloride transport. In this case, the increase is more pronounced in the first month of carbonation where 7.96 mm of carbonation depth leads to an increase of 26% in chloride coefficient diffusion.

In the same way, for FA concretes (and deeper carbonation depths), under the conditions studied, the combined action increases the chloride diffusion coefficient. But, for this case, the coefficient of diffusion has increased (Figure 7). In the first month the increase is 44%, in the second month it is 191% and the third month has the biggest increase, 249%.

![Figure 7: Chloride diffusion coefficient for FA concrete, group A and group B](image)

Figure 8 shows the relationship between the chloride diffusion coefficient with carbonation depth in FA concrete.
This increase in chloride diffusion coefficient can be related with the increase in large capillary pores, caused by carbonation, providing a "privileged way" for chlorides. [20, 21] studied the effect of carbonation on pore structure and found that, after carbonation, there is a reduction in total porosity but the proportion of large capillary pores was increased slightly for the OPC pastes and much more significantly for FA pastes. Furthermore, the carbonated concrete has its chloride binding capacity reduced which means that there are more free chlorides.

Despite the large increase in the diffusion coefficient occurred in the FA concrete, it is important to say that the use of this material continues to be beneficial in environments containing chlorides because it presents diffusion coefficients quite lower than OPC concrete, Figure 6. However, the use of FA in environments containing chlorides and CO₂, at the same time, should be considered with care.

Based on present results, it can be pointed out that carbonated concrete works as a "privileged way" for chlorides penetration. It is quite evident for FA concrete as we can see in Figure 9, that shows the FA concrete halves (group A) after finished the test. It is possible to see that chlorides crossed completely the carbonated front but reached depths only a few millimeters deeper than the carbonation front.
5 CONCLUSIONS

Based on the results achieved it is possible to conclude, for the studied conditions, that:

- Carbonation has an influence on the chloride ion diffusion coefficient for OPC and FA concrete. Carbonated concrete works as a "privileged way" for chlorides penetration;
- For OPC concrete the carbonation increases the chloride diffusion coefficient up to 26%;
- For FA concrete the carbonation sharply increases the chloride diffusion coefficient. This increase achieves 249% for three months in carbonation chamber. Furthermore, the diffusion coefficient increases with the carbonation depth increase.

REFERENCES


INFLUENCE OF CONDUCTION OF STEEL BAR ON ELECTROCHEMICAL MEASUREMENT OF REINFORCED CONCRETE STRUCTURE

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Key words: Corrosion, Electrical conductivity, Electrochemical measurement

Abstract. In this study, to clarify the effect of electrical conductivity on the progress of corrosion of steel bars in concrete structures, the electrochemical properties of the connection between non-corroded and corroded steel bars in the concrete was experimentally investigated. Two types of test specimen were prepared in this study; the parallel-bar specimen in which corroded steel bar and non-corroded steel bar were arranged in parallel and the cross-bar specimen in which corroded steel bar and non-corroded steel bar were arranged in cross. The half-cell potential, the impedance spectroscopy and the corrosion current density were measured. Comparing the change in half-cell potential of the corroded steel bar and non-corroded steel bar, by electrical conduction, the steel bar that was not actually corroded was evaluated as the corroded steel bar. The impedance spectroscopy of the non-corroded steel bar was decreased by connecting with the corroded steel bar. From these experimental results, the non-corroded steel bar was affected by the polarization.
1 INTRODUCTION

Corrosion of steel bars occurs easily due to non-uniform chloride ion concentration in and around the part where the bars intersect (see Figure 1). In that part, the non-corroded steel bar acts as the localized cathode while the corroded steel bar acts as the anode. Consequently, it is thought that the corrosion current density concentrates in that part due to connection between non-corroded and corroded steel bars. However, given the huge number of intersecting parts in a structure, it is difficult to measure the electrochemical properties of individual intersecting parts because of the electrochemical intervention of each intersection and steel bar.

To clarify the effect of electrical conductivity on the progress of corrosion of steel bars in concrete structures, the electrochemical properties of the connection between non-corroded and corroded steel bars in the concrete was experimentally investigated.

Figure 1: Corrosion of steel bar embedded in concrete

2 EXPERIMENTAL PROCEDURE

2.1 Specimens

Concrete was prepared using ordinary Portland cement (density: 3.15 g/cm$^3$, Blaine fineness: 3320 cm$^2$/g). Fine aggregate was river sand (density: 2.60 g/cm$^3$, fineness modulus: 2.75) while coarse aggregate was crushed sandstone with G$^{\text{max}}$ of 20 mm (density: 2.71 g/cm$^3$, fineness modulus: 6.77). The concrete mix proportions are shown in Table 1. The slump of fresh concrete was 10 ± 2.5 cm and the air content was 4.5 ± 1.5%. The chloride ion concentration in the specimen was 0% or 10% by weight of water (chloride ion concentration of concrete unit volume 0 or 16.5 kg/m$^3$).

The steel bar (SS400) was 16 mm in diameter and 250 mm in length, and the embedded area of the electrode was 125.6 cm$^2$. The steel bars were electrically integrated through connection with lead wires.

A half segment of the parallel specimen (P) and three-quarters of the cross specimen (C) were embedded in concrete having 10% chloride ion concentration added into the mix in order to promote steel bar corrosion, as shown in Figure 2.

The specimens were sealed and cured for 14 days. The specimens were then exposed to 15 cycles of wet-dry exposure, consisting of sprinkling with saturated Ca(OH)$_2$ water for 3 days followed by drying for 4 days per cycle.
2.2 Measurement

Half-cell potential, polarization curves and impedance spectroscopy were measured. Stainless steel of 100×50 mm was used as a counter electrode. Ionic contact with Ag/AgCl (SSE) was used as a reference electrode. Measurements were conducted using a potentiogalvanostat (P/G, Hokudodenko: Hz-7000), as shown in Figure 3.

2.2.1 Half-cell potential

The measurements were conducted using a high impedance voltmeter (1TΩ), since the half-cell potential was measured across a concrete cover thickness of 20 mm and the concrete was kept in the wet condition. The half-cell potential was measured at the time of measuring the impedance spectroscopy.
2.2.2 Electrochemical impedance spectroscopy (EIS)

Electrochemical impedance spectroscopy (EIS) measurements were made with a three-electrode system. The frequencies were scanned from 10 mHz to 10 kHz, applying a potential of 10 mV and measuring at 5 steps/decade [1]. Numerical fitting of the impedance data was made using specific software.

2.2.3 Polarization curves

The scan was started at the half-cell potential to the cathodic limit, and was conducted in the anodic direction until reaching the anodic limit. In addition, the polarization curve excluding the resistance of the concrete was obtained by considering the decrease in voltage. As a result, the cathodic curve and anodic curve were obtained. The relationship between electric current and voltage for the anode and cathode is given by Equation (1). The coefficients in Equation (1) were calculated using the measurement results. The corrosion current can be obtained from the intersection of the two lines.

\[ \eta = a + b \log i \]  

where \( \eta \): overvoltage (mV), \( i \): current density (\( \mu \text{A/cm}^2 \)), \( a, b \): fixed number specific to an electrode reaction, (mV) (mV/decade).

3 RESULTS AND DISCUSSION

3.1 Half-cell potential

Figures 4 and 5 show the temporal change of the half-cell potential. Figure 4 shows that the chloride ion concentration in the \( \text{P} \)–\( \text{Corr} \) shifted to slightly less noble in the range of −400 to −600 mV. According to the ASTM standard test method [2] (\( E \leq -230 \text{ mV vs. SSE} \)), it is thought that the corrosion of steel bars has a probability of more than 90%. On the other hand, in the case of specimens not containing chloride ions, the \( \text{P} \)–\( \text{N3} \) shifted to −200 to 0 mV and therefore no corrosion is expected.

Figure 5 shows that the potential of cross specimens (\( \text{C} \)–\( \text{Corr1} \) and \( \text{C} \)–\( \text{N2} \)) shifted to −300 to −400 mV. It can be seen that the half-cell potential in this case is almost the same. Comparing the change in half-cell potential of the corroded steel bar and non-corroded steel bar, by electrical conduction, the steel bar that was not actually corroded was evaluated as the corroded steel bar. The reason is that both the reference electrode and counter electrode were at the position affected by the macro-cell current (see Figure 6). As a result, in the case of the non-corroded steel bar connected with the corroded steel bar, the half-cell potential could not be correctly measured.

3.2 Impedance spectroscopy

Figures 7 and 8 show the measurement results of the impedance spectroscopy. Figure 7 shows that the impedance spectroscopy of \( \text{P} \)–\( \text{N3} \) is larger than that of \( \text{C} \)–\( \text{N2} \). It is thought that the impedance spectroscopy of the non-corroded steel bar (\( \text{C} \)–\( \text{N2} \)) connecting with the corroded steel bar became small due to the effect of electrical conductivity.
Figure 4: Half-cell potential (Parallel)

Figure 5: Half-cell potential (Cross)

Figure 6: Setting position of the reference electrode

Figure 7: Impedance spectroscopy (Non-corroded steel bar, 11cycle)

Figure 8: Impedance spectroscopy (Corroded steel bar, 11cycle)

Figure 8 shows that the corroded steel bar in the P-Corr and C-Corr1 was almost the same. Also, the impedance spectroscopy of the corroded steel bar was smaller than that of the non-corroded steel bar.

When measuring the impedance spectroscopy, we expect the situation seen in Figure 9. From the measurement results, it was found that the impedance spectroscopy of the non-corroded steel bar was decreased by connecting with the corroded steel bar. This phenomenon may be due to the fact that the measurement area includes both the non-corroded steel bar and the corroded steel bar, as in Figure 10. Therefore, it is necessary to estimate the actual measurement area of the steel bar, especially in the case of the non-corroded steel bar connecting with the corroded steel bar.
3.3 Corrosion current density

Figures 11 and 12 show the results of the polarization curves after 11 cycles. The calculated corrosion current density using the measurement results is shown in Table 2. The corrosion current density of C-N2 is larger than that of P-N3. This is also due to the effect of electrical conductivity, which can be observed in the impedance spectroscopy results. In the case of the corroded steel bar, the corrosion current density of both P-Corr1 and C-Corr1 was larger than that of the non-corroded steel bar (P-N3 and C-N2).

![Figure 11: Polarization curves (Non-corroded steel bar, 11 cycle)](image1)

![Figure 12: Polarization curves (Corroded steel bar, 11 cycle)](image2)

<table>
<thead>
<tr>
<th>Measurement point</th>
<th>Corrosion current density(µA/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-N3</td>
<td>2.04</td>
</tr>
<tr>
<td>C-N2</td>
<td>3.11</td>
</tr>
<tr>
<td>P-Corr1</td>
<td>8.78</td>
</tr>
<tr>
<td>C-Corr1</td>
<td>7.72</td>
</tr>
</tbody>
</table>

3.4 Summary

The experimental findings related to the influence of electrical conductivity on the electrochemical measurement are summarized in Table 3. In the case of the half-cell potential of the cross-bar containing chloride ions, all steel bars shifted to $-400$ mV. The impedance spectroscopy of the non-corroded steel bar was decreased by connecting with the corroded steel bar. From these experimental results, the non-corroded steel bar was affected by the polarization.
Table 3: Summarize experimental results

<table>
<thead>
<tr>
<th></th>
<th>Including chloride ion</th>
</tr>
</thead>
<tbody>
<tr>
<td>The half-cell potential</td>
<td></td>
</tr>
<tr>
<td>Parallel</td>
<td>Non-Corroded steel bar was −200 to 0mV</td>
</tr>
<tr>
<td></td>
<td>Corroded steel bar was −400 to −600mV</td>
</tr>
<tr>
<td>Cross-bar</td>
<td>All steel bar shifted to −400mV</td>
</tr>
<tr>
<td>Impedance spectroscopy</td>
<td></td>
</tr>
<tr>
<td>Parallel</td>
<td>The corroded steel bar was smaller than that of the non-corroded steel bar.</td>
</tr>
<tr>
<td>Cross-bar</td>
<td>The non-corroded steel bar connecting with the corroded steel bar became small due to the effect of electrical conductivity.</td>
</tr>
<tr>
<td>Corrosion current density</td>
<td>Cross-bar</td>
</tr>
<tr>
<td></td>
<td>The corrosion current density of non-corroded steel bar connecting with the corroded steel bar became large.</td>
</tr>
<tr>
<td></td>
<td>→Non-corroded steel bar was affected by the corroded steel bar (polarization).</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

- The half-cell potential of the non-corroded steel bar shifted to less noble by connecting with the corroded steel bar. It is thought that both steel bars are affected by the polarization.
- The impedance spectroscopy of the non-corroded steel bar was decreased by connecting with the corroded steel bar. This may be due to the fact that the measurement area includes both the non-corroded steel bar and the corroded steel bar.
- The corrosion current density of the corroded steel bar was about 2–4 times larger than that of the non-corroded steel bar. It is thought that a combination of low $S_A$ and high $S_C$ increases the corrosion current density.

ACKNOWLEDGMENTS

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REFERENCES

INFLUENCE OF TEMPERATURE HISTORY ON CHLORIDE PENETRATION OF CONCRETE USING WASTE-DERIVED AGGREGATE

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Key words: High Temperature History at Early Age, Chloride Penetration, Molten Slag Fine Aggregate, Porous Ceramic Waste Aggregate

Abstract. The present study aims at investigating the effect of high temperature history simulating a temperature generated in mass concrete structure on resistance to Cl\(^-\) penetration into concrete using the waste-derived aggregates. Six types of concretes were prepared. Portland blast-furnace slag cement type B was used and water to cement ratio of all concretes was 40% to enhance durability under severe chloride environment. Unit water content was 165 kg/m\(^3\) and sand-total aggregate ratio was 45.0vol.%. The MS, PCCA and PCFA were used and their replacement ratios were 30vol.%, 12vol.% and 15vol.%, respectively. Specimens were sealed after casting and stored in a room where the temperature history, whose maximum temperature was 70°C, was controlled. Specimens stored at 20°C were also prepared to make clear the temperature effect. Specimens were submerged at the age of 28 days in sodium chloride solution at a concentration of 10mass% for 182 days.

As a result, apparent diffusion coefficients of Cl\(^-\) (D\(_{ap}\)) for all concrete subjected to high temperature history were larger than that for concrete cured at 20°C. Besides, MS was observed to be effective in decreasing the D\(_{ap}\), especially when concrete was subjected to high temperature history. In addition, the distributions of Cl\(^-\) in concrete at the age of 100 years were calculated by using the D\(_{ap}\) and the depth from the surface at which the concentration of Cl\(^-\) reached the threshold value for corrosion initiation were evaluated. The depths of concrete using waste-derived aggregates were at least 10% smaller than that of concrete without waste-derived aggregate. This fact shows that waste-derived aggregate can improve the resistance to Cl\(^-\) penetration into concrete if they are used appropriately.
1 INTRODUCTION

Waste materials have been desired to be utilized as concrete materials in construction fields to preserve the environment. In concrete field, a lot of waste material has been used not only in production of cement but also as aggregates to contribute to sustainable society.

Molten slag aggregate is one of waste-derived aggregate. Municipal waste is incinerated and the incinerated ash is melted to reduce the volume, and then molten slag is produced as granulated slag. The molten slag aggregate was researched as aggregate for concrete over recent decades and their applicability were reported [1, 2, 3]. However, the use of molten slag aggregate was limited to be used as fine aggregate to plain and reinforced precast concrete members with the design compressive strength under 35 N/mm² in Japanese Industrial Standard (hereafter, JIS) A 5031 “Melt-solidified slag aggregate for concrete derived from municipal solid waste and sewage sludge” [4]. Accordingly, more investigation on properties of concrete using molten slag aggregate has been needed to use it more widely. It was reported to be effective in enhancing the resistance to chloride penetration probably owing to its latent hydraulicity [5].

Porous ceramic waste aggregate is also one of waste-derived aggregates. It is derived from roof tile waste, which is emitted about 10,000 ton per year in the western Japan. The porous ceramic waste aggregate has relatively higher water absorption and lower crushing value, and it has been researched as an internal curing agent. It was reported to be effective as an internal curing agent in reducing autogenous shrinkage and developing compressive strength of concrete [6][7]. In addition, it was implied that the porous ceramic waste aggregate could improve the resistance of concrete against chloride penetration [8].

According to these previous studies, these waste-derived aggregate mentioned above can be applicable to structural concrete as well as can improved properties of concrete. On the other hand, although it is well known that high temperature history can affect properties of concrete, few investigations on the influence on resistance against chloride penetration in concrete using waste-derived aggregate have been carried out.

The present study aims at investigating the resistance to chloride penetration into concrete using the waste-derived aggregate, targeting at MS, porous ceramic waste fine and coarse aggregate, when the concrete is subjected to high temperature history at early age.

2 EXPERIMENT PROCEDURES

2.1 Materials and mix design

Table 1 shows the properties of materials used in the study. Portland blast furnace slag cement type-B, which met the standards set in JIS R 5211 [9], was used as cementitious materials. Crushed liparite sand, molten slag fine aggregate (hereafter, MS), and porous ceramic fine aggregate (hereafter, PCFA) were used for fine aggregate, while crushed liparite gravel and porous ceramic coarse aggregate (hereafter, PCCA) were used for coarse aggregate. The size of PCFA and PCCA were from 1 mm to 5 mm and from 5 mm to 13 mm, respectively, and both of them were immersed in water for more than 7 days before drying to saturated surface-dry condition to apply as internal curing agents.
Table 1: Properties of materials

<table>
<thead>
<tr>
<th>Materials</th>
<th>Type</th>
<th>Notation</th>
<th>Density (g/cm³)</th>
<th>Specific surface area (cm²/g)</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cementitious material</td>
<td>Portland blast-furnace slag cement Type B</td>
<td>BB</td>
<td>3.04</td>
<td>3,840</td>
<td>-</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>Crushed liparite sand</td>
<td>S</td>
<td>2.60</td>
<td>-</td>
<td>2.28</td>
</tr>
<tr>
<td></td>
<td>Porous ceramic fine aggregate</td>
<td>PCFA</td>
<td>2.26</td>
<td>-</td>
<td>8.92</td>
</tr>
<tr>
<td></td>
<td>Molten slag fine aggregate</td>
<td>MS</td>
<td>2.77</td>
<td>-</td>
<td>0.42</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>Crushed liparite gravel</td>
<td>G</td>
<td>2.64</td>
<td>-</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>Porous ceramic coarse aggregate</td>
<td>PCCA</td>
<td>2.27</td>
<td>-</td>
<td>9.20</td>
</tr>
</tbody>
</table>

Table 2 lists the mixture proportions of the concretes prepared in this study. The water to cementitious materials ratio for all concretes was 40%, and the unit water content was 165 kg/m³. Fine aggregate was replaced with MS at 30vol.% and/or with PCFA at 15vol.%, while coarse aggregate was replaced with PCCA at 12vol.%. The amount of water absorbed into 12vol.% PCCA was the same as that into 15vol.% PCFA.

Table 2: Mixture proportions

<table>
<thead>
<tr>
<th>Name of Specimen</th>
<th>W/C (%)</th>
<th>Air (%)</th>
<th>s/a (%)</th>
<th>Replacement ratio (vol.%)</th>
<th>Unit content (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BB40</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>-</td>
<td>BB S MS PCFA G PCCA</td>
</tr>
<tr>
<td>BB40MS</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>30</td>
<td>165 413 766 0 0 952 0</td>
</tr>
<tr>
<td>BB40PCFA</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>15</td>
<td>165 413 651 0 100 952 0</td>
</tr>
<tr>
<td>BB40MSPCFA</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>30 15</td>
<td>165 413 421 245 100 952 0</td>
</tr>
<tr>
<td>BB40PCCA</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>12</td>
<td>165 413 766 0 0 839 97</td>
</tr>
<tr>
<td>BB40MSPCCA</td>
<td>40</td>
<td>4.5</td>
<td>45.0</td>
<td>30 12</td>
<td>165 413 536 245 0 839 97</td>
</tr>
</tbody>
</table>

2.2 Temperature conditions

All specimens were sealed with aluminum adhesive tape after casting and stored in a temperature-controlled chamber, which provided a high temperature history up to the age of 7 days and then at 20°C. The temperature history was a simulated temperature at the point where it shows the highest maximum temperature in massive concrete structure with same mixture proportion of BB40. In addition to the specimens subjected to the high temperature history at early age, specimens stored at 20°C just after casting were also prepared in the case of BB40, BB40PCFA and BB40MSPCFA for comparison. Both temperature histories were shown in Figure 1. H-concrete and 20-concrete stand for concrete subjected to high temperature history and concrete stored at 20°C just after casting, respectively.
2.3 Chloride diffusion test

Cylinder samples having the diameter of 100 mm and the height of 50 mm were taken from cylinder specimens having the diameter of 100 mm and the height of 200 mm with a diamond-cutter at the age of 28 days to avoid the effect of casting surface. Then, all surfaces except for one end surface exposed to chloride solution were coated with an epoxy resin. The samples were immersed in NaCl solution at concentration of 10mass% after immersion for 1 day in water.

At the immersion period of 182 days, the samples were taken from the NaCl solution and split to measure the depth of chloride penetration by spraying AgNO₃ solution. After that, the samples were sliced at each around 2.5 mm from the surface and each sliced sample was milled into powder. The total chloride content at each depth of concrete was determined by boiling 1 g of the powder in 1 mL of H₂O₂ acidified with 50 mL of 2.0 mol/L HNO₃. The solution was cooled down after boiling, and filtered with membrane filter. After that, the volume of the filtered solution was adjusted to 200 mL by adding deionized water. Samples were analyzed for the chloride concentration by using an ion chromatography.

The diffusion coefficient was obtained by fitting the measured profile to the solution of Fick’s second law and initial chloride concentration as shown Eq.(1).

\[
C(x,t) = C_0 \left\{ 1 - \text{erf} \left( \frac{x}{2(D_{ap} \cdot t)} \right) \right\} + C_i
\]

where, \(C(x,t)\) = total chloride concentration at depth of \(x\) (mm) at age of \(t\) (year), \(C_0\) = chloride concentration at the surface (kg/concrete-m³), \(\text{erf}\) = error function, \(D_{ap}\) = apparent chloride diffusion coefficient (mm²/year), \(C_i\) = initial chloride concentration in concrete.
3 RESULTS AND DISCUSSION

3.1 Apparent chloride diffusion coefficient

Profile of total chloride concentration is shown in Figure 3. The profile were measured from the surface to the depth of more than 25 mm, while the penetration depths measured by using AgNO₃ were from 8 mm to 13 mm. In the case of each temperature condition, no significant difference among mixture proportions was observed, whereas the shape of profiles in the case of H-concrete was more gradual than that in the case of 20-concrete. It means that using waste-derived aggregate (MS, PCFA, and PCCA) for concrete subjected to high temperature history did not affect adversely the resistance against chloride ingress.

![Figure 3: Profile of total chloride concentration in concrete at the immersion period of 182 days in the case of H-concrete (a), in the case of 20-concrete (b)](image)

Apparent chloride diffusion coefficient was obtained from the profile as shown in Figure 3, where the value for the correlation coefficient for each fitting was greater than 0.98. The fitting curves and profiles of BB40MS and BB40PCFA in the case of H-concrete were illustrated in Figure 4 as examples.

The apparent chloride diffusion coefficients are described in Figure 5. In the case of H-concrete, apparent chloride diffusion coefficient of every concrete using waste aggregate was smaller than that of BB40, although in the case of 20-concrete that of BB40PCFA or BB40MSPCFA was same or larger than that of BB40. It can be said that the damage by temperature history on concrete could be reduced by using MS, PCFA and PCCA, resulting in the smaller diffusion coefficient of concrete using waste aggregate than BB40 when concrete was subjected to high temperature history. In addition, MS was observed to be effective in decreasing the $D_{ap}$, especially when concrete was subjected to high temperature history.
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![Graph showing chloride concentration and diffusion coefficient](image)

**Figure 4:** Fitting curves and profiles of BB40MS and BB40PCFA H-concrete

**Figure 5:** Apparent chloride diffusion coefficient

### 3.2 Estimation of distribution of chloride concentration

According to Eq. (1), distribution of chloride concentration at the age of 50 years and 100 year was calculated by using apparent chloride diffusion coefficient obtained in section 3.3. Chloride concentration at the surface of 13 kg/m³ was used, which is designed value at the
most severe condition on standard specifications by Japan Society of Civil Engineers (hereafter, JSCE) [10]. In addition, initial chloride concentration of 0.3 kg/m$^3$ was used based on the same standard specification.

Calculated distributions of chloride concentration in BB40 and BB40PCFA at the age of 100 years are shown in Figure 5 to investigate the influence of high temperature history at early age. It can be seen that the high temperature history influence adversely on the resistance against chloride penetration, and the influence in BB40 was larger than BB40PCFA. The concentration of chloride which reached the threshold value for corrosion initiation (hereafter, $C_{lim}$) was added as red broken line. The concentration of 2.06 kg/m$^3$ was calculated according to the standard specification of JSCE [10]. In the case of BB40, the depth at which the concentration of chloride reached $C_{lim}$ in H-concrete and 20-concrete were 182 mm and 118 mm, respectively. It means high temperature history increased the depth by a factor of 1.5. On the other hand, in the case of BB40PCFA, the depth of H-concrete and 20-concrete were 165 mm and 142 mm, respectively, which indicates that high temperature history increased the depth by a factor of 1.2.

![Figure 5: Estimation of chloride content distribution in BB40 and BB40PCFA at the age of 100 years](image)

The depth at which the concentration of chloride reached $C_{lim}$ in the case of all mixtures at the age of 50 years and 100 years are shown in Figure 6. The depths of concrete using waste-derived aggregates were around 10% - 30% smaller than that of concrete without waste-derived aggregate. This result can support that waste-derived aggregate can improve the resistance to chloride penetration into concrete if they are used appropriately.
4 CONCLUSIONS

The aim of present study is investigating the applicability of waste-derived aggregate to massive structural concrete using portland blast furnace slag cement type B to use them more widely for sustainable society. Concrete was subjected to high temperature history simulating the temperature in massive concrete structure. The following conclusions can be drawn within the limit of the study.

- High temperature history at early age can affect adversely the resistance of concrete against chloride ingress and it can double the apparent diffusion coefficient of concrete using portland blast furnace slag cement type B.
- The apparent chloride diffusion coefficient of concrete using waste-derived aggregate was smaller than that of concrete without waste-derived aggregate. When the depth at which the concentration of chloride reached the threshold value for corrosion initiation at the age of 50 years and 100 years were evaluated, using waste-derived aggregates were at least 10% smaller than that of concrete without waste-derived aggregate.
- Consequently, waste-derived aggregate can be applied for concrete which can be subjected to high temperature history. In addition, the waste-derived aggregate could improve the resistance against chloride ingress.

REFERENCES


LONG-TERM EFFECTS OF THE HARDENING TEMPERATURE AND RELATIVE HUMIDITY ON THE MICROSTRUCTURE AND PROPERTIES OF MORTARS WITH ACTIVE ADDITIONS

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Key words: Ground Granulated Blast Furnace Slag, Fly Ash, Microstructure, Durability, Relative Humidity, Temperature.

Abstract. In recent years the use of active additions on cement, like ground granulated blast furnace slag and fly ash, has become very popular because they have many environmental benefits. The main advantages are the reduction of CO₂ emissions and the lower energy consumption during the cement production, as well as the use of an industrial waste. The use of these wastes also means an economical benefit for the cement industry. Many studies show that in laboratory conditions this kind of materials has good service properties, even better than Portland cement. However, real structures are usually hardened in different conditions depending on their geographical location. These different environmental conditions may influence the development of the microstructure and the service properties of slag and fly ash cement mortars. In this work, the behaviour of mortars made with an ordinary Portland cement and with cements which incorporate slag and fly ash, was tested. These mortars were exposed to different temperature and relative humidity conditions during their hardening. The development of the microstructure of mortars and the changes of their service properties were studied at different hardening ages until 5 years. The mortars made with slag and fly ash cements have shown a good behaviour compared with ordinary Portland cement mortars. The temperature and relative humidity have an influence on microstructure and properties of all cement types studied. As a preliminary conclusion, cements with slag and fly ash hardened under non-optimal conditions can have good service properties at high hardening ages, even better than Portland cement.

1 INTRODUCTION

The use of mineral admixtures in the cement manufacture has many environmental benefits, like the reduction of CO₂ emissions and the use of an industrial waste. The particular cases of ground granulated blast furnace slag and fly ash, and their effect on the properties of the cementitious materials is a topic of study. Many studies show that in laboratory conditions this kind of materials has good service properties, even better than Portland cement [1]. This fact is due to the development of the hydration reactions of the slag and pozzolanic reactions of fly ash with the portlandite produced in the hydration of clinker. These reactions densify the pore structure of concrete due to the formation of additional CSH phases, so the pore size
distribution is shifted toward finer pores (pore refinement) [1].

Durability properties of cementitious materials are known to be directly related to the microstructure of these materials [2]. Then, the use of slag and fly ash cements also reduces the permeability of concrete and improves their resistance to aggressive ion ingress [3]. Real structures are usually hardened in different environmental conditions depending on their geographical location. The different temperature and especially the different relative humidity [3] present in the environment may influence the development of hydration reactions of slag and pozzolanic reactions of fly ash. This influence may cause a different microstructure [4] and, as a consequence different service properties of these concretes and mortars, such as the diffusion coefficient of chlorides [5, 6] and compressive strength [7].

In this work, mortars made with three different commercial cement types, an ordinary Portland cement, a ground granulated blast furnace slag cement and a fly ash cement, exposed to different environmental conditions, were tested. The development of their microstructure and their changes in durability-related properties were studied at different hardening ages until 1900 days (5 years), as a function of the hardening environment.

2 EXPERIMENTAL SETUP

2.1 Sample preparation

Mortar samples were prepared using an ordinary Portland cement (OPC), CEM I 42.5 R (CEM I from now on), a Portland cement with fly ash (content of fly ash from 21 to 35%), CEM II/B-V 42.5 R (CEM II from now on), and a ground granulated blast furnace slag (GGBS) cement (content of GGBS between 66-80% of total binder), III/B 42.5 L/SR (CEM III from now on), according to the Spanish standard UNE EN 197-1 [8]. The water to cement ratio was 0.5. Fine aggregate was used according to the Spanish standard UNE EN 196-1 [9]. The aggregate to cement ratio was 3:1 for all the mortars.

Cylindrical specimens were prepared. They were cast in cylindrical moulds of 10 cm diameter and 15 cm height. Samples were kept in 95% RH chamber and 20ºC for 24 hours. After that time they were demoulded and they were cut into cylinders of 1 and 5 cm thick. Environmental conditions (RH and temperature) were managed with glycerol solutions introduced in hermetically sealed containers. Solutions were prepared according to the German standard DIN 50 008 part 1 [10]. Containers were introduced into a chamber with controlled temperature.

The environments studied were three: environment A (100% RH and 20ºC, optimum laboratory condition), environment B (85% RH and 15ºC) and environment C (65% RH and 20ºC). Environment B represents the Atlantic climate, present in the north part of Iberian Peninsula (Spain and Portugal), while environment C is representative of Mediterranean climate, present in the east part of Iberian Peninsula. Tests were performed at several ages until 1900 days (5 years approximately).

2.2 Mercury intrusion porosimetry

In order to study the microstructure of mortar samples, mercury intrusion porosimetry was used. This is a well-known and very used technique [11], in spite of being reported the main
problems of employing it [12]. Samples were oven dried for 24 hours at 105°C before the test. Two measurements were made on each sample. The porosimeter employed was an Autopore IV 9500 from Micromeritics. The total porosity was studied.

2.3 Impedance spectroscopy

The impedance measurements on the mortar samples were carried out using the impedance analyser Agilent 4294A. This analyser allows capacitance measurements in the range from \(10^{-14}\) F to 0.1 F, with a maximum resolution of \(10^{-15}\) F. Impedance spectra of mortar samples were obtained in the frequency range from 100 Hz to 100 MHz, using a non-contacting method [11]. The electrodes were circular (\(\Omega=7\) cm) and made of flexible graphite, attached to a copper piece with the same diameter.

The obtained impedance spectra were validated using the Kramers–Kronig (K–K) relations, to ensure causality, linearity and stability of the measurements made [13]. Measured data were fitted to the equivalent circuit proposed by Cabeza et al. [11], which includes two time constants. This circuit has been widely used for different type of materials, like cements with active additions [2, 3]. The circuit includes several parameters, however in this work only the capacitance \(C_1\) was studied. This capacitance is related to the solid fraction in the sample. The fitting of the measured data to the model proposed is made using a Simplex optimization method which is described elsewhere [14, 15]. Four different cylindrical samples of 1 cm thick were tested for each cement type and environment.

2.4 Capillary absorption test

The capillary absorption test was performed according to Spanish standard UNE 83.982 [16]. The result obtained is the capillary suction coefficient. Before the test, samples were completely dried for 12 hours in an oven at 105°C, in spite of Rilem recommendation [17] that suggests a saturation degree of 70% on samples. The election of complete drying was made in order to accelerate the test and to prevent changes in the microstructure not a result of the hardening environment, especially at early ages. Three different cylindrical samples of 5 cm thick were tested for each cement type and environment.

2.5 Forced migration test

The forced migration test was performed according to Finnish standard NT Build 492 [18]. The result of the test is the non-steady-state chloride migration coefficient \(D_{NTB}\). Samples were saturated for 24 hours before the test according to ASTM Standard C1202-97 [19]. Three different cylindrical samples of 5 cm thick were tested for each cement type and environment.

3  RESULTS AND DISCUSSION

3.1 Microstructure results

The results of microstructural characterization of the three types of mortar studied are depicted from Figure 1 to Figure 6 for each environment.

For environment A (Figures 1 and 2), total porosity decreased and capacitance \(C_1\)
increased with time for CEM I mortars. Both parameters showed a similar trend for CEM II mortars, although their capacitance $C_1$ developed an important growth at larger ages. For samples made with CEM III, total porosity showed a high decrease between 7 and 28 days, and the main rise of capacitance $C_1$ has been observed between 7 and 60 days. In the long-term the total porosity of slag mortars continued decreasing and their capacitance $C_1$ carry on rising, although at a lower rate. These results could mean that a higher RH accelerates the development of the hydration and pozzolanic reactions, especially the hydration of slag [3]. With these reactions, new solids appear quicker, then total porosity decreases at early ages and capacitance $C_1$ rises. The decrease of total porosity and the growth of capacitance $C_1$ for CEM II samples are delayed in comparison to CEM III samples. This could be related to the delaying of the development of pozzolanic reactions of fly ash respect to the hydration of clinker and slag [20].

**Figure 1.** Results of total porosity for mortars exposed to environment A.

**Figure 2.** Results of capacitance $C_1$ for mortars exposed to environment A.

**Figure 3.** Results of total porosity for mortars exposed to environment B.

**Figure 4.** Results of capacitance $C_1$ for mortars exposed to environment B.
For environment B (Figures 3 and 4) total porosity hardly changed with time or showed a small decrease for the majority of the samples studied. The capacitance $C_1$ kept practically constant or increased slowly with age for all types of mortars. The slower development of microstructure observed for environment B, could be related with the lower temperature of this condition (15ºC). In an environment with low temperature the hydration reactions of clinker and slag slow down [21] and the formation of solids is also slower. If the clinker hydration is slower, the pozzolanic reactions of fly ash will start later. Then, more time was needed to observe their effects in microstructure development, as suggested the fact that total porosity kept constant between 7 and 180 hardening days for CEM II mortars, and it hardly decreased since then until 1900 days. Besides, this result could mean that the lower temperature may also slow down the development of pozzolanic reactions of fly ash.

For environment C (Figures 5 and 6), total porosity of CEM I samples kept practically constant until 28 days, decreased since then until 90 days, increased between that age and 1900 days. The capacitance $C_1$ for CEM I mortars increased until 30 days approximately, since then until 120 days it kept constant, and it decreased in the long-term. Total porosity of CEM II specimens followed practically the same evolution with time as CEM I ones until 180 days. However, it decreased at greater ages. The capacitance $C_1$ of CEM II samples kept practically constant until 120 days, and it fell at 180 days, but it rose since then until 1900 days. For CEM III samples, total porosity remained constant until 28 days, decreased between 28 and 90 days, and increased at 180 days, falling in the long-term. On the other hand, for slag cement mortars the capacitance $C_1$ showed the highest values of all types of cement studied since 50 days, and at later ages this capacitance hardly decreased or remained constant.

The results observed for environment C, may be interpreted as that the lower RH slows down hydration reactions of clinker and slag [6], then the decrease of total porosity happens later (since 28 days hardening), and the increase of capacitance $C_1$ is slower. The similar evolution of total porosity for CEM I and II samples until 90 days of age, could mean that the
degree of development of pozzolanic reactions of fly ash is very low at that hardening time. This could be related with slowing down of the hydration reactions of clinker, due to lower RH in the environment. The products of these hydration reactions are necessary to start the pozzolanic reactions of fly ash [20], so it seems that under condition C more time is necessary to observe the beneficial effects on microstructure of replacing clinker by fly ash, as indicated the long-term results of CEM II mortars. The increase of total porosity between 90 and 180 days and the decrease of capacitance $C_1$ at the same age observed for most of the samples, could be due to the formation of shrinkage microcracks as a consequence of lower RH in the environment [22]. Nevertheless, in the case of mortars with active additions, the continuous development of slag hydration and fly ash pozzolanic reactions brings the formation of new solid phases and the consequent reduction of porosity in the long-term, despite the abovementioned possible formation of shrinkage microcracks.

Finally, it is important to emphasize that the addition of fly ash and ground granulated blast furnace slag improved the microstructure of mortars for all the studied environments after 1900 hardening days, compared to ordinary Portland cement.

### 3.2 Durability-related properties

The results of capillary suction coefficient ($K$) and non-steady-state chloride migration coefficient ($D_{NTB}$) are depicted from Figure 7 to Figure 12.

**Figure 7.** Capillary suction coefficient results for mortars exposed to environment A.

**Figure 8.** Results of non-steady-state chloride migration coefficient for environment A.

For environment A (Figures 7 and 8), in general the coefficient $K$ decreased with time for CEM I, II and III samples. The $D_{NTB}$ coefficient had smaller values for samples prepared using CEM II than using CEM I at later ages, but the CEM III ones showed the lowest values of this coefficient between 7 and 1900 days. This result could be expected from the microstructure characterization. The $D_{NTB}$ decreased with the hardening age until 90 days for the majority of mortars hardened in condition A, mainly in the case of mortars with active additions.
For environment B (Figures 9 and 10), the coefficient $K$ and the $D_{NTB}$ decreased with time for all mortars studied. However, the decrease of both parameters was slower than observed for environment A.

![Environment B](image)

**Figure 9.** Capillary suction coefficient results for mortars exposed to environment B.

![Environment B](image)

**Figure 10.** Results of non-steady-state chloride migration coefficient for environment B.

For environment C (Figures 11 and 12), the coefficient $K$ decreased with age for CEM I and CEM III samples (mainly between 28 and 90 days). For CEM II, this coefficient hardly decreased until 180 days, but it showed an important fall from that age to 1900 days. Regarding the coefficient $D_{NTB}$, CEM I and II samples presented a decreasing tendency of this coefficient between 7 and 28 days of age. In general, this tendency continued until 1900 days for CEM II mortars. However, for CEM I samples $D_{NTB}$ strongly increased since 28 hardening
days. This high increase was not observed for CEM II and III samples hardened under environment C. As could be expected from microstructure results, the development of shrinkage microcracks in the samples, as a consequence of lower RH in the environment, could also facilitate the ingress of chlorides, especially in the case of CEM I mortars. This effect of microcracking could also be produced in CEM II and III mortars, but probably their smaller volume of capillary pores [3] makes more difficult the ingress of chlorides, as indicated by their lower $D_{NTB}$ in the long-term. Furthermore, another fact that can help to explain the good resistance of CEM II and III mortars to chloride ingress is the higher binding capacity of slag cement, as compared to Portland cement. This binding capacity is due to the high content of calcium aluminates brought by the slag and fly ash [23]. Finally, CEM III mortars showed the lowest $D_{NTB}$ coefficient at the majority of the ages for condition C.

Again, it is shown that a very high RH combined with high enough temperature in the environment make easier the development of hydration and pozzolanic reactions, and then, the coefficient $K$ and especially the coefficient $D_{NTB}$ decrease quickly. On the other hand, if temperature in the environment is lower, the development of hydration and pozzolanic reactions is slower, as has been discussed previously for microstructure results. Then, the durability-related parameters decrease slowly and more time is needed to reach similar values than observed for an optimum laboratory condition. Nevertheless, if RH is lower and environmental temperature is high enough, hydration reactions of clinker and slag and pozzolanic reactions of fly ash also develop slower, and the coefficient $K$ and the coefficient $D_{NTB}$ decrease later. In spite of that, in the very long-term, $D_{NTB}$ reaches very low values for CEM II and III, compared to CEM I ones. At greater ages (more than 90 days), the influence of lower RH has more effect for CEM I samples, and their $D_{NTB}$ increases. As it has been already explained, this fact could be a consequence of combination between shrinkage effects and higher volume of capillary pores, which show the direct relation between microstructure and durability-related-properties, especially the chloride migration coefficient.

4 CONCLUSIONS

The main conclusions that can be obtained from the results previously discussed can be summarized as follows:

- The development of pozzolanic reactions of fly ash is delayed respect to the hydration of clinker and slag. This makes that the benefits of replacing clinker by fly ash in microstructure and durability-related properties will be shown at higher hardening ages.

- A low temperature in the environment slows down the development of hydration and pozzolanic reactions. Then, the development of the microstructure is slower and it is needed more time to reach similar durability-related properties than for an optimum laboratory condition.

- The improvement of the durability of mortars with active additions is delayed in environments with relative humidity lower than 100%, but after 5 hardening years these properties have reasonable good values, especially the chloride ingress resistance. In these environments, it seems that degree of development of fly ash pozzolanic reactions was very low until 90 or 180 days hardening. However, since
those ages the effect of these reactions on durability of CEM II mortars was more evident.

- Relative humidity lower than 100% can produce shrinkage microcracking in mortars at greater hardening ages. The effect of this microcracking is higher for CEM I mortars as they are exposed to a lower relative humidity, and it mainly produces an important fall of their chloride ingress resistance.

- The addition of fly ash and ground granulated blast furnace slag improved the microstructure of mortars for all the studied environments after 5 hardening years, compared to ordinary Portland cement.

- Cements with ground granulated blast furnace slag and fly ash, hardened under environmental conditions of Atlantic and Mediterranean climates, can have good service properties in the very long-term.

5 ACKNOWLEDGEMENTS

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REFERENCES


MECHANICAL PROPERTIES AND DURABILITY OF MORTARS CONTAINING NANOSILICA AND RICE HUSK ASH

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Key words: Nanosilica, rice husk ash, mortar, mechanical properties, durability, sustainability.

Abstract. Reactive materials such as nanosilica and rice husk ash when incorporated in mortars and concrete mixtures improve the mechanical properties and enhance their durability. In the present investigation, the effect of nanosilica hydrosols and rice husk ash on the compressive strength, chloride ion permeability, capillary absorption, and electrical resistivity of single and binary blended mortars was investigated. Results showed that the addition of nanosilica improved mortar performance, while rice husk ash did not have a significant influence and mainly attributed to lower strength and durability for binary blended mortars, compared to single mixes containing nanosilica at early ages. However, binary mixtures displayed the best performance regarding the mechanical properties and durability at longer ages.

1 INTRODUCTION

Civil structures are usually subjected to harsh environmental conditions. One of such aggressive agents is chloride ion that has the ability to impair the strength, serviceability and beauty of structures. Currently, the most widely used mineral additives for improvement of concrete properties and reducing the destructive effects of chloride ions are silica fume and fly ash, while the most commonly utilized nano-powders are nano-SiO$_2$, nano-Al$_2$O$_3$ and nano-Fe$_2$O$_3$ [1]. Nano-SiO$_2$ is synthetically produced in the form of ultra-fine particles of amorphous silica and could lead to better results in comparison with micro-SiO$_2$ [2,3]. The addition of nanosilica was found to increase the compressive strength development of mortars. Some researchers [4-11] have reported a progressive increase of compressive strength with increasing nanosilica content, while others [12-16] obtained optimal proportions for lower dosages of additive due to agglomeration caused by difficulties to disperse the higher percentage of particles during mix.

The small particle size of nano-SiO$_2$ provides a larger surface area, which speeds up the
rate of cement hydration and pozzolanic reactions with calcium hydroxide crystals, which are arrayed in the interfacial transition zone and produces C–S–H gel. Thus, the size and amount of calcium hydroxide crystals are significantly decreased, and the early age strength of the hardened cement paste is increased. [17-19] Furthermore, chloride penetration mortars containing nanosilica have also greatly decreased [11, 20]. The addition of nanosilica not only increases compressive strength which is mainly attributed to the reduction in total porosity but also results in a finer pore structure that reduces permeability [21, 22].

2 EXPERIMENTAL PROGRAM

2.1 Materials

The Portland cement used in this study was of type I-425 according to the Iranian national standards 389. Its chemical composition and physical properties is shown in Table 1.

Rice husk was collected from rice fields of Gilan, located in the north of Iran and was incinerated at a controlled temperature of 650°C and time duration of 60 min, using the RHA furnace at Amirkabir University of Technology. After combustion, the ash was pulverized to the same size as cement particles by using a vibrating ball mill. The results of XRF and XRD of the RHA are given in Table 1 and Figure 1, respectively.

Two types of nanosilica hydrosols with different specific surface areas, including Levasil 100/40 and Levasil 300/25 supplied by H.C. Stark Gmbh, were used in this study. These products, according to the manufacture’s data, have specific surface areas of 100 and 300 m²/g, mean particle sizes of 36.2 and 12.9 nm and mass concentrations of 40 and 25%, respectively. The physical and chemical properties of the nanosilicas are shown in Table 2.

Table 1: Chemical composition and physical properties of cement and rice husk ash.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Cement (%)</th>
<th>RHA(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>59.8</td>
<td>1.68</td>
</tr>
<tr>
<td>SiO₂</td>
<td>22.52</td>
<td>88.2</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.24</td>
<td>0.12</td>
</tr>
<tr>
<td>MgO</td>
<td>4.16</td>
<td>0.62</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.86</td>
<td>0.22</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.12</td>
<td>0.32</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.78</td>
<td>1.92</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.43</td>
<td>0.03</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>-</td>
<td>0.73</td>
</tr>
<tr>
<td>Loss of ignition</td>
<td>0.85</td>
<td>5.84</td>
</tr>
<tr>
<td>C₃S</td>
<td>45.31</td>
<td>-</td>
</tr>
<tr>
<td>C₃S</td>
<td>25.53</td>
<td>-</td>
</tr>
<tr>
<td>C₃A</td>
<td>11.75</td>
<td>-</td>
</tr>
<tr>
<td>C₄AF</td>
<td>7.35</td>
<td>-</td>
</tr>
<tr>
<td>Blaine specific surface area (cm²/gr)</td>
<td>2848</td>
<td>2848</td>
</tr>
</tbody>
</table>
A polycarboxylate based superplasticizer (Tamcem 23) was employed to aid the dispersion of nano-particles and RHA and achieve the desired workability.

River sand with maximum aggregate size of 4.75mm was graded according to the requirements of ASTM C77 (Table 3). The resultant standard graded sand had a specific gravity of 2500 kg/m$^3$, water absorption of 3.16% and fineness modulus of 3.49.

### Table 2: Chemical composition and physical properties of nanosilicas.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Levasil 100/40 (%)</th>
<th>Levasil 300/25 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>0.43</td>
<td>0.35</td>
</tr>
<tr>
<td>Loss of ignition</td>
<td>&lt;1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Moisture content</td>
<td>60</td>
<td>75</td>
</tr>
<tr>
<td>Surface area (m$^2$/g)</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>Particle size (nm)</td>
<td>36.2</td>
<td>12.9</td>
</tr>
<tr>
<td>pH</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>L100</td>
<td>L300</td>
</tr>
</tbody>
</table>

### Table 3: Graded sand according to ASTM C77.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>ASTM C778</th>
<th>experimental program</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.18 mm (No.16)</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>600µm (No.30)</td>
<td>96 to 100</td>
<td>100</td>
</tr>
<tr>
<td>425µm (No.40)</td>
<td>65 to 75</td>
<td>65</td>
</tr>
<tr>
<td>300µm (No.50)</td>
<td>20 to 30</td>
<td>20</td>
</tr>
<tr>
<td>150µm (No.100)</td>
<td>0 to 4</td>
<td>0</td>
</tr>
</tbody>
</table>

### 2.2 Procedures

The mortar mixtures for compressive strength test specimens were designed with a
constant water/binder ratio of 0.485 and sand/binder ratio of 2.75, according to the ASTM C109 standard. Capillary absorption, RCMT and electrical resistivity test specimens were also prepared with the same requirements and a total of 15 mortar mixtures were prepared with 2.5% and 5% (by weight) replacement of cement with nanosilica and a 5% to 20% (by weight) replacement with rice husk ash. The mixture proportion is provided in Table 4, in which the percentage of each pozzolan is followed by its abbreviation. As seen in Table 4, the slump is kept at a range of 110-140 mm and the difference in water demand of various mixes was accounted for by the use of required amounts of the super plasticizer.

Table 4: Mortar mixture proportions (kg/m³).

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement</th>
<th>Water</th>
<th>Sand</th>
<th>Nano-SiO₂</th>
<th>RHA</th>
<th>Super plasticizer</th>
<th>Flow Table (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>1000</td>
<td>569</td>
<td>2666</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>131</td>
</tr>
<tr>
<td>2.5L100</td>
<td>975</td>
<td>519</td>
<td>2666</td>
<td>84</td>
<td>-</td>
<td>1.2</td>
<td>130</td>
</tr>
<tr>
<td>2.5L300</td>
<td>975</td>
<td>478</td>
<td>2666</td>
<td>121</td>
<td>-</td>
<td>2.6</td>
<td>120</td>
</tr>
<tr>
<td>5L100</td>
<td>950</td>
<td>468</td>
<td>2666</td>
<td>168</td>
<td>-</td>
<td>2.4</td>
<td>126</td>
</tr>
<tr>
<td>5L300</td>
<td>950</td>
<td>388</td>
<td>2666</td>
<td>242</td>
<td>-</td>
<td>4</td>
<td>114</td>
</tr>
<tr>
<td>5RHA</td>
<td>950</td>
<td>569</td>
<td>2666</td>
<td>-</td>
<td>50</td>
<td>1.3</td>
<td>131</td>
</tr>
<tr>
<td>10RHA</td>
<td>900</td>
<td>569</td>
<td>2666</td>
<td>-</td>
<td>100</td>
<td>1.8</td>
<td>132</td>
</tr>
<tr>
<td>15RHA</td>
<td>850</td>
<td>569</td>
<td>2666</td>
<td>-</td>
<td>150</td>
<td>2.2</td>
<td>130</td>
</tr>
<tr>
<td>20RHA</td>
<td>800</td>
<td>569</td>
<td>2666</td>
<td>-</td>
<td>200</td>
<td>2.9</td>
<td>130</td>
</tr>
<tr>
<td>2.5L100+10RHA</td>
<td>875</td>
<td>519</td>
<td>2666</td>
<td>84</td>
<td>100</td>
<td>2.5</td>
<td>135</td>
</tr>
<tr>
<td>2.5L300+10RHA</td>
<td>875</td>
<td>478</td>
<td>2666</td>
<td>121</td>
<td>100</td>
<td>3.8</td>
<td>125</td>
</tr>
<tr>
<td>2.5L100+15RHA</td>
<td>825</td>
<td>519</td>
<td>2666</td>
<td>84</td>
<td>150</td>
<td>3.2</td>
<td>137</td>
</tr>
<tr>
<td>2.5L300+15RHA</td>
<td>825</td>
<td>478</td>
<td>2666</td>
<td>121</td>
<td>150</td>
<td>4.2</td>
<td>116</td>
</tr>
<tr>
<td>5L100+10RHA</td>
<td>850</td>
<td>468</td>
<td>2666</td>
<td>168</td>
<td>100</td>
<td>3</td>
<td>132</td>
</tr>
<tr>
<td>5L300+10RHA</td>
<td>850</td>
<td>388</td>
<td>2666</td>
<td>242</td>
<td>100</td>
<td>7</td>
<td>112</td>
</tr>
</tbody>
</table>

Cubes of 50 mm were cast and compacted by tamping the mortar 32 times in two layers. These specimens were used for compressive strength and capillary absorption test. Also, cylinders with the diameter of 100 mm and the height of 200 mm were cast for RCMT and electrical resistivity test and compacted using the vibrating table. After casting, the specimens were covered to minimize water evaporation. They were demolded after 24 h, and cured in water (CH-saturated) at 23 ± 2 °C until testing.

2.3 Testing methods

2.3.1 Compressive strength

Compression tests were performed at 3, 7, 28 and 90 days in accordance to ASTM C 109. At each age, three 50 mm cubic mortar specimens were tested for determining compressive strength.

2.3.2 Rapid chloride migration test (RCMT)

The RCMT test was performed according to the method described in NT Build 492 [30] at the ages of 28 and 90 days. For each mixture, 3 specimens have been tested according to the
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RCMT method in order to measure their chloride resistivity. In this method, the diffusion of chloride ions into 50 mm thick cement mortar disks is accelerated by applying an electrical potential for duration of 24 h. In this procedure, the magnitude of applied potential is selected based on initial current passed through the specimen. At the end of the test, the specimens are axially split and 0.1 M AgNO₃ solution is sprayed on to one of the freshly split sections. The non-steady-state migration coefficient is then calculated according to NT Build492.

2.3.3 Surface electrical resistivity

The surface resistivity of mortar specimens was measured using the four point Wenner Probe at ages of 7, 28 and 90 days. In this non-destructive method, a current is applied to the two outer probes and the potential difference is measured between the two inner probes.

2.3.4 Capillary absorption

The capillary absorption test was carried out according to the procedures described in BS EN 480-5:2005 at the ages of 7 and 28 days. For each mixture, three 50 mm cubic mortar specimens were dried in the oven at 50°C for 10 days until all the pores were dried of water. They were then put on rods in a water bath in such a way that one surface of each specimen was immersed in water for no more than 3±1 mm. Other surfaces were covered with impermeable tape to prevent penetration of water. Weight of absorbed water was then measured at ages of 3, 6, 24 and 72 hr.

3 RESULTS AND DISCUSSIONS

3.1 Compressive strength

The average compressive strength results of various mixes as determined from three cubic specimens are given in Table 5. To provide a better comparison, the normalized strength results relative to the control mix are presented in Figure 2.

The high enhancement of compressive strength for mortars containing L300 is due to their high surface area and reactivity. Rapid consumption of Ca(OH)₂ crystals, which was formed during hydration of Portland cement specially at early ages, has greatly improved the strength of L300 mortars at the ages of 3 and 7 days. However, with the progress of hydration, the increase of strength due to the incorporation of such nano-particles gradually diminishes and L100 mortars prove to be more efficient at higher ages. For instance, the improvement in compressive strength of mortars incorporating 5% L100 compared to the control mix improves from 17.4% at the age of 3 days to 36.8% at the age of 90 days.

Regarding the compressive strength of mortar specimens containing RHA, it can be inferred from the results that the highest values belong to the mixtures containing 10% level of cement replacement at the ages of 3 and 7 days and 15% replacement at the ages of 28 and 90 days. However, due to the slow reactivity of RHA particles, improvement of strength compared to the control mix was only witnessed at the age of 90 days for 10 and 15% replacement of cement.

Furthermore, binary mixes containing nanosilica and RHA displayed relatively low values of compressive strength at earlier ages due to the negative effect of RHA and high performances at ages of 28 and 90 days. For instance, 5%L100+5RHA demonstrated the most
significant influence on enhancing the compressive strength at higher ages (35.7% increase in strength at 90 days) whereas low strength values were observed for this mix at earlier ages (2.2% decrease in strength at 3 days).

Table 5: compressive strength results of mortar mixes.

<table>
<thead>
<tr>
<th>Mortar Mix</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 days</td>
</tr>
<tr>
<td>OPC</td>
<td>36.8</td>
</tr>
<tr>
<td>2.5L100</td>
<td>40.8</td>
</tr>
<tr>
<td>2.5L300</td>
<td>46.4</td>
</tr>
<tr>
<td>5L100</td>
<td>43.2</td>
</tr>
<tr>
<td>5L300</td>
<td>49.6</td>
</tr>
<tr>
<td>5RHA</td>
<td>22.4</td>
</tr>
<tr>
<td>10RHA</td>
<td>31.2</td>
</tr>
<tr>
<td>15RHA</td>
<td>28</td>
</tr>
<tr>
<td>20RHA</td>
<td>20.8</td>
</tr>
<tr>
<td>2.5L100+10RHA</td>
<td>31.2</td>
</tr>
<tr>
<td>2.5L300+10RHA</td>
<td>39.2</td>
</tr>
<tr>
<td>2.5L100+15RHA</td>
<td>28</td>
</tr>
<tr>
<td>2.5L300+15RHA</td>
<td>35.2</td>
</tr>
<tr>
<td>5L100+10RHA</td>
<td>36</td>
</tr>
<tr>
<td>5L300+10RHA</td>
<td>41.6</td>
</tr>
</tbody>
</table>

Figure 2: Normalized compressive strength results relative to control mix.

3.2 RCMT

The results of rapid chloride migration test at the ages of 28 and 90 days are presented in
Table 6. Normalized migration coefficients relative to the control mix are also displayed in Figure 3. Comparison of the data for different nano mixes shows lower chloride permeability for nanosilicas with lower surface area. In addition, the increase in replacement level of cement has led to higher chloride resistivity. Nanosilica particles possess high pozzolanic activity and as a result, the hydration of cement is accelerated and larger volumes of reaction products are formed. Also SiO₂ nanoparticles could recover the particle packing density of the blended cement, directing to a reduced volume of larger pores in the cement paste.

The ability of RHA mixtures to reduce chloride intrusion into mortar is clearly evident from the obtained results, where mortars containing RHA outperform control mixes. As the replacement level of RHA increases from 5 to 20% by weight, lower migration coefficients are observed. It is found, in general, that the rate of strength development is slower for mixtures containing RHA at early ages whereas the rate of increase in chloride penetration resistance is significant between 28 and 90 days.

From the data in Table 6, it is evident that there are significant reductions in chloride ion permeability in binary blended mortars compared to RHA mixes. This enhancement in durability is essentially due to the improvement or degeneration of the pore structure of mortars. By the addition of nanosilica and RHA, the pore structure of concretes is refined and therefore lower migration coefficients are obtained. It should be noted that 5L100+10RHA mix demonstrated higher resistivity among other mixtures, where RCMT coefficients were decreased by 586.1% compared to control mortar at the age of 90 days.

Table 6: Migration coefficient value of mortar mixes.

<table>
<thead>
<tr>
<th>Mortar Mix</th>
<th>Migration Coefficient (×10⁻¹² m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>28 days</td>
</tr>
<tr>
<td>OPC</td>
<td>35.4</td>
</tr>
<tr>
<td>2.5L100</td>
<td>10.5</td>
</tr>
<tr>
<td>2.5L300</td>
<td>19.6</td>
</tr>
<tr>
<td>5L100</td>
<td>7.8</td>
</tr>
<tr>
<td>5L300</td>
<td>13.7</td>
</tr>
<tr>
<td>5RHA</td>
<td>32.5</td>
</tr>
<tr>
<td>10RHA</td>
<td>27.7</td>
</tr>
<tr>
<td>15RHA</td>
<td>19.1</td>
</tr>
<tr>
<td>20RHA</td>
<td>15.7</td>
</tr>
<tr>
<td>2.5L100+10RHA</td>
<td>13.1</td>
</tr>
<tr>
<td>2.5L300+10RHA</td>
<td>16.8</td>
</tr>
<tr>
<td>2.5L100+15RHA</td>
<td>11.5</td>
</tr>
<tr>
<td>2.5L300+15RHA</td>
<td>16.2</td>
</tr>
<tr>
<td>5L100+10RHA</td>
<td>5.6</td>
</tr>
<tr>
<td>5L300+10RHA</td>
<td>15.7</td>
</tr>
</tbody>
</table>
3.3 Electrical resistivity

The results of electrical resistivity at 7, 28 and 90 days are presented in Table 7. Figure 4 shows the relative enhancement in resistivity in comparison to the control mortar.

The results indicate higher electrical resistance in mortars containing nanosilica and RHA at the ages of 28 and 90 days compared to other mixes. In fact, the replacement of cement with nanosilica is more efficient at 28 days, where the electrical resistivity is improved by 328.6% for 5L100+10RHA specimen compared to the control mix. Nanosilica accelerates the formation of C-S-H gels at earlier ages due to its ultra fine particle size and high specific area. Nevertheless, results show a decrease in electrical resistance development with the progress of hydration.

On the other hand, RHA mixes decrease resistivity at the age of 7 days due to their slow reactivity and low pozzolanic activity at earlier ages and demonstrate higher values of electrical resistance at the ages of 28 and 90 days. The resistivity curves also show incremental trends by increasing the replacement level of RHA.

Similar to the RCMT results, the addition of nanosilica with lower specific area has resulted in better performance and the incorporation of RHA together with nanosilica in binary blended mixes has improved resistivity at the ages of 28 and 90 days compared to single mixes containing nanosilica.
### Table 7: Electrical resistance of mortar mixes.

<table>
<thead>
<tr>
<th>Mortar Mix</th>
<th>Resistivity (kΩ.cm)</th>
<th>7 days</th>
<th>28 days</th>
<th>90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>4.8</td>
<td>5.6</td>
<td>10.2</td>
<td></td>
</tr>
<tr>
<td>2.5L100</td>
<td>5.1</td>
<td>9.3</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>2.5L300</td>
<td>4.8</td>
<td>6.4</td>
<td>10.2</td>
<td></td>
</tr>
<tr>
<td>5L100</td>
<td>9.2</td>
<td>20</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>5L300</td>
<td>8.3</td>
<td>12</td>
<td>18.3</td>
<td></td>
</tr>
<tr>
<td>5RHA</td>
<td>2.5</td>
<td>6.4</td>
<td>12.1</td>
<td></td>
</tr>
<tr>
<td>10RHA</td>
<td>2.8</td>
<td>8.5</td>
<td>16.3</td>
<td></td>
</tr>
<tr>
<td>15RHA</td>
<td>3</td>
<td>10</td>
<td>19.5</td>
<td></td>
</tr>
<tr>
<td>20RHA</td>
<td>3.3</td>
<td>12</td>
<td>24.7</td>
<td></td>
</tr>
<tr>
<td>2.5L100+10RHA</td>
<td>5.8</td>
<td>15</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>2.5L300+10RHA</td>
<td>5.6</td>
<td>11</td>
<td>22</td>
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</tr>
<tr>
<td>2.5L100+15RHA</td>
<td>8</td>
<td>18</td>
<td>29</td>
<td></td>
</tr>
<tr>
<td>2.5L300+15RHA</td>
<td>7.6</td>
<td>13</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>5L100+10RHA</td>
<td>9.2</td>
<td>24</td>
<td>37</td>
<td></td>
</tr>
<tr>
<td>5L300+10RHA</td>
<td>8.9</td>
<td>15</td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 4:** Normalized electrical resistance relative to control mix.

### 3.4 Capillary absorption

The results for capillary absorption testing of mortars at different time intervals are shown in Table 8. As seen from the table, the addition of nanosilica has slightly decreased water absorption into specimens and the increase in replacement levels has further improved capillary absorption. As mentioned previously, fine nano particles display better performance at earlier ages due to their higher surface area and therefore the lowest water absorption rate
was reported for 5L300 mix. It is interesting to note that the effect of RHA on sorptivity at the age of 7 days was negligible and the addition of RHA to nanosilica mixtures had little effect on the improvement of capillary absorption.

Pozzolan additives and nano particles in particular, create more nucleation sites which could produce more C-S-H gel at earlier ages. Moreover, non reactive particles with Ca(OH)$_2$ crystals, can act as fillers to further compact the microstructure and reduce the volume of larger pores in the cement paste.

Increasing the age of capillary absorption test from 7 to 28 days has slightly improved the performance of RHA mixtures and the increase of RHA content from 5 to 20% has led to higher loss in capillary absorption. For instance, 20RHA mix shows 17.1% reduction in the value of water absorption after 72 hr compared to control specimen. Also, binary samples show better performance compared to other specimens at earlier ages, whereas the highest drop of capillary absorption is of 5L100+10RHA mix and equal to 39.2%. Comparing this result to the 33.1% reduction in sorptivity for 5L300+10RHA mix at the age of 7 days demonstrates the efficiency of larger nanosilica particles at higher ages.

### Table 8: Capillary absorption values of mortar mixes.

<table>
<thead>
<tr>
<th>Mortar Mix</th>
<th>Water absorption (gr)</th>
<th>7 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3 hr</td>
<td>6 hr</td>
</tr>
<tr>
<td>OPC</td>
<td></td>
<td>10.5</td>
<td>12.4</td>
</tr>
<tr>
<td>2.5L100</td>
<td></td>
<td>9.7</td>
<td>11.6</td>
</tr>
<tr>
<td>2.5L300</td>
<td></td>
<td>8.8</td>
<td>11</td>
</tr>
<tr>
<td>5L100</td>
<td></td>
<td>7.5</td>
<td>10.2</td>
</tr>
<tr>
<td>5L300</td>
<td></td>
<td>7</td>
<td>10</td>
</tr>
<tr>
<td>5RHA</td>
<td></td>
<td>10.7</td>
<td>13</td>
</tr>
<tr>
<td>10RHA</td>
<td></td>
<td>10.6</td>
<td>12.8</td>
</tr>
<tr>
<td>15RHA</td>
<td></td>
<td>10.7</td>
<td>12.6</td>
</tr>
<tr>
<td>20RHA</td>
<td></td>
<td>10.7</td>
<td>12.6</td>
</tr>
<tr>
<td>2.5L100+10RHA</td>
<td></td>
<td>9.5</td>
<td>11.5</td>
</tr>
<tr>
<td>2.5L300+10RHA</td>
<td></td>
<td>8.9</td>
<td>11.3</td>
</tr>
<tr>
<td>2.5L100+15RHA</td>
<td></td>
<td>9.9</td>
<td>11.8</td>
</tr>
<tr>
<td>2.5L300+15RHA</td>
<td></td>
<td>9.2</td>
<td>11.5</td>
</tr>
<tr>
<td>5L100+10RHA</td>
<td></td>
<td>7.1</td>
<td>10.2</td>
</tr>
<tr>
<td>5L300+10RHA</td>
<td></td>
<td>6.7</td>
<td>9.8</td>
</tr>
</tbody>
</table>

### 4 CONCLUSIONS

The influence of nanosilica hydrosols with different surface areas of 100 and 300 m$^2$/g and rice husk ash was investigated on the rate of strength development and chloride resistance of mortar mixes. The incorporation of nanosilica, especially at higher replacement levels, resulted in enhanced compressive strength compared to the control mixes. This was due to the high pozzolonic reactivity of nano-particles with calcium hydroxide crystals which leads to the production of secondary gel. With the progress of hydration, nanosilicas with lower specific area outperform finer particles, whereas 5100 mix demonstrated the highest increase in strength among nanosilica blended mortars. Single mixtures containing RHA witnessed
low strength development at earlier ages and with the progress of pozzolanic reactions, 10RHA and 15RHA mixes slightly improved strength performance at the age of 90 days. Similarly, binary mixes showed lower strength results compared to nanosilica mortars at ages of 3 and 7 days, while, 5L100+10RHA mix significantly improved performance at ages of 28 and 90 days. Analogues results were obtained for capillary absorption test.

With regards to RCMT, decline in early age chloride resistance of RHA mortars was not encountered and the non-steady-state migration coefficient was significantly decreased with the addition of nanosilica. Among mortars, 5L100+10RHA mix displayed the highest chloride resistivity. The results of electrical resistance showed similar trends as to RCMT, with only a slight difference in the performance of RHA mortars at the age of 7 days. However, in general, replacement levels of nanosilica had a higher influence on electrical resistance, whereas nanosilicas with lower specific areas displayed lower values of migration coefficient.

REFERENCES


MECHANICAL PROPERTIES OF CONCRETE REINFORCED WITH RECYCLED STEEL FIBERS: A CASE STUDY

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Key words: Recycled steel fibers, FRC, toughness, flexural behavior

Abstract. According to recent data [1], every year approximately 2.5 million tons of used tires (about 250 million units) are either recycled or recovered in Europe. Generally, end of life tire (ELT) enters a waste management system based on product/material recycling and/or energy recovery. The application of ELTs in civil engineering (such as foundation for roads and railways, draining material, erosion barriers, etc.) in 2013 was 11% of ELTs sent to material recovery. However, the most important material recovery is represented by recycling of rubber granulates and powder (82%).

In this context, the experimental work herein discussed is part of a wider scientific investigation on the application of steel fibers recycled from ELTs as discontinuous reinforcement in concrete matrix. As a matter of fact, the main role of the steel fibers within the matrix is to control the crack opening and propagation, increasing the overall ductility of concrete elements [2, 3]. As soon as the cracks widen out, the toughness increases and the cracking process is modified from brittle to ductile, allowing the material to redistribute the stresses. This cracking control also improves the durability of the material.

On the basis of previous research studies [4, 5] and also considering the available literature [6, 7] the main issue of the proposed research is to extend the characterization of the recycled steel fibers and to develop the study of their structural behavior when applied for a new eco-friendly concrete. Using these recycled steel fibers in concrete production could ensure a surplus value in terms of environmental and ecological benefits and consequently a significant reduction in landfiling of ELTs. Fresh and hardened properties of concrete reinforced with recycled steel fibers are discussed as well as the post-cracking behavior properties. The obtained results evidenced the good behavior of the proposed material when compared with concrete reinforced with industrial steel fibers. A concrete pavement was finally realized applying the proposed reinforced mixture with recycled steel fibers. As discussed in the following, this application was successful and it fulfilled the specific requested properties.

1 INTRODUCTION

Sustainability in concrete technologies is nowadays a long debated topic by the scientific...
community because concrete is the worldwide most used construction material with a 
estimated production of over 25 billion tons, increasing every year. It is clear that an 
appropriate use of this material, as well as the introduction of recycled raw materials in its 
production could ensure significant contribution to a sustainable development. There are 
several reasons for its great diffusion: the constitutive materials (water, cement and 
aggregates) are available everywhere and they are relatively inexpensive, its production is 
simple and its use covers large variety of structural applications. The ordinary, unreinforced 
concrete is characterized by low tensile strength and low tensile ultimate strain, hence it is 
typically a brittle material. In the last decades there has been a growing interest in concrete 
reinforced with low volume fractions of short fibers (FRC). It is well recognized that the 
introduction of this discontinuous fiber reinforcement improves mechanical characteristics of 
the material, supplying a good resistance to crack opening and propagation. Thus, the typical 
brittleness of the plain concrete can be overcome.

In the present experimental work a concrete mix reinforced with steel fibers recycled from 
ELTs was optimized and then applied for a prototype of concrete pavement. The use of 
concrete reinforced with steel fibers is particularly suitable in hyper-static structures, among 
which the industrial pavements on ballast are an example, as the fiber reinforcement enhance 
the concrete’s post-cracking residual strength increasing the load bearing capacity of the 
structure. In the available literature it has been well deepened how the reinforcement of the 
ordinary concrete matrix with discontinuous fibers modifies its typical brittleness into a 
ductile behavior, improving its toughness and its post-cracking residual strength [8, 9, 10].

This experimental work is a part of a research project, based on the development and the 
characterization of fiber reinforced concrete with RSF (Fig.1). The main aim is the design of 
new eco-friendly concrete followed by environmental and ecological benefits as well as a 
significant reduction in landfilling of the waste tires.

![Figure 1: Recycled steel fibers from ELTs](image)

After the characterization of the materials, the definition of the mechanical properties of 
the concrete for the selected application, the optimization of the adequate mix-design and the 
mechanical characterization of compressive strength and post-cracking behavior were done.

The realized mixtures showed good properties both in the fresh and hardened state and also 
the post-cracking behavior of the reinforced mixtures confirmed the effectiveness of the fiber 
presence. The final part of the project was the realization of the prototype of pavement in real 
scale partially reinforced with steel fibers from ELTs, which was successfully realized and it 
is still regularly used.
2 EXPERIMENTAL OVERVIEW

The case herein discussed is focused on the construction of a concrete pavement into an industrial warehouse usually used, by the company partner of this project, as a parking for vehicles and trucks (Fig. 2). The technical service of the company provided the mixture usually produced for the realization of analogous pavements. On the basis of this mixture, several tests were done in a laboratory scale in order to optimize the mixture when recycled steel fibers were introduced. The main targets of this optimization procedure, requested by the company, were an adequate workability, a good mechanical strength after 28 days and the pumpability of the fresh mixture, which is defined as the ability of the mixture to be casted with a pumping system. In the end, the selected parameters for this case study were a workability class at least S3 (fluid concrete), according to [11], a compressive strength after 28 days, $R_{cm(28)}=30$ MPa.

The warehouse has a total surface of about 590 m$^2$. Taking into account the total volume of concrete needed and the maximum capacity of the mixer, the surface was divided into three zones. The former was made by the optimized ordinary mixture, the second zone was realized with the optimized mixture reinforced with the RSF and the latter was realized with the mixture originally provided by the company. This choice was done for comparison purpose.

![Figure 2: Company’s warehouse](image)

3 MATERIALS CHARACTERIZATION

As well-known, adding fibers into a concrete matrix generally reduce its rheological properties. This aspect becomes even more significant when RSF from ELTs are introduced into the matrix, as the high dimensional variability of these fibers is the main cause of their balling tendency within the mixture that consequently reduces the workability of the mix.

As a matter of fact, the main parameter to be controlled during the development of the most suitable mixture was the workability, as well as the pumpability, which were significantly influenced by the particular type of used fibers.

RSF are characterized by different geometrical dimensions and they also present many irregular wrinkles (Fig.1). Thus, a geometrical characterization analysis of the fibers was performed on a sample of 1,000 specimens, randomly extracted after the shredding process of the tires. The analysis is essential to evaluate the scatter of the lengths and diameters of the fibers, which affects the tendency of a fiber reinforced mixture to produce balling of fibers in...
the freshly mixed state. The fiber diameters varied between 0.11 mm and 2.8 mm and the class with the higher frequency was 0.20-0.25 mm (21.8%), as shown in Fig. 3. The average value of the diameter was 0.33 mm (C.O.V.=42.3%).

![Figure 3: Diameters’ frequency](image)

The length of the reinforcement is referred to the distance between the outer ends of the fibers. The class including the major number of fiber length was 5-15 mm (65.1%), as shown in Fig. 4. The average value of the fiber length was 12.6 mm (C.O.V.=50.0%). At the end of the analysis the equivalent aspect ratio was determined equal to 41 (C.O.V.=52.9%).

![Figure 4: Lengths’ frequency](image)

4 M.I.X-DESIGN

The main aspect monitored during the optimization of the mix was the tendency of the RSF to bundle in the fresh state [4, 5, 13]. For that reason, several variations were applied to the original mix-design provide by the company in order to match the requested properties.
Ordinary Portland cement, according to the requirements of the National Standard [12], was used for the mixtures. A high-range water-reducing admixture was added to improve the fresh concrete workability. Water, two different grain sizes of limestone aggregates and locally available sand were also used (Fig. 5). Concrete reinforced with industrial steel fibers (ISF) was also realized for comparison purposes with hooked end fibers (length/diameter=50).

Once obtained the optimal rheological and mechanical properties of the plain concrete, the reinforced concrete was optimized. In this case a further slight variation of the aggregate distribution was needed in order to achieve good pumpability. After the definition of the optimal mixtures, the mechanical behavior of the proposed concrete was evaluated respect to both plain concrete and industrial fiber reinforced concrete.

The mixing stages were carried out by introducing the aggregates and then, gradually, water and cement. Then the admixture was introduced and at the end, when the mixture reached a sufficient workability, the fibers were added. This procedure allows achieving a homogeneous distribution of the fibers into the concrete due to the friction produced during the mixing process.

In the Table 1 the details of each final concrete mix, which was named with its own code (Plain Concrete: PC; Industrial Steel Fiber Concrete: ISFC; Recycled Steel Fiber Concrete: RSFC), are shown.

<table>
<thead>
<tr>
<th></th>
<th>PC</th>
<th>ISFC</th>
<th>RSFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (0-4)</td>
<td>[kg/m$^3$]</td>
<td>1160</td>
<td>1150</td>
</tr>
<tr>
<td>Aggregate I (4-8)</td>
<td>[kg/m$^3$]</td>
<td>219</td>
<td>200</td>
</tr>
<tr>
<td>Aggregate II (8-16)</td>
<td>[kg/m$^3$]</td>
<td>457</td>
<td>486</td>
</tr>
<tr>
<td>Fly ash</td>
<td>[kg/m$^3$]</td>
<td>60</td>
<td>65</td>
</tr>
<tr>
<td>Water reducing admixture</td>
<td>[kg/m$^3$]</td>
<td>3.0</td>
<td>3.7</td>
</tr>
<tr>
<td>Fibers</td>
<td>[%v]</td>
<td>---</td>
<td>20.3</td>
</tr>
<tr>
<td></td>
<td>[kg/m$^3$]</td>
<td>---</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Figure 5: Granulometric distribution
The cement content (CEM II A-LL 42.5R), the water content and the w/c ratio were constant for each mix, equal to 300 kg/m³, 180 l/m³ and 0.58 respectively. It was also used a water reducing admixture (Creactive No.W.A. 135), which allowed higher fluidity avoiding segregation. For a better comparison of the reinforced concrete mixtures, the content of steel fibers was constant in terms of %v (volume of concrete), taking into account the different densities of the two utilized types of fibers: 6.9 kg/dm³ for RSF and 7.82 kg/dm³ for ISF.

5 EXPERIMENTAL RESULTS

The main results of the fresh and hardened characterization of the realized mixtures are shown in the following Table 2. The optimized mixtures comply with the requested properties both in the fresh and in the hardened state. A minimum of two slump tests were performed on the fresh concrete for each mixture and the class of consistency was identified according to [11]. The percentage of entrapped air in the concrete sample was measured according to [14] as well as the fresh concrete density. The compressive strength was measured according to [15]. It can be observed that compressive strength for reinforced concrete specimens is slightly higher than that found for the control mix, while closer values were found for both recycled and industrial steel fibers. However, the obtained results confirm that adding fibers into the concrete matrix has minor effects on the compressive strength of the reinforced concrete, as discussed in the available literature [9, 16, 17, 18].

<table>
<thead>
<tr>
<th></th>
<th>PC</th>
<th>ISFC</th>
<th>RSFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump [mm]</td>
<td>190</td>
<td>170</td>
<td>200</td>
</tr>
<tr>
<td>Fluidity grade</td>
<td>S4</td>
<td>S4</td>
<td>S4</td>
</tr>
<tr>
<td>Entrapped air [%]</td>
<td>3.2</td>
<td>3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>Fresh density [kg/m³]</td>
<td>2387</td>
<td>2399</td>
<td>2416</td>
</tr>
<tr>
<td>$R_{c(28)}$ [MPa]</td>
<td>43.6 ± 0.2</td>
<td>48.8 ± 0.8</td>
<td>47.8 ± 0.3</td>
</tr>
<tr>
<td>Hardened density [kg/m³]</td>
<td>2408</td>
<td>2400</td>
<td>2393</td>
</tr>
</tbody>
</table>

In addition to this general characterization of the mixtures, the post-cracking behavior was evaluated by means of flexural tests on notched beams, according to [19]. It is well recognized that the main role of the steel fibers within the concrete matrix is to control the crack opening and propagation, increasing the overall ductility of concrete elements [2, 3]. The addition of steel fibers contributes to modify the brittle concrete into a ductile material, able to withstand large deformations before losing its bearing capacity. Toughness is the mechanical parameter that most clearly distinguishes fiber reinforced concrete from ordinary unreinforced concrete. Basically flexural toughness can be defined as the area under the complete load-deflection curve. The experimental test carried out within this project consists in a four point flexural test on notched beams (150 mm x 150 mm x 600 mm) under crack mouth opening displacement (CMOD) control at a constant rate of 50 μm/min. Three full-bridge resistive clip-on gauges were used for experimental measurements: one clip gauge was applied at the notch mouth and the others on the two opposite faces of the beam at the height of notch tip, (Fig. 6). The former gauge monitored the CMOD and the others recorded the CTOD (Crack Tip Opening Displacement).
In the present study nine beams (three for each mixture) were realized and tested. The meaningful curves Load vs. CTOD for plain concrete (PC), industrial fiber reinforced concrete (ISFC) and recycled fiber reinforced concrete (RSFC) are shown in Fig. 7.

Figure 6: Flexural test set-up

Figure 7: Flexural behavior: a) plain concrete; b) industrial steel fiber concrete; c) recycled steel fiber concrete
The obtained results confirm the positive effect of the fibers on the post-cracking behavior, even when recycled fibers from ELTs are used. The typical brittleness of the unreinforced concrete (Fig. 7a) is modified into a more ductile behavior (Fig. 7b and Fig. 7c for ISF and RSF respectively): plain concrete has no residual strength, the specimens with industrial steel fibers are characterized by an average residual load of 3 kN, while the average residual load with recycled fibers is equal to 4 kN.

6 PAVEMENT PROTOTYPE

The final part of the project was focused on the application of the optimized mixtures for the realization of pavements in real scale in the company’s production plant. The main effort to be managed during this last phase was the proper mixing of the components according to the optimal mix (Tab.1). As well-known, the laboratory conditions are very far from the real conditions when operating in a production plant. The first effort was related to the control of the aggregates’ humidity, which is very important in order to avoid more water than needed into the matrix. The second aspect was the different environmental conditions (temperature and relative humidity), which vary day by day. Finally, the last effort was to control the pumpability of the reinforced mixture when the pump system was used. These three aspects were considered and monitored, modifying mixtures in order to preserve the requested properties. In the following Fig. 8 the main steps of the pavement’s realization are shown.

The day after casting, the pavement was completed with a quartz finishing treatment of the
external surface. In the following Fig. 9, the final aspect of the pavement is shown.

![Figure 9: Pavement after casting: a) no cracks after 24 hours; b) presence of steel fibers on the surface after 24 hours; c) crack at the continuity of casting after 28 days; d) presence of steel fibers on the surface after 28 days](image)

The pavement is currently used by the company as parking for trucks, cars, etc. (Fig. 10), thus it will be possible to monitor its behavior under real loading conditions. No defects have been detected yet, although the presence of fibers on the external surface could represent an issue to be solved. The monitoring will continue for the next years in order to collect complete results concerning the discussed application. The most important data that can be collected concerning the different behavior between the zones realized with ordinary concrete and the zones realized with concrete reinforced with steel fibers from ELTs.

![Figure 10: Currently use of the prototype pavement](image)
7 CONCLUSIONS

On the basis of the experimental results shown in the present paper, the following conclusions can be summarized:

- Although only a slight increase in compressive strength was noticed, the principal effect of the fibres is to bridge the cracks developing into the matrix, increasing the ductility of the concrete element;
- Both types of reinforced concrete (with RSF and ISF) exhibit better post-cracking properties than the unreinforced concrete and their behaviors are comparable. As expected, the softening branch of the reinforced concrete curve is significantly modified, both in terms of ultimate strain, residual stress and stress gradient;
- RSFC evidenced a high scatter in the post-cracking stage under flexural tests: probably, the different distribution of the fibres at the broken section governs the behavior of the specimens;
- It has been proved that the proposed concrete could be used for its application in concrete pavements, as long as the necessary precautions are taken in terms of adequate mix-design, workability and pumpability properties.
- The results confirmed that recycled steel fibers from ELTs could be an alternative as reinforcement of concrete matrix. However, there is still the need of further research work to widen the experimental database and to better characterize the proposed material.

ACKNOWLEDGEMENTS

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MODIFIED EXPANDED CLAY LIGHTWEIGHT CONCRETES FOR THIN-WALLED FLOATING STRUCTURES

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Key words: Floating structures, Lightweight concretes, Modifiers, Durability

Abstract. Shipbuilding reinforced concrete is developing in many countries around the world. Floating docks, marinas, hotels and houses are built of concrete. More often, heavy concrete shipbuilding is used for floating concrete structures. Using expanded clay lightweight concretes reduces the weight of the construction. At reducing the weight load lifting of the floating structure is increased. The use of lightweight concrete improves people’s occupation and equipment on a reinforced concrete vessel.

Properties of shipbuilding expanded clay lightweight concretes were studied. Methods of experiment planning were used. The following factors of composition were varied: the amount of sulphate-resistant Portland cement; the concentration of the hydrophobic additives in the processing of gravel, the amount of waterproofing additives; the amount of superplasticizer; the amount of fiber. The experiment was conducted on a 5-factors optimal plan. All tested lightweight aggregate mixtures had the same high mobility.

It is found that the hydrophobic treatment gravel increased slightly compressive strength of expanded clay lightweight concrete. Maximum strength modified concrete saturated with water is more than 40 MPa. Fiber increases the tensile strength of the concrete. For the construction of floating structures the most important indicator of quality is a watertight concrete. Concrete with the Portland cement content of 500 kg/m\(^3\) when processing hydrophobization porous gravel has water-resistance not lower W6. When the amount of cement is 600 kg/m\(^3\), concrete with additives and hydrophobization gravel have water-resistance not below then W8. Such concrete can be used for outdoor structures.

We investigated the frost resistance of concrete. We found that the shipbuilding expanded clay lightweight concrete with cement content of 500 kg/m\(^3\) or more have frost-resistance not lower than F450. Due to hydrophobic lightweight aggregate the frost-resistance is increased by 50-100 cycles, due to the introduction of fiber - 50 cycles. Maximum frost-resistance of modified expanded clay lightweight concrete is F600.

The average density of shipbuilding expanded clay lightweight concrete in water saturated state is 1745..1855 kg/m\(^3\). Such density corresponds to the requirements of industry standards. When using hydrophobised gravel concrete density is reduced to 20-30 kg/m\(^3\), humidity of the concrete is also reduced. Expanded clay lightweight concrete on hydrophobized gravel has a
lower thermal conductivity.

Optimal compositions of shipbuilding expanded clay lightweight concrete were chosen. The technology of application of the modified shipbuilding lightweight concrete and heavy concrete was developed.

INTRODUCTION

Shipbuilding concrete is often used for parking floating structures. Nowadays floating docks, wharves, houses and hotels, oil platforms are built of concrete [1,2]. Concrete floating structures are durable, because they can be used up to 80-100 years without thorough repairs. This term is almost two times more than the life of the steel structures.

Concrete floating structures operated under severe conditions. These concretes are subjected to wetting and drying, changes in temperature from -30 to +50°C, the pressure of water and ice, effects of salt, algae and organisms. That is, they have the effect of all three types of corrosion by the V. Moskvin [3] and other effects.

Shipbuilding heavy concrete with granite rubble is more often used. However, during the construction of the concrete ships expanded clay lightweight concrete is used. Using expanded clay lightweight concrete increases the load capacity of reinforced concrete ships. Also, lightweight aggregate structure significantly improves the comfort of the people and work conditions of technological equipment in the premises of the ship [4].

Shipbuilding expanded clay lightweight concrete proved its effectiveness over many years of use. First lightweight aggregate ship "S.S. Selma" was built in 1919. The thickness of the ship's hull was about 4 to 5 inches, the thickness of the protective layer of about 16 mm [5,6]. The average strength of concrete at 28 days was 393 kg/cm². After the accident in 1922, the tanker was partially flooded, but his hull remained satisfactory. Surveys have shown that for 90 years the strength of concrete in some designs doubled.

During World War II, 104 concrete ships were built [4]. Specialists from the United States were surveyed four aggregate concrete hulls of ships at the age from 55 to 80 years. They showed a high durability of the material in an aggressive marine environment [7]. Besides durability experts indicate good maintainability of structures made of expanded clay lightweight concrete and safety of dry cargo.

Expanded clay lightweight concrete floating structures were also built in Ukraine. Expanded clay floating docks were built on concrete shipbuilding plant "Pallada" [8]. Surveys were conducted of the given floating structures. Surveys have shown the durability of lightweight aggregate concrete structures under various conditions. [9] Most of these docks are still in operation [10].

Advances in technology permit to increase the durability and mechanical properties of expanded shipbuilding clay concrete. Therefore, studies of the properties of shipbuilding expanded clay lightweight concrete modifiers and a fiber is important.

MATERIALS AND METHODS

We used following materials: sulfate-resistant Portland cement M400, expanded clay gravel fractions 5-10 mm and 2.5-5 mm, quartz sand with fineness modulus 2.7, superplasticizer S-3, bridging agent Penetron Admix, silicon organic liquid 136-157M, polypropylene fiber Baucon (fiber diameter of 18.7 micron, fiber length - 12 mm).
Hydrophobization gravel was carried out by dipping into the emulsion silicone fluid. Studies of the shipbuilding expanded clay concrete properties were conducted using experimental design techniques [11]. The following factors had varied compositions:

- $X_1$ – sulfate-resistant Portland cement, from 400 to 600 kg/m$^3$;
- $X_2$ – concentration of the silicone fluid in the emulsion during processing of gravel, from 0 to 1.6%;
- $X_3$ – bridging the addition of Penetron Admix, 0 to 2% of cement weight;
- $X_4$ – additive superplasticizer S-3, from 0.5 to 0.9% of cement weight;
- $X_5$ – polypropylene fibers Baucon, from 0 to 1.2 kg/m$^3$.

The experiment was conducted on a 5-factors 27 point optimal plan.

RESULTS

All expanded clay lightweight concrete mixes have mobility equal 2 ± 0.5 cm. This amount of water in the mixture was adjusted, that is dependent on the composition of the concrete. Analysis of changes in the amount of water in the mixtures revealed that increasing the amount of Portland cement and additive superplasticizer S-3 reduces the W/C. Introduction of additives Penetron Admix does not affect the W/C mixture, and the use of fiber significantly increases the W/C. Hydrophobization gravel reduces water/cement aggregate concrete mixes by 10-20% due to the reduction of water absorption of lightweight aggregate.

Strength of shipbuilding expanded clay concrete determined under normal conditions and in water-saturated state [12]. Influence of factors on the strength of expanded clay lightweight concrete composition in water-saturated state describes the experimental statistical model:

$$f_{ck,\text{cube}}(\text{MPa}) = 37.05 + 5.03x_1 \pm 0x_1^2 - 0.79x_1x_2 + 0.29x_1x_3 \pm 0x_1x_4 - 0.23x_1x_5$$
$$- 3.13x_2 - 3.01x_2^2 + 0.31x_2x_3 - 0.36x_2x_4 \pm 0x_2x_5$$
$$+ 0.22x_3 \pm 0x_3^2 - 0.16x_3x_4 - 0.43x_3x_5$$
$$+ 0.78x_4 - 2.51x_4^2 + 0.40x_4x_5$$
$$- 0.17x_5 - 0.36x_5^5$$

The analysis of this model shows that the introduction of Penetron Admix and fiber has little effect on the compressive strength of expanded clay lightweight concrete shipbuilding. Effect of amount of Portland cement, superplasticizer C-3 and the concentration of the silicone fluid in the processing gravel shown in Fig.1.a

The diagram shows that the strongest there is concrete, which was introduced in 0.7-0.8% superplasticizer S-3. Hydrophobization gravel at a concentration of 0.7-0.8% silicone fluid increases the strength expanded clay lightweight concrete. Increasing the concentration of silicone fluids is more negatively affected by 0.8% since filler deteriorates adhesion to the matrix.

In the compressive strength shipbuilding expanded clay at equilibrium moisture content (Fig.1.b) varying the composition of the factors in the studies has a similar effect. All expanded clay concrete has high water resistance - the coefficient of softening the material above 0.85. When an amount of Portland cement 500 kg /m$^3$ $K_W$ is not less than 0.87, and in the case of hydrophobic treatment is not less than 0.88.
For structures of floating installations one of the main indicators of quality is the water-resistance of concrete. In experimental statistical model diagram type "cubes on the square" is built. This diagram shows the influence of factors on the composition of the water-resistance is shown in Fig.2.

The diagram shows that the amount of Portland cement is less than 500 kg/m$^3$ water-resistance expanded clay lightweight concrete is from W2 to W6. Such compositions may be used only in the pantoon baffles and interior of floating structures premises.

Compositions with amounts of Portland cement 500 kg/m$^3$ at gravel processing hydrophobizator a concentration of about 0.8% are water-resistance below W6. When the amount of cement is 600 kg/m$^3$, water-resistance of concrete is not less than W8. Compositions with the amount of superplasticizer S-3 0.7-0.8% have the maximum water-resistance. The effectiveness of the introduction of additives Penetron Admix is approximately equal for all compositions. Introduction 2% Penetron Admix improves the water-resistance on approximately 2 atmospheres.

Frost-resistance is an important quality indicator the longevity and sustainability of concrete floating structures. Frost-resistance affects the durability of concrete in the climatic conditions of Ukraine and other countries with a temperate climate. Research has shown that shipbuilding expanded clay concrete using at least 500 kg/m$^3$ of cement have frost-resistance not less than F450. Due to frost hydrophobic expanded clay lightweight concrete increased by 50..100 cycles due to fiber by another 50 cycles. Frost modified expanded clay lightweight concrete with a fiber reaches F600.

On account of lightweight concrete use a reduction in the weight of concrete occurs. This increases load capacity of vessels, including floating docks. The average density of expanded clay lightweight concrete after water saturation is from 1745 to 1855 kg/m$^3$. That density of
concrete complies with the industry standard. Increasing the amount of Portland cement and additives superplasticizer S-3 increases the density of expanded clay. By treating hydrophobic treatment gravel average density "wet" materials decreased by 30..40 kg/m³. This is due to the repulsion of water from the gravel.

Figure 2: influence of factors on the composition of water-resistance of the shipbuilding expanded clay lightweight concrete

The thermal conductivity has an important indicator of the quality of shipbuilding expanded clay lightweight concrete. Lightweight concrete provides comfort and indoor temperature balance in the vessel. It is found that the amount of additive Penetron Admix and fiber does not affect the thermal conductivity of the material. Fig. 3 shows a diagram of the effect on the amount of Portland cement, additives superplasticizer S-3 and the concentration of the silicone fluid on the thermal conductivity of expanded clay lightweight concrete.
Andrey V. Mishutin, Sergii O. Kroviakov, Nikolay V. Mishutin and Vladimir L. Bogutsky

Figure 3: Effect of the amount of Portland cement, superplasticizer C-3 and the concentration of the silicone fluid in the thermal conductivity of the shipbuilding expanded clay lightweight concrete

The diagram shows that increasing the amount of cement increases the thermal conductivity of 0.02-0.03 W/m*K. A similar increase in thermal conductivity expanded clay lightweight concrete occurs when increasing the amount of superplasticizer S-3. The greatest decrease in thermal conductivity is due to the hydrophobic treatment of gravel. When using an emulsion with 0.8% silicone fluid thermal conductivity expanded clay lightweight concrete reduced by 0.09-0.10 W/m*K. Using an emulsion with a concentration of 1.6% reduces in thermal conductivity of 0.11-0.13 W/m*K. Given the impact of silicone fluid to all the physical and mechanical properties of expanded clay lightweight concrete can be recommended to limit its concentration to 0.6-0.8%.

Select the optimum compositions shipbuilding expanded clay lightweight concrete. Optimal technological methods of manufacture and use of expanded clay lightweight concrete were developed for thin floating hydraulic structures. «Regulations on the preparation of modified shipbuilding technologies for the manufacture of thin-walled expanded clay lightweight concrete floating structures and floating docks» were developed and approved.

CONCLUSIONS

- on the results of 5 factors experiment Shipbuilding expanded clay lightweight concrete with high strength (35..45 MPa), water-resistance (W6..W10) and frost-resistance (up to F600) is obtained;
- a modified expanded clay lightweight concrete provides increasing of structures of floating installations durability;
- a modified expanded clay lightweight concrete complies with Maritime Register and can be used for the construction of parking ships and other floating structures;
- by replacing heavy shipbuilding concrete to expanded clay lightweight concrete, increases load-carrying capacity of a ship, in particular a floating dock, and increase people's comfort and technological equipment;
- optimal compositions of shipbuilding expanded clay lightweight concrete were chosen; the technology of application of the modified shipbuilding lightweight concrete and heavy concrete was developed;
- "Regulations on the preparation of modified shipbuilding technologies for the manufacture of thin-walled expanded clay lightweight concrete floating structures and floating docks" were developed and approved.

REFERENCES
PERMEABILITY OF HYBRID CONCRETE FOR SUSTAINABLE BRIDGE DECK PAVEMENT

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Key words: permeability, hybrid concrete, bridge deck, pavements

Abstract. The purpose of this paper was to investigate the effect of latex and silica fume in hybrid concrete containing organic latex and inorganic silica fume. To obtain this goal, a series of rapid chloride permeability tests were performed according to ASTM C 1202 with the main experimental variables of latex contents (3, 5 and 7%) and silica fume contents (6, 7 and 8%).

The compressive strengths of hybrid concrete were measured to be between 38 and 44 MPa at 28 days and 41 to 50 MPa at 56 days. The flexural strengths were measured to be between 5.9 and 7.5 MPa at 28 days. The permeability of hybrid concretes is very low at all mixtures. The passed charges were less than 600 Coulombs at 56 days for all mixtures, which is categorized as a “very low” permeable material according to the criteria indicated at ASTM C 1202[2]. This “very low” category is obtained at a latex-modified concrete or internally-sealed concrete, which means that the hybrid concrete is well internally sealed by latex film and filled by finer silica fume. The rapid chloride permeability was not affected by air content nor spacing factor, even though all mixtures showed ‘very low’ permeability. This might be explained by that the structure of the paste of latex-modified concrete is such that the micro pores and voids normally occurring in Portland cement system are partially filled with the latex film that forms during curing.

1 INTRODUCTION

Concrete used in bridge deck pavement, exposed to severe weather or to de-icing salt, must be virtually impermeable or water tight. Concrete permeability may be the most relevant concrete property affecting its durability, especially under exposure to aggressive environment. To determine the chloride permeability of concrete, many engineers have increasingly been using AASHTO T277, "Rapid Determination of the Chloride Permeability of Concrete," and specifying a coulomb rating of 1,500 or less to ensure low permeability
concrete [1]. This method is equivalent to ASTM C 1202 and the most accepted test to determine the relative permeability of concrete [2].

Many attempts have been adopted to decrease water permeability or to increase water tightness. One of the most popular ways are latex-modified concrete (LMC) and high performance concrete (HPC) containing silica fume. Since the inception of latex-modified Portland concrete for bridge deck overlay in 1957, thousands of bridges have been overlaid using LMC in the United States [3]. Since the introduction into Korea in 2000, hundreds of new bridges are being overlaid with LMC [4]. Silica fume is most commonly used supplementary cementitious material which consists of very fine vitreous particles with a surface area between 13,000 and 30,000 m²/kg and its particles are approximately 100 times smaller than the average cement particles. The high performance concrete containing very finer silica fume results in an impermeable concrete [5].

A common LMC contains 15% of latex solid against cement content, and this result in very expensive one [3, 4]. The main roles of latex in a concrete are filling voids and bridging effect by latex film [6]. A high performance concrete with silica fume contains 7 to 10 % of silica fume. The main roles of silica fume in a concrete are filling voids and Pozzolan effect. A high bridge concrete, which contains both of organic latex and inorganic silica fume, might both effects of bridging and Pozzolan [7].

The purpose of this paper was to investigate the effect of latex and silica fume on the rapid chloride permeability in a hybrid concrete containing organic latex and inorganic silica fume.

2 EXPERIMENTAL PROCEDURES

2.1 Materials

A Portland cement was used in this study. To obtain the best optimized mix proportions of hybrid concrete both of latex and silica fume were carefully selected from a series of preliminary tests. Latexes were made of 48 percent solid suspension of styrene butadiene rubber. The proportions of styrene and butadiene were 66 percent and 34 percent, respectively. Silica fume tested was a powder type, which was a very fine powder consisting mainly of spherical microspheres of mean diameter about 0.15 microns, with a very high specific surface area (20,000–30,000 m²/kg), with a content of 94% of SiO₂.

The maximum size of coarse aggregate was 13mm considering the thickness of bridge deck overlay. The coarse and fine aggregates were crushed limestone and natural sand, respectively. Both of fine and coarse aggregates satisfied the given Korean specifications. The specific gravities of coarse aggregate and fine aggregates were 2.61 and 2.63, respectively.
2.2 Sample Preparation

The main experimental variables were latex contents (3, 5 and 7%) and silica fume contents (6, 7 and 8%). Nine mix design, each with a fixed water-cement ratio of 39%, binder content of 390kg/m³, and 56% of S/a, were used. An additional defoamer of 1.0% and super plasticizer of 1.0% were mixed to lower air entrainment and to ensure workability of hybrid concrete, respectively. The concrete mixtures for hybrid concrete are shown in Table 1.

Three kinds of samples were cast for each mix: 100×200 mm cylinders for compressive strength test, 100×100×400mm beams for flexural strength test. After casting, all molded specimen were covered with water-saturated burlap, and left in the casting room at 22 Celsius and 50±2 % relative humidity. After 24 hours, the specimens were demolded and cured at a controlled curing room at the temperatures of 20 Celsius and 80% relative humidity until testing [3, 6]. A concrete cylinder specimen, having 100x200mm, was cut into cylinder having 50mm thickness with a water-cooled diamond saw, for rapid chloride permeability test. The compressive strength and flexural strength were measured according to KS F 2405 and KS F 2408, respectively.

Table 1. Mix proportion of hybrid concrete

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Unit Weight (kg/m³)</th>
<th>SP (%)</th>
<th>Antifoamer (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water</td>
<td>Cement</td>
<td>Silica Fume</td>
</tr>
<tr>
<td>S6L3</td>
<td>139.4</td>
<td>367</td>
<td>23.4</td>
</tr>
<tr>
<td>S6L5</td>
<td>131</td>
<td>367</td>
<td>23.4</td>
</tr>
<tr>
<td>S6L7</td>
<td>122.5</td>
<td>367</td>
<td>23.4</td>
</tr>
<tr>
<td>S7L3</td>
<td>139.4</td>
<td>363</td>
<td>27.3</td>
</tr>
<tr>
<td>S7L5</td>
<td>131</td>
<td>363</td>
<td>27.3</td>
</tr>
<tr>
<td>S7L7</td>
<td>122.5</td>
<td>363</td>
<td>27.3</td>
</tr>
<tr>
<td>S8L3</td>
<td>139.4</td>
<td>359</td>
<td>31.2</td>
</tr>
<tr>
<td>S8L5</td>
<td>131</td>
<td>359</td>
<td>31.2</td>
</tr>
<tr>
<td>S8L7</td>
<td>122.5</td>
<td>359</td>
<td>31.2</td>
</tr>
</tbody>
</table>

2.3 Permeability Test

The samples were epoxy-coated on the sides and vacuum-saturated before being tested. The total charges were recorded at a series of times. The rapid chloride permeability is a kind of measure of a concrete’s resistivity, and not a permeability. However, this resistivity is a very useful and simple index of permeability resistance of concrete. The rapid chloride permeability test was performed according to ASTM C 1202 (equivalent to AASHTO T277) to evaluate the relative permeability of latex modified concretes [1, 2].
3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Compressive Strength Development

The Figure 1 shows the compressive strength developments of hybrid concrete with latex contents and silica fume contents, at 7 days, 28 days and 56 days. The compressive strengths were measured to be between 38 and 44 MPa at 28 days and 41 to 50 MPa at 56 days. The compressive strengths increases as silica fume content increases at all levels and at all curing days. This is well expected by filling effect and Pozzolan effect of silica fume. However, there is no trend in latex content variation. This is, also, well anticipated in compressive strength development because of the flexibility of the latex component named by Butadiene. The latex films between the hydrated cement and aggregates, specially the flexibility of Butadiene, affected to compressive failure mode. However, the latex film might act as bridging between aggregate and cement paste.

3.2 Flexural Strength Development

Figure 2 shows the flexural strength developments at 28 days of hybrid concrete with latex contents and silica fume contents. The flexural strengths were measured to be between 5.9 and 7.5 MPa at 28 days. It increases as silica fume content and latex content increase at both cases. This is well expected by filling effect and Pozzolan effect of silica fume and bridging effect of latex films between aggregate and cement paste.
3.3 Rapid Chloride Permeability

Figure 3 and Figure 4 compares the air content of fresh and hardened hybrid concrete and air spacing factor, respectively. The air is higher at fresh concrete than that at the hardened. The differences of those two are between 0.5 to 3.1 %, but they show a similar trend. The spacing factors of the hybrid concretes were measured to be between 212 and 576μm.
Figure 5 shows the charge passed, as a test result of rapid chloride permeability according to ASTM C 1202, at 56 days of hybrid concrete with latex contents and silica fume contents. The permeability of hybrid concrete did not show a trend as latex content or silica fume content increased. The permeability of hybrid concretes is very low at all mixtures. The specified criteria[2], a coulomb rate of 1,000 or less, to ensure low permeability concrete could be met at all mixtures after 56 days curing. The passed charges were less than 600 Coulombs at 56 days for all mixtures. Hybrid concrete of latex and silica fume is categorized as a “very low” permeable material according to the criteria indicated at ASTM C 1202, as shown in Table 1[2]. This “very low” category is obtained at a latex-modified concrete or internally-sealed concrete, which means that the hybrid concrete is well internally sealed by latex film and filled by finer silica fume.

Figure 6 compares air content of hardened concrete, air spacing factor, and the charge passed, which shows no special relationship or trends. As a result, the rapid chloride permeability was not affected by air content nor spacing factor, even though all mixtures showed ‘very low’ permeability.

Air void content could be an important factor for concrete permeability. Air contents of hybrid concrete were measured to be between 2.2 to 5.0% at hardened concrete. The excessive air content was controlled by adding a deformer. The air quantity at hybrid concrete was similar at normal concrete. Even though one hybrid concrete had more air contents than the other, it shows less permeability as shown in the previous paragraph. The rapid chloride permeability was not affected by air content. This is due to followings: The structure of the paste of latex-modified concrete is such that the micro pores and voids normally occurring in Portland cement system are partially filled with the latex film that forms during curing; The
greater resistance to chloride intrusion is attributable to the finer silica fume and lower w/c and a plastic film the latex produce within the concrete, which inhibits the movement of chlorides. Another reason could be explained as shown in Figure 7. The inside air void surfaces in concrete are covered by latex film, and this makes water impermeable through air void in hybrid concrete, while water could be permeable through air void in OPC.

<table>
<thead>
<tr>
<th>Charge Passed (coulombs)</th>
<th>Chloride Ion Penetrability</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 4,000</td>
<td>High</td>
</tr>
<tr>
<td>2,000 - 4,000</td>
<td>Moderate</td>
</tr>
<tr>
<td>1,000 - 2,000</td>
<td>Low</td>
</tr>
<tr>
<td>100 - 1,000</td>
<td>Very Low</td>
</tr>
<tr>
<td>&lt; 100</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

Figure 5. Rapid chloride permeability of hybrid concrete
Figure 6. Comparison of air content, spacing factor and charge passed

Figure 7. Schematic Comparison of Water Permeability between OPC and LMC
4. CONCLUSIONS

- The compressive strengths of hybrid concrete were measured to be between 38 and 44 MPa at 28 days and 41 to 50 MPa at 56 days. The flexural strengths were measured to be between 5.9 and 7.5 MPa at 28 days.

- The permeability of hybrid concretes is very low at all mixtures. The passed charges were less than 600 Coulombs at 56 days for all mixtures, which is categorized as a "very low" permeable material according to the criteria indicated at ASTM C 1202[2]. This "very low" category is obtained at a latex-modified concrete or internally-sealed concrete, which means that the hybrid concrete is well internally sealed by latex film and filled by finer silica fume.

- The rapid chloride permeability was not affected by air content nor spacing factor, even though all mixtures showed ‘very low’ permeability. It could be explained as that the structure of the paste of latex-modified concrete is such that the micro pores and voids normally occurring in Portland cement system are partially filled with the latex film that forms during curing. The greater resistance to chloride intrusion is attributable to the finer silica fume and lower w/c and a plastic film the latex produce within the concrete, which inhibits the movement of chlorides.

REFERENCES

POROSITY AND RESISTIVITY MEASUREMENT OF ACCELERATED
CURED GEOPOLYMER AND CONVENTIONAL CONCRETE

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Key words: Geopolymer concrete, Porosity, Sorptivity, Resistivity

Abstract. This study is aimed to evaluate and compare the pore structure and resistivity of
thermal cured fly ash based geopolymer and Portland cement concretes. A wide range of
practical thermal curing regimes were applied to the test specimens and materials properties
were measured. Test results shows that although thermal curing increases the early-age
strength of the Portland cement system, it leads to a lower long-term strength and higher
porosity compared to the standard moist cured specimens. On the contrary, geopolymer
concrete can benefit from the thermal curing to develop very high early-age strength as well
as maintaining its long-term compressive strength. Also, the porosity of properly heat-cured
gopolymer concrete is much lower than its counterpart accelerated-cured Portland cement
concrete and very close to that of the water cured Portland cement concrete.

1 INTRODUCTION

Over the last decades, geopolymer concrete (GPC) has emerged as a novel engineering
material with the potential to become a substantial alternative to Portland cement (OPC)
concrete in construction and building industry. The term ‘geopolymer’ was first introduced by
Davidovits in 1979 to represent the inorganic polymers resulting from the reaction of
materials containing aluminosilicate with an alkali solution to produce an inorganic polymer
binder. The production of silico-aluminate based geopolymer requires source materials that
are rich in silica and alumina content, such as fly ash, slag, metakaolin, etc.

Currently, accelerated curing is widely being used where high early-age strength is
required; such as precast industry. Thermal curing is known as reaction accelerator which
significantly increases the early-age strength of Portland cement based systems compared to
the ambient curing [1]. However, thermal curing has shown [1] to have a detrimental effect on
the long term strength of OPC based systems. The same authors have previously studied the
possibility of using geopolymer concrete in order to overcome the negative impact of thermal
curing on the long-term strength of OPC concrete. A wide range of thermal curing and their
influence on short term and long term strength of GPC and OPC concrete has been
comprehensively studied [1]. The results showed that geopolymer concrete can benefit from
the thermal curing to develop very high early-age strength as well as maintaining its long-
term compressive strength. A proper heat-cured GPC (e.g. cured at 75°C for 18-24 hours) can
obtain the 28-day compressive strength of its counterpart standard-cured OPC concrete after only 24 hours and maintains this strength for the rest of its measured lifespan [1].

Although the above mentioned conclusion is very valuable, the pore structure of the resulting material is also to be investigated in order to have a better understanding of the long term performance of the thermal-cured GPCs compared to the Portland cement based systems.

The pore structure of a material is important in determining the durability characteristics. Sorptivity provides an indication of the pore structure and their connectivity (capillary network) which is a major factor influencing aggressive ions penetration into the concrete when exposed to the severe environments (e.g. marine environment). Lower sorptivity delivers a higher resistance to concrete towards water absorption. A high sorptivity coefficient indicates the existence of a highly connected porous structure or low tortuosity of the pore network. McCarter et al. [2] reported that minimizing the capillary sorptivity is essential in order to reduce the ingress of aggressive ions (e.g. chloride or sulfate) into concrete, which can cause serious damage.

The electrical resistivity of water saturated concrete is another measure of the concrete pore connectivity. Concrete resistivity is a volumetric property that indicates the ability to transport electrical charge through the material [3]. A concrete containing less interconnected pores with a more tortuous path will have a higher electrical resistivity as it would be more difficult for ions to pass through these kinds of pore network. Accordingly, the measurements of concrete electrical resistivity can provide a good indication of its permeability since both properties are mainly controlled by the tortuosity of the pore network. More compacted microstructures which tend to have more complex and refined pore networks result in a higher tortuosity and a lower permeability, whereas, less-dense microstructures resulting for instance from a higher water to binder ratio would have greater porosity, lower tortuosity and higher permeability [4]. The higher the porosity the lower the resistivity, due to higher volumetric fraction of pores [3]. Hence, the concrete electrical resistivity provides a reasonable indications of the pore connectivity and therefore of the concrete’s resistance to penetrations of aggressive substances (e.g. salt water having cl– or atmospheric CO2) [3]. Although the pore network connectivity and the moisture content in the pore network is the major factor determining the electrical resistivity of concrete, however, the existence and motion of ions such as Na+, K+ and OH– in the pore solution should also be considered. In concrete, electrolytic conduction is the main phenomenon of electricity transport since aggregates conductivity is very low [5]. Thus, the ions concentration in the pore network plays a significant role in concrete electrical resistivity measurements.

Moreover, the concrete resistivity can be used as a measure of corrosion rate of embedded steel reinforcement [6–12]. Generally, a low concrete resistivity is correlated to rapid chloride penetration and to a high corrosion rate [9]. Browne [6] previously studied the relationship between corrosion rate and electrical resistivity of concrete and found that the concrete resistivity must be kept above the range of 5 to 10 kΩ.cm to prevent corrosion of reinforcement steel. In many particular cases, it has been shown that concretes with high resistivity have a low chloride diffusion coefficients [10], [11], [13]. Gonzalez et al. [7] suggested that the corrosion rate was negligible when concrete resistivity was higher than 10 kΩ.cm.

The aim of this study is to experimentally investigate the influence of 12 different thermal curing on pore structure of a low-calcium fly ash based geopolymer concrete and compare the
results to that of OPC concrete. Volume of permeable voids, sorptivity and electrical resistivity were measured at the age of 28 days.

2 MATERIALS AND MIXING

The binder used in this study to make geopolymers is combination of three constituent aluminosilicate source materials, namely, 70% class F fly ash, 20% ultra-fine fly ash and 10% slag (by mass). The low-calcium type fly ash branded as Blue Circle Fly Ash by Boral was sourced from Eraring Power Station in New South Wales, Australia. The ultra-fine fly ash branded as Kaolite High-Performance Ash (HPA) by Cement Australia was obtained from Callide Power Station in Queensland, Australia. Finally, ground granulated blast furnace slag (GGBFS) was supplied by Australian Steel Mill Services, Port Kembla, New South Wales, Australia. The chemical compositions of the binders determined by X-ray fluorescence (XRF) analysis in accordance with AS 1141.6.2 test method are listed in Table 1. The amorphous content of fly ash, Kaolite HPA and GGBFS was found to be 85.2%, 81.2% and 95.8%, respectively.

<table>
<thead>
<tr>
<th>Oxide</th>
<th>FA [wt. %]</th>
<th>Kaolite HPA [wt. %]</th>
<th>GGBFS [wt. %]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon dioxide, SiO₂</td>
<td>66.56</td>
<td>45.14</td>
<td>31.52</td>
</tr>
<tr>
<td>Aluminium oxide, Al₂O₃</td>
<td>22.47</td>
<td>33.32</td>
<td>12.22</td>
</tr>
<tr>
<td>Iron oxide, Fe₂O₃</td>
<td>3.54</td>
<td>11.99</td>
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<td>Calcium oxide, CaO</td>
<td>1.64</td>
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<tr>
<td>Potassium oxide, K₂O</td>
<td>1.75</td>
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<tr>
<td>Sodium oxide, Na₂O</td>
<td>0.58</td>
<td>0.07</td>
<td>0.21</td>
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<td>1.37</td>
<td>4.62</td>
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<td>0.56</td>
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<td>Sulphur trioxide, SO₃</td>
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</tbody>
</table>

The alkaline activator used is a mixture of aqueous solution of sodium hydroxide (NaOH) and commercially available sodium silicate (Na₂SiO₃) grade D in a mass ratio of 2.5:1 (Na₂SiO₃ to NaOH). The sodium hydroxide solution used was obtained by dissolving the technical grade NaOH pellets of 98% purity in tap water. These pellets have a molecular weight of 40, a specific gravity of 2.1 and a pH of approximately 14. The concentration of sodium hydroxide solution used is 12 molar (M) consisted of 480 grams (g) of NaOH pellets per liter of NaOH solution or 361 g of NaOH pellets per kg of NaOH solution. The sodium silicate solution has a chemical composition by mass of 14.7% Na₂O, 29.4% SiO₂ and 55.9% H₂O with a modulus ratio (Mₙ) of 2 (Mₙ=SiO₂/Na₂O=2), specific gravity of 1.53 and a pH of 12.9 (values provided by supplier, PQ Australia). Both alkaline solutions were prepared and mixed together 24 hours prior to usage.

Coarse aggregate used is crushed basalt sourced from Dunmore quarry in New South Wales, Australia, with a maximum nominal size of 10 mm, the specific gravity of 2.8 and
water absorption of 1.6%. The fine aggregate used is Sydney sand with a specific gravity of 2.65 and water absorption of 3.5%. All aggregate was first kiln dried at 105°C for a period of 48 hours and then prepared to saturated surface dry (SSD) prior to batching. The drying procedure, which removes all the moisture content of the aggregate and also the organic particles in the sand, could help to adjust more accurately the mix water and consequently enhance the consistency of the GPC.

In order to make the conventional concrete for the purpose of comparison, ordinary Portland cement conforming to the requirements of ASTM Type I was used. The mix proportions are summarized in Table 1. For OPC concrete, mixing was performed in accordance with AS 1012.2; however, to make geopolymer concrete mix, the solid constituents were firstly dry mixed in a vertical pan mixer for 5 minutes to achieve a uniform dispersion. The alkaline activator solution and water were then gradually added and further mixed for 15 minutes.

Table 2: Mix proportions

<table>
<thead>
<tr>
<th>Materials</th>
<th>GPC</th>
<th>OPCC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[wt. %]</td>
<td>[kg/m3]</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>50.1</td>
<td>1221.2</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>25.5</td>
<td>620.8</td>
</tr>
<tr>
<td>Fly ash</td>
<td>11.1</td>
<td>271.6</td>
</tr>
<tr>
<td>Kaolite HPA</td>
<td>3.2</td>
<td>77.6</td>
</tr>
<tr>
<td>GGBFS</td>
<td>1.6</td>
<td>38.8</td>
</tr>
<tr>
<td>Portland cement</td>
<td>-</td>
<td>0.0</td>
</tr>
<tr>
<td>12 M NaOH solution</td>
<td>2.3</td>
<td>55.3</td>
</tr>
<tr>
<td>Sodium silicate solution</td>
<td>5.7</td>
<td>138.7</td>
</tr>
<tr>
<td>Free water</td>
<td>0.5</td>
<td>13.3</td>
</tr>
<tr>
<td>High-range water reducer (HWR)</td>
<td>-</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Three different curing regimes were applied to the GPC and the OPCC specimens: lime-water curing (LWC), sealed ambient curing (SAC) and heat curing (HC). Lime-water curing is only applied for OPCC by placing the demoulded samples in lime-saturated water in a controlled room at a temperature of 23 ± 2 °C until the testing date. Sealed ambient curing and heat curing were applied for both GPC and OPC concrete specimens. For the SAC, specimens were kept sealed inside the mould until the testing day in a controlled room at a temperature of 23 ± 2 °C. In order to monitor the quality of the sealing system, specimens’ mass were measured before and after the curing period and the rate of moisture loss was observed to be negligible (< 0.1%). For the heat curing, one hour after finishing of the casting, sealed specimens were moved to the oven. Twelve different heat curing conditions applied to the identical concrete samples, which involved three different temperatures of 60, 75 and 90°C and four curing periods of 8, 12, 18 and 24 hours. After completion of the heat curing period, samples were demoulded and stored in a controlled room at a temperature of 23 ± 2°C until the testing date.
3 TEST METHODS

Water absorption (ASTM C642-13), apparent volume of permeable voids - VPV (ASTM C642-13) and sorptivity (ASTM C1585-13) tests are conducted to measure the transport properties of concretes after 28 days of age. Surface resistivity (AASHTO TP-95) and bulk resistivity tests are also performed at 28 days. The compressive strength of the test specimens measured in accordance with ASTM C39.

4 RESULTS AND DISCUSSION

4.1 Compressive strength

The compressive strength of geopolymer and their counterpart Portland cement concretes at 28 days of age are summarised in Table 2. Each of the test result presented corresponds to the mean value of three test specimens. Individual strengths were well within the range of ±5.0% of the mean value.

<table>
<thead>
<tr>
<th>Mix designation</th>
<th>Curing temperature °C</th>
<th>Heat curing duration h</th>
<th>$f_{c,28}$ [MPa ± SD]</th>
</tr>
</thead>
<tbody>
<tr>
<td>GPC-SAC</td>
<td>23-sealed</td>
<td>N.A.</td>
<td>41.7±0.9</td>
</tr>
<tr>
<td>GPC-60D-8h</td>
<td>60</td>
<td>8</td>
<td>27.4±0.9</td>
</tr>
<tr>
<td>GPC-60D-12h</td>
<td>60</td>
<td>12</td>
<td>37.8±0.3</td>
</tr>
<tr>
<td>GPC-60D-18h</td>
<td>60</td>
<td>18</td>
<td>45.6±0.4</td>
</tr>
<tr>
<td>GPC-60D-24h</td>
<td>60</td>
<td>24</td>
<td>50.0±0.8</td>
</tr>
<tr>
<td>GPC-75D-8h</td>
<td>75</td>
<td>8</td>
<td>44.8±0.2</td>
</tr>
<tr>
<td>GPC-75D-12h</td>
<td>75</td>
<td>12</td>
<td>53.9±0.5</td>
</tr>
<tr>
<td>GPC-75D-18h</td>
<td>75</td>
<td>18</td>
<td>60.0±0.1</td>
</tr>
<tr>
<td>GPC-75D-24h</td>
<td>75</td>
<td>24</td>
<td>62.3±0.2</td>
</tr>
<tr>
<td>GPC-90D-8h</td>
<td>90</td>
<td>8</td>
<td>52.2±0.7</td>
</tr>
<tr>
<td>GPC-90D-12h</td>
<td>90</td>
<td>12</td>
<td>58.6±0.9</td>
</tr>
<tr>
<td>GPC-90D-18h</td>
<td>90</td>
<td>18</td>
<td>59.8±0.2</td>
</tr>
<tr>
<td>GPC-90D-24h</td>
<td>90</td>
<td>24</td>
<td>60.7±1.2</td>
</tr>
<tr>
<td>OPCC-LWC</td>
<td>23-lime water</td>
<td>N.A.</td>
<td>64.5 ± 1.2</td>
</tr>
<tr>
<td>OPCC-SAC</td>
<td>23-sealed</td>
<td>N.A.</td>
<td>52.9 ± 1.0</td>
</tr>
<tr>
<td>OPPC-60D-8h</td>
<td>60</td>
<td>8</td>
<td>42.6 ± 0.1</td>
</tr>
<tr>
<td>OPPC-60D-12h</td>
<td>60</td>
<td>12</td>
<td>47.6 ± 0.7</td>
</tr>
<tr>
<td>OPPC-60D-18h</td>
<td>60</td>
<td>18</td>
<td>47.6 ± 1.9</td>
</tr>
<tr>
<td>OPPC-60D-24h</td>
<td>60</td>
<td>24</td>
<td>49.4 ± 0.9</td>
</tr>
<tr>
<td>OPPC-75D-8h</td>
<td>75</td>
<td>8</td>
<td>45.2 ± 1.5</td>
</tr>
<tr>
<td>OPPC-75D-12h</td>
<td>75</td>
<td>12</td>
<td>45.4 ± 0.4</td>
</tr>
<tr>
<td>OPPC-75D-18h</td>
<td>75</td>
<td>18</td>
<td>46.0 ± 0.5</td>
</tr>
<tr>
<td>OPPC-75D-24h</td>
<td>75</td>
<td>24</td>
<td>48.9 ± 0.1</td>
</tr>
<tr>
<td>OPPC-90D-8h</td>
<td>90</td>
<td>8</td>
<td>46.1 ± 1.7</td>
</tr>
<tr>
<td>OPPC-90D-12h</td>
<td>90</td>
<td>12</td>
<td>43.1 ± 0.4</td>
</tr>
<tr>
<td>OPPC-90D-18h</td>
<td>90</td>
<td>18</td>
<td>43.7 ± 0.5</td>
</tr>
<tr>
<td>OPPC-90D-24h</td>
<td>90</td>
<td>24</td>
<td>43.8 ± 0.9</td>
</tr>
</tbody>
</table>
The compressive strength of the geopolymer concretes increases with the curing temperature from 60°C to 90°C. The reason for this increase is that at higher temperatures the geopolymerization degree is higher which results in the formation of higher amount of reaction products [14]. Infrared spectrometric analysis conducted by Rovnaník [14] showed that regardless of the curing regime adopted, some of the aluminosilicate source materials will remain unreacted in the hardened GPC. However, the amount of these unreacted solids reduces by applying a higher curing temperature. The reason is that the fly ash grains are not fully dissolved during geopolymerization process and the reaction is taking place at the surface layer of the solid particles to form primary geopolymer gel [15]. The further geopolymerization rate is then controlled mainly by diffusion of hydroxide and silicate ions through the primary geopolymer gel, which is affected mainly by curing temperature at the early-stage of the reaction [14].

However, for a longer period of heat curing (18 hours and 24 hours) the optimum strength is achieved at 75°C. This complies with previous studies [9,10] reporting that curing geopolymer concrete for longer periods of time at elevated temperatures (e.g. 90°C) weakens its microstructure resulting in a lower compressive strength. Among all the different curing durations and temperatures, the GPC cured at 75°C for a period of 24 hours shows the highest compressive strength at 28 days of 62.3 MPa. However, reducing the heat curing duration from 24 hours to 18 hours does not lead to a significant reduction in strength (around 4% reduction). As a result, 18 hours heat curing at 75°C could be considered as the optimum economical heat curing condition for low-calcium FA-based geopolymer concrete.

The 28-day compressive strength of the GPC, heat-cured at 75°C for 18 hours, is about 20% higher than the maximum strength of heat-cured OPCC (49.4 MPa), and 15% higher than that of the ambient cured Portland cement concrete. In the case of OPC concrete, results in Table 2 shows that except for the samples heat-cured for only 8 hours, increasing the curing temperature from 60°C to 90°C corresponds with a lowering of the 28-day compressive strength. Furthermore, contrary to the geopolymer, the compressive strength of Portland cement concrete is not very sensitive to the change in the heat curing temperature (60°C to 90°C) and curing duration (8 hours to 24 hours). The difference between the lowest and the highest strength of OPCCs at 28 days is only around 16% (of the lowest value) while this value is 127% for the geopolymer concrete.

### 4.2 Volume of permeable voids (porosity)

The volume of permeable void (VPV) results are shown in Figure 1. In the case of geopolymer concretes, test results show that inappropriate curing condition such as curing temperatures lower than 75°C and curing durations lower than 18 hours would increase the volume of permeable voids. The optimum VPV (13.7-13.8%) is obtained for 18 and 24 hours curing at 75 and 90°C.

For the OPC concrete, it can be observed that heat-curing, increases the porosity of the matrix compared to the ambient curing condition. However, the volume of permeable voids is decreasing by increase in the curing temperature and duration. The best heat-cured Portland cement concrete had a VPV of 15.4% while this value for standard lime-water cured (LWC) and ambient-sealed cured samples are 13.5% and 14.9%, respectively.
Previous studies [18] report that the porosity of the fly ash based geopolymer concretes is higher than that of the slag based or fly ash/slag blended GPCs and their counterpart Portland cement concrete. The significantly higher amounts of voids identified in specimens with high fly ash contents (> 50 wt.%) is attributed to the nature of the gel type forming in the binder [19]. The N–A–S–H (Na₂O–Al₂O₃–SiO₂–H₂O) binding type gels which dominate the microstructure of the fly ash-based geopolymer is less dense compared to the C–A–S–H (CaO–Al₂O₃–SiO₂–H₂O) forming in slag-based geopolymer concrete [19] and the C–S–H in OPC concrete [20]. Provis et al. [21] stated that, the presence of more bound water, induced by the presence of Ca in the system, provides greater pore-filling capacity to this type of gel than N-A-S-H geopolymer type gels. However, in this study, the VPV of the properly heat cured fly ash-based geopolymers is very similar to that of the standard lime-water cured Portland cement concrete (13.7% compared to 13.5%).

![Figure 1: Average porosity (VPV) of GPC and OPCCs at 28 days](image)

4.3 Sorptivity

Figure 2 is presenting the initial rate of water absorption or the sorptivity coefficient of different GPC and OPC samples. The sorptivity coefficient (Si) which is the initial slope of the sorptivity curves up to the saturation limit (up to 6 hours in this study) is obtained by using equation 1 where \( I \) is the absorption and \( t \) is the time. The correlation coefficient (R²) of all the test results is greater than 0.99.

\[
I = S_i \sqrt{t}
\]

It is evident that the sorptivity coefficient of geopolymer concretes is decreasing by increase in the curing temperature and time up to 24 hours at 75°C. This is possibly due to the formation of more densified geopolymer network resulting in a decreased pore size or more tortuous pore network.

However, the overall trend of sorptivity coefficient \( S_i \) of the heat cured Portland cement concretes differs totally from the geopolymers. In cement concrete specimens, the sorptivity coefficient increases with increasing the curing temperature and time. Curing OPC concrete at
the elevated temperatures substantially affects the tortuosity characteristics of the C–S–H phase by creating a more extended capillary pore network. The sorptivity coefficient of heat cured OPCC at 90°C for 24 hours is 365% more than that of the ambient-cured (OPCC-SAC) specimen (10.6×10⁻³ mm/s⁰.⁵ compared to 2.6×10⁻³ mm/s⁰.⁵).

Figure 2: Initial sorptivity at 28 days for GPC and OPC concretes

Capillary sorption is associated with the total porosity and the tortuosity and size of the pore network [18]. To better understand the pore size distribution of the tested specimens, mercury intrusion porosimetry (MIP) has performed on a couple of selected samples. One geopolymer and one Portland cement paste sample having the same composition as GPC and OPC concrete used in the study (excluding aggregate) have been prepared. The geopolymer paste cured at 75°C for 18 hours and the Portland cement paste was cured in sealed ambient condition. The MIP results are presented in Figure 3.
Figure 3: Pore structure distribution of heat-cured geopolymer and ambient cured OPC mortars determined by mercury intrusion porosimetry (MIP)

The total porosity (open pores) of samples could be calculated using the volume of mercury intruded at the maximum experimental pressure as shown in Figure 3-a. The total volume of intruded mercury for OPC paste is slightly lower than that of the geopolymer paste (0.155 compared to 0.185 ml/g). This shows lower total porosity for ambient-cured OPC compared to the heat-cured geopolymer which is in line with the total water absorbed at the end of the sorptivity test for the counterpart concrete samples. The total water absorbed at the end of the sorptivity test was slightly lower for ambient cured OPC concrete compared to the GPC cured at 75°C for 18 hours (2 mm versus to 2.6 mm).

The pore size distribution of geopolymer sample versus the cement paste is shown in Figure 3-b. The differential curve for heat-cured geopolymer presents several peaks reflecting pore sizes with diameter in the range of 3 to 13 nanometres with a critical or threshold peak at around 4 nm. These can be referred to the gel pores as earlier reported by Ma et al. [22]. For the heat-cured geopolymer sample, no other significant peaks are seen representing the larger size pores (capillary pores) that have been observed for the studied cement mortar. Such microstructure is distinctly different from that of cement paste. The homogeneous gels in the geopolymer matrix occupying most of the bulk space and the formation of the typical capillary pores (which can generally be observed in cement paste) could not be found in the geopolymer sample.

Although doubts exists on the accuracy of MIP method to reflect the true pore size distribution of cement based materials [23], the permeability and diffusion characteristics of materials are reported [24] to be closely related to threshold pore diameter. The threshold pore diameter has been defined as the diameter of pore that completes the first connected pore pathway in materials [25].

4.4 Surface and bulk resistivity

Surface and bulk resistivity tests were conducted on vacuum-saturated GPC and OPC concretes at the age of 28 days and results are presented in Table 4. The lowest and highest
resistivity values for geopolymers were recorded for the specimens cured at 60°C for 8 hours and cured 75°C for 24 hours, respectively. Increasing the curing temperature from 60°C to 75°C and the curing time from 8 hours to 24 hours increases the surface and bulk resistivity of geopolymer concrete specimens by more than 270%. This means that the resistivity of geopolymer concrete is strongly influenced by the curing regime and is susceptible to the change in the initial heat-curing temperature and duration.

However, as shown in Table 4, the resistivity of GPCs are by far less than their counterpart OPC concrete. Although the low resistivity facilitates the ionic current between anodic and cathodic sites during the corrosion process and is reported [26] to increase the corrosion rate of steel reinforcement in Portland cement concrete, this yet to be sufficiently investigated for geopolymer concrete. The low resistivity observed for the GPC specimens studied here is most likely related to the free metallic ions (i.e. Na⁺) in the pore solution that could carry more electrical current comparing to the Portland cement structure.

<table>
<thead>
<tr>
<th>Curing condition</th>
<th>Surface Resistivity [kΩ.cm]</th>
<th>Bulk Resistivity [kΩ.cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature [ºC]</td>
<td>GPC</td>
<td>OPCC</td>
</tr>
<tr>
<td>60</td>
<td>8</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>3.5</td>
</tr>
<tr>
<td>75</td>
<td>8</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>5.0</td>
</tr>
<tr>
<td>90</td>
<td>8</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>3.2</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>4.0</td>
</tr>
<tr>
<td>23 ºC – 28 days lime water cured</td>
<td>N.A.</td>
<td>12.5</td>
</tr>
<tr>
<td>23 ºC – 28 days ambient cured</td>
<td>2.5</td>
<td>12.0</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

- Depending on the heat curing regime, fly ash-based geopolymer concrete samples developed compressive strengths ranged between 27.4 - 62.3 MPa at 28 days. The compressive strength increased with the increase in the curing temperature up to 75°C and curing duration up to 24 hours.
- Inappropriate curing condition such as curing temperatures lower than 75°C and curing durations lower than 18 hours would increase the volume of permeable voids in geopolymer concrete. The optimum VPV (13.7-13.8%) is recorded for 18 and 24 hours curing at 75°C and 90°C. The VPV of the properly heat cured fly ash-based geopolymers is very similar to that of its counterpart Portland cement concrete cured.
Geopolymer concretes heat-cured at 60°C showed higher sorptivity compared to the ambient cured samples. Heat-curing of fly ash-based GPCs at 75 and 90°C for 18 to 24 hours results in a lower sorptivity than the ambient curing thereby indicating lower porosity or less interconnected capillary pores. This is in line with the results obtained for volume of permeable voids (VPV).

For saturated geopolymer concrete, the resistivity increases with an increase of the curing temperature and time. The highest resistivity for GPCs was achieved with specimens cured for 18 to 24 hours at 75°C. Heat curing reduces the resistivity of Portland cement concrete.

ACKNOWLEDGMENT

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REFERENCES


Concrete, 1997.


POZZOLANIC MATERIALS OBTAINED THROUGH A TREATMENT METHODOLOGY OF LANDFILLS. CHARACTERIZATION OF NEW CEMENTS AND DURABILITY OF CONCRETES

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Keywords: Blended cements, pozzolanicity, landfills, hydration, durability

Abstract. This study introduces a separation protocol or technology and verifies the suitability of reusing materials recovered from landfills as active additions in cement manufacture. A landfill located in Spain, nearby coal-fired power plant, was used to study the possible pozzolanicity of the wastes deposited there. Physical-mechanical trials were run to compare the performance of the laboratory cement (75 % OPC, 25 % fractions with lowest particle sizes, with and without unburnt carbon) to the results found for cement bearing conventional fly ash (CEM II A-V). The results obtained shown that the behaviour exhibited by these new cements was comparable to the performance in standardised cement.

Also the durability of concretes prepared with these new cements was established. The concretes were submitted to durability tests of chlorine penetration, sulphate attack and leaching process. In all cases, the new concretes exhibited a comparable behaviour to concrete prepared with OPC cement

This methodology can be applied in other landfills in the world.
1 INTRODUCTION

Construction is an activity essential to economic development. The most durable and economically viable construction material is concrete, whose main industrial component, cement, is characterised by its significant contribution - around 7% of the worldwide total - to anthropogenic CO₂ emissions [1]. The equilibrium among economic, environmental and raw material-related factors that once made Portland cement a highly competitive product has been upset by the awareness of the effect of greenhouse gases, in this case CO₂, on climate change.

There are consolidated strategies for lowering CO₂ emissions in cement production: a) replacement of clinker with mineral additions such as pozzolans or blast furnace slag; b) enhancement of the energy efficiency of cement manufacture; c) use of alternative fuels; and d) development of more eco-efficient cements with low or nil Portland cement content.

One of the pozzolans used for this purpose is siliceous fly ash, a by-product of coal-fired power generation. Its standard use as a cement addition is beneficial for technical, economic and environmental reasons, albeit subject to the requirements laid down in European standard EN-197-1:2011. These particles affect cement in different ways. By pozzolanic reaction, that means, the portlandite (Ca(OH)₂) generated during clinker combines at room temperature with the active silica and alumina in the ash to yield a C-S-H gel (with low C/S than the C-S-H gel forming when the silicates in portland cement clinker hydrated) and aluminate hydrates similar to the ones forming in conventional clinker hydration. These additional reaction products increase the density and reduce the permeability of mortars and concretes. Also the spherical shape of fly ash particles favours the workability of cementitious systems [2-4].

In Spain, specifically, each steam power plant has a landfill, some of whose materials may be pozzolanic. According to several inventories and catalogues (www.igme.es), anywhere from 13 000 to 20 000 abandoned power plant and mining landfills are scattered across the country. The environmental problems posed are of deep concern to the Spanish Directorate General of Industry, Energy and Mines, which has recommended that project briefs and projects should be drawn up to restore the land affected and research should be conducted into reusing these materials for energy or geomechanical purposes.

In a previous study [5] a new separation protocol technology was proposed and verified suitability of reusing materials recovered from landfills as active mineral additions in cement manufacture. In the present investigation a new step is going up, studying the cement hydration process and the durability of concrete prepared from these blended cements.

2 EXPERIMENTAL PART

The landfill chosen for this study is located on a hillside north of a steam power plant just 5 km NE of the centre of Aliaga, a town in the Spanish province of Teruel. The separation process and the Laboratory-scale plant to fraction landfill materials were described in [5].
2.1. Materials

The sample selected after separation process from the landfill was the fractions D-i < 100 μm previously eliminated the unburnt carbon. Also a CEM I 42.5 R/SR cement (OPC) with Blaine specific surface of 400 m²/kg, as well as a control fly ash (FA) routinely used at the cement plant to prepare CEM II/V-A-B cements were used as reference materials. The chemical composition of all these materials is presented in Table 1.

<table>
<thead>
<tr>
<th></th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>TiO₂</th>
<th>P₂O₅</th>
<th>MnO</th>
<th>L.o.I.</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>20.9</td>
<td>3.9</td>
<td>4.0</td>
<td>62.8</td>
<td>1.7</td>
<td>3.3</td>
<td>0.2</td>
<td>--</td>
<td>0.1</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>FA</td>
<td>41.9</td>
<td>3.9</td>
<td>21.9</td>
<td>6.4</td>
<td>1.1</td>
<td>0.9</td>
<td>1.5</td>
<td>--</td>
<td>0.3</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>D-i</td>
<td>38.1</td>
<td>26.6</td>
<td>17.1</td>
<td>3.5</td>
<td>0.6</td>
<td>4.7</td>
<td>--</td>
<td>1.6</td>
<td>0.6</td>
<td>&lt;0.1</td>
<td>&lt;0.1</td>
<td>2.0</td>
</tr>
</tbody>
</table>

L.o.I: Lost of ignition

2.2. Test conducted in cement pastes
- Pozzolanic reactivity (Frattini Test) (European standard EN-196-5:2011): Pozzolanic reactivity trials were conducted on laboratory-prepared blends containing 75/25 wt%, respectively, of cement and FA and fraction D-i.
- Physical-chemical charactarisation of cements (European standard EN-197-1:2011): These trials were conducted on blends containing 75 % cement and 25 % of fractions D-i or FA.

2.3. Concrete preparation and durability test conducted
In concrete preparation two cements were used: CEM FA (75% OPC and 25% FA) and CEM D-i (75% OPC and 25% D-i). 24 concrete cubic blocks of 150 mm x 150 mm x 150 mm were prepared for each type of cement. Dosages used are shown in Table 2. All concretes were cured in a climate chamber at 25°C ± 2 °C with 99% of relative humidity. At this age their compressive strengths were determined using an Ibertext press.

<table>
<thead>
<tr>
<th></th>
<th>Concrete D-i (95l)</th>
<th>Concrete CV (65l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (kg)</td>
<td>10.2</td>
<td>10.2</td>
</tr>
<tr>
<td>FA (kg)</td>
<td>---</td>
<td>3.4</td>
</tr>
<tr>
<td>D-i (kg)</td>
<td>3.4</td>
<td>0.0</td>
</tr>
<tr>
<td>Sand (kg)</td>
<td>37.7</td>
<td>37.7</td>
</tr>
<tr>
<td>Gravel (kg)</td>
<td>41.1</td>
<td>41.1</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>7.4</td>
<td>7.4</td>
</tr>
<tr>
<td>Admixture (kg)</td>
<td>0.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>
Ready-mixed concretes were tested on performance and durability tests:

- Metal leaching using the Duch standard EA NEN 7375:2004 “Determination of leaching of inorganic components with the diffusion test”. ICP determinations were carried out in a VARIAN 725 ES-ICP inductively-coupled plasma optical emission spectrometer
- Sulphate attack test. The ASTM C1012/C1012M-09 regulation was adapted for concretes as it is specific for mortar test tubes.
- Chloride penetration test. The ASTM C1543-10a standard was followed.

3 RESULTS AND DISCUSSION

3.1. Pozzolanic reactivity (Frattini Test)

Figure 1 plots the 8- and 15-day [OH] mM/L versus [CaO] mM/L values obtained with the pozzolanic test conducted [6]. Data points positioned underneath the curve are indicative of lower CaO than OH⁻ ion concentration: i.e., of Ca (OH)₂ capture and hence pozzolanicity. The same pozzolanicity study was conducted also on an aluminosiliceous fly ash (FA) routinely used at the cement plant to prepare CEM II A-B/V-type cements (Table 1).

![Figure 1: Pozzolanicity data](image)

The findings also showed that fraction D-i pozzolanicity was at least comparable to and on occasion greater than the reactivity recorded for conventional aluminosiliceous fly ash (FA).

3.2. Physical-chemical characterisation of cements

Table 3 gives the water/cement ratios for suitable consistency, the initial and final setting times, the expansion values and mechanical strength behaviour of the cements prepared with 25 % D-i and FA.
The cement samples containing fractions D-i exhibited around 28% higher water/cement values than the cement made with conventional fly ash. Those data denote their greater capacity to fix water, perhaps due to their composition and surface characteristics. The initial and final setting times in these cements, around 1 hour less than in CEM FA. All the cements also confirmed to the expansion requisites for all exhibited values of fewer than 10%.

According to the findings, the two cements met the physical and mechanical requirements laid down in European standard EN 197-1:2011.

Table 3: Physico-chemical characterization of cements

<table>
<thead>
<tr>
<th></th>
<th>L/S</th>
<th>Initial setting time</th>
<th>Final setting time</th>
<th>Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM D-i</td>
<td>0.31</td>
<td>4 h 30 min</td>
<td>7 h 00 min</td>
<td>&lt;10 %</td>
</tr>
<tr>
<td>CEM FA</td>
<td>0.25</td>
<td>5 h 30 min</td>
<td>7 h 40 min</td>
<td>&lt;10 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>2 days</th>
<th>7 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexural strength (MPa)</td>
<td>Compressive strength (MPa)</td>
<td>Flexural strength (MPa)</td>
</tr>
<tr>
<td>CEM D-i</td>
<td>4.1 ± 0.1</td>
<td>17.2 ± 0.6</td>
<td>6.2 ± 0.2</td>
</tr>
<tr>
<td>CEM FA</td>
<td>4.0 ± 0.1</td>
<td>16.4 ± 0.5</td>
<td>6.0 ± 0.5</td>
</tr>
</tbody>
</table>

3.3. Concrete durability

The main concrete durability tests are following presented.

3.3.1. Leaching tests

The leaching agent pH was measured at different times of the test. Results obtained are shown in Table 4. The basic pH in early times of this test is due to leaching of alkaline (Na⁺, K⁺) and alkaline earth ions (Ca²⁺, Mg²⁺).

Table 4. pH of the leachate of D-i concrete and CV concrete

<table>
<thead>
<tr>
<th>Time</th>
<th>pH Concrete D-i</th>
<th>pH Concrete FA</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 hours</td>
<td>11.1</td>
<td>9.1</td>
</tr>
<tr>
<td>1 day</td>
<td>11.6</td>
<td>8.0</td>
</tr>
<tr>
<td>4 days</td>
<td>10.7</td>
<td>8.2</td>
</tr>
<tr>
<td>10 days</td>
<td>9.9</td>
<td>9.7</td>
</tr>
<tr>
<td>16 days</td>
<td>8.0</td>
<td>9.8</td>
</tr>
<tr>
<td>36 days</td>
<td>7.8</td>
<td>9.6</td>
</tr>
</tbody>
</table>
These findings show that the D-i concrete is neither toxic nor dangerous. The following elements were leached: aluminium, calcium, potassium, magnesium, sodium, sulphur, strontium and zinc, neither of which are included as toxic or dangerous, even chrome was found in a smaller quantity to the one included in the limits of the toxic elements of EPA [7].

On the basis of the ICP-OES results, the leaching of the different elements was characterized applying the Godbee et al. equations adapted by Conner [8]. The calculated parameter was \( L_i \) (parameter which measures the resistance of a waste to be leached) which is the leaching indicator.

\( L_i \) values related to time are represented and shown in Figure 2. It can be noted that all the leaching rates studied have values of more than 6, which shows that both D-i and CV concretes behave as good confiners of those elements inside its matrix.

![Figure 2: \( L_i \) values leached from D-i and FA Concretes](image)

### 3.3.2. Sulphate Attack and Chlorine Diffusion

The performance to the sulphate attack and dissemination of chlorides of FA and D-i concretes is fully comparable.

No modifications in the dimensions of the test tubes where observed until the end of the test (90 days), variations on their mechanical strengths were neither observed. Table 5 shows the values of the compression strength on concrete test tubes after both tests at 14 days age.

In Figure 3a you can see the appearance of the D-i concrete test tube after being in contact for 56 days with the sodium sulphate solution (4.4 wt% \( \text{Na}_2\text{SO}_4 \)) complying ASTM C1012/C1012M-09 Standard. In Figure 3b it is observed that the D-i concrete test tube does not show any color variation after being tinted with \( \text{AgNO}_3 \), after being tested 56 days following the ASTM C1543-10a Standard.
Table 5: Strength evolution of FA and D-i concrete test tubes after durability tests.

<table>
<thead>
<tr>
<th>Concrete FA</th>
<th>28d (before test)</th>
<th>14d (after test)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>34.3 MPa</td>
<td>41.6 Mpa</td>
</tr>
<tr>
<td>Chlorine</td>
<td></td>
<td>40.5 Mpa</td>
</tr>
<tr>
<td>Sulphates</td>
<td></td>
<td>38.8 Mpa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete D-i</th>
<th>28d (before test)</th>
<th>14d (after test)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>42.5 MPa</td>
<td>42.2 Mpa</td>
</tr>
<tr>
<td>Chlorine</td>
<td></td>
<td>43.6 Mpa</td>
</tr>
<tr>
<td>Sulphates</td>
<td></td>
<td>50.5 Mpa</td>
</tr>
</tbody>
</table>

Figura 3: A: D-i test tubes after being attacked by sulphates for 56 days. B: D-i test tubes after Chlorine Diffusion and AgNO₃ treatment.

4 CONCLUSIONS

The main conclusions extracted from the paper research are:

- This study introduces a separation protocol or technology and verifies the suitability of reusing materials recovered from landfills as active additions in cement manufacture. The possible inclusion of these alternative additions would contribute to lowering the consumption of natural resources (pozzolans and raw meal) and reducing the industry’s carbon footprint, but would require greater flexibility in European standard EN-197-1:2011 to accommodate a wider range of pozzolans. This methodology can be applied in other landfills in the world.
- It has been confirmed the new addition produced from this technology has pozzolanic activity and all physical and mechanical requirements laid down in European standard EN 197-1:2011.

- D-i concrete is neither toxic nor dangerous. The following elements were leached: aluminum, calcium, potassium, magnesium, sodium, sulphur, strontium and zinc, neither of which are included as toxic or dangerous, even chrome was found in a smaller quantity to the one included in the limits of the toxic elements of EPA.

- Regarding L_{i} values (parameter which measures the resistance of a waste to be leached) all the leaching rates studied have values of more than 6, which shows that D-i concrete behave as good confiners of those elements inside its matrix.

- D-i test tubes concretes attacked by sulphate maintained their dimensional stability after 90 days of testing. Strength of test tubes does not decrease after being attacked by sulphates.

- No evidence of chloride penetration is shown, in any age of the test, after treating D-i concretes with a AgNO_{3} solution. Strength of concrete test tubes did not decrease, at any age studied, after being attacked by NaCl.

ACKNOWLEDGEMENTS

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REFERENCES


PRELIMINARY ASSESSMENT OF DURABILITY OF A LOW CARBON CONCRETE MADE WITH LIMESTONE CALCINED CLAY PORTLAND CEMENT

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Key words: Cement, Concrete, Environment, Durability

Abstract. Through a collaborative research effort between the Ecole Polytechnique Federal de Lausanne and CIDEM a new kind of cementitious binder has been developed based on the synergy between calcined clays and limestone. Industrial trials carried out in Cuba during 2013-2015 have proven that the new cement is very effective even with clinker content as low as 50%. This paper presents preliminary results of a full program of assessment of durability of concrete cast with the new cement. Concrete elements have been cast and placed on an exposure site in the northern coast of Cuba. Cores have been taken, and have been tested for chloride ingress and migration, carbonation and air permeability. Concrete made with the cement containing calcined clays and limestone appears to have a more tortuous pore structure, and an increased chloride binding capacity due to the presence of alumina phases, which helps decreasing the movement of chloride ions through the matrix. For the same reason, and despite the low free lime content, this concrete shows a reduced carbonation compared to the one made with Portland cement, at age 15 month, placed on seashore in the hot and humid tropical zone. Air permeability measurements confirm the lower permeability of the concrete subject of study; this is very consistent with the results of chloride ingress and migration and carbonation.

1 INTRODUCTION

Worldwide demand for cement and concrete has increased exponentially in the last twenty years and which is a result of the combination of strong ongoing trends such as the growth of...
population and the increased need for infrastructure and housing. The cement industry has faced significant pressure and made efforts to improve its production efficiency as well as its environmental impact \cite{1}. The real potential for improvement relies on clinker substitution by Supplementary Cementitious Materials (SCMs) \cite{2,3}, but the problem is that the availability of these materials is limited and thus the potential to reduce energy and emissions through clinker substitution is compromised \cite{4-6}.

Kaolinitic clays, however, are widely available in the tropical belt of the planet. Extensive research work has proven the reactivity of kaolinitic clays when fired at temperatures between 600-800 degrees Celsius \cite{7,8}. It has recently been proven that clays with relatively low kaolinite content (under 50\%) could also be very reactive \cite{9,10}.

Further, it has been demonstrated that the synergy existing between calcined clays and limestone enables clinker substitution rates of 50\% without compromising neither the early and late strength nor the durability of the cement \cite{11,12}. A ternary, clinker-calcined clays-limestone blend cement with clinker substitution rate up to 50\% has been developed and produced at different scales, including a full industrial trial at a cement plant, with use on general purpose construction applications in Cuba and India. \cite{13,14}

This paper presents preliminary evaluation of the durability of concrete made with the cement produced during the industrial trial in Cuba. The tests were realized on concrete specimens laid on the seashore in a highly aggressive environment. Chloride, carbonation and air permeability were the main experiments carried out.

## 2 MATERIALS AND EXPERIMENTAL PROCEDURES

The ternary blend cement was produced at the cement plan Siguaney in Cuba during an industrial trial in August 2013. Results of the physical, chemical and mechanical tests performed on the blended cement produced are presented in Table 1. \cite{13}.

Table 1- Summary of the chemical, physical and mechanical properties of the experimental cement produced

<table>
<thead>
<tr>
<th>Oxides composition [%]</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>IR</th>
<th>LOI</th>
<th>CaO free</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>27.3</td>
<td>4.6</td>
<td>4.6</td>
<td>49.8</td>
<td>1.3</td>
<td>3.7</td>
<td>12.6</td>
<td>7.1</td>
<td>0.9</td>
<td>98.4</td>
</tr>
<tr>
<td>Average proportion of the components [%]</td>
<td>Clinker</td>
<td>Calcined clay/ limestone</td>
<td>Gypsum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50.0</td>
<td>41.1</td>
<td>8.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Physical properties</th>
<th>Retained 90µm sieve %</th>
<th>Blaine test cm²/g</th>
<th>Bulk density g/cm³</th>
<th>Normal consistency %</th>
<th>Initial setting Min.</th>
<th>Final setting Hours</th>
<th>Volum. stability mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0</td>
<td>4190</td>
<td>2.9</td>
<td>25.0</td>
<td>135</td>
<td>2.9</td>
<td>0.3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Compressive strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 days</td>
<td>7 days</td>
</tr>
<tr>
<td>11.0</td>
<td>17.5</td>
</tr>
</tbody>
</table>
25 MPa Concrete was made using Cuban calcareous aggregates. The series M381 and M25 were made with the new cement, while the series M32 was cast with Cuban Ordinary Portland Cement, type I with 88% of clinker, known as P35. Table 2 presents details of the mix design and the mechanical properties of the concrete made.

Table 2: results of characterization of concrete mixes subject of this study

<table>
<thead>
<tr>
<th>ID</th>
<th>Target Strength</th>
<th>Cement kg/m3</th>
<th>w/c</th>
<th>Slump cm</th>
<th>date</th>
<th>Strength MPa</th>
<th>porosity %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3d</td>
<td>7d</td>
</tr>
<tr>
<td>M381</td>
<td>25 Mpa</td>
<td>360</td>
<td>0.47</td>
<td>8</td>
<td>02/11/2013</td>
<td>-</td>
<td>21</td>
</tr>
<tr>
<td>M25</td>
<td>25 Mpa</td>
<td>360</td>
<td>0.47</td>
<td>12</td>
<td>06/02/2014</td>
<td>-</td>
<td>18.5</td>
</tr>
<tr>
<td>M32</td>
<td>25 Mpa</td>
<td>300</td>
<td>0.40</td>
<td>8</td>
<td>10/02/2014</td>
<td>17.40</td>
<td>-</td>
</tr>
</tbody>
</table>

Concrete was placed on concrete blocks, which were laid at an exposure site at the northern coast of Cuba, see Figure 1.

Figure 1: exposure site at the northern coast of Cuba

Figure 2: test set-up for chloride migration

The experimental program carried out was the following:
• Chloride migration test following the multiionic transport model STADIUM® [15]. A constant DC voltage is applied; the current as well as the voltage between the two sides of the tested sample are daily measured over 14 days. (Figure 2). In addition, the chloride concentration is regularly measured in the downstream bath. The water accessible porosity value was measured according to norm ASTM C642. Additionally, after the migration test, the samples were gradually ground and their total chloride content determined. The tortuosity of the system was preliminary assessed through the formation factor [16,17] defined as the ratio of the pore solution electrical conductivity to the bulk (solid and pore solution) conductivity (the inverse of the τ coefficient calculated by Stadium). A method to estimate pore solution conductivity from the concentration of alkali and hydroxyls ions in the pore solution while bulk conductivity is calculated from the initial resistivity measured in the migration test.

• Carbonation depth, measured by spraying the exposed surfaced with phenolphthalein. Cores of the elements were taken. The cores were split in two parts so have the carbonated part exposed.

• In-situ measurement of air permeability using the Torrent method: it measures the coefficient of air-permeability kT with the PermeaTORR instrument.[18] The coefficient of air-permeability kT (10⁻¹⁶ m²) is calculated as function of the increase in pressure recorded in the measuring chamber. The test duration ranges between 2 and 6 min.

3 DISCUSSION OF RESULTS

3.1 Chloride
Cores of concrete were taken, slices 100 x 27 mm were prepared (with replica) and placed at the STADIUM cell, and a voltage was applied to accelerate ions through the concrete matrix. Based on the data from measurement of current, the formation factor was calculated according to the equation (1):

\[ \gamma = \frac{\sigma_p}{\sigma_b} \]  

Where:

\( \sigma_p \) : Electrical conductivity of the pore solution

\( \sigma_b \) : Electrical conductivity of the matrix (solid and solution)

It is equivalent to the ratio of the self-diffusion coefficient to the microstructural diffusion coefficient, and it characterizes por structure. In this work, it was used to assess the refinement of the pore structure produced using the new cementitious system, since the difference between the two conductivities is due to the restricted pathways through which the current is constrained in the bulk conductivity measurement. [16] Thus, it could be a way to estimate tortuosity of the system.
Figure 3 presents the current measurements on the samples tested (one replica for each), during the time of the migration test. Specimens of concrete made with the blended cement containing calcined clays and limestone (M381 and M25), had an acceleration voltage of 18 volts, and despite this they showed higher values of current measured at all times compared with the reference series M32, where 8 volts voltage was applied. This indicate the denser pore network and the reduced ion transport of the series M381 and M25.

Figure 4 presents the results of the formation factor calculated with the initial current measurement presented above, to make sure that changes in the pore solution have not yet taken place. All series produced with the new cement (M381 and M25), despite the higher voltage applied had a bigger formation factor than the series made with Portland cement (M32), thus confirming that concrete made with the new cement had a more tortuous and probably more refined pore network. The formation factor in concrete made with the blended cement was 3 times higher than in concrete made with Portland cement.
Figure 5: chloride profiles (left) total chloride, (right) free chloride

Figure 5 presents the chloride profiles of the outer part of the cores taken. Total and free chloride is measured, in order to have an idea of the chloride binding capacity of the systems incorporating more alumina through the clays \([19,20]\). The series M381 and M25, made with the new cement, had in all depths far less chloride than the series M32 cast with Portland cement. Chloride binding effect is clearly observed in all series of concrete cast with cement containing calcined clays, thus indicating that further to the improvement on the pore structure, chloride binding could be an interesting factor in these systems.

Figure 6: carbonation depth for concrete laid at exposure site 15 months

### 3.2 Carbonation

Figure 6 presents the results of measuring carbonation depth at the cores taken approximately 15 months after concrete was cast and laid for exposure. Despite the fact that the system clinker-calcined clays-limestone is known to be low in free lime, which is consumed in the
pozzolanic reaction \cite{11}, concrete made with this kind of cement shows a reduced carbonation depth compared with concrete made with Portland cement. The main reasons for this could be the pore refinement produced through the synergy of calcined clay and limestone, which reduces diffusion of diluted carbon through the concrete matrix \cite{21}. Further studies of carbonation, at later ages should be done.

### 3.3 Air permeability

Table 3 presents the results of measuring air permeability measure in concrete cylinders taken during sampling of the production, which were kept at air curing for 3 months. $k_T$ is series M381 and M25 qualifies as low permeability (see Table 4), and it is approximately 3 times lower than the series M32 made with Portland cement, which has moderate permeability. This is consistent with the results in chloride migration and carbonation described above, and it indicates the pore refining effect produced by the synergy between calcined clays and limestone.

<table>
<thead>
<tr>
<th>ID sample</th>
<th>$k_T$</th>
<th>log $k_T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M381</td>
<td>0.023</td>
<td>0.34</td>
</tr>
<tr>
<td>M25</td>
<td>0.1</td>
<td>0.28</td>
</tr>
<tr>
<td>M32</td>
<td>0.34</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 4: categories of permeability based on air permeability measurements \cite{22}

<table>
<thead>
<tr>
<th>Class</th>
<th>$k_T$ ($10^{-16}$ m$^2$)</th>
<th>Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td>PK1</td>
<td>&lt;0,01</td>
<td>Very low</td>
</tr>
<tr>
<td>PK2</td>
<td>0,01 – 0,10</td>
<td>Low</td>
</tr>
<tr>
<td>PK3</td>
<td>0,10 – 1,00</td>
<td>Moderate</td>
</tr>
<tr>
<td>PK4</td>
<td>1,00 – 10,00</td>
<td>High</td>
</tr>
<tr>
<td>PK5</td>
<td>&gt; 10.00</td>
<td>Very high</td>
</tr>
</tbody>
</table>

### 4 CONCLUSIONS

The following conclusions are outlined:

- Calculation of formation factor based on the initial current measurement in migration tests is 3 times higher proves that concrete series made with a ternary cement based on clinker-calcined clay-limestone compared to concrete having similar strength but made with Portland cement, thus proving that the incorporation of calcined clays and limestone do increase tortuosity of the pore system. Chloride profiling confirms that less chloride has entered the matrix in the former, probably due to the refined pore structure plus the capability of the system of binding more chlorides due to the increase of alumina phases in the matrix.

- Despite the lower amount of free lime that is consumed in the pozzolanic reaction, the system containing calcined clay and limestone presents 2-3 times lower carbonation rate at the age and conditions evaluated (15 months, seashore in the hot and humid tropical zone). This is probably caused by the pore refinement, which reduces
diffusion of diluted carbon through the concrete matrix. This result is however not conclusive; further studies at longer term should be carried out.

- Measurement of air permeability coefficient $k_T$ aided with the Torrent method prove a reduction of around 3 times of permeability in the concrete matrix made with the ternary cement containing calcined clay and limestone. This is very consistent with the two conclusions drawn above, and could confirm the beneficial effect of using calcined clays and limestone as Supplementary Cementitious Materials.

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PREVENTING REINFORCEMENT CORROSION IN CRACKED CONCRETE BY SELF-REPAIR

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Key words: Self-healing, Polyurethane, Capsules, Chloride-induced corrosion, X-ray radiography

Abstract. Corrosion of steel reinforcement is one of the most detrimental attack mechanisms for reinforced concrete structures. The presence of cracks, which are inextricably linked to reinforced concrete, accelerates this attack mechanism. Within this study it was aimed to heal cracks by an autonomous mechanism which is triggered upon crack appearance. During an accelerated corrosion test on mortar samples with embedded reinforcement bars, it was shown that the presence of cracks indeed accelerates the onset of corrosion and that crack healing in both, the traditional, manual way and the proposed autonomous way delay the onset of corrosion. This difference in corrosion behavior was proven by visual evaluation of the rebar surface, by means of X-ray radiography and by gravimetrical measurements which were performed after the accelerated corrosion test was finished.

1 INTRODUCTION

Worldwide about 5 tons of steel get lost every second due to corrosion [1]. For reinforced concrete, relying on the reinforcement bars to bear the tensile stresses, corrosion is one of the most dangerous attack mechanisms. Especially when cracks are created in the concrete matrix, a high risk for chloride- or carbonation-induced corrosion exists. Although repair of cracks is expensive, it has been estimated that the additional indirect expenses due to traffic jams, loss in production,… are about ten times higher compared to the direct costs [2]. Self-repair of cracks can thus have a high economic impact as the indirect costs as well as a part of the direct costs can be avoided. In addition, self-repair will lead to more durable concrete structures when the risk for reinforcement corrosion can be decreased.

Research on self-repair of cracks in concrete is going on now for several years within the Magnel Laboratory for Concrete Research in Belgium. Different approaches to obtain self-repairing properties are investigated among which the use of calcium carbonate precipitating bacteria [3]; the use of embedded hydrogels which take up water entering into the cracks and
block the crack through swelling [4]; the use of fibres to restrict the crack width [5] and the use of supplementary cementitious materials which hydrate more slowly compared to cement [5], both to improve the natural crack healing phenomenon, named autogenous healing, in concrete; and the use of encapsulated healing agents which are released at the moment of crack creation [6].

The crack healing efficiency is then evaluated by performing water permeability tests [7] or capillary water absorption tests [8] on cracked samples with and without self-healing properties and by comparing the water flow rate through the cracks or the water ingress into the cracks for both test series. The crack closure efficiency of each of the aforementioned techniques has been evaluated by microscopic investigation of the crack mouth at the concrete surface [5] or by X-ray computed tomography of the inner side of the crack [6, 9, 10] before and after healing. Regain of the mechanical properties, such as strength and stiffness, is evaluated by performing a reloading test of the samples after self-healing of the crack has taken place and by comparing the obtained mechanical properties to those obtained for similar cracked samples without self-healing properties [6]. The outcome of these studies led us conclude that the aforementioned approaches are successful to obtain crack healing in an autonomous way.

However, the most important aim of self-healing concrete is to obtain more durable concrete structures. Therefore, in the present study it was investigated whether self-repair of cracks can prevent reinforcement corrosion. Mortar samples with and without self-healing properties and with an embedded reinforcement bar were exposed to a chloride solution and subjected to an accelerated corrosion test. To obtain autonomous crack healing, capsules with healing agent were embedded in the matrix and capsule breakage was triggered upon crack formation. Corrosion of the reinforcement steel was for all test series evaluated visually and by means of X-ray radiography. Comparison of the radiographs obtained for the different test series clearly demonstrated that both manual repair and self-repair of cracks can delay the onset of corrosion of cracked concrete.

2 MATERIALS AND METHODS

2.1 Self-healing mechanism

The self-healing mechanism which was used within this study consists of capsules filled with healing agent which are embedded in the cementitious matrix. The healing agent used, was a one component, polyurethane (PU) based agent which is commercially available (HA Flex SLV AF, De Neef Conchem). Tubular glass capsules with an inner diameter of 3.00 mm, an outer diameter of 3.35 mm and a length of 35 mm were filled with the healing agent. First one end of the capsules was sealed by means of methyl methacrylate (MMA) glue. Then, the healing agent was injected in the capsules by means of a syringe with a needle after which the second end was sealed with MMA glue. The capsules were positioned inside the mortar samples and expected to break and trigger autonomous crack healing at the moment of crack formation.

2.2 Reinforced, cracked mortar samples with(-out) self-healing properties

Smooth reinforcement bars with a diameter of 8 mm were used as reinforcing steel in the
samples. The bars were used as received but before placement in the moulds they were degreased. The bars were positioned along the length and in the centre of 40 mm x 40 mm x 160 mm moulds. Before placement of the mortar, actions were taken to provide one test series with self-healing properties and to obtain standardized cracks.

Self-healing properties were obtained for one test series by positioning the capsules filled with healing agent inside the moulds. Therefore, one couple of plastic wires was connected to the furthest sides of the mould at a distance of 12 mm below the top of the mould. Three capsules were positioned onto these wires at the positions shown in Figure 1.A and B.

Standardized cracks were obtained by placing metal plates with a thickness of 300 µm inside the mortar at the moment the samples were prepared. The metal plate, with a width of 160 mm, was oriented in the same way as the rebar and introduced to such a depth that it just reached the top of the rebar. For the samples with self-healing properties, the glass capsules were positioned through these plates and therefore three holes with a diameter of 3.5 mm, where the capsules went through, were introduced in the plates before placement [11].

After preparation of the moulds, the mortar mixture was made. Mortar with a sand to cement ratio of 3:1 and a cement to water ratio of 2:1 was used. The moulds were filled with mortar carefully in order to prevent breakage of the capsules and movement of the plates used to obtain standardized cracks. Afterwards, the moulds were placed in an air-conditioned room with a temperature of 20°C and a relative humidity of more than 90%. One day later all samples were demoulded and placed in the same air conditioned room until the time of testing.

To obtain the final test samples, the 40 mm x 40 mm x 160 mm prisms were reduced to three 40 mm x 40 mm slices with a thickness of about 10 mm as shown in Figure 1.C. For each test series under investigation three samples were tested.

![Figure 1](image)

**Figure 1**: Test samples used for the accelerated corrosion test. Position of the glass capsules with embedded healing agent inside the mould (A) and through the metal plate (B). Final test samples were obtained by reducing 40mmx40mmx160mm prisms to three 40mmx40mmx10mm prisms (C).
2.3 Crack formation and crack healing

Standardized cracks were obtained by removing the metal plates at the time of demoulding. This technique allowed to have similar cracks with a crack width of 300 µm for all of the test samples.

For one test series, cracks were healed manually through injection of the PU healing agent into the cracks by means of a syringe with a needle. Injection of the cracks was continued until they seemed to be completely filled with the healing agent. For the test series with embedded capsules, crack healing was obtained in an autonomous way. At the moment the metal plates were removed in order to create the crack, the glass capsules, which went through these plates, were broken. Due to breakage of the glass capsules the healing agent was released inside the crack and cured upon contact with the moisture available in the matrix.

2.4 Accelerated corrosion test

In this study corrosion of the rebar was induced by the ingress of chlorides via the cracks and/or the cementitious matrix. Therefore, the bottom, cracked side of the test samples was exposed to a chloride solution with a concentration of 165 g NaCl per litre. It was decided to use such a high chloride concentration in order to speed up the corrosion process. In order to obtain unidirectional ingress of the solution into the samples, the lower part of the samples was covered until a height of 1 cm with aluminum butyl tape in such a way that only the bottom side of the sample came into contact with the solution. Exposure of the samples to the chloride solution was performed by placing each sample in a separate plastic container with a lid. The top side of the samples was glued onto the lid and the solution level in each container was the same so that the submersion level was equal for all samples.

Further acceleration of the corrosion process was obtained by applying a potential difference over the length of the rebar in a similar way as performed in the study of Michel et al. [12]. Therefore, an electric connection was made with the rebar (working electrode) by means of an insulated copper wire which was attached to the rebar at one side of the sample and connected to the cathode of a DC regulator. As counter electrode, an stainless steel mesh was placed at the bottom of the container, inside the chloride solution. The counter electrode was connected to the anode of the DC regulator. The electrical connection between the working and the counter electrode was realized by the chloride solution in which the samples were submersed. The potential difference applied during the accelerated corrosion test, which lasted for 70 hours, was 15V. The applied potential difference will certainly exceed the natural condition, however, as the main aim of this study is to compare the efficiency of manual and autonomous crack healing and to find out to what extent the onset of corrosion can be delayed due to crack healing, we believe that the applied potential difference is acceptable.

2.5 Visual evaluation of the corroded rebars

During the accelerated corrosion test, rebars were inspected visually from the side of the samples at regular time steps. In order to allow clear visualization of the rebars, a circular opening was provided inside the plastic containers. In addition, these openings prevented condensation at the sides of the plastic containers which certainly had to be avoided for both the visual evaluation and for the X-ray radiographic measurements.
2.6 X-ray radiographic analysis

X-ray radiographic measurements were performed at the Centre for X-ray Tomography at Ghent University (UGCT) [13] using the cone beam setup of the HECTOR scanner [14] which was developed in collaboration with the company X-ray engineering BVBA (www.xre.be). An aluminum filter with a thickness of 1 mm was employed to block low-energetic X-rays at the source to reduce beam hardening. The X-ray tube provided a voltage of 200 kV with a power of 20 W. Ten subsequent radiographic scans, with an exposure time of 0.5 seconds, were taken when the samples were exposed for 2 hours, 4 hours, 6 hours, 24 hours, 30 hours and 70 hours to the accelerated corrosion test.

2.7 Gravimetrical measurements

After completing the accelerated corrosion test the corroded rebars were removed from the mortar samples and the mass loss due to corrosion was determined gravimetrically for each of the three samples of all test series. The mortar was removed by means of a hammer and the obtained rebars were cleaned before they were weighed by rinsing them. The initial mass of the rebars was calculated from the rebar length and diameter and the density of the steel reinforcement. The mass measured after corrosion was compared to the initial mass in order to determine the mass loss.

3 RESULTS

In Figure 2, pictures are shown from the side surface of one representative sample of each test series.

![Figure 2: Visual evaluation of the rebars during exposure to the accelerated corrosion test.](image-url)
It can be clearly seen that for the sample with the untreated crack, the sodium chloride solution immediately entered the crack when the sample was put in the solution and reached the position of the rebar. After two hours of exposure to the accelerated corrosion test, onset of corrosion was clearly noticed for the latter test series. For the series with manually and autonomously healed cracks the rebar was still intact. Two more hours later, even more corrosion products were noticed for the series with untreated cracks while the two other test series still remained undamaged. It is also clear from the pictures that for the test series with manually and autonomously healed cracks, the healing agent inside the crack still prevented the sodium chloride solution to reach the embedded rebar. For the series with the manually healed crack a dark stain is visible under the rebar, however, this is caused by the healing agent which partly covered the side of the sample after injection. After 30 hours exposure to the accelerated corrosion test, samples from all test series started to corrode, however, for the samples with the manually healed crack much less corrosion products were deposited compared to the samples from the two other test series.

In Figure 3 radiographic images are shown of one representative sample of each test series. The rebars, which highly attenuate the X-rays, are represented as a black circle and the mortar is seen as the grey shaded area around it. Underneath the rebars, a white line is noticed in the radiographs which is caused by the standardized crack in the samples. The dark grey narrow lines above the rebar which are noticed in the images are caused by the attenuation of the copper wire which was used to make the electrical contact for the accelerated corrosion test. At the start, the radiographic images look similar for all test series. For the samples with manually and autonomously healed cracks one would expect to see some healing agent inside the crack. However, this is not noticed from the radiographs as the polymeric healing agent hardly attenuates the X-rays and thus no discrimination can be made between an untreated crack and a crack filled with PU without additional image processing.

![Figure 3](image-url)

**Figure 3**: Radiographic image of one representative sample of each test series for different time steps during the accelerated corrosion test.
After two hours of accelerated corrosion, deposits of corrosion products can be seen as a dark grey stain underneath the rebar of the samples with the untreated crack. This is in agreement with the visual evaluation of the rebar at the side of the sample. After four hours of accelerated corrosion, additional information can be gained from the radiographs, next to the deposition of corrosion products, which can also be noticed externally. It is noticed that after four hours, pitting of the rebar starts which can be seen as a spot at the bottom of the rebar, close to the crack, which colors grey instead of black. This means that material disappears at this position because less attenuation of the X-rays is obtained. As the accelerated corrosion test evolves over time, more deposits are noticed and the pitting corrosion process continues. The state of the sample with the manually healed crack remains unchanged for up to 30 hours of accelerated corrosion. For the samples with the autonomously healed crack, from 24 hours onwards, some deposits of corrosion products can be noticed at the right hand side of the rebars. The two latter findings are again in agreement with the conclusions drawn from the visual evaluation.

The samples of all test series started to corrode heavily after 70 hours exposure to the accelerated corrosion test. A lot of deposited corrosion products can be seen and cracking of the mortar matrix is noticed as white zones appear within that area. While for the samples of each test series a reduction of the steel cross section is noticed, it can be seen that this reduction remains highest for the sample with the untreated crack which started to corrode much earlier compared to the two other test series.

In order to make a quantitative comparison between the resistance against corrosion for the different test series, the reduction in steel radius due to pitting corrosion was measured from the radiographs. Therefore, the grey values were plotted along a line which was oriented vertically at the position of the crack and went towards the center of the rebar. A clear decrease in grey value was noted when moving from a point in the sample where mortar or air was present to a point in the sample where the steel rebar was present. So based on the grey value level, a clear distinction could be made between the different materials present in the sample. However, as pitting corrosion started, a shift in the grey values towards the higher values was noticed over time. A value of ‘6000’ was taken as threshold value and the reduction in steel radius was calculated. The outcome of this calculation is represented in Figure 4.

The bar chart in Figure 4 represents the mean value obtained from the analysis of three samples. For the samples with the untreated crack, the steel radius is already decreased due to pitting after two hours exposure to the accelerated corrosion test. When corrosion continues, reduction of the steel radius goes on gradually. For the samples with manually and autonomously healed cracks, the reduction in steel radius is almost nihil and remains even slightly positive, due to the measurement accuracy, during the first hour of the accelerated corrosion test. Only for the samples with the autonomously healed crack, after 24 and 30 hours exposure to the accelerated corrosion test a slightly negative value, which means reduction of the steel radius, is noticed. As already seen before, after 70 hours exposure to the test, the samples of all test series started to corrode and to show a decrease in steel radius. However this decrease was highest for the samples with the untreated cracks.
After finishing the corrosion test and removing the corroded rebars from the mortar samples, the decrease in mass of the corroded rebars was determined by weighing. As already seen before, the samples of all test series started to corrode when the accelerated corrosion test was continued for 70 hours. This finding is confirmed by the graph shown in Figure 5. The embedded rebars of all test series showed some decrease in mass. This decrease was, however, significantly different for the series with the untreated cracks (35%) compared to the series with the manually (17%) healed cracks. The series with the autonomously healed cracks had a somewhat higher mass decrease (24%) compared to the samples with manually healed cracks, however, these mean values were not significantly different.

**Figure 4:** Reduction in reinforcement steel radius during the accelerated corrosion test.

**Figure 5:** Mass loss of the rebars after 70 hours exposure to the accelerated corrosion test. Error bars represent the standard error (n=3).

4 CONCLUSIONS

Autonomous healing of concrete cracks by means of encapsulated polyurethane is an efficient mechanism to prevent the ingress of aggressive agents into cracks and to delay the onset of corrosion.

In the current study, standardized cracks were introduced and the crack healing mechanism
was triggered at the moment the metal plates were removed from the mortar. However, it should be mentioned that previous research has shown that the proposed self-healing mechanism also works efficiently when realistic cracks are created [6].

In this and in previous studies glass capsules were used to encapsulate the healing agent. However, we would like to say that in ongoing research we are searching for encapsulation materials which have evolving properties. At first, these encapsulation materials should be flexible so they can be added to the concrete during mixing, while upon concrete hardening they should become more brittle so they easily break when cracks appear.

It can also be concluded that X-ray radiography is a very efficient technique to evaluate the status of a rebar inside a cementitious material. Pitting of the reinforcement bar could be clearly noticed on the radiographs as the X-ray attenuation and thus the grey values changed.

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PUMPABILITY OF SUSTAINABLE SCC MIXTURES

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Key words: Civil Engineering, Concrete Pumping, Concrete Pumpability, Self-Compacting Concrete

Abstract. Self-Compacting Concretes (SCC) are sustainable materials because they replace large amounts of cement by the use of low embodied energy and CO₂ mineral additions. The addition of such fine materials not only enhances the workability of the mix, but also enlarges the durability due to the reduction of intrinsic permeability. However, the addition of fine materials may alter the behaviour of the mix during pumping, making it shear-thickening and requiring unexpectedly high pumping pressures in some cases. These effects make necessary the development of some kind of quality control methodology for assessing the performance of these mixes during pumping. This paper presents preliminary results obtained in an ongoing experimental program on the behaviour of SCC mixes during pumping and early ages (EA); with the overall goal of identifying a set of parameters usable for quality control while pumping those mixes. Results obtained so far, suggest that the function linking pressure gradient with discharge rate is not linear for some SCC mixes, making not directly applicable QC methods based on the assumption of a linear behaviour. Results also show that the selected EA indicators allow a proper assessment of the changes which affect later SCC hardened performance.

1 INTRODUCTION

With few exceptions, pumping is currently the method of choice for concrete placement, due to its flexibility, high placement rates and ease of industrialization. However, predicting the pumping pressure needed for a given mix, installation and discharge rate is not straightforward[11]. Although in the daily practice it is assumed that pumping pressure is
correlated with workability\cite{2}\cite{3}, and therefore the slump test is used as Quality Control (QC) indicator, it is known that this is not true in all cases\cite{4}\cite{5}\cite{6}\cite{7}.

It is commonly admitted that classic concrete mixes in open channels follow Bingham’s constitutive law\cite{4}\cite{9}\cite{10}\cite{11}\cite{12}\cite{13}\cite{14}\cite{15}, which is of linear type. In pipes, the same model can be used. The constitutive law, together with the geometry of the pipe, allows estimating the speed of the fluid at any point. The integration of the resulting velocity field conducts to the Buckingham equation, which relates the discharge rate \( q \) with the pressure gradient \( p_x \) along the axis of the pipe; equation that under certain conditions can be solved and simplified, leading to a linear solution of the form \( p_x = k_1' + k_2' q \)\cite{5}\cite{8}. Coefficients \( k_1' \) and \( k_2' \) can be determined quite straightforwardly when the concrete pump is properly instrumented, and their value may be used as basis for an online pumping QC method with higher performance than the simple slump test.

Nevertheless, other mixes, among them SCC, may follow nonlinear constitutive laws\cite{11}\cite{13}\cite{15}. It has been proposed that Herschel-Bulkley’s law is the one that fits better the behaviour of most concrete mixes\cite{11}. Unfortunately, when using this law, the integration of the velocity field to obtain an analytical expression relating the pressure gradient and the discharge rate is not possible in general, although an approximated numerical integration can be performed\cite{13}.

Therefore, the simplifications leading to a linear expression for \( p_x \) cannot be applied, and in consequence the QC method proposed by the authors\cite{5}, based on the assumption of a linear behaviour, may not be appropriate to these types of mixes.

In order to propose a better but also simplified approach, an experimental program on SCC-type mixtures pumpability has been carried out to analyse the material behaviour during pumping, early ages and hardened performance. The main goal of this research is to identify the main parameters for accurate pumping quality control related both to the pumping process and the effects of pumping on the material performance at early ages (24 hours after casting)\cite{17}\cite{18}\cite{19}. While the first helps to control the process itself the second will allow predict if hardened concrete performance will be as designed\cite{17}\cite{18}.

The whole program is structured in three stages. The first step includes the mixtures design, the definition of the main flow equations and a laboratory experimental study to identify the main parameters to be controlled during and after pumping. In a second stage, a real scale testing setup will be instrumented and several SCC samples will be pumped. In the last part of the investigation, laboratory and real scale test results will be compared and the main parameters under study will be evaluated.

On this paper, focus was set on justifying the concrete flow modelling adopted as well as on showing the accuracy of pumping as well as EA indicators for deriving insights about the process itself and related the pumped concrete future performance.

2 MODELLING OF CONCRETE FLOW IN PIPES: PRESSURE-FLOW RELATIONS

Due to its heterogeneous nature, the mathematical modelling of concrete flow is quite complex. It is commonly accepted\cite{9}\cite{10} that when concrete flow in open channels -for example, when it is poured in a formwork- it can be modelled as a Bingham fluid, with shearing happening in most part of its mass.

A Bingham fluid is defined by the law:
\[ \tau = \tau_0 + \mu \dot{\gamma} \] (1)

Where \( \tau \) is the shear stress, \( \tau_0 \) the yield stress, \( \mu \) the dynamic viscosity and \( \dot{\gamma} \) the shear rate. The fundamental parameters -yield stress \( \tau_0 \) and dynamic viscosity \( \mu \) - of the Bingham model can be measured using concrete rheometers\(^9\)\(^{10}\)\(^{11}\)\(^{12}\).

However, when flowing under pressure in closed channels -pipes- concrete flows in a quite different way\(^1\). It has been observed experimentally in conventional vibrated concretes\(^3\)\(^6\)\(^{15}\) that shear occurs mainly in a thin limit layer in the interface between the wall of the pipe and the central core, which moves as an almost rigid solid. The limit layer acts as a lubricating film between the pipe wall and the concrete core, reducing notably the friction forces between them, and in fact allowing concrete to be pumped. In this case the relation between the pressure gradient needed to maintain a certain flow is mainly determined by the rheo-tribological properties of said limit layer.

On the other hand, recent work\(^13\)\(^{14}\) shows that other types of concretes, like SCC, may suffer shear in a significant part of their mass when pumped. In this case, not only the properties of the limit layer have effect on the pumping pressure, but also the rheological properties of the whole mass of concrete. Nevertheless, in both cases this same Bingham model can be used for computing the relation between pressure gradient and flow, although with some particularities that are discussed below.

### 2.1 Conventional concrete flow

When Bingham fluid flows in a pipe, shear only occurs in the outer layer of fluid, while the central part -the one closest to pipe’s axis- moves in a solid-like manner, without suffering shear. For these fluids, the pressure gradient and flow relation is defined in implicit form by Buckingham-Reiners equation:

\[ q = \frac{\pi a^4 p_x}{8 \mu} \left[ 1 - \frac{4}{3} \left( \frac{2\tau_0}{ap_x} \right) + \frac{1}{3} \left( \frac{2\tau_0}{ap_x} \right)^4 \right] \] (2)

where \( q \) is the volumetric flow, \( a \) is the radius of the pipe, \( p_x \) the pressure gradient (\( dp/dx \)).

### 2.2 Self-Compacting Concrete flow

In other types of concrete, and in especial in SCC mixes, it has been found\(^13\)\(^{14}\) that the thickness of the layer suffering shear is significant in relation with the radius of the pipe, and also the presence of a lubricating layer with properties different from that of the bulk of concrete mass. On the other hand, the mix may not behave as a Bingham fluid, but respond to another type of law, like Herschel-Bulkley’s\(^{11}\):

\[ \tau = \tau_0 + k \dot{\gamma}^n \] (3)

where \( k \) is consistency index and \( n \) the flow index (for \( n = 1 \), the above reduces to Bingham’s model). Depending on the value of parameter \( n \), shear-thinning or shear-thickening effects may appear. However, Herschel-Bulkley’s law has several issues that hinder its application\(^{15}\). Among them, it is the fact that the integration of the velocity field
does not conduct in general to an analytical solution, and convergence issues when performing numerical integration. On the other hand, shear-thinning or shear-thickening effects can be accounted for using a modified Bingham constitutive law, although leading to much more complex expressions than Buckingham’s equation\textsuperscript{[13]}.

2.3 Simplification of $p_x\text{-}q$ relationship

From the technological point of view, expressing the discharge rate $q$ as a function of the pressure gradient $p_x$ is not as convenient as the opposite, that is, expressing $p_x$ as a function of $q$.

In conventional concretes, where the thickness of the outer layer suffering shear -the so-called interface or lubricating layer- is much smaller than the radius of the pipe, equation (2) can be simplified to the linear expression\textsuperscript{[5][8]}:

$$p_x = k'_1 + k'_2 q$$ \hspace{1cm} (4)

where $k'_1$ and $k'_2$ can be expressed as function of the fundamental rheo-tribological parameters $\tau_{\text{oi}}$ and $\mu_i$ of the interface layer\textsuperscript{[8]}:

$$k'_1 = \frac{8\tau_{\text{oi}}}{4a}$$ \hspace{1cm} (5)

$$k'_2 = \frac{4\mu_i}{\pi a^4}$$

For concretes responding to other constitutive laws, the process conducting to establishing equation (4) cannot be applied, and it is much more complex establishing the relation between $p_x$ and $q$. Nevertheless, the equation (4) can be generalized adding a quadratic term:

$$p_x = k'_1 + k'_2 q + k'_3 q^2$$ \hspace{1cm} (6)

The coefficients of this last equation shall also be related to the fundamental rheological and rheo-tribological parameters of the mix, although this relation cannot be expressed in general in analytic form.

However, for practical QC purposes, both (4) and (6) equations are useful enough, as the values of parameters $k'_1$, $k'_2$ and eventually $k'_3$ can be measured quite easily if the concrete pump is instrumented, and used for comparing the pumping performance of mixes or detecting potential issues in certain installations. For example, a positive value in $k'_3$ indicates a shear-thickening behavior that may cause problems at high discharge rates. Conversely, negative values of $k'_3$ indicate a shear-thinning effect, which, if taken into account, could allow a safe reduction in pipe diameters or a higher discharge rate for a given diameter.

3 REAL SCALE TESTING

3.1 Mixtures Design

The components and mixture compositions used for the sample preparation are summarised in Table 1. The Reference mixture (M1) was adjusted in the laboratory to achieve
a minimum spread diameter of 600 mm (M2). Afterwards, the mixture was manufactured in an industrial mixer with 3 m$^3$ of capacity (MR1) that also required a water adjustment (MR2_0). This last sample was used in the full scale pumping test, were 10 different batches of the MR2_0 (MR2_1-MR2_10) sample were pumped. Two samples of batch 3 (MR2_3) were taken for early age (EA) monitoring and hardened properties evaluation: One before pumping and the other after pumping. In one of the batches another mix, dubbed MR3 was used, which was intentionally adjusted to test the limits of pumpability by changing the grading in the fraction (4-20), increasing the proportion of coarser aggregates and, consequently, decreasing the amount of sand.

### Table 1. Mixture compositions (kg/m3).

<table>
<thead>
<tr>
<th>Batch Material</th>
<th>Laboratory</th>
<th>Full scale</th>
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<tbody>
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<td></td>
<td>M1</td>
<td>M2</td>
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<td>CEM I 52,5 SR</td>
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<td>Gravel (4-20)</td>
<td>980</td>
<td>980</td>
</tr>
<tr>
<td>HRWRA</td>
<td>5.5</td>
<td>7</td>
</tr>
<tr>
<td>W/(c+FA)</td>
<td>0.38</td>
<td>0.40</td>
</tr>
</tbody>
</table>

### 3.2 Experimental methods

To verify the feasibility of the approach implemented for determining pumpability indicators$^{[5][6]}$ a test setup consisting of an standard stationary pump (PM1408D) and a pumping circuit, comprising an horizontal section of 10 m followed by a rubber hose, 3 m long, connected to a waste dump (Figure 1) was used. The pump was instrumented with pressure sensors and displacement transducers to allow monitoring pumping rate. All sensors were connected to a data recording device, acquiring the data in real time in a laptop computer. From the records so obtained, the values of volumetric flow and the steady state pumping pressure corresponding to that volumetric flow were extracted$^{[6]}$.

![Figure 1. Pumping experimental set-up](image)

Additionally, the flow ability of the fresh samples was tested with the Slump flow test
(According to ASTM C1611) and the environmental conditions were recorded (temperature – T- and relative humidity – RH-). Moreover, the fresh samples were placed in a plastic mould of 150x100x80 mm were internal temperature, p-wave Ultrasonic pulse velocity (UPV) and two capillary pressure sensors were used to monitor them during 8-14 hours[18].

Some mixtures were also monitored in a shrinkage channel of internal dimensions of 500x100x50 mm[17]. Free shrinkage under external conditions was measured on the samples placed some under controlled environmental conditions (MR1 and MR2_0) and other tested at the pumping site (MR2 batch 3 Un-pumped –MR2_3U- and Pumped —MR2_3P-). The samples at the laboratory were subjected to a 3 m/s airflow in order to amplify the dimensional instability that is related to the microstructure formation and the mechanical capacity development at EA[17].

The samples were air cured and were also tested hardened using non-destructive characterization techniques. UPV was measured on 100 mm samples and surface air permeability tests using the Figg method (Porositycope™)[19] and micro-porosimetry measured by nitrogen adsorption were carried out.

4 RESULTS

4.1 Pumping Tests

This section reports the results related with pump-ability, that is, the relationship between pressure and volumetric flow when pumping several batches of the mix under analysis (MR2_3). The pumping characteristics of this SCC mix were somehow different from that of previously tested conventional vibrated concrete[5].

While in some cases (for example Figure 2 (a)), a linear behaviour was observed, in others, most possibly due to inevitable batch-to-batch minute differences, a mild shear-thickening behaviour is observed in the following batch (Figure 2 (b)), suggesting a lack of pumpability robustness. Pressure gradient at 25 m³h⁻¹ was the same, but not between 8-12 m³h⁻¹. Anyway, pumping pressure was in the normal range to be expected for this type of SCC.

The shear-thickening behaviour is more evident when testing mix MR3, which needed much higher pumping pressure gradient was for the same discharge rate, thus showing a lower pumpability.

![Figure 2. Pressure gradient / discharge rate relationship for a DN125 pipe, mix MR2_X](image)
4.2 Early Age Tests

This section reports the EA performance of the SCC mixtures under real external conditions: MR2, Un-Pumped and Pumped.

Figure 4 summarises the EA parameters measured on the SCC sample manufactured (MR2_0). The sample was monitored in a room with controlled environmental conditions because the extreme external conditions did not allow a complete monitoring. The temperature in the room was set on 26°C, although the average temperature of the 15 hours was 24.5°C, the initial temperature was 32°C and the minimum value was 21°C. The extremely high initial temperature of the sample (48.7°C) and the large difference regarding the environmental temperature produced an unusual curve shape for this type of mixtures. Instead, the thermal difference between sample and environment was plotted, because is easier to identify the change in the tendency that corresponds to the beginning of the cement hydration (fairly initial setting time)[17].

Three key-points could be identified: a) Beginning of the hydration process and solid microstructure formation at 50 minutes; b) SCC stiffening and air entrainment in the pore network at 6 hours; c) End of stiffening process, drainage of water available in the pore network and dimensional stability at 8 hours.
The un-pumped sample (MR2_3U) was taken from the concrete after mixing and the experimental results of temperature, capillary pressure and UPV are plotted in Figure 5. Two key-points were identified: a) The beginning of the hydration process and the initial formation of the solid structure was identified at 2 hours; b) SCC stiffening at 6 hours.

Figure 5. Early age parameters of the MR2_3U SCC mixture.

Figure 6 summarises the experimental results of the EA monitoring of the Pumped sample (MR2_3P). This sample was taken from the same batch as the un-pumped sample and both samples were submitted to the same conditions. The experimental setup included temperature, capillary pressure, UPV and dimensional stability. Three key-points were identified: a) The dimensional instability and the increase of the negative pressure on one of the capillary sensors were measured at 30 minutes; b) The beginning of the cement hydration and the solid microstructure formation occurred at 1 hour and 40 minutes after casting; c) SCC stiffening was recorded at 6 hours.

Figure 6. Early age parameters of the MR2_3P SCC mixture.
4.3 Hardened State Tests

The hardened samples were tested using UPV, air permeability non-destructive techniques. The experimental results are reported in Table 2. The UPV values were similar in all the samples, although the increase of water to cement ratio of the compositions led to a reduction of the UPV. The same occurred when the sample was pumped.

Table 2. Hardened parameters of SCC samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>UPV (m/s)</th>
<th>Air permeability (seg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M 1</td>
<td>3766</td>
<td>--</td>
</tr>
<tr>
<td>M 2</td>
<td>3543</td>
<td>16.3</td>
</tr>
<tr>
<td>MR 1</td>
<td>4110</td>
<td>7.4</td>
</tr>
<tr>
<td>MR 2_0</td>
<td>3766</td>
<td>29.0</td>
</tr>
<tr>
<td>MR2_2U</td>
<td>4018</td>
<td>55.6</td>
</tr>
<tr>
<td>MR2_2P</td>
<td>3689</td>
<td>14.3</td>
</tr>
</tbody>
</table>

The air permeability did not follow the same pattern. The loss of flow ability due to the hot conditions observed when the sample was produced at the concrete plant (MR1) compared to that produced at the laboratory (M2) can explain the increase of air permeability associated to the reduction of the Figg time. The adjustment of the mixture to achieve larger flow ability explains the reduction of permeability. It must be highlighted that the samples were not compacted. The samples taken in the full scale initial test pointed a reduction of the Figg time due to the pumping process.

Figure 7. Microporosity distribution on SCC hardened samples.
Figure 7 plots the microporosity distribution of the three real scale SCC samples produced: One mix at the concrete plant and two transported to the construction site (un-pumped and pumped). Transportation to the construction site reduced the amount of capillary porosity (between 100 and 1000), while pumping again increased it.

5 CONCLUSIONS

- The samples analysed exhibited a behaviour ranging from linear to slightly rheopectic, not unexpected in SCC mixes. Although in the range of flows considered (up to 50% of the nominal flow of the pump), pumping pressure was within acceptable limits and pressure needed for pumping at higher rates may be unexpectedly high, especially if mix design is not corrected.

- The approach proposed for assessing the pumping performance of SCC mixes is giving promising results, as it is able of detecting potential issues derived from shear-thickening behaviour or the need of improving mix design, apparent in the noticeable increase of pressure gradient and shear-thickening for mix MR3.

- The three SCC samples monitored showed a similar performance at early ages. In all cases, the beginning of the solid microstructure formation (fairly initial setting) followed the thermal stabilization. The material stiffening occurred around 6 hours after casting. Therefore, neither the environmental conditions tested nor the cast in place procedure (pumped and un-pumped samples) substantially influenced the mechanical capacity development of the material at early age.

- However, both the environmental conditions and the cast-in-place procedure influenced the beginning of the hydration process, the dimensional stability and the solid microstructure formation during the first 6 hours. Pumping also speeded up the reaction and an increase on the sample temperature was recorded.

- Two main effects of pumping were identified: on one hand the temperature increase, which would explain the speed up of the process, and on the other hand the lack of enough water in the pore network which brought forward the beginning of the dimensional instability.

- For the compositions considered, the environmental conditions and the casting procedures in this study, it can be said that the beginning of the chemical-physical changes could happened as soon as 40 minutes after casting. The solid microstructure formation could start between 1.5 and 2 hours. Higher temperature of the components or the environment would speed up the process.

- The hardened characterization also showed the effect of the environmental conditions and pumping. The samples with lower flow ability showed lower UPV and larger air permeability. When the samples taken from the full scale test were compared, it was observed that the pumped sample had a lower UPV, related to a lower concrete stiffness, and the air permeability was larger, which could affect the long term performance of the material.

6 ACKNOWLEDGEMENTS

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REFERENCES


PUNCHING SHEAR STRENGTH OF CONCRETE SLABS REINFORCED WITH RECYCLED STEEL FIBRES FROM WASTE TIRES

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Key words: Fibre Reinforced Concrete, Flat Slab, Punching Shear, Recycled Tires, Steel Fibres, Experimental investigation

Abstract: The increasing amount of waste tires around the world and their landfilling present a continuous problem to be solved. Currently, in some countries and unions of countries there are directives that prohibit landfilling and imply that constituent materials of waste tires must be recycled.

The main goal of the presented research was to test the ability of sorted recycled tire steel fibres to perform in the same way as industrial steel fibres in flat slabs loaded in punching. Research included investigation of behavior of reinforced concrete slabs made of several mixes including plain concrete, mix with industrial steel fibres and mix with industrial and recycled steel fibres – hybrid fibres. Nine small-scale flat slabs were tested and results indicate that the use of hybrid fibres can improve the punching shear strength and deformation capacity of concrete slabs. Furthermore, slabs reinforced with hybrid fibres showed better behavior when compared to slabs reinforced only with industrial steel fibres.

1 INTRODUCTION

Flat slabs, i.e. slabs supported directly on columns, without beams between the columns, generally transfer a significant concentrated load that affects a relatively small area. The issues in design of flat slabs are mostly governed by quite high deflections in serviceability conditions and punching shear failure in the ultimate limit state [1]. The punching failure mode is generally a brittle failure mode that can lead to progressive collapses because of the adversely redistributed loads. Finally, such an event can lead to the load bearing capacity loss of the entire structure [2]. There are many reinforcement solutions available for increasing the
punching shear capacity, such as bent-up bars, stirrups, double headed shear studs, structural steel shear heads, shear bands, lattice shear reinforcement and UFO punching preventers [3]. Furthermore, there are solutions available for increasing the punching shear capacity of existing flat slab systems, i.e. post-installed punching shear reinforcement [4]. In the last couple of decades, the use of steel fibre reinforced concrete (SFRC) for increasing the punching shear capacity has been researched (e.g. [5], [6], [7], [8], [9], [10]). Researches in these studies generally concluded that the use of fibre reinforcement is a viable alternative for increasing the punching shear capacity of slab-column connections. Furthermore, they noticed an increase in deformational capacity of SFRC slabs which was explained by the bridging effect of the fibres after the concrete matrix cracked. Overall, it can be concluded that steel fibres are able to maintain the integrity of the slabs, i.e. to mitigate or even prevent the brittle failure caused by punching shear [11].

Besides industrially manufactured steel fibres (MSF), recycled tire steel fibres (RTSF) as a by-product in the recycling process of waste car and truck tires came in focus of the researchers in the last decade (e.g. [12], [13], [14], [15], [16], [17], [18], [19]). The main subject in most of the studies in this area were material properties of concrete reinforced with RTSF or with mixed MSF and RTSF (hybrid fibres – HF; hybrid steel fibre reinforced concrete – HFRC). The latter is assessed as necessary as RTSF alone are not able to bridge macro cracks because of their relatively short length. Some studies were also performed in order to assess the practical application of RTSF in the concrete industry (e.g. [20], [21], [22]). It must be noted that one should differentiate the unsorted RTSF and sorted (classified) RTSF with known geometrical and mechanical properties.

This paper presents an experimental study to evaluate the effect of manufactured and hybrid (mixed manufactured and sorted recycled) steel fibres on the punching shear resistance and deformation capacity of flat slabs. A total of nine small-scale flat slabs were tested using three different concrete mixes: plain concrete mix, SFRC mix and HSFRC. Experimental results are analysed and performance of steel fibres for this kind of application is assessed.

2 DESCRIPTION OF THE EXPERIMENTAL INVESTIGATION

2.1 Materials and slab specimens

Three concrete mixes were used in the presented research: plain concrete mix (mix id PC), SFRC mix with 40 kg of manufactured steel fibres per m³ of concrete (mix id 40M0R) and HSFRC mix with 20 kg of manufactured and 20 kg of recycled steel fibres per m³ of concrete (mix id 20M20R). The concrete mix composition of the mentioned mixes is shown in Table 1.

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>Cement (kg/m³)</th>
<th>Water (l/m³)</th>
<th>w/c ratio</th>
<th>Aggregate, total (kg/m³)</th>
<th>Superplasticizer (kg/m³)</th>
<th>MSF (kg/m³)</th>
<th>RTSF (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>370</td>
<td>170</td>
<td>0.46</td>
<td>1840</td>
<td>2.22</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>40M0R</td>
<td>370</td>
<td>170</td>
<td>0.46</td>
<td>1790</td>
<td>2.22</td>
<td>40</td>
<td>0</td>
</tr>
<tr>
<td>20M20R</td>
<td></td>
<td></td>
<td></td>
<td>1790</td>
<td>2.22</td>
<td>20</td>
<td>20</td>
</tr>
</tbody>
</table>
The following river aggregate fractions were used: 0 – 4 mm crushed, 4 – 8 mm and 8 – 16 mm. The cement type CEM II/A-M 42,5 N was used. Also, superplasticizer was used in order to achieve the target consistency class S4. Properties of the used manufactured and sorted recycled steel fibres are shown in Table 2. Both fibre types are also shown in Figure 1.

Table 2: Properties of the manufactured and recycled fibres used in the research

<table>
<thead>
<tr>
<th>Fibre type</th>
<th>Geometrical shape</th>
<th>Producer</th>
<th>Length L (mm)</th>
<th>Diameter D (mm)</th>
<th>Aspect ratio L/D</th>
<th>Tensile strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>manufactured</td>
<td>straight with hooked ends</td>
<td>ArcelorMittal</td>
<td>35.0 (+2/-3)</td>
<td>0.55 (+0.04)</td>
<td>64</td>
<td>1200</td>
</tr>
<tr>
<td>recycled, sorted</td>
<td>irregular (wavy)</td>
<td>Twincon</td>
<td>20.0 (+2.0)</td>
<td>0.15 (+0.04)</td>
<td>166</td>
<td>2850</td>
</tr>
</tbody>
</table>

Figure 1: Manufactured steel fibres (MF) on the left and sorted recycled steel fibres (RTSF) on the right

Figure 2: Reinforcement of the slabs

Cube specimens (150 mm) were used to determine the compressive strength \( f_c \) of concrete according to the standard EN 12390-3:2009/AC:2011 [23]. Cylindrical specimens (150 mm x 300 mm) were used to determine the splitting tensile strength \( f_{ct} \) of concrete according to the standard EN 12390-6:2009 [24]. The obtained average concrete strengths are shown in Table 3. Steel reinforcement bars of 14 mm in diameter and with yield strength \( R_{p0.2} = 560 \) MPa, tensile strength \( R_m = 637 \) MPa and elastic modulus \( E_s = 200 \) GPa was used. Mechanical properties of the used reinforcement bars were determined experimentally according to the standard EN ISO 15630-1:2010 [25].

Each of the three concrete mixes was used to cast three identical slab specimens. Therefore, a total of nine slabs with dimensions of 1.5 m x 1.5 m x 0.125 m were tested. Each slab was casted from different batch, so concrete properties were determined for batch
separately. Adequate compaction was achieved by using a table vibrator. All the data regarding each particular slab are shown in Table 3. Tensile reinforcement ratio was 1.5% in all slabs and rebars of 14 mm in diameter were spaced at 11 cm. The steel fabric mesh consisting of 5.5 mm rebars spaced at 15 cm in both directions was placed on the compression side. The protective layer of concrete was 1.5 cm on the tension side and 2.0 cm on the compression side, while the effective depth of the slabs amounted to 9.6 cm. The arrangement of slabs reinforcement in plan and cross-section is shown in Figure 2.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Concrete mix</th>
<th>Concrete compressive strength $f_c$ (MPa)</th>
<th>Concrete tensile splitting strength $f_{ct}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-1</td>
<td>PC</td>
<td>50.4</td>
<td>3.5</td>
</tr>
<tr>
<td>S2-2</td>
<td></td>
<td>52.8</td>
<td>3.2</td>
</tr>
<tr>
<td>S2-3</td>
<td></td>
<td>54.1</td>
<td>3.1</td>
</tr>
<tr>
<td>S3-1</td>
<td>40M0R</td>
<td>43.1</td>
<td>3.0</td>
</tr>
<tr>
<td>S3-2</td>
<td></td>
<td>42.3</td>
<td>3.3</td>
</tr>
<tr>
<td>S3-3</td>
<td></td>
<td>44.8</td>
<td>2.9</td>
</tr>
<tr>
<td>S4-1</td>
<td>20M20R</td>
<td>46.9</td>
<td>3.8</td>
</tr>
<tr>
<td>S4-2</td>
<td></td>
<td>47.9</td>
<td>4.1</td>
</tr>
<tr>
<td>S4-3</td>
<td></td>
<td>47.0</td>
<td>4.1</td>
</tr>
</tbody>
</table>

2.2 Test setup and measurements

Slabs were supported by eight discrete spatially hinged supports placed on steel columns of circular cross-section, which were distributed in the radius of 75 cm from the slab center. The slabs were loaded in displacement control mode ($v = 0.6$ mm/min) using a hydraulic static-dynamic testing machine, 600 kN in capacity. Load was applied in the center of the slabs via steel plate of dimensions 13 cm x 13 cm x 3 cm. To prevent stress concentration in the edges of this plate, a thin HDF pad (high-density fiberboard of approx. 850 kg/m$^3$) of equal plan dimensions was placed between the steel plate and the concrete slab. Figure 3 shows the described test setup.

The loading force and various other slab behavior parameters were continuously measured during the testing. Measurement positions are shown in Figure 4. Twelve linear variable differential transformers (LVDTs) were used for measurement of vertical displacements on top and bottom surfaces of the slabs. On the top side, displacements were measured along two axes in one-eighths and one-fourths of the span. To control the displacement symmetry, displacements were also measured in one-fourths of additional two axes. On the bottom side, displacements were measured along one axis only. Since the LVDTs for displacement measurement were placed on exact distances between each other, these measurements were also used for the calculation of slabs slope. Measurement points were located in one-fourths, one-eighths and one-half of the span. Radial strains were measured on the compression side of the slabs using LVDTs placed on measuring base of 100 mm. Sensors were located along two
axis, two sensors per each axis. The occurrence of decompression (reduction of compression strain) on these measurement points is an indicator of development of shear cracks, i.e. of the oncoming punching shear failure.

**Figure 3: Test setup**

![Test setup image]

**Figure 4: Measurement points on slabs**

![Measurement points on slabs image]

3 PUNCHING TESTS RESULTS AND DISCUSSION

3.1 Load – deflection response

Figure 5 represents the load versus displacement diagrams for all the tested slabs. Displacement marked D0A was measured in the center of the bottom side of the slabs (see Figure 4). Table 4 shows relevant obtained test results, where $F_{\text{max}}$ is the maximum applied...
load (punching load) and \( \delta_{F_{\text{max}}} \) is the corresponding D0A displacement, \( F_{\text{max, ratio}} \) is the ratio between maximum average load for SFRC (HSFRC) slabs and RC slabs, \( \delta_{F_{\text{max, ratio}}} \) is the ratio between average D0A displacement at maximum load for SFRC (HSFRC) slabs and RC slabs. The observed failure mode for each slab is also included in this table.

**Table 4: Maximum load and deflection results obtained in the punching tests**

<table>
<thead>
<tr>
<th>Slab</th>
<th>( F_{\text{max}} ) (kN)</th>
<th>( F_{\text{max, av}} ) (kN)</th>
<th>( \delta_{F_{\text{max}}} ) (mm)</th>
<th>( \delta_{F_{\text{max, av}}} ) (mm)</th>
<th>( F_{\text{max, ratio}} )</th>
<th>( \delta_{F_{\text{max, ratio}}} )</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-1</td>
<td>345.16</td>
<td>325.62</td>
<td>14.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2-2</td>
<td>303.33</td>
<td>295.00</td>
<td>11.54</td>
<td>13.04</td>
<td>-</td>
<td>-</td>
<td>Punching</td>
</tr>
<tr>
<td>S2-3</td>
<td>328.37</td>
<td></td>
<td>13.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3-1</td>
<td>361.69</td>
<td></td>
<td>14.38</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3-2</td>
<td>359.86</td>
<td>357.89</td>
<td>14.40</td>
<td>14.27</td>
<td>1.10</td>
<td>1.09</td>
<td>Punching</td>
</tr>
<tr>
<td>S3-3</td>
<td>352.14</td>
<td></td>
<td>14.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4-1</td>
<td>376.55</td>
<td></td>
<td>15.80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4-2</td>
<td>341.96</td>
<td>367.10</td>
<td>14.33</td>
<td>14.94</td>
<td>1.13</td>
<td>1.15</td>
<td>Punching</td>
</tr>
<tr>
<td>S4-3</td>
<td>382.80</td>
<td></td>
<td>14.69</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 5:** Load – center displacement diagrams for all the tested slabs
The presented results show a clear increase of stiffness and punching capacity with the introduction of steel fibres in the concrete mix. Punching load was higher for both SFRC and HSFRC slabs when compared to referent RC slabs, in average for 10 % and 13 %, respectively. However, due to the relative low content of fibres in this research (40 kg per m³ of concrete), failure of the slabs in punching mode was not prevented.

In terms of displacement at maximum (punching) load, similar trends were observed as in case of punching capacity of the slabs. This displacement increased for 9 % and 15 % in case of SFRC and HSFRC slabs when compared to referent RC slabs.

Generally, it is worth noting that HSFRC slabs showed better results than the SFRC slabs. These results evidence that sorted recycled tire steel fibres can be used as an equivalent replacement for manufactured steel fibres. Of course, further extensive research is needed in order to study larger spectrum of different HSFRC mixes.

3.2 Load – strain and load – slope response

The following Figure 6 and Figure 7 represent load versus strain (mark RD) and load versus slope (mark Ψ) diagrams for all the tested slabs. Position of these measurement points can be seen in Figure 4. Concrete compressive strain decrease presents the advancement of shear cracks in the slab, i.e. the announcement of the oncoming punching failure.

![Figure 6: Load – RD1C strain diagrams for all the tested slabs](image-url)
Regarding the measured slopes, it is visible that SFRC and HSFRC slabs had bigger slopes (higher deformation capacity) at maximum load, when compared to referent RC slabs. Again, note that HSFRC slabs showed better behavior in this respect than the SFRC slabs.

![Load – ΨG 2B-3B slope diagrams for all the tested slabs](image)

### 3.3 Inspection of the slabs after failure

After failure, each slab was inspected and cracks on the tensile surface were marked in order to study the influence of fibres on crack pattern. Furthermore, one slab per each group (concrete mix) was saw-cut in order to study the influence of fibres on the shear crack inclination. Figure 8 shows one slab per each group with its crack pattern where it can be seen that these patterns are somewhat similar, but concrete protective layer on SFRC slab and HFRC slab remained integral unlike the one on the PC slab.

![Cracking pattern for slabs (form left to right): S2-3, S3-3, S4-2](image)
Figure 9 shows comparison of shear crack inclination between PC, SFRC and HSFRC slab after the slabs were saw cut. Shear crack angles were approximately 36° for PC slab, 24° for SFRC slab and 27° for HSFRC slab. From this data, it is visible that fibres have a tendency of decreasing the shear crack angle and moving it away from the column, i.e. including a larger portion of concrete to resist the punching action.

4 CONCLUSIONS

In this study, the results of punching shear tests carried out on flat slabs with different concrete mixes (PC – plain concrete, SFRC – steel fibre reinforced concrete, HSFRC – hybrid steel fibre reinforced concrete) are presented and discussed. Three small-scale slabs were tested per each concrete mix. SFRC mix included 40 kg of manufactured fibres per m$^3$ of concrete while HSFRC mix included 20 kg of manufactured and 20 kg of sorted recycled tire steel fibres (RTSF) per m$^3$ of concrete.

Steel fibres in SFRC and HSFRC slabs improved the punching shear resistance when compared to PC slabs. In addition, fibres caused higher stiffness and deformation capacity of the mentioned FRC slabs. Furthermore, although the crack pattern was somewhat similar in all types of slabs, slabs with FRC remained integral while the concrete protective layer in PC slabs fell apart in some regions of the slabs. Also, after the failure, some of the tested slabs were saw cut in order to study the influence of fibers on the angle of the shear crack. This
angle was smaller (relative to the horizontal) in case of SFRC and HSFRC slabs which indicates that fibres have a tendency of moving the shear crack away from the column, i.e. including a larger portion of concrete to resist the punching action.

Slabs with HSFRC showed better behavior than the SFRC slabs in many aspects that were analyzed in this research. Therefore, sorted RTSF have proven to be a quality replacement for a certain amount of manufactured steel fibres in FRC, i.e. to be an economical, ecological and sustainable alternative for the manufactured steel fibres. Popularization of this type of sustainable concrete mixes will help finding and developing new applications for RTSF in the concrete industry. Hence, extensive research is still necessary in order to study the performance of recycled tire steel fibres on a large spectrum of different HSFRC mixes and applications.

ACKNOWLEDGEMENTS
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Robust Design and Durability of CO₂-reduced Concrete with high Amount of Supplementary Cementitious Materials

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Key words: performance based design concept, green concrete, durability, robustness, water efficiency, granular optimization

Abstract. The current paper introduces a new innovative concept for a performance-based design of Green-Concrete. Considering cement production causes about 5-7 % of anthropogenic greenhouse-gases, new concepts are required to reduce CO₂-burden in concrete. While several approaches to follow greener production routes in cement production already exist, there are further strategies to reduce CO₂-burden by partly replacing cement content by supplementary cementitious materials (SCM) or fillers in concrete necessary. These strategies bases on replacement of cement content in concrete by SCM, while loss of concrete performance can be partly compensated by reducing water content, through optimizing solid composition and increasing superplasticizer (SP) concentration. The step wise approach of the described concept enables the user to choose the raw materials by their availability and performance (workability, mechanical properties and durability). One main challenge is the description of concrete performance, due to concrete with high amount of SCM doesn’t follow usually used criteria like strength, w/c-ratio or cement content to ensure durability. Therefor new performance curves are introduced, which enables the user to characterize the performance of SCM in concrete and choose concrete composition by evaluating the necessary cement content for an application task. Due to higher level of optimization concrete with high amount of SCM are supposed to react more sensitive to unexpected changes in temperature and water content. Therefore initial tests have been carried out to characterize robustness of developed concrete compositions compared to ordinary concrete.

1 INTRODUCTION

The concrete production is CO₂-intensive, which is mainly caused by cement clinker production [1]. Considering climate goals and availability of resources the reduction and efficient use of cement clinker in concrete is a main challenge for concrete research in future. Due to several investigation projects in the past one powerful and flexible solution for the reduction of cement clinker content in concrete has been pointed out by replacing cement content by Supplementary cementitious materials or inert fillers (for shorter description in the current paper both will be meant by SCM) [2, 3, 4]. Due to lower reactivity of SCM a loss of
performance is expected with increased reduction of cement. Therefore, the reduction of water by using higher amount of superplasticizer (SP) and granular optimization of solid composition are used to partly compensate loss of performance. While concrete is a cheap mass product the chosen strategy has to consider the availability of raw materials, if a significant reduction of CO$_2$ should be aimed. In this case the strategy needs to react in a flexible way, if changes in raw materials (cement type, type and amount of SCM) occur.

In current paper the basic approaches and a new strategy for the design of concrete with high amount of SCM is described. The method considers the availability of raw materials and their interaction in concrete for performance and optimizes the concrete composition by quick and easy applicable mortar tests. Due to high reduction level of cement the resulting concrete composition doesn’t fit to conventional rules to predict sufficient durability, especially cement content. Additionally the conventional models and correlation based on strength and w/c-ratio doesn’t fit for new concrete with high amount of SCM. Thus, new strategies to describe performance in concrete like the concept of equivalent concrete performance need to be used. In the current paper, the results of comprehensive durability tests are shown. The resulting performance is described by performance curves which can be used to evaluate the potential of SCM for cement reduction in concrete. Applying requirements of a practical application task, the necessary cement content can be determined, additionally.

Since the resulting concrete compositions inhale low water and cement content and composition is optimized, they are supposed to react in sensitive way, if unexpected differences in water content and variations in temperatures occur. Thus, the current paper describes initial test results for selected concrete with high amount of SCM compared to conventional concrete composition.

2 DESIGN BASICS

2.1 availability of raw material and performance

While concrete is a cheap mass product, the production has to be based on the type, quality and mass of locally available raw materials [5]. If further transport and costs are required for selected ones, the potential is questionable. In consequence, the availability of raw materials is a key parameter for the design of CO$_2$-reduced concrete. Already today high amount of conventional SCM (fly ash or granular groundblast furnace slag) is used. But resources of high reactive SCM are limited [5]. Thus make it necessary to use further alternative SCM.

Due to lower reactivity of SCM the cement replacement is limited to a necessary level for the required performance of an application task. To additionally increase the reduction of cement content a further optimization of composition to lower water demand and the efficient use of water for workability can be used.

2.1 use of high amounts of superplasticizer

One key solution is an increased use of superplasticizer (SP). Due to the dispersion of fine particles water demand of the mixture can be reduced without losing workability. Thus, an increased solid concentration can be achieved, while excess water increases waterfilm thickness around particles. Further investigations show the influence of the dispersion effect by measuring the liquid demand of a solid composition for varying SP concentration [6].
2.2 granular optimization of mortar

A second key solution is a granulometric optimization of solid composition, which also leads to a lower water demand. Several strategies are known to increase the packing density by an optimization of the granulometric properties of the grading curves, e.g. [2, 4]. But recent tests at the Institute of Building Materials Science at Leibniz University of Hannover show that there is a more practicable method for optimizing the efficient use of water [7].

Figure 1 shows the relation between different optimization strategies for a mortar composition. An optimization of packing density fills the cavity volume and the water content can be reduced for the excess water. At the same time, points of contact between particles are increasing, which causes an enhanced viscosity. In that case, it is obvious that the coarse aggregates can hardly move next to each other (mortar level). To improve the workability, an excess of fine particles is a powerful solution to create a buffering layer, which could provide a larger distance between the coarse particles, while maintaining water content.

The addition of fine particles leads to an increased water demand due to higher specific surface (paste), which is partly compensated by a decreasing demand of water film thickness because of the buffering layer (mainly mortar). This works like a ball bearing between coarse grains. The effect disappears, when the buffering layer has reached a specific thickness. The hindrance of movability between the coarse particles is low. The additional buffer is not able to compensate an addition of water demand caused by increasing surface [7].

In addition to increased movability, the larger surface helps to buffer water in a thinner water layer, if undefined variations of water dosage occur. Thus, the robustness may increase.

The model results in a new method for optimizing water efficiency of mortar compositions for the design of high performance concrete. Recent comparative test results show a better particle movability of such mixtures by using same amount of liquid (SP and water). In current investigation, the method is used to optimize mortar compositions by determining binder to sand ratio.
3 PERFORMANCE BASED DESIGN CONCEPT

3.1 Design Strategy

Considering the before mentioned design principles, a new performance based design concept has been developed [9]. The design concept is systematically divided in four steps:

- Choosing locally available raw materials, especially binder components (cement and SCM) and steps of variation in binder composition
- Optimizing granular mortar composition for water-efficiency by using defined binder composition
- Optimizing superplasticizer concentration near to saturation point in mortar composition
- Evaluating mortar and concrete performance in performance tests for mechanical properties und durability, by using performance curves.

The stepwise approach enables to consider the availability of raw materials and the interaction of solid and fluid component. Using a systematical, stepwise reduction of cement content, the performance of concrete can be described by substitution level of concrete.

The use of concept will be shown within application.

3.2 Application

In current investigation, new concrete with reduced CO₂-burden have been developed following the described design steps [8,9]. There for, locally available raw materials were chosen. In first step, a high reactive ordinary Portland cement CEM I 52,5 R is combined with different conventionally used and alternative SCM like fly ash (FA), trass powder (TR) or limestone powder (LS). The cement content is partly reduced by one of chosen SCM in steps of 20 Vol.-% from 100 Vol.-% clinker to 20 Vol.-% clinker and 80 Vol.-% SCM. The different combinations are defined as new binder and added to an initial solid mortar composition with locally available sand (max. grain size: 2mm).

In the next step, the optimal binder content for the given sand has been evaluated by optimizing volumetric binder to sand ratio (b/s) in mortar, using the method for increasing the

Figure 2: the four step optimization process for mortar compositions (left) and the resulting graph for granular optimization (right); here shown for pure OPC and sand

In the next step, the optimal binder content for the given sand has been evaluated by optimizing volumetric binder to sand ratio (b/s) in mortar, using the method for increasing the
water-efficiency of mortar composition [7]. Therefore, quick and easy mortar tests are carried out by using the slump flow for indicator of movability. First, a liquid (combination of water and superplasticizer (SP) in ratio of 20 % SP/w) is dosed for a slump flow with Hagerman-cone of 20-25 cm. Due to high concentration of SP optimal dispersion of fine particles is reached, like it is aspired by using higher concentration of SP in strategies for CO₂-reduced concrete. Thus, the application of high SP-concentration ensures the consideration of SP-influence on optimized b/s [6]. After reaching targeted slump flow, solid and liquid content is held up constant while b/s is varied in steps of 10 kg/m³. Measuring slump flow after every step of variation and drawing results over b/s leads to new movability curves, see fig. 2.

The peak of curve describes the optimal b/s for water-efficiency and depends on combination of type and volumetric ratio of cement and SCM in binder, fig. 3. Further the chosen type of sand can also significantly influence the resulting b/s [7].

Figure 3: Resulting graphs for granular optimization of binder / sand ratio for:
- binder compositions with (a) OPC and fly ash, (b) OPC and trass powder

Figure 4: liquid demand of different solid mortar compositions by increasing SP concentration

In next step, the defined solid mortar composition is combined with different liquid (combinations of SP and water) in order to evaluate the compatibility of SP and solid
composition for optimized flowability, viscosity and slump flow over time of handling. The water demand for a specific slump flow can be lowered by increasing SP-concentration. With further addition of SP the effect of dispersion decreases since only little efforts are achieved. In that case, the saturation point is reached. For the displayed SP-type an effective SP-concentration is achieved within a dosage between 2.5 %/w and 5 %/w. Both concentrations are further used for the current investigation.

In the last step concrete is designed by using the optimized mortar compositions and choosing a suitable grading curve for gravel. Table 1 shows a selection of resulting concrete.

Table 1: overview of resulting concrete compositions (concrete for robustness tests marked with *)

<table>
<thead>
<tr>
<th>Z/ZS</th>
<th>Ref*</th>
<th>FA</th>
<th>limestone</th>
<th>Trass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vol-%</td>
<td>22.5/78.5</td>
<td>60/40</td>
<td>40/60</td>
<td>20/80*</td>
</tr>
<tr>
<td>type of cement</td>
<td>-</td>
<td>CEM</td>
<td>CEM I 52.5 R</td>
<td></td>
</tr>
<tr>
<td>cement content</td>
<td>kg/m³</td>
<td>290</td>
<td>283</td>
<td>204</td>
</tr>
<tr>
<td>type of SCM</td>
<td>-</td>
<td>Fly ash</td>
<td>Limestone</td>
<td>Trass powder</td>
</tr>
<tr>
<td>SCM content</td>
<td>kg/m³</td>
<td>60</td>
<td>131</td>
<td>212</td>
</tr>
<tr>
<td>Aggregates</td>
<td>kg/m³</td>
<td>1766</td>
<td>1766</td>
<td>1778</td>
</tr>
<tr>
<td>Water</td>
<td>kg/m³</td>
<td>175</td>
<td>142</td>
<td>125</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>kg/m³</td>
<td>2.8</td>
<td>6.8</td>
<td>6.3</td>
</tr>
<tr>
<td>w/b</td>
<td>-</td>
<td>0.5</td>
<td>0.34</td>
<td>0.31</td>
</tr>
<tr>
<td>w/c</td>
<td>-</td>
<td>0.6</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>SP/w</td>
<td>%</td>
<td>1.6</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>consistency</td>
<td>Cm</td>
<td>46</td>
<td>50</td>
<td>53</td>
</tr>
<tr>
<td>Density</td>
<td>kg/dm³</td>
<td>2.32</td>
<td>2.38</td>
<td>2.38</td>
</tr>
<tr>
<td>Aquiv. CO₂</td>
<td>kg/m³</td>
<td>163</td>
<td>278</td>
<td>201</td>
</tr>
</tbody>
</table>

3 CONCRETE PERFORMANCE AND EVALUATION

3.1 Motivation and testing

Due to the high reduction level of cement the normally used descriptive method to predict durability can no longer be used. Additional investigations show the relation of strength and durability for conventional concrete is no longer valid for concrete with high amount of supplementary materials [10]. Thus, make it necessary to use new concepts for evaluation of performance.

Table 2: testing procedures (strength and durability)

<table>
<thead>
<tr>
<th>Tests</th>
<th>Storage</th>
<th>time of test</th>
<th>Test conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>7 days water, then climate chamber 20°C, 65 %</td>
<td>2, 28, 56 days</td>
<td>-</td>
</tr>
<tr>
<td>Carbonation</td>
<td>28 days</td>
<td>2 % CO₂, 20°C, 65 % H, 28 Tage</td>
<td></td>
</tr>
<tr>
<td>Frost attack</td>
<td>56 days</td>
<td>CIF, 56 FTC</td>
<td></td>
</tr>
<tr>
<td>Chloride migration</td>
<td>Water</td>
<td>56 days</td>
<td>RCM-Test</td>
</tr>
</tbody>
</table>
A well working approach represents concept of equivalent performance (EN 206-1). Therefore the performance of CO₂-reduced concrete is compared with performance of conventionally used concrete or objective limits from standards. To evaluate the performance of CO₂-reduced concrete comprehensive investigations have been carried out for properties of fresh and harden state (mechanical properties and durability) [10, 11]. The current paper focuses on durability. Table 2 shows the parameters of test procedures.

### 3.2 resulting performance and practical application

To describe the performance results for concrete with a specific available SCM the test results for performance criteria are drawn over substitution level of cement. Like it is shown in figure 6, the resulting performance is decreasing with replacement of cement and depends on chosen SCM and degree of optimization, f. e. choosing lower SP-concentration. Thus, the performance curves enable to describe and compare the performance of available SCM.

![Figure 5: performance curves, describing strength, depth of carbonation, freeze thaw attack and chloride migration of concrete with high amount of SCM](image)
In figure 6 performance curves for strength and selected durability parameters are shown. For a specific application task the required limits are included in diagrams, which make enable to evaluate the necessary cement content for a chosen SCM. The comparison of test results and usual application limits like a strength requirement of C25/30 for an outdoor-application show, even a high substitution level of 80 Vol-% of cement by a low reactive trass powder can reach this limit. Due to the missing objective limits for carbonation, reference concrete, like it is conventionally used in Germany, has been tested and the reference value examines limit. In this case, only lower reduction level of cement is allowed to fulfill the requirement of equivalent performance. In case of freeze-thaw resistance and chloride migration in Germany additional limits from standards exist. Comparing the test results with limits of standards even a high substitution level of 80 Vol-% can reach the required limit for XF3. For chloride migration the tested CO2-reduced concrete using limestone powder isn’t suitable, because of assessed high migration coefficients. Even trass powder can only reach substitution level of 60 % for an adequate chloride coefficient. The use of FA reaches high substitution levels up to 80 Vol.-%. In consequence it is obvious that for different application tasks the use of different SCM is expedient. The described performance curves enable to choose right ones.

4 ROBUSTNESS

4.1 Motivation

The design process bases on mixture optimization to partly compensate performance loss by cement reduction. Due to application of high SP-concentration and granular optimization, mixtures tend to be more sensitive to variations in water content and temperature.

Like it is known from SCC the variation in water content and temperature can cause significant changes in properties of fresh state [12]. Due to increased variation in w/c-ratio and low cement content also significant changes in properties of harden state may occur, additionally. Under low temperatures hydration of binder may slow down, which would require an increased after treatment of concrete. On the other hand, higher temperatures may cause significant increase in hydration of cement, which may result in increased early age properties. Particular attention should be paid to dry out process of concrete, which may interrupt subsequent hardening of slowly reactive SCM.

To initially describe the changes in concrete properties of fresh and harden state, variations in water content (+15 l/m³) and temperature (10°C, 20 °C and 35 °C) to one reference and three alternative concrete with low CO2-burden have been carried out. The mixtures are signed by * in table 1.

4.2 Test procedures

In a first test series, the marked concrete compositions from table 1 are applied to variation of temperatures in three steps (10°C, 20°C, 35 °C). The flowability over time is measured. Additional specimens have been produced for strength development. The storage is carried out, compared to table 2.

In a second test series, concrete are mixed with 15 l/m³ excess water to describe the influence of uncontrolled variations in water content, for example caused by inadequate measured humidity of aggregates. The executed tests have been the same.
4.3 Robustness for variation in temperature

Like it is shown in figure 6 the resulting slump flow over time after mixing differs a lot. Excepting the results for concrete with high amounts of trass powder, the main influence on slump flow is observed for increased temperatures. While the initial slump flow for reference and trass powder concrete decreases with higher temperatures, the slump flow for concrete with high amount of FA increases. This might be described by the high amount of PCE-SP combined with low water content. Additionally, a limit of slump flow of about 35 cm is applied to describe a minimum of workability. It gets obvious that even under higher temperature the workability will maintain till 60 Min. Only trass powder can’t reach this limit.

The results for strength development are shown in figure 7. For better comparison, strength is related to the strength after 28 days. Additionally the relative strength development at temperatures of 10 and 35 °C is normed to the relative strength of 20°C for every time step, fig. 7. Strength development decreases for lower temperatures. The reference mixture and the concrete with FA using lower SP concentration have lower reduction in strength than concrete with high SP-concentration. In consequence the strength after 28 days is reduced about 20 % for reference and for concrete with FA using low SP-concentration. Concrete with high SP-concentration loses about 40 % in tests. Thus, further tests have to be carried out to describe reaction of concrete with high amount of SCM and SP and make it necessary to give suggestions for preventing loss of strength during low temperatures.

Applying higher temperatures (35 °C) to concrete mixtures the strength development is increased. Concrete with high amount of SCM develop much faster than the reference. The resulting strength after 28 days lies between 10-30% lower than strength by 20°C, which might be explained by faster dehydration. The largest decrease is observed for reference concrete, which may lead to a lower relevance of influence of higher temperatures for CO2-
reduced concrete. The results show the significance of arrangements for preventing fast dehydration by after treatment.

Figure 7: relative strength development for 20°C and relative strength development normed to strength by 20°C for changes in temperature (10°C and 35°C) in the first 7 days

4.4 Robustness for variation in water content

The results for variations in water content are shown in figure 8. The increase in water content leads to a significant increase of slump flow, while the characteristics of the slump flow over time after mixing doesn’t change. Relating growth of slump flow to initial slump flow by regular water dosage and drawing the coefficient over SP-concentration shows an increased grow of slump flow with increased SP-concentration. However, the expected risk of sedimentation and bleeding with increased water over dosage can only be confirmed for reference and concrete with trass powder. The highly reduced water content and the addition of paste during granular optimization reduce water film thickness around particle significantly in concrete with FA. Thus, water over dosage is handled by a lower increase of water film thickness, which would lead to segregation and bleeding. In consequence the robustness is increased compared to reference with lower powder and higher regular water content.

Like it is mentioned before, the increase in water content may cause significant changes in w/c-ratio even under high substitution level. This might cause also the significant change in strength over time. In figure 9 the relative changes in strength are shown. Like it is expected, CO2-reduced concrete tend to be more sensitive for water over dosage than conventional concrete mixtures. But it is also obvious that conventional concrete with CEM III/A even lose 20% of strength by water over dosage and the gap between the optimized mixtures isn’t as big as expected.

Further tests to describe the behavior of durability are still in progress but showing the
same consequences than strength development.

Figure 9: slump flow over time for selected concrete compositions with 0 l/m³ and 15 l/m³ water over dosage. Relative growth of slump flow related to SP-concentration in water.

Figure 10: relative strength development normed to strength defined conditions for water over dosage of 15 l/m³

5 CONCLUSIONS

The current paper introduces a new performance based design concept for the development of concrete with high amount of supplementary cementitious materials. The main conclusions can be summarized as:

- A powerful solution to reduce CO₂-burden of concrete is to replace cement by SCM
- The chosen design concept has to consider the availability, quality and performance of raw materials.
- For partly compensating the performance loss of concrete a new method for optimizing the water efficiency of mortar composition can be used. The excess of paste volume achieve an increased movability of coarse aggregates and the robustness in case of water over dosage is improved.

- New performance curves can be used to describe the performance of concrete with high amount of SCM. Based on the concept for equivalent performance the curves can be used to evaluate the necessary cement content.

- The low water and cement content and high SP-concentration can lead to an increased sensibility to changes in temperatures and variations in water content, which may cause significant changes of fresh concrete properties especially slump flow. Additionally changes in strength can occur. To prevent a decrease in strength the water content needs to be observed carefully and after treatment needs to be adapted to temperature conditions.

- Further tests for robustness of durability properties will be executed in future.

REFERENCES


STEEL CORROSION IN RECYCLED AGGREGATE CONCRETE CONTAINING AMINO ACID

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Key words: Recycled Aggregate, Amino Acid, Arginine, Chloride-induced Corrosion of Steel, Pore Solution of Mortar

Abstract. Recycled aggregate concrete has been focused on due to the shortage of aggregates for concrete and the effective utilization of resources. So, this study experimentally investigated the possibility of the recycled aggregate reinforced concrete containing Arginine which would improve both the living environment for fishes in the sea and the resistance performance against chloride induced corrosion of steel in concrete. In this study, immersion test of reinforced concrete specimens into salt water and chemical analysis of pore solution extracted from mortar specimens by high compression were carried out.

As a result, the protection performance against steel corrosion in the recycled concrete was not much improved by the dosage of Arginine. However, the combination of the admixture of Arginine and fly ash as the substitute of the fine aggregate was most effective to improve the resistance performance against chloride penetration into concrete and consequent steel corrosion in concrete. Moreover, it was confirmed that the concentration of hydroxide ions in pore solution increased while that of chloride ions decreased with the dosage of Arginine, which would result in the enhancement of the protection performance against steel corrosion in concrete.
1 INTRODUCTION

Recently, it has been reported that wave-dissipating concrete blocks containing Arginine, a kind of amino acid, could effectively form marine periphytic algae on the concrete surface in the actual seawater area, which would improve the living environment for fishes in the sea [1]. Such Arginine mixed concrete has mainly applied as plain concrete. However, the application of Arginine mixed concrete to reinforced concrete structures may be effective due to the high basicity of Arginine [2] which would enhance the protection performance of the concrete against the steel corrosion in concrete.

On the other hand, recycled aggregate concrete has been focused on due to the shortage of aggregates for concrete and the effective utilization of resources. So, this study experimentally investigated the possibility of the recycled aggregate reinforced concrete containing Arginine from the viewpoint of the resistance performance against chloride induced corrosion of steel in concrete. In this study, immersion test of reinforced concrete specimens into salt water and chemical analysis of pore solution extracted from mortar specimens by high compression were carried out. Moreover, a by-product from thermal power plant, fly ash was mixed into a part of specimens as a substitute of fine aggregate, expecting enhancement of durability against chloride induced corrosion of steel.

2 EXPERIMENTAL PROGRAM

2.1 Mixture proportions of concrete and materials

Six mixture proportions of concrete used in this study are shown in Table 1. As the names of concrete, initial ‘N’ means normal aggregate concrete and initial ‘R’ means recycled aggregate concrete. When ‘A’ is included in the name, the concrete contains Arginine that was 3% of cement mass. In this study, Arginine was solved into mixing water for NA, RA and RFA, while only for NAP, Arginine was mixed as a powder like cement. In the case of RFA, in addition to Arginine, fly ash was admixed as a substitute of 20 vol.% of fine aggregate. Water to cement ratio (W/C) of all cases concrete was 55%. The mix proportions of mortar were same as the mix proportions of concrete but coarse aggregate and chemical admixtures were not mixed. NaCl was solved into mixing water of mortar so that 3.0 kg/m³ of Cl⁻ was premixed in concrete.

<table>
<thead>
<tr>
<th>Name</th>
<th>W/C (%)</th>
<th>s/a (%)</th>
<th>Unit mass (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>W</td>
</tr>
<tr>
<td>N</td>
<td>55</td>
<td>48</td>
<td>170</td>
</tr>
<tr>
<td>NA</td>
<td>55</td>
<td>47</td>
<td>170</td>
</tr>
<tr>
<td>NAP</td>
<td>55</td>
<td>47</td>
<td>170</td>
</tr>
<tr>
<td>R</td>
<td>55</td>
<td>48</td>
<td>170</td>
</tr>
<tr>
<td>RA</td>
<td>55</td>
<td>47</td>
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</tr>
<tr>
<td>RFA</td>
<td>55</td>
<td>42</td>
<td>170</td>
</tr>
</tbody>
</table>

*WRA: Water reduce agent, **AEA: AE agent
Table 2: General information of materials

<table>
<thead>
<tr>
<th>Materials</th>
<th>Name</th>
<th>Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>C</td>
<td>Ordinary portland cement (density: 3.16 g/cm³, specific surface area: 3280 cm²/g, R₂O: 0.56%)</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>NS</td>
<td>Normal fine aggregates (density: 2.59 g/cm³)</td>
</tr>
<tr>
<td></td>
<td>RS</td>
<td>Recycled aggregate (density: 2.23 g/cm³, size is less than 5 mm)</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>NG</td>
<td>Normal aggregate (density: 2.66 g/cm³, Gₘₐₓ: 15 mm)</td>
</tr>
<tr>
<td></td>
<td>RG</td>
<td>Recycled aggregate (density: 2.34 g/cm³, Gₘₐₓ: 15 mm)</td>
</tr>
<tr>
<td>Fly ash</td>
<td>FA</td>
<td>Type II fly ash classified in JIS A 6201 (density: 2.33 g/cm³, specific surface area: 4150 cm²/g, SiO₂: 63.2 %, ignition loss: 1.8 %)</td>
</tr>
</tbody>
</table>

General information of materials used in this study is shown in Table 2. Recycled aggregates were made by crashing of existing PC piles. Recycled aggregates under the size of 5 mm were classified as the recycled fine aggregates and of 5-15 mm were classified as the recycled coarse aggregates.

2.2 Preparation and curing of specimens

Reinforced concrete specimens for steel corrosion monitoring were 100×100×300 mm prisms with a steel bar φ13 SR235 (JIS number) at their cover depth of 25 mm from the exposed surface. One edge of a steel bar was exposed from concrete surface and coated by epoxy resin, while the embedded length of the steel bar in concrete was 270 mm. φ 100×200 mm concrete cylinders were prepared for the compressive strength test. Moreover, φ 50×100 mm mortar cylinders were made for the analysis of extracted pore solution. After the casting and demolding, these specimens were sealed and kept in the room of constant temperature as 20 ºC for 28 days or 91 days.

2.3 Tests

RC specimens after curing were immersed into the 10 % NaCl solution at the constant temperature of 20 ºC. Before the immersion, concrete surfaces of the specimen were coated by epoxy resin except for one exposed surface. One cycle of the immersion test consists of the immersion period for 10 days and the drying period for 4 days. Electrochemical indexes of steel corrosion were measured at the end of each cycle during the immersion test of RC specimens. After finishing the immersion test for 4 months, concentration values of amino acid in the salt water were measured using the amino acid analyzer.

As the electrochemical indexes, half-cell potential, polarization resistance of steel in concrete and resistivity of cover concrete were measured regularly. As a reference electrode for these electrochemical monitoring, saturated silver chloride (Ag/AgCl) was used. Polarization resistance was measured by the rectangular wave electric current polarization method, as the deference of impedances at 800 Hz and 0.1 Hz of electric current frequency. Resistivity was obtained as the impedance at 800 Hz of electric current frequency.

At the curing time of 91 days, 5 ml or more amount of pore solution in mortar cylinders were extracted by the loading of high compressive pressure. Chemical analysis of such pore solution was carried out immediately after the extraction for measuring the concentration of OH⁻, Cl⁻ and several kinds of amino acid.
3 COMPRESSIVE STRENGTH OF CONCRETE

The results of compressive strength test using concrete at the age of 91 days and air content values are shown in Figure 1. According to a past research [3], compressive strength of concrete showed 3 to 10% decrease with the dosage of Arginine. From Figure 1, NA containing Arginine shows about 10% increase of compressive strength from the case of N without Arginine. The reason of such difference of compressive strength may be caused by the difference of air content.

On the other hand, in the cases of recycled-aggregate concrete, the compressive strength values are lower than those of the normal-aggregate concrete due to the lower quality of recycled aggregate [4]. Furthermore, the dosage of Arginine (RA) causes the small decrease of the strength. However, in the case of fly-ash-admixing concrete (RFA), the compressive strength becomes remarkably higher than the case of R. The possible reasons of such increase of compressive strength are not only the lower air content due to the fly-ash mixing but also the promotion of the pozzolanic reaction of fly ash affected by the high basic property of Arginine.
4 ELECTROCHEMICAL EVALUATION OF STEEL CORROSION IN RC SPECIMENS IMMERSED INTO SALT WATER

The variation curves of half-cell potential and polarization resistance of steel in concrete specimens during the immersion test into salt water are shown in Figure 2 and Figure 3 respectively. Moreover, the variation curves of resistivity of cover concrete are shown in Figure 4.

As shown in Figure 2, in the cases of normal-aggregate concrete, the values of half-cell potential generally decrease and reach the region of high possibility of corrosion defined by ASTM C876-91 after the immersion for 4 months. However, NA and NAP with the dosage of Arginine show the slower decrease of the potential than the case of N, which may suggest the improvement of the corrosion tendency due to the dosage of Arginine with high basic property.

On the other hand, recycled-aggregate concrete cases R and RA show the rapid drop of the potential entering the corrosion region defined by ASTM within the initial 1 month of immersion test. Since the recycled aggregates employed in this study were low quality ones classified in class L defined by JIS A 5023, the penetration of Cl\(^-\) through the interface between the cement paste and the recycled aggregate would be accelerated. However, when both of the fly ash and the Arginine were mixed into the recycled concrete (RFA), the decrease of the potential is suppressed and the value doesn’t enter the corrosion region even when the immersion test for 4 months completed. From these results, it can be said that the admixing of fly ash and the Arginine into recycled concrete would result in the higher resistance performance against chloride-induced corrosion than the case of normal concrete.

From Figure 3, half-cell potential and polarization resistance are strongly correlated, namely, following the decrease of the half-cell potential which means the increase of corrosion possibility, the polarization resistance values are decreasing which means the enhancement of the steel corrosion rate. Especially in the cases of recycled concrete, RA shows slower decrease of the polarization resistance than the case of R. It may suggest that the steel corrosion rate in concrete can be suppressed by the dosage of the Arginine even in the cases of recycled concrete.
Some past papers reported the influence of the dosage of amino acid on the steel corrosion in concrete [6]. According to these papers, some sorts of amino acid have the effect to suppress the steel corrosion in concrete or in salt water [2]. As the mechanism, the suppression of the corrosion pitting formation due to the adsorption of amino acid on the steel was suggested. However, the further research is necessary to clarify the detailed mechanism of the suppression of the steel corrosion.

According to Figure 4, the resistivity values are strongly affected by the quality of the aggregates. The resistivity values of the recycled concrete are smaller than those of the normal concrete. These results show that the recycled concrete has the relatively porous pore structure around the interface between cement past and recycled aggregate. However, in the case of RFA, remarkably high resistivity over the case of the normal concrete is observed and that resistivity is growing with time, which would be caused by the formation of the dense pore structure with the progress of the pozzolanic reaction. Such results of concrete resistivity well agree with the results of half-cell potential and polarization resistance.

5 CHEMICAL ANALYSIS OF PORE SOLUTION EXTRACTED FROM MORTAR

5.1 Concentration of anions

Measured OH\(^{-}\) concentration, Cl\(^{-}\) concentration and Cl\(^{-}\)/OH\(^{-}\) molar ratio in pore solution expressed with the high compressive pressure at the age of 91 days are shown in Figure 5 and Figure 6 respectively. Since the mixture proportion of mortar is defined as the mixture proportion of concrete without coarse aggregate, 3.0 kg/m\(^3\) of premixed Cl\(^{-}\) in concrete correspond to 4.75 kg/m\(^3\) in mortar specimens.

As shown in Figure 5, OH\(^{-}\) concentration values in pore solution of mortar with the dosage of the Arginine are higher than those of non-Arginine specimens. Such an increase of OH\(^{-}\) concentration in pore solution would be caused by the high basicity property of the Arginine. Furthermore, the Cl\(^{-}\) concentration values of mortar with the dosage of the Arginine are lower than those of non-Arginine cases. The detailed mechanism is not clarified yet but a past research pointed out that some kinds of amino acid affected cement hydration products, which may bind Cl\(^{-}\) ions in pore solution.

As for the dosage state of the Arginine, NAP mixing the powder Arginine show the higher OH\(^{-}\) concentration and the lower Cl\(^{-}\) concentration than those of NA mixing the all Arginine.
solved in mixing water. When the Arginine is mixed as the powder, a part of the Arginine remains as a state of crystal which may strongly affect the composition of ions in pore solution.

Regarding the difference of aggregates, recycled aggregate mortar R shows the lower $\text{OH}^-$ concentration and lower $\text{Cl}^-$ concentration than those of normal aggregate mortar N probably due to the carbonated cement paste attached to the recycled aggregate. However, RA with the dosage of the Arginine results in the much higher $\text{OH}^-$ concentration and the lower $\text{Cl}^-$ concentration than those of R. Moreover, RFA with the admixing of fly ash results in a little lower $\text{OH}^-$ concentration due to progress of the pozzolanic reaction but the lower $\text{Cl}^-$ concentration than those of RA. The reason of this is presumed that the Arginine would accelerate the pozzolanic reaction of fly ash and the reaction products may incorporate $\text{Cl}^-$ ions in pore solution [7].

The $\text{Cl}^-/\text{OH}^-$ molar ratio is generally regarded as an index to evaluate the initiation of steel corrosion in mortar or concrete, then the larger this index is the severer corrosion condition of steel is. According to Figure 6, the dosage of the Arginine results in the decrease of $\text{Cl}^-/\text{OH}^-$ molar ratio because of the rise of $\text{OH}^-$ concentration and the fall of $\text{Cl}^-$ concentration. From these result, it can be said that the dosage of the Arginine could mitigate the steel corrosion condition under the constant content of chlorides in concrete. The protection effect against steel corrosion due to the dosage of the Arginine observed in the immersion test into salt water may be caused by such a change of the ionic composition in pore solution. Moreover, fly-ash mixing into concrete or mortar generally increase the $\text{Cl}^-/\text{OH}^-$ molar ratio due to the fall of the $\text{OH}^-$ concentration caused by the pozzolanic reaction. So, the Arginine could be the admixture for fly ash concrete to improve the durability since the $\text{Cl}^-/\text{OH}^-$ molar ratio of RFA kept a small value in this study.

5.2 Concentration of amino acid

The concentration values of some kinds of amino acid are shown in Figure 7. The Arginine gradually changes to the other sort of amino acid, the Ornithine or the Citrulline under the high alkalinity circumstance like in concrete. In Figure 7, total amount of the Arginine was calculated assuming that all of the detected Ornithine, Citrulline and ammonia are generated

![Figure 7: Amino acid content in pore solution](image-url)
from the Arginine. From Figure 7, the highest total amount of the Arginine is detected in pore
solution of RFA followed by the cases of NA and RA. NAP results in the lowest total amount
of the Arginine. The mechanism that fly-ash mixing accelerates the concentration of amino
acid in pore solution is not clarified so far. Further investigation is necessary including the
mechanism of the pozzolanic reaction affected by the Arginine.

6 CONCLUSIONS

(1) As a result of compressive strength test, in the case of recycled aggregate concrete, the
compressive strength decreased a little with the dosage of Arginine but remarkably
increased with the dosage of both Arginine and fly ash as a substitute of fine aggregate.

(2) The protection performance against steel corrosion in the recycled concrete was not much
improved by the dosage of Arginine. However, the combination of the admixture of
Arginine and fly ash as the substitute of the fine aggregate was most effective to improve
the resistance performance against chloride penetration into concrete and consequent
steel corrosion in concrete.

(3) It was confirmed that the concentration of hydroxide ions in pore solution increased while
that of chloride ions decreased with the dosage of Arginine, which would result in the
enhancement of the protection performance against steel corrosion in concrete.

(4) The highest concentration of amino acid in pore solution was measured in the case of fly
ash mortar.

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on the nucleation, growth, and stability of ettringite [Ca$_3$Al(OH)$_6$]$_2$(SO$_4$)$_3$ \cdot 26$H$_2$O,
STUDY OF THE BEHAVIOR OF CONCRETE WITH RECYCLED POLYPROPYLENE FIBERS

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Key words: Plastic Shrinkage, Recycled polypropylene fibers, Concrete control, Fiber reinforced concrete

Abstract. This study aims to characterize the behavior of concrete reinforced with recycled synthetic fibers produced from polypropylene waste materials. The fact that these fibers are made of polypropylene guarantees that they are compatible with the alkalinity of the concrete. The objective is to analyze the effectiveness of these fibers when used to control the cracking due to shrinkage in concrete.

The test program includes plastic shrinkage (ASTM C 1579) tests for determining the effect on controlling shrinkage. Seven different concrete mixes have been analyzed using several types of fibers and two different dosages for the recycled synthetic fibers.

The tests performed in this work show promising results in the reduction of plastic shrinkage cracking using the synthetic fibers from recycled waste materials. This study could yield in the introduction of a new type of environmentally friendly commercial fiber, since the plastic wastes represent a high percentage of the total amount of wastes.

1 INTRODUCTION

The use of waste materials is becoming more widespread in construction. Due to the high volume of source materials used in this area, it is important to replace as many as possible with recycled materials to reduce the environmental impact of the exploitation of raw materials.

World production of plastics in 2013 was 299 million tons, of which about 60 are in Europe (of which 24.7 tons became waste). The polypropylene accounts for 19% of them, being the most demanded type of plastic[1], therefore it should be noted the importance of recycling it. The justification for the use of recycled plastics is not only environmental, but also it has an important economic component. The demand for raw materials for manufacturing products is becoming higher, therefore the possibility of reusing these materials reduces industry costs significantly.

The use of recycled fibers is not very common in the construction industry right now are few authors have worked on the use of recycled plastics in structural concrete, such investigations are very important because they are a residue which represents most of the total volume of waste, some studies showing results in pavements with the use of recycled polypropylene fibers from industrial waste compared with that of concrete reinforced with virgin polypropylene fibers[2] these fibers are treated and extruded to give them the same way
as the commercial fibers, unlike the fibers used in this study.

The shrinkage control is important because it involves the appearance of cracks. If not considered adequately in the design phase, it affects the durability of concrete and, therefore, the lifetime of the whole structure. One of the important factors when considering concrete shrinkage is the thickness of the pieces, the smaller the thickness, the higher shrinkage will suffer the structure, so the importance of this phenomenon is greater in surface elements, such as pavements, plates and slabs. Cracks due to shrinkage are not a structural risk by themselves, but favoring the aggression of external factors that penetrate through them to the reinforcement, causing corrosion. The synthetic fibers, unlike metal, does not suffer damage from corrosion. They may be of materials such as polypropylene, nylon, polyvinyl alcohol, etc., and has to check its compatibility with concrete. These fibers are characterized by having a large number of fibers per kg (about 2 · 10^3 times more than macro synthetic fibers), however they do not have structural behavior. They monofilament fibers, which are thought to provide effective crack resistance. These is due to their capability to retain solids inside concrete matrix, avoiding the rising of a significant amount of water to the free surface of concrete, which would evaporate quickly, leading to shrinkage and cracking of the concrete.

The objective of this study is to analyze the behavior of concrete reinforced with recycled polypropylene fibers, produced from out of use transport bags for its application in slabs. These bags are made of polypropylene, as well as some commercial fibers, ensuring compatibility with the alkalinity of the concrete.

Recycled polypropylene fibers (RPF) are characterized by continuous size distribution with greater presence of small fibers, similar to those used in shrinkage control (Figure 1).

![Figure 1. Appearance of recycled polypropylene fibers.](image)

The phenomenon of shrinkage is influenced by various factors: age, this phenomenon occurs to a great extent at an early age; the water / cement ratio; the type and nature of the aggregates; the thickness, etc. Therefore there are several types of retraction that affect concrete, one is plastic shrinkage. Plastic shrinkage is a problem to be considered in elements where the thickness is much smaller than the other two dimensions, such as the case of flooring, tiles, etc. That is reason to focus the study in this type of retraction, and how it affects the concrete.

2 EXPERIMENTAL PROGRAM

This study aims to analyze the fresh properties of concrete made with RPF. A standard method for evaluating plastic shrinkage (ASTM – C 1579) are made. Measurements of
temperature, humidity and wind speed were made throughout the test. Environmental conditions were forced in a chamber with heater, fan and de-humidifier, and were registered automatically with Termo – Higro – Anemometer device. Total duration of the test is 24 hours.

Control concrete was used as a base for fiber reinforced concretes. A concrete susceptible to be affected by shrinkage was chosen to better compare the effect of different fibers. (Table 2). Nine different concretes were tested with five different types of fibers, four types of commercial fibers, combining two sizes: macro-fibers (MA) and micro-fibers (mi), and two materials: steel (St) and synthetic (Sy) are chosen as reference to compare the results obtained with Recycled Polypropilene Fibers (RPF) added in four proportions (Table 3).

Figure 2 shows the fibers used.

![Different fibers used.](image)

**Table 1. Characteristics of fibers**

<table>
<thead>
<tr>
<th>Designation</th>
<th>Macro Synthetic Fiber</th>
<th>Macro steel</th>
<th>micro synthetic</th>
<th>micro steel</th>
<th>Recycled polypropilene</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (mm)</td>
<td>48</td>
<td>35</td>
<td>12</td>
<td>12</td>
<td>10-40</td>
</tr>
<tr>
<td>Diameter(mm)</td>
<td>0,74</td>
<td>0,55</td>
<td>0,031</td>
<td>0,2</td>
<td>-</td>
</tr>
<tr>
<td>slenderness (mm)</td>
<td>65</td>
<td>64</td>
<td>387</td>
<td>60</td>
<td>-</td>
</tr>
<tr>
<td>Density (kg/m3)</td>
<td>910</td>
<td>7850</td>
<td>910</td>
<td>7850</td>
<td>910</td>
</tr>
<tr>
<td>Form</td>
<td>twisted</td>
<td>Crimped - end</td>
<td>Straight</td>
<td>monofilament</td>
<td>Flat</td>
</tr>
</tbody>
</table>
Table 2. Mix proportions

| Material  | Cem II 45.5 R | Water | Fine Aggregate | Coarse aggregate 4/6 | Coarse aggregate 6/10 | Filler | Superplasticizer (%)
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>kg/m³</td>
<td>350</td>
<td>220</td>
<td>900</td>
<td>350</td>
<td>450</td>
<td>50</td>
<td>0.6</td>
</tr>
</tbody>
</table>

The dosages for every type of commercial fiber have been chosen based on commonly used dosages in construction. The amount of RPF fibers starts from doubling the quantity of commercial fiber typically used for shrinkage control and taking as upper limit 10 kg/m³.

Table 3. Fiber proportions

<table>
<thead>
<tr>
<th>Fibers</th>
<th>Commercial fibers</th>
<th>Recycled fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain</td>
<td>MA Sy</td>
</tr>
<tr>
<td>Dosage (kg/m³)</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>%</td>
<td>0</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Type and number of tests performed for each type of concrete are shown in Table 4. The table shows the minimum number of tests scheduled for each type of concrete. Occasionally some tests have been repeated to confirm the results.

Table 4. Type of tests performed and number of specimens tested.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Test</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>Compressive strength 7 days</td>
<td>3</td>
</tr>
<tr>
<td>mi Sy</td>
<td></td>
<td></td>
</tr>
<tr>
<td>mi St</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MA Sy</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MA St</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RPF (1.8 kg)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RPF (3.6 kg)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RPF (5 kg)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RPF (10 kg)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Plastic Shrinkage</td>
<td>1</td>
</tr>
</tbody>
</table>

3 METHODOLOGY

Since the study focuses on the early age behavior of concrete, compressive strength tests are performed at 7 days. The compressive strength test is performed according to UNE EN 12390-3. To carry out this study, 300mm cylindrical specimens are made with 150mm diameter.

For plastic shrinkage test, 355 ± 10 mm by 560 ± 15 mm specimens are made with a depth of 100 ± 5 mm according to ASTM C 1579. These specimens are subjected to restrain conditions by one piece of metal sheet with two restraints placed 90 ± 2 mm inward from each end of the mould and one central stress riser. To increase the restriction of movement of the
concrete mass, the mould was modified by placing steel hooks on the sides. During plastic shrinkage tests, environmental conditions are kept stable with values of 35 ± 2 °C relative humidity between 36 and 63% and a wind speed of between 4.5 and 8 m/s.

Wind Speed for plastic shrinkage test was achieved by placing a wind tunnel within the chamber, which it has a required temperature and humidity by a fan, a heater and a dehumidifier placed therein. For the launch of the test, several tests were performed to achieve the right conditions. During the plastic shrinkage test an optical microscope is placed over the wind tunnel, and it is faced to the specimen through a window opened on the upper face of the tunnel. Pictures are periodically taken throughout the test and stored in the measurement equipment to afterwards determine the age and evolution of the cracking on the specimen. (Figure 3).

4 RESULTS AND DISCUSSION

A quantitative analysis of the results has been performed to determine the feasibility of incorporating recycled synthetic fibers to concrete. A control concrete has been used as a reference to compare the quality of the synthetic fibers in reducing the cracking due to shrinkage. Cracking width obtained in plastic shrinkage test has been measured for that purpose.

The fibers influence the slump of concrete. This influence can be corrected by slightly increasing the additive dosage amount, since the dosage of plain concrete is 0.6% by the weight of cement. The compressive strength of the specimens tested is not affected significantly by the use of RPF fibers, the values are between 29.83 and 34.87 as shown in Table 5.

Table 5. Slump and Compressive strength results.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Plain</th>
<th>RPF (1,8kg)</th>
<th>RPF (3,6kg)</th>
<th>RPF (5kg)</th>
<th>RPF (10kg)</th>
<th>miSy</th>
<th>miSt</th>
<th>MASy</th>
<th>MASst</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump</td>
<td>13</td>
<td>18</td>
<td>14</td>
<td>13</td>
<td>5</td>
<td>21</td>
<td>14</td>
<td>18</td>
<td>17</td>
</tr>
<tr>
<td>fc 7d (Mpa)</td>
<td>31,06</td>
<td>31,82</td>
<td>34,87</td>
<td>30,4</td>
<td>30,7</td>
<td>29,83</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>fc 28d (Mpa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>34,37</td>
<td>34,77</td>
<td>35,57</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Taking as a reference the commercial fibers to shrink control (mi Sy), with high dosages of RPF are achieved and even exceeded the values obtained with the reference fibers. From 5 kg/m³ of RPF results that are below the threshold set by commercial synthetic fibers to control plastic shrinkage are achieved, meaning that they are able to effectively control shrinkage. The appearance of cracks obtained in the different samples can be seen in the Figure 5. RPF reinforced concrete shows a crack appearance similar to that mi Sy reinforced concrete, they may have a use as fibers for plastic shrinkage control bringing hope to incorporate environmentally friendly commercial fibers.

The average temperature is similar in every test, with a deviation as small as 2.7ºC. This values are within the limits defined in ASTM C - 1579 (36±3ºC), except for the plain concrete. This can be done to a failure on the thermostat, however the crack width obtained is big enough to be used as a reference to determine the capability of the fibers for shrinkage control. Regarding the relative humidity, abnormal data appears on the concretes reinforced using dosages of 1.8 and 3.6 kg/m³. This was most probably caused by the weather conditions during both tests. In the remaining tests the humidity is kept between 60% and 63.2%. These values are much higher than the maximum humidity allowed in ASTM standards, however due to the environmental conditions of the laboratory it was not possible to reduce the humidity below 60% and therefore it was decided to keep it around that value. (Table 6).

**Table 6.** Plastic shrinkage parameters and results for the different type and amount of fibers.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Dosage (kg/m³)</th>
<th>% of fibers by the weight of cement</th>
<th>Crack width (mm)</th>
<th>Time at the first crack (min)</th>
<th>Average Temperature (ºC)</th>
<th>Average Humidity (%)</th>
<th>Wind Speed (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>0</td>
<td>0</td>
<td>1,00</td>
<td>64</td>
<td>*31,20</td>
<td>63,24</td>
<td>7,97</td>
</tr>
<tr>
<td>RPF (1.8 kg)</td>
<td>1.8</td>
<td>0.2</td>
<td>0,60</td>
<td>121</td>
<td>36,70</td>
<td>36,46</td>
<td>6,87</td>
</tr>
<tr>
<td>RPF (3.6 kg)</td>
<td>3.6</td>
<td>0.4</td>
<td>0,40</td>
<td>148</td>
<td>34,03</td>
<td>40,74</td>
<td>7,18</td>
</tr>
<tr>
<td>RPF (5kg)</td>
<td>5</td>
<td>0,55</td>
<td>0,25</td>
<td>87</td>
<td>34,44</td>
<td>60,37</td>
<td>5,06</td>
</tr>
<tr>
<td>RPF (10kg)</td>
<td>10</td>
<td>1.11</td>
<td>0,15</td>
<td>28</td>
<td>36,30</td>
<td>60,58</td>
<td>4,52</td>
</tr>
<tr>
<td>mi Sy</td>
<td>0,9</td>
<td>0,1</td>
<td>0,28</td>
<td>116</td>
<td>34,82</td>
<td>60,02</td>
<td>6,82</td>
</tr>
<tr>
<td>mi St</td>
<td>35</td>
<td>1,3</td>
<td>0,30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MA Sy</td>
<td>5</td>
<td>0,55</td>
<td>0,25</td>
<td>134</td>
<td>36,16</td>
<td>60,27</td>
<td>4,63</td>
</tr>
<tr>
<td>MA St</td>
<td>35</td>
<td>1,3</td>
<td>0,10</td>
<td>45</td>
<td>36,33</td>
<td>60,04</td>
<td>*2,40</td>
</tr>
</tbody>
</table>

All fibers used contribute to shrinkage cracking control. In general, the greater the percentage of fibers by the weight of cement, the lower the maximum crack width. In the majority of cases the time at the first crack is delayed crack in the fiber reinforced specimens, while an appearance of crack is observed at very early ages when the amount of 1% of fibers is exceeded. The highest crack width is obtained, logically, in the plain concrete and the smallest width in the concrete reinforced with 10kg/m³ of RPF. The concrete reinforced with the reference fiber (mi Sy) gives a crack width of 0.28mm. (Figure 4).
5 CONCLUSIONS

- This study has shown the effectiveness of fiber obtained from industrial waste, which is an environmental breakthrough, achieving give new life to a material that otherwise would go to landfill.

- The results obtained in ASTM 1579 plastic shrinkage tests provide promising results when using high dosages of synthetic recycled fibers (around 5 – 10 kg/m³). Crack width can be reduced by 75% for shrinkage cracks even when using only 5 kg/m³, in
comparison with control concrete.
- The influence that the fibers have on the slump of concrete can be corrected by slightly increasing the superplasticizer additive dosage, without affecting the mechanical strength of concrete.
- RPF reinforced concrete shows a crack appearance similar to that of Sy reinforced concrete, they may have a use as fibers for plastic shrinkage control bringing hope to incorporate environmentally friendly commercial fibers.

References

SUSTAINABILITY ANALYSIS OF STEEL FIBRE REINFORCED CONCRETE SLABS
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Key words: Concrete, Slabs, Steel Fibre Reinforced Concrete

Abstract. Fibre reinforced concrete has been used in concrete structures without any additional reinforcement when the design is determined by transient load stages (e.g. precast segments for tunnels), in elements with favourable boundary conditions or structures subjected to low load levels (e.g. pavements or pipes). The material has been more recently applied as the primary reinforcement in elements subjected to higher load levels such as slabs. As a result of the experience gained in this type of application the American Concrete Institute (ACI) has published a report on the design and construction of steel fibre reinforced concrete (SFRC) elevated slabs. Despite these advances, in some cases fibres have not been used as primary reinforcement in concrete slabs due to economic reasons. However, in most cases the comparison of this solution with other alternatives such as traditional reinforcement has been made considering only direct material costs disregarding indirect costs, social and environmental factors. Considering the above, the aim of this study is to present a method to evaluate the sustainability of concrete slabs by means of the multi-criteria decision making approach for assessing sustainability MIVES.
1 INTRODUCTION

Fibre reinforcement technology and knowledge has advanced significantly in the past years and, as a result, has expanded to other applications besides the traditional (e.g. pavements or precast elements). In particular, steel fibre reinforced concrete (SFRC) has been successfully employed in flat slabs of several buildings in Europe [1] and in Spain [2] with the fibres as the only or the main reinforcement. Such achievement represents a step forward in the applications of SFRC not only in civil engineering but also in the field of architecture.

The technical advantages of using SFRC in this typology of structures have been extensively studied [3-5], among them are the increased toughness and ductility, the cracking control or the enhanced performance in case of dynamic effects or impacts. Besides these, other advantages related with the construction should be considered when analysing the possibility of using SFRC. Given that fibres are added in the concrete plant when the concrete mix is manufactured, the amount of work on site decreases significantly. The operations of preparing, handling and placing the traditional reinforcement are reduced to localized areas where the traditional reinforcement might be necessary. As a result, the execution time of the structure is also reduced. Furthermore, the use of SFRC makes the vibration of the concrete unnecessary and the occupational safety is improved due to the lack of risks associated to the handling of the traditional reinforcement.

Although the experiences in several buildings mentioned above have confirmed these advantages and proved the technical feasibility of using SFRC in concrete slabs, its use is often not possible due to a partial picture of the benefits reported. In fact, in most cases the decision whether to use steel fibres or traditional reinforcement is made based on the direct material costs of the reinforcement, disregarding the overall costs, social aspects or environmental factors.

Considering the above, the present document aims at proposing a method for the sustainability assessment of concrete slabs using MIVES (a multi-criteria decision making approach for assessing sustainability). The method proposed is used to evaluate two different reinforcement solutions for concrete slabs of an office building in Spain taking into account economic, environmental and social aspects. Furthermore, a sensitivity study is conducted to analyse different scenarios.

2 METHOD FOR ASSESSING THE SUSTAINABILITY OF CONCRETE SLABS

2.1 General features

The method developed in the present study is based on the method MIVES [6-11]. This model requires defining three fundamental aspects: (1) the boundaries of the system that determine the scope of the analysis, (2) the requirements tree, the criteria and the indicators involved in the decision-making process and (3) the value functions that convert the attributes or physical units associated with each indicator into one-dimensional values from 0 to 1. The Analytic Hierarchy Process (AHP) method [12] is used to assign an appropriate weight to each element: requirements, criteria and indicators.
2.2 Requirements tree

The requirements tree defined in the method proposed (see Table 1) consists of the three requirements (R) generally associated with the sustainability according to the United Nations [13]: economic, environmental and social. These requirements are articulated in 5 criteria (C) and 9 indicators (I). The indicators were selected to be representative (discriminators between solutions) and independent of each other to ensure a proper evaluation.

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Criteria</th>
<th>Indicator</th>
<th>Units</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1 Economic ((\lambda_R = 60%))</td>
<td>C1 Construction costs ((\lambda_C = 100%))</td>
<td>I1 Execution cost ((\lambda_I = 85%))</td>
<td>€/m²</td>
<td>DS</td>
</tr>
<tr>
<td>R2 Environmental ((\lambda_R = 20%))</td>
<td>C2 Resources consumption ((\lambda_C = 33%))</td>
<td>I1 Reinforcing steel ((\lambda_I = 40%))</td>
<td>Kg/m²</td>
<td>DCx</td>
</tr>
<tr>
<td>R3 Social ((\lambda_R = 20%))</td>
<td>C3 Effects on the constructor ((\lambda_C = 80%))</td>
<td>I1 Risks during construction ((\lambda_I = 80%))</td>
<td>Attributes</td>
<td></td>
</tr>
<tr>
<td></td>
<td>((\lambda_C = 30%))</td>
<td>I3 Energy ((\lambda_I = 30%))</td>
<td>MJ/m²</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>((\lambda_C = 67%))</td>
<td>I3 CO₂ emissions ((\lambda_I = 100%))</td>
<td>Kg/m²</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>((\lambda_C = 20%))</td>
<td>I4 Noise pollution ((\lambda_I = 20%))</td>
<td>Db</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>((\lambda_C = 20%))</td>
<td>I5 Third-party discomfort ((\lambda_I = 100%))</td>
<td>Attributes</td>
<td></td>
</tr>
</tbody>
</table>

DS: decreasing S-shape; DCx: decreasing convex

The **economic requirement** (R₁) is defined by a single criterion that is the **construction costs** (C₁). This criterion is evaluated through two indicators: the **execution cost** (I₁), which involves the costs associated with the construction of the concrete slabs, and the **non-conformity cost** (I₂) that correspond to the costs derived from imperfections or errors during the construction. The execution cost is calculated taking into account the formwork (materials and assembly), steel rebars, steel fibres, concrete, labour and auxiliary resources and facilities. The non-conformity costs are evaluated through attributes. For that, an assessment of the risks that increase the probability of a non-conforming product is conducted.

The **environmental requirement** (R₂) is evaluated through two criteria: the **resources consumption** (C₂) and the **emissions** (C₃). The resources consumption is defined by the consumption of **reinforcing steel** (I₁), of **water** (I₄) and **energy** (I₅), whereas for the emissions only CO₂ emissions are considered. The reinforcing steel consumption is evaluated considering the total amount of steel required for the concrete slabs, either longitudinal and transversal reinforcement for the traditional solution or steel fibres for the FRC. In terms of water consumption, only the water used in the manufacturing of the materials and the water used on site are considered. The energy consumption is obtained from the available databases. Finally, the emissions criterion is assessed by quantifying the amount of CO₂ released during the manufacturing of the concrete and the steel required for the construction of the slab.
The social requirement \((R_3)\) is determined by means of two criteria: the effects on the constructor \((C_5)\) and the third-party effects \((C_5)\). Two indicators are considered in the effects on the constructor: the risks during construction \((I_7)\) and the noise pollution \((I_8)\). The risks during construction in the different stages of the construction process are assessed by means of attributes. For that, the risks during concrete pouring and vibration of the slab are analyzed qualitatively. Notice that the use of steel fibres may cause cuts and lesions to the workers when they protrude on the surface. The third-party effects are evaluated with one indicator, the third-party discomfort \((I_9)\). Three variables are used to quantify this indicator, namely the noise pollution, the discomfort for the bystanders and effects on the traffic.

The indicators previously described are assigned value functions in order to evaluate the sustainability index \((I_s)\) of the alternatives, which ranges from 0.0 to 1.0 and normally remains below 0.8. This approach was already applied in previous studies [6, 14-17]. This function transforms physical units of each indicator (e.g. €/m², kg/m², dB) into dimensionless values ranging from 0 to 1. These values represent the sustainability or satisfaction of each indicator. Eq.(1) shows the general form of a value function.

\[
I_{\text{ind}}(X) = A + B \left( 1 - e^{-K_i \frac{|X_{\text{ind}} - X_{\text{min}}|}{C_i}} \right)^{P_i} \tag{1}
\]

In Eq.(1), \(B\) is the value of \(I_{\text{ind}}\) for \(X_{\text{min}}\); \(X_{\text{min}}\) is the minimum abscissa value in the indicator interval assessed; \(X\) is the abscissa value for the indicator assessed; \(P_i\) is a shape factor which defines whether the curve is concave \((P_i<1)\), convex \((P_i>1)\), linear \((P_i=1)\) or S-shaped \((P_i>1)\) (see Figure 1); \(C_i\) approximates the abscissa at the inflexion point; \(K_i\) tends towards \(I_{\text{ind}}\) at the inflexion point; \(B\), the factor that prevents the function from exceeding the range (0, 1), is obtained by Eq.(2), \(X_{\text{max}}\) being the abscissa value of the indicator that gives a response value of 1 for increasing value functions.

\[
B = \left[ 1 - e^{-K_i \frac{|X_{\text{max}} - X_{\text{min}}|}{C_i}} \right]^{-1} \tag{2}
\]
The form of the value functions assigned to each indicator (see Table 1) is a decreasing S-shape curve (DS) for I₁, I₄-I₆ and I₈ and a decreasing convex curve (DCx) for I₃.

The weights (λ) of each requirement, criterion and indicator are assigned by applying the AHP method and represent the importance of each element in the requirement tree. Notice that the weights assigned to the requirements in Table 1 correspond to a situation where the economic requirement is strongly prioritized in order to simulate a more entrepreneurial view of the analysis or to take into account a situation of financial recession on the part of the authority or agency that has to take the decision and make the investment. Even though this scenario may be realistic, and therefore considered in the study, it is hard to accept from the sustainability point of view. This scenario will be referred to as base scenario (E₀). For the purpose of this study; two additional scenarios prioritizing other requirements will also be considered in order to evaluate the sensitivity of the method (see section 4).

3 CASE STUDY: OFFICE BUILDING IN ARRASATE-MONDRAGÓN

3.1 Description of the structure

The structure selected for the case study is the project of the head office building of LKS in Arrasate-Mondragón (Guipúzcoa), which is a pioneer experience in Spain in the use of steel fibres as the main reinforcement in concrete slabs. All the data corresponding to the geometry, materials, etc. used in the present study was obtained from a Doctoral Thesis [18].

The building has 4 floors and a semi-basement that provides access to the offices. The four façades are oriented with the cardinal points. The construction involves the basement (floor -1), the ground floor and three other floors. The dimensions of the basement below ground level are 43.0 x 20.0 m, whereas at ground level the dimensions are 23.0 x 20.0 m. The total built surface of the building is 3506 m²; from which 862 m² correspond to the basement and 661 m² to each of the other four floors above ground.

The structural solution adopted for the design consists of a grid of 8.0 m x 8.0 m with round and rectangular reinforced concrete columns supported directly on slabs of 30 cm of thickness corresponding to the slabs of the ground floor, floors 1, 2 and 3. This type of floor is constructed around a central core that contains the facilities and services of the building, which means that the slabs have openings. A top view of this type of floor is shown in Figure 2. The columns are aligned horizontally in sections A, B, C and D and separated 8.0 m, 4.5 m and 5.4 m, respectively, and vertically in sections 1, 2, 3, 4 and 5, which separated 7.8 m. Notice that the largest areas of the grid (8.0 m x 7.8 m) are located between sections A and B. The slab of the basement (floor -1) has a similar distribution as the one shown in Figure 2, however it includes two more sections horizontally (sections 6 and 7), adding 10.0 m more to the lateral dimension of the building. Each floor of the building has a perimetral terrace for maintenance that extends 1.15 m from the concrete structure. These terraces are made of metallic structure.
3.2 Initial structural solution

The initial structural solution consisted in a concrete with a characteristic compressive strength of 30 MPa reinforced with traditional steel bars with characteristic yield strength of 500 MPa. The main difference between the initial solution and the proposal of SFRC is the reinforcements in the slabs of the ground floor and floors 1, 2 and 3. The design and the traditional reinforcement for the foundations, the basement (floor -1), the walls of the basement and the concrete columns remain the same in both solutions.

The traditional reinforcement in the slabs supported by the concrete columns may be classified in the following groups of reinforcement:

- **Bottom base reinforcement**: located in the entire surface at the bottom of the slab in order to bear positive bending moments. It consists of a grid rebars of 12 mm of diameter separated 15 cm. This configuration is the same for all floors.
- **Top base reinforcement**: located in the entire surface at the top of the slab in order to bear negative bending moments. This reinforcement presents the same configuration as the previous one and is the same for all floors.
- **Bottom reinforcement**: located at the bottom of the slab only in certain areas where the bottom base reinforcement is not enough to resist positive bending moments.
• **Top reinforcement**: located at the top of the slab only in certain areas such as columns or cantilevers where the top base reinforcement is not enough to bear the negative moments.

• **Punching transversal reinforcement**: located over the concrete columns in the shape of a cross with a variable number of branches in order to bear tangential stresses and to avoid punching failure.

• **Ring-beam or edge-beam reinforcement**: longitudinal and transversal reinforcement located in the external perimeter of the slab and at the edges of the openings of the slab (for the facilities).

Table 2 presents the total amount of traditional reinforcement (in kg) in each floor and the resulting steel content in kg/m$^3$. The reinforcement is grouped in base reinforcement (BaR), bottom reinforcement (BR), top reinforcement (TR), punching transversal reinforcement (PR) and ring-beam reinforcement (RR).

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Ground floor</th>
<th>Floor 1</th>
<th>Floor 2</th>
<th>Floor 3</th>
<th>Roof floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>BaR</td>
<td>19565 kg</td>
<td>14550 kg</td>
<td>14550 kg</td>
<td>14550 kg</td>
<td>14550 kg</td>
</tr>
<tr>
<td>BR</td>
<td>1366 kg</td>
<td>210 kg</td>
<td>210 kg</td>
<td>210 kg</td>
<td>1125 kg</td>
</tr>
<tr>
<td>TR</td>
<td>3479 kg</td>
<td>2075 kg</td>
<td>2075 kg</td>
<td>2075 kg</td>
<td>2733 kg</td>
</tr>
<tr>
<td>PR</td>
<td>2820 kg</td>
<td>2456 kg</td>
<td>2456 kg</td>
<td>2456 kg</td>
<td>2456 kg</td>
</tr>
<tr>
<td>RR</td>
<td>781 kg</td>
<td>781 kg</td>
<td>781 kg</td>
<td>781 kg</td>
<td>781 kg</td>
</tr>
<tr>
<td>Amount (kg/m$^3$)</td>
<td>111 kg/m$^3$</td>
<td>107 kg/m$^3$</td>
<td>107 kg/m$^3$</td>
<td>107 kg/m$^3$</td>
<td>117 kg/m$^3$</td>
</tr>
</tbody>
</table>

The budget for the structure (without taxes) in the initial solution is 1,060,000€, which is divided into different concepts: foundations, walls and slab of the basement (325,000€), concrete columns (75,000€), slabs for ground floor and floors over the ground (510,000€) and metallic structure (150,000€).

### 3.3 Solution with SFRC

The alternative solution is based on the substitution of the steel rebars by steel fibers, where possible, maintaining the same thickness of the slab and the characteristic compressive strength of the concrete. The main goal is to reduce the cost associated to the structure, which is significantly affected by the labor and the preparation and placement of the traditional reinforcement. The steel fibers used present circular cross-section of 1.3 mm, a length of 50 mm and a wavy geometry along its length. The SFRC is produced in a concrete plant, adding 100 kg/m$^3$ of fibres to the concrete mix.

The use of traditional reinforcement as a complement for the steel fibres was considered necessary in certain areas of the slabs due to their singular geometry. Some examples are the edges of the grid (the four corners of the slab), openings in the slabs, perimetral cantilevers, cantilevers with façade loads or in the embedment of the slab of the ground floor with the wall of the basement. In all cases, the steel rebars exhibit a yield strength of 500 MPa. Furthermore, additional anti-progressive collapse rebar (APC) where included to avoid that...
local failure may lead to collapse of the entire structure such as reported in some buildings
with a structure of slabs supported on concrete columns [20].

Table 3 presents the total amount of reinforcement (kg) in each floor and the resulting steel
content in kg/m$^3$. The reinforcement is grouped in reinforcement for positive moments in
edges of the grid (ER), the complementary reinforcement in certain areas (CR), the APC
reinforcement (ApCR) and the steel fibres (SF).

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Ground floor</th>
<th>Floor 1</th>
<th>Floor 2</th>
<th>Floor 3</th>
<th>Roof floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>ER</td>
<td>512 kg</td>
<td>1025 kg</td>
<td>1025 kg</td>
<td>1025 kg</td>
<td>2131 kg</td>
</tr>
<tr>
<td>CR</td>
<td>2233 kg</td>
<td>1930 kg</td>
<td>2058 kg</td>
<td>1832 kg</td>
<td>2674 kg</td>
</tr>
<tr>
<td>ApCR</td>
<td>3103 kg</td>
<td>2627 kg</td>
<td>2627 kg</td>
<td>2627 kg</td>
<td>4378 kg</td>
</tr>
<tr>
<td>Amount (kg/m$^3$)</td>
<td>23 kg/m$^3$</td>
<td>29 kg/m$^3$</td>
<td>29 kg/m$^3$</td>
<td>29 kg/m$^3$</td>
<td>49 kg/m$^3$</td>
</tr>
<tr>
<td>SF</td>
<td>100 kg/m$^3$</td>
<td>100 kg/m$^3$</td>
<td>100 kg/m$^3$</td>
<td>100 kg/m$^3$</td>
<td>100 kg/m$^3$</td>
</tr>
<tr>
<td>Total amount (kg/m$^3$)</td>
<td>123 kg/m$^3$</td>
<td>129 kg/m$^3$</td>
<td>129 kg/m$^3$</td>
<td>129 kg/m$^3$</td>
<td>149 kg/m$^3$</td>
</tr>
</tbody>
</table>

Notice that the APC reinforcement represents additional rebars that could have also been
included in the initial solution. If this reinforcement is excluded from the calculations, the
amount of traditional reinforcement of this solution reduces significantly, ranging between 11
and 26 kg/m$^3$.

3.5 Concrete mixes

The concrete mixes considered for the initial solution with RC and the alternative solution
with SFRC are presented in Table 4. The differences observed in the mix respond to the need
to compensate the loss of workability of the fresh concrete due to the addition of fibres.
Therefore, changes in the content of cement, water and aggregates are detected. Furthermore,
fly ash was also added to the mix with steel fibres.

<table>
<thead>
<tr>
<th>Components</th>
<th>Characteristics</th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (kg/m$^3$)</td>
<td>CEM I</td>
<td>300</td>
<td>400</td>
</tr>
<tr>
<td>Aggregates (kg/m$^3$)</td>
<td>-</td>
<td>1905</td>
<td>1850</td>
</tr>
<tr>
<td>Water (kg/m$^3$)</td>
<td>-</td>
<td>165</td>
<td>185</td>
</tr>
<tr>
<td>w/c (-)</td>
<td>-</td>
<td>0.55</td>
<td>0.41</td>
</tr>
<tr>
<td>Admixture (kg/m$^3$)</td>
<td>Fly ash</td>
<td>-</td>
<td>120</td>
</tr>
<tr>
<td>Fibres (kg/m$^3$)</td>
<td>Steel</td>
<td>-</td>
<td>100</td>
</tr>
</tbody>
</table>

3.5 Evaluation of the indicators

The indicators where evaluated according to Eq.(1) and the resulting values are presented
in Table 5. Notice that, for the evaluation of the risks during construction ($I_7$), a high
workability of the concrete mix was assumed for the initial solution with reinforced concrete
(RC). Moreover, a normal execution control was considered for the assessment of the non-
conformity costs ($I_2$).
Table 5: Values of the indicator ($X_i$) for each alternative

<table>
<thead>
<tr>
<th>Indicators</th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_1$ Execution cost (€/m$^2$)</td>
<td>0.251</td>
<td>0.433</td>
</tr>
<tr>
<td>$I_2$ Non-conformity cost (-)</td>
<td>0.674</td>
<td>0.740</td>
</tr>
<tr>
<td>$I_3$ Reinforcing steel (kg/m$^2$)</td>
<td>0.523</td>
<td>0.281</td>
</tr>
<tr>
<td>$I_4$ Water (m$^3$/m$^2$)</td>
<td>0.687</td>
<td>0.516</td>
</tr>
<tr>
<td>$I_5$ Energy (MJ/ m$^2$)</td>
<td>0.821</td>
<td>0.686</td>
</tr>
<tr>
<td>$I_6$ CO$_2$ emissions (kg/m$^2$)</td>
<td>0.595</td>
<td>0.275</td>
</tr>
<tr>
<td>$I_7$ Risks during construction (-)</td>
<td>0.248</td>
<td>0.437</td>
</tr>
<tr>
<td>$I_8$ Noise pollution (Db)</td>
<td>0.504</td>
<td>1.000</td>
</tr>
<tr>
<td>$I_9$ Third-party discomfort (-)</td>
<td>0.461</td>
<td>1.000</td>
</tr>
</tbody>
</table>

3.6 Sustainability indices ($I_s$) for each alternative

The parameters that define the value function of each indicator are presented in Table 6. These values were agreed in seminars with experts and complemented by the criteria reported in the literature. The sustainability indices are presented in Table 7 based on the requirements tree defined (see Table 1), the values of $X_i$ obtained for each solution (Table 6) and the parameters of the value functions presented in Table 4. Notice that the values of Table 7 correspond to the base scenario ($E_0$).

Table 6: Values of $I_s$ and $I_R$ obtained for each alternative

<table>
<thead>
<tr>
<th></th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_s$</td>
<td>0.378</td>
<td>0.485</td>
</tr>
<tr>
<td>$I_{R1}$</td>
<td>0.314</td>
<td>0.479</td>
</tr>
<tr>
<td>$I_{R2}$</td>
<td>0.616</td>
<td>0.349</td>
</tr>
<tr>
<td>$I_{R3}$</td>
<td>0.332</td>
<td>0.640</td>
</tr>
</tbody>
</table>

The results presented in Table 6 show that the solutions that use steel fibres as an alternative to steel bars result in a higher $I_s$ value, particularly 28.3% higher. The better performance of the SFRC solution is the result of two factors: the reduction of the overall costs and the noise pollution or discomfort to the labour and third-party.

4 SENSITIVITY STUDY

The sensitivity of the method proposed is analysed in this section by considering two additional scenarios. The weights assigned for each scenario are detailed subsequently:

- $E_0$ ($\lambda_{R1} = 60\%$, $\lambda_{R2} = 20\%$ $\lambda_{R3} = 20\%$) simulated a situation in which the economic requirement is prioritized over the others.

- $E_1$ ($\lambda_{R1} = 45\%$, $\lambda_{R2} = 35\%$ $\lambda_{R3} = 20\%$) assigns more weight to the environmental requirement ($R_2$) at the expense of the economic requirement.

- $E_2$ ($\lambda_{R1} = 45\%$, $\lambda_{R2} = 20\%$ $\lambda_{R3} = 35\%$) assumes a greater importance of the social requirements by assigning more weight than in the base scenario at the expense of the economic requirement.
Notice that the new scenarios maintain the same weight values for the criteria (\(\lambda_C\)) and indicators (\(\lambda_I\)) as those used in scenario \(E_0\) (see Table 1). The results of the sensitivity analysis are shown in Table 7.

Table 7: Values of \(I_s\) for each scenario of the sensitivity analysis

<table>
<thead>
<tr>
<th></th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E_0)</td>
<td>0.378</td>
<td>0.485</td>
</tr>
<tr>
<td>(E_1)</td>
<td>0.423</td>
<td>0.466</td>
</tr>
<tr>
<td>(E_2)</td>
<td>0.381</td>
<td>0.509</td>
</tr>
</tbody>
</table>

The results in Table 7 reveal that the solution with SFRC presents higher values of \(I_s\) in all the scenarios, thus revealing it is more sustainable than the solution with RC. This difference represents 28.3% in scenario \(E_0\), 10.2% in scenario \(E_1\) and 33.6% in scenario \(E_2\). Nevertheless, the values obtained are considerably low, which indicates that improvements still can be made in order to reach the desired levels of satisfaction (or sustainability). The highest \(I_s\) for the original solution with RC corresponds to scenario \(E_1\), in which the environmental requirements present higher weight than the social requirements but lower than the economic.

5 CONCLUSIONS

The present study proposes a method for the analysis of the sustainability of concrete slabs taking into account economic, environmental and social factors based on MIVES. The model allows comparing and prioritising alternative solutions while minimising the subjectivity in the decision-making process. The method is applied to a the real case of an office building in Spain originally designed with reinforced concrete but finally constructed with a SFRC solution. The conclusions drawn from the sustainability indices \(I_s\) are presented subsequently:

- The substitution of the traditional reinforcement with steel fibres yields higher values of \(I_s\) in all the scenarios considered in the present study, thus revealing that the solution of SFRC is more sustainable.
- The analysis of each of the requirements reveals that the degree of satisfaction for the solution with SFRC is highest for the social requirement, followed by the economic requirement and, finally, by the environmental requirement.
- The higher value of \(I_{R1}\) and \(I_{R3}\) in the case of the SFRC is explained by the reduction of labour and execution time and the reduction of the noise pollution and discomfort both for the labour and third-party, respectively.

ACKNOWLEDGEMENTS

The authors acknowledge Dr. Aitor Maturana, Associate Professor at the University of the Basque Country, his consent to use data from his scientific work in order to conduct the present study.
REFERENCES


THE INFLUENCE OF METAKAOLIN AND NATURAL ZEOLITE ON THE RHEOLOGY, ENGINEERING AND DURABILITY PROPERTIES OF HIGH STRENGTH SELF-COMPACTING CONCRETE AT EARLY AGE

ICCS16

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Keywords: HSSCC, Metakaolin, Zeolite, Engineering and Durability properties.
Abstract. The effect of two admixtures; metakaolin and natural zeolite in high strength self-compacting concrete, HSSCC at the early ages are experimentally investigated. For this aim, the rheological, engineering and durability (transport) properties of prepared mixtures are studied. Five SCC formulations are considered, covering the same class resistance of 50 MPa for HSSCC using a constant w/c_m (0.4): SCC mix with limestone filler and without additive (HSL) as the control concrete, two mixes with 10% and 15% (cement weight) of metakaolin in replacement of cement (HSM10, HSM15) and another two with zeolite in replacement of cement (HSZ10, HSZ15). At early age, there is a correlation between the durability properties such as the chloride migration coefficient in non-steady state, the electrical resistivity, the capillary absorption and the water penetration. Compared to control concrete, both pozzolans improves the durability (transport) properties of the studied SCCs involving a better resistance against the penetration of aggressive ions. Also, with increasing the percentage of pozzolans, such improvement is more remarkable.

1 INTRODUCTION

The concept of self-compacting concrete, SCC was born in Japan in the late 1980s. Environmental problems caused by 5-7% of CO₂ from the cement industry are constantly growing. Consequently, the use of pozzolan as a partial or total substitution of Portland cement in concrete production and particularly in self-compacting concretes which need high
volume of cement paste can be an important step in improving the environment. According to the importance of the durability of structures that are in the Persian Gulf side and industrial areas in Iran, with considering they are constantly exposed to aggressive agents. Must therefore, always find economic additions and resistant to destructive factors in considering that the latter is one of the issues addressed for Iranian researchers. Consequently, the purpose of this article is to study the influence of metakaolin manufactured in Tehran (capital of Iran) and zeolite manufactured in Semnan (central of Iran) on the rheological, mechanical and durability properties of high strength SCC (HSSCC).

2 MATERIALS

In this experiment five formulations were studied, covering the same class resistance of 50 MPa for HSSCC. Five types of SCC mixes consisting of constant w/cm (0.4) are as followed: SCC mix with limestone filler and without additive (HSL) as the control concrete, two mixes with 10% and 15% (cement weight) of metakaolin in replacement of cement (HSM10, HSM15) and another two with zeolite in replacement of cement (HSZ10, HSZ15). Crushed angular granite material with a nominal maximum size of 12 mm and well-graded natural siliceous sand with a nominal maximum size of 4 mm were used as coarse and fine aggregates respectively. The mix design is presented in Table 1. By increasing the replacement level of additives from
10 to 15%, the viscosity of fresh concrete and the amount of superplasticizer required to achieve the desired slump flow diameter (70±3cm) is also increased. This last is very remarkable in mixture of HSZ15 (Table 1).

Table 1: Mix design of HSSCC mixtures for 1m³ (kg/m³)

<table>
<thead>
<tr>
<th>Mix name</th>
<th>W</th>
<th>C</th>
<th>G</th>
<th>S</th>
<th>LP</th>
<th>SP</th>
<th>MK</th>
<th>Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSL</td>
<td>180</td>
<td>450</td>
<td>790</td>
<td>790</td>
<td>150</td>
<td>4.85</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>HSM10</td>
<td>180</td>
<td>405</td>
<td>790</td>
<td>790</td>
<td>150</td>
<td>4.7</td>
<td>45</td>
<td>-</td>
</tr>
<tr>
<td>HSM15</td>
<td>180</td>
<td>382.5</td>
<td>790</td>
<td>790</td>
<td>150</td>
<td>6.53</td>
<td>67.5</td>
<td>-</td>
</tr>
<tr>
<td>HSZ10</td>
<td>180</td>
<td>405</td>
<td>790</td>
<td>790</td>
<td>150</td>
<td>8.94</td>
<td>-</td>
<td>45</td>
</tr>
<tr>
<td>HSZ15</td>
<td>180</td>
<td>382.5</td>
<td>790</td>
<td>790</td>
<td>150</td>
<td>21.62</td>
<td>-</td>
<td>67.5</td>
</tr>
</tbody>
</table>

3 RESULTS AND DISCUSSIONS
3.1 Fresh state

In order to evaluate the effects of pozzolans and filler on properties of SCC, fresh state tests were performed (Table 2).

Table 2: Test results of fresh concrete

<table>
<thead>
<tr>
<th>Mix. name</th>
<th>Slump flow</th>
<th>L-box</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dia. (mm)</td>
<td>T₅₀ (Sec.)</td>
</tr>
<tr>
<td>HSL</td>
<td>720.8</td>
<td>2.03</td>
</tr>
<tr>
<td>HSM10</td>
<td>686.7</td>
<td>5.0</td>
</tr>
<tr>
<td>HSM15</td>
<td>718.3</td>
<td>2.7</td>
</tr>
<tr>
<td>HSZ10</td>
<td>685</td>
<td>3.45</td>
</tr>
<tr>
<td>HSZ15</td>
<td>695</td>
<td>2.02</td>
</tr>
</tbody>
</table>
3.2 Hardened state
3.2.1 Compressive strength

The average strength of three samples tested at each five ages is shown in Figure 1. The results indicated that the compressive strength of the concrete ($f'_{c}$) containing zeolite is lower than the HSL at all ages. At 28 day, the compressive strength of HSM10 is slightly higher than the HSL specimen whereas, from 3 day age, the compressive strength of HSM15 is higher than the HSL and HSM10.

![Figure 1: Compressive strength of SCC mixes](image)
3.2.2 Split tensile strength and modulus of elasticity

Modulus of elasticity of concrete ($E_c$) mixes and the tensile strength test of concrete ($f_{td}$) in an indirect way was measured in accordance to ASTM C469 and ASTM C496, respectively. It is a usual practice to consider $E_c$ and $f_{td}$ as a factor time's square root of $f'_c$, like as in Eq. (1) and Eq. (2):

$$E_c = \alpha f'_c^{0.5}$$  \hspace{1cm} (1)
$$f_{td} = \beta f'_c^{0.5}$$  \hspace{1cm} (2)

The values of $\alpha$ and $\beta$ and the standard deviation ($\sigma$) for each specimen is presented in Tables 3 and 4. Comparison of SCC mixes indicated that using metakaolin resulted in higher modulus of elasticity for both two different percentages and higher tensile strength compared to either natural pozzolan zeolite mixes or control mix.

Table 3: Modulus of elasticity of SCC mixes at 28 day age

<table>
<thead>
<tr>
<th>SCC Mix.</th>
<th>$f'_c$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSL</td>
<td>43.57</td>
<td>2.9×10^4</td>
<td>4393.43</td>
</tr>
<tr>
<td>HSM10</td>
<td>44.25</td>
<td>3.2×10^4</td>
<td>4810.53</td>
</tr>
<tr>
<td>HSM15</td>
<td>50.64</td>
<td>3.4×10^4</td>
<td>4777.84</td>
</tr>
<tr>
<td>HSZ10</td>
<td>40.55</td>
<td>2.5×10^4</td>
<td>3925.94</td>
</tr>
<tr>
<td>HSZ15</td>
<td>28.00</td>
<td>2.0×10^4</td>
<td>3779.64</td>
</tr>
</tbody>
</table>
Table 4: Cylindrical compressive and tensile test results

<table>
<thead>
<tr>
<th>Mix. name</th>
<th>$f'_c$ (MPa)</th>
<th>$\sigma$ (MPa)</th>
<th>$\varepsilon_d$ (MPa)</th>
<th>$\sigma$ (MPa)</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSL</td>
<td>43.57</td>
<td>2.30</td>
<td></td>
<td></td>
<td>0.34</td>
</tr>
<tr>
<td>HSM10</td>
<td>44.25</td>
<td>2.40</td>
<td></td>
<td></td>
<td>0.36</td>
</tr>
<tr>
<td>HSM15</td>
<td>50.64</td>
<td>6.06</td>
<td>2.63</td>
<td>0.29</td>
<td>0.37</td>
</tr>
<tr>
<td>HSZ10</td>
<td>40.55</td>
<td>1.90</td>
<td></td>
<td></td>
<td>0.29</td>
</tr>
<tr>
<td>HSZ15</td>
<td>28.00</td>
<td>1.70</td>
<td></td>
<td></td>
<td>0.32</td>
</tr>
</tbody>
</table>

3.3 Permeable voids

The test to determine the permeability pore volume was performed according to ASTM C 642. The test was carried out at 7, 14, 28 days of curing on cylindrical specimen ($\varnothing 100 \times H50$ mm). The results have shown that mixes containing metakaolin exhibit low permeable porosity comparing to control mix. This result can be explained by the effect of metakaolin to reduce and refine the porosity due to its pozzolanic action. Mixes containing zeolite increase the permeable porosity comparing to control mix (Figure 2). The reason can be attributed to the low activity index of this admixture and to the rheological behavior at fresh state of the mixes which required excessive demand of superplastizer involving an increase of the entrained air [1].

![Figure 2: Volume of permeable pore (voids, %) of HSSCC studied.](image-url)
3.4 Capillary absorption (sorptivity)

This test was carried out on cylindrical specimen (Ø 100 × H 50 mm) based on the AFREM procedure [2]. The coefficient of sorptivity (S) after 0.5, 1.2, 4, 8, 24, 48, 72 hr, for different mixes at 7, 14 and 28 days of aging are presented in Figure 3. Both types of pozzolans reduced sorptivity at the different tested ages. This effect is remarkable with the increase of the percentage of pozzolan replacement. Zeolite pozzolan exhibits better performances than metakaolin. This result can be explained by the refined porosity due to the reticular shape and by the high specific surface of zeolite.

![Figure 3: Sorptivity of HSSCC studied at different ages to (mm/h⁰.⁵).](image)

3.5 Diffusion of chloride ion

In this study, the NT- BUILD 492 method [3] was used to investigate chloride ion migration in non-steady-state conditions under electric field. The average values of three measurements of chloride migration coefficient for each mix are presented in
Figure 4. It is clear that, at the early ages of 3 and 7 day, due to low activity of pozzolans, no improvement in the trend of reducing the amount of chloride migration is occurred. However, at longer ages of 14 and 28 day, both admixtures at the two different percentages of pozzolans, reduced the chloride migration coefficient comparing to control mix. The quality of concrete is also evaluated based on the measurement of the current passing through the sample (Figure 5).

**Figure 4**: Results of non-steady-state migration coefficient of chloride ion

**Figure 5**: Relation between the initial current and (Dnssm).
3.6 Electrical resistivity

Figure 6 gives the results of electrical resistivity of the different mixes at 28 day of curing obtained, based on FM 5-578 [4]. Incorporation of the two admixtures involves an increase in electrical resistivity. The electrical resistance can be related to the penetration of chloride as shown in figure 7. Consequently, if the electrical resistance of concrete increase, the coefficient of chloride migration coefficient will decrease [5].

![Figure 6: Bulk electrical resistivity results (kΩ cm).](image)

![Figure 7: Relation between the electrical resistivity and (Dnssm).](image)
3.7 Water penetration

Water penetration test was carried out based on the BS EN 12390-8 [6] and the results are presented in Figure 8. As it is seen, replacing Portland cement by 10% and 15% of metakaolin or zeolite led to a reduction of the water penetration depth comparing to control mix.

![Figure 8: The water penetration depth in HSSCCs at 28 days of curing.](image)

4 CONCLUSIONS

1. Both admixtures used in this study, metakaolin and zeolite, have increased the demand of superplasticizer and the viscosity of the fresh SCC.
2. SCC mixes containing metakaolin showed better mechanical properties comparing to control and zeolite mixes.
3. At the age of 14 and 28 days, both admixtures at 10 and 15 percentages have decreased sorptivity and chloride migration coefficient.
4. A correlation has been established between electrical resistivity and chloride migration coefficient ($R^2 = 0.91$).
5. This study has shown the beneficial use of metakaolin in SCC in terms of mechanical and durability properties. The use of zeolite seems to be an interesting admixture for durability aspects.

6 REFERENCES


VARIOUS DURABILITY ASPECTS OF CEMENT PASTES AND CONCRETES WITH SUPPLEMENTARY CEMENTITIOUS MATERIALS

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Key words: Concrete, Supplementary Cementitious Materials, Durability indicators, Mercury Intrusion Porosimetry

Abstract. The use of supplementary cementitious materials (SCMs) as a constituent for concrete receives considerable attention, due to the lower CO₂ emission of these materials compared to the production of classic Portland cement. Furthermore, concretes incorporating SCMs show some improved durability properties. SCMs are mainly pozzolanic materials (Fly Ash or Metakaolin) or alkali-activated materials such as ground granulated blast slag (GGBS).

In this paper, the durability of concretes and cement pastes, which incorporate SCMs as partial replacement of cement, has been investigated in comparison with the CEM-I material one. Here, SCMs are Metakaolin, fly ash and GGBS. Water porosity, chloride migration and diffusion, electrical resistivity and natural and accelerated carbonation tests have been performed, in particular in order to assess durability indicators. The changes in the durability properties with the water curing time (from 7 days to more than 1 year) have been investigated. Coupled aggressions are also studied on these materials. For example, the materials are put in contact with chloride and sulphate solutions after partial carbonation. In addition, some aspects of the microstructure and of the pore structure are investigated (for example by Mercury Intrusion Porosimetry), in order to better understand the results obtained relatively to durability indicators.

The results show an evolution of the properties as a function of the cement replacement ratio by SCMs: the durability with regard to chloride penetration is improved but the resistance to carbonation is reduced (whatever the SCM type and content).
1 INTRODUCTION

Corrosion of steel rebars is one of the main causes of reinforced concrete degradation. This corrosion is due to carbonation or chloride ingress. If the corrosion due to chloride ions takes place more often in marine environments or in the presence of deicing salts, carbonation occurs systematically, in a more or less high degree, depending of the environmental conditions (relative humidity, temperature…). Durability indicators are generally required in order to evaluate these degradation risks [1-5].

Supplementary cementitious materials (SCMs) are included into concrete to substitute a part of cement in order to reduce the CO₂ footprint. SCMs are mainly pozzolanic materials including industry waste such as ground granulated blast slag (GGBS) and fly ash (FA) [7-8]. Metakaolin (MK) is also known for having similar effects. In concretes, these SCMs can contribute to improve strength and durability of these materials [9-10].

This study focuses on the durability indicators of SCMs cement materials such as water porosity, apparent chloride diffusion coefficient (D_{app,Cl}), electrical resistivity and portlandite amount [11-12], in addition to compressive strength. The resistance to the carbonation process (in replacement of gaz permeability as durability indicator) and chloride binding isotherms (CBIs) are also evaluated. In addition, microstructural study (XRD, thermal analyses, NMR spectroscopy and MIP) is performed to better understand the results on the durability indicators.

2 EXPERIMENTAL

Various concretes were designed with the same clinker and granular skeleton using siliceous aggregates. The main constituents of the cement and SCMs are given in table 1. The binder content and the water-binder ratio (w/b) of the concretes are equal respectively to 300 kg/m³ and 0.53 for all the mixtures. The studied binders are CEM I (OPC with 97% clinker), CEM III/A (with 62% GGBS) denoted CEM III GGBS(62%), CEM III/C (with 82% GGBS) denoted CEM III GGBS(82%), CEM I + 30% FA denoted CEM I FA(30%), CEM I + 10% MK denoted CEM I MK(10%) and CEM I + 25% MK denoted CEM I MK(25%). Cement pastes (w/=0.50) with the same binders were also prepared.

Table 1 : Chemical composition of the cement and SCMs tested (%) and Bogue calculation (with MK as metakaolin and FA as fly ash).

<table>
<thead>
<tr>
<th></th>
<th>CaO</th>
<th>SiO₂</th>
<th>Fe₂O₃</th>
<th>Al₂O₃</th>
<th>TiO₂</th>
<th>MgO</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>MnO</th>
<th>SO₃</th>
<th>Cl</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I</td>
<td>62.53</td>
<td>19.54</td>
<td>2.90</td>
<td>4.98</td>
<td>0.30</td>
<td>0.84</td>
<td>0.30</td>
<td>0.82</td>
<td>0.09</td>
<td>2.97</td>
<td>0.05</td>
</tr>
<tr>
<td>CEM III/A</td>
<td>49.77</td>
<td>29.86</td>
<td>1.28</td>
<td>8.10</td>
<td>0.46</td>
<td>4.61</td>
<td>0.40</td>
<td>0.56</td>
<td>0.16</td>
<td>2.29</td>
<td>0.32</td>
</tr>
<tr>
<td>CEM III/C</td>
<td>45.70</td>
<td>32.00</td>
<td>1.00</td>
<td>9.90</td>
<td>0.50</td>
<td>5.80</td>
<td>0.63</td>
<td>0.54</td>
<td>0.20</td>
<td>2.00</td>
<td>0.33</td>
</tr>
<tr>
<td>MK</td>
<td>0.00</td>
<td>66.29</td>
<td>4.29</td>
<td>21.30</td>
<td>1.12</td>
<td>0.25</td>
<td>0.84</td>
<td>0.49</td>
<td>0.00</td>
<td>0.08</td>
<td>0.00</td>
</tr>
<tr>
<td>FA</td>
<td>0.00</td>
<td>51.59</td>
<td>6.58</td>
<td>23.78</td>
<td>1.03</td>
<td>0.49</td>
<td>1.09</td>
<td>3.05</td>
<td>0.11</td>
<td>3.05</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Many water-curing times are chosen (7, 28, 90, 180 and 365 days) in order to take into account the evolution of the materials as a function of the age. In addition to compressive strength tests, various durability tests are performed on these concretes and pastes: water
porosity, chloride migration and diffusion [12], electrical resistivity [13] and natural and accelerated carbonation tests [14]. In addition, chloride binding isotherms (CBIs) are obtained by equilibrium method [11]. Finally, microstructure is characterized by mercury intrusion porosimetry (MIP) [12], XRD [15], TGA-DTA [14] and $^{29}$Si and $^{27}$Al nuclear magnetic resonance magical angle spinning spectroscopy (NMR MAS) [15].

3 RESULTS AND DISCUSSION

3.1 Compressive strength

At the young age (see figure 1), MK concrete has a higher compressive strength than others concretes which is consistent with previous studies [16] and related to high reactivity of MK. The high reactivity of MK at young age is related to their aluminate phases and their high surface area. MK pozzolanic reactions consume portlandite and produce C-S-H and C-S,Al-H denser [17]. Furthermore, MK also plays an accelerator on hydration of C$_3$S by the nucleating effect due to the increase of fines in a cementitious matrix. The filler effect could have an impact by increasing the compactness of concrete, and thus improving the mechanical properties. However, at the long term, the maximum compressive strength of MK concrete (65 MPa) is still lower than the CEM I concrete (80 MPa).

![Figure 1: Compressive strength of concretes in function of water-curing time. The values in % correspond to the hydration rate obtained by $^{29}$Si NMR (1-Q$^0$) on cement pastes.](image)

In contrast, concretes with GGBS or fly ash have a lower resistance at young age. The compressive strength increases with the water-curing time, depending on the nature of the SCMs. The compressive strength of FA concrete continues to increase after one year of water-curing. The CEM III GGBS(82%) concrete has the lowest compressive strength at the long term. Mechanical strength is related to the formation of C-S-H during hydration. Therefore, it
is interesting to have information about hydration process of anhydrous phases. When compressive strength is compare to hydration rate (obtained by $^{29}$Si NMR [15], see figure 1), these two properties increase with water-curing time for a same binder. However, at the same water-curing time, two concretes may have the same compressive strength but with two different hydration rates: for example, the compressive strength of CEM I and CEM III GGBS(62%) concretes are 75 MPa at one year and hydration rates are respectively 93% and 61%. As expected, the results show that the hydration process plays the main role in the development of mechanical properties (the quantity of C-S-H produced). In addition, there are the filler effect, increasing the compactness and certainly what can be defined as the "quality" of C-S-H network, depending on how they are produced (pozzolanic reactions, incorporation of Al...).

3.2 Porosity

Water porosity of all concretes decreases with water-curing time. The higher decrease is for the two GGBS concretes. Formation of denser C-S-H during pozzolanic reaction (in FA concrete) or slag alkali-activation (in GGBS concrete) needs time in comparison to clinker hydration (see hydration rates in figure 2).

These results are confirmed by pore size distribution of cement pastes obtained by MIP (see figure 3). The main peak of the distribution (50 nm) observed on CEM I decreases whereas the cement replacement by SCM increases. It can due to the filler effect but also to the formation of C-S-Al-H denser inside the porous during SCM reactions (pozzolanic and alkali-activated) since the peak in this area decreases between 28 and 365 days. This formation of C-S-Al-H are also seen by $^{29}$Si NMR [15]. Indeed, the average length of the C-S-H chains is longer in SCM cement pastes in comparison to CEM I one (see figure 3). It appears that C-S-H chains polymerise in SCM cement pastes. It is probably due to the presence of Al substituted to Si in C-S-H.
3.3 Chloride diffusion apparent coefficient ($D_{\text{app,Cl}}$)

Less porosity for SCM concretes has consequences on other durability properties such as chloride diffusion. Indeed, it appears in figure 4 that the chloride apparent coefficient ($D_{\text{app,Cl}}$) of all SCM concretes is lower compared to CEM I one at the long term. MIP shows a finer pore network for SCM cement pastes compared to CEM I one (see figure 3) explaining the results on $D_{\text{app,Cl}}$. However, the evolution of $D_{\text{app,Cl}}$ in function of water-curing time depends on the type of SCM used. The stabilization value is reached between 7 and 28 days for CEM I, 28 and 56 days for MK concretes, 91 and 180 days for GGBS concretes and after one year for FA concrete. As for compressive strength, pozzolanic reactions need times to improve $D_{\text{app,Cl}}$ in FA concrete.

These results obtained by migration tests are similar to those obtained by diffusion tests (see comparison in figure 4), for a same material after 365 days of water-curing.
Nevertheless, the observed differences are larger between these two tests for MK cementitious materials. For example, a factor 5 is reached for $D_{\text{app,Cl}}$ of CEM I MK(25%) obtained by migration and diffusion tests. In general, it is considered [12] that $D_{\text{app,Cl}}$ obtained by the migration depends only on the porous network. Indeed, during the migration test, interactions (between chloride and the cementitious matrix) are very weak and can be often neglected. In contrast, $D_{\text{app,Cl}}$ from diffusion tests depends in part on these interactions and also the porous network which explain why $D_{\text{app,Cl}}$ obtained by the migration are higher to $D_{\text{app,Cl}}$ obtained by diffusion tests. Consequently, CEM I MK(25%) seems to have a higher capacity for chloride binding which is confirmed by CBIs presented in figures 5A and 5B. In addition, XRD and $^{27}$Al NMR spectroscopy [15] show that, for a same NaCl concentration of the solution, CEM I MK(25%) produces more Friedel’s salt than CEM I (see figure 5C) and physical binding (accessed by combination of NMR and CBIs results) is also higher.

![Figure 5](image.png)

Figure 5: Concretes and cement CBIs (in left) and diffractogramms (in right) of crushed cement pastes in contact of NaCl solution (1.5 M). E ettringite, FS Friedel’s salt, P portlandite.

### 3.4 Resistivity

Resistivity tests show contrasted results (see figure 5A in right) in opposite to some other study [13]. There are two groups here. The first group, the resistivity of both CEM I and MK concretes does not evolve with water-curing time and stay inferior to 200 ohm.m. The second group, the resistivities of GGBS concretes and FA concrete evolve slowly with water curing time but reach high values after one year of water-curing. Figure 5B shows chloride apparent coefficients as the resistivity inverse function in order to verify the correlation reported in [13]. If the values at 7 days water-curing are removed (except for CEM I FA(30%)), a linear relation is found for each concrete. The hydration of materials at 7 days water-curing continues to evolve (including during the tests) and this is probably why the correlation between resistivity and chloride apparent coefficient is not relevant here.
3.5 Portlandite amount

For cement pastes, portlandite amount decreases when the substitution increases whatever the type of SCM (see figure 6).

Portlandite amount is only produced by the clinker and other binders contains less clinker than CEM I (dilution effect). In addition, the pozzolanic reactions in MK pastes or FA pastes consume portlandite. In order to separate the effect of the dilution and the consumption of portlandite (due to pozzolanic reactions or alkali-activation), the mass proportion of...
Portlandite compared to the clinker (Mp) is calculated (see green values in figure 6). For GGBS cement pastes, Mp are similar to those of CEM I cement pastes confirming that GGBS needs aqueous calcium hydroxide medium to dissolve but calcium hydroxide are not consumed during C-S-H formation. Therefore, portlandite amount depends only on the dilution effect in GGBS cement materials. In contrast, large amounts of portlandite are consumed by pozzolanic reactions in MK and FA cement materials. After one year of water-curing, more than 40% of portlandite amount (compared to CEM I Mp) are used in these reactions for CEM I MK(25%).

3.6 Carbonation depth and weight increase

The main consequence of portlandite amount decrease is that SCM concretes are less resistant than concrete based CEM I, against a same carbonation treatment (see figure 8A). The carbonation depths are function of the cement replacement by SCM. In addition, an increase of water-curing time reduces the depth of carbonation for all concretes.

Crushed cement pastes were carbonated until mass constant in chamber (HR 65%, 1.5% CO2 and T=20°C), the weight increases (%Wi) are given in the table 2. It depends on the type and the cement replacement percentage. One hypothesis is that it is determined by the amount of carbonatable material and in particular the quantity of C-S-H + Portlandite. However, this hypothesis does not explain why the CEM I FA(30%) had the lowest %Wi of all carbonated cement pastes. Indeed, the portlandite amount (see figure 6) in CEM I FA(30%) is close to that of the GGBS(62%) cement paste. The main cause for the %Wi is the formation of CaCO3. Nevertheless, the amount of calcium depends on the type of binder. Mass proportion of CaO by type binder is 62% for CEM I, 47% for CEM I MK(25%), 50% for CEM III

Figure 8: Carbonation depth (in left) measured by phenolphthalein tests on concretes after various water-curing times and natural (NC) or accelerated carbonation (AC) (1.5 % CO2, 65% RH and T=20°C). Comparison between % of weight increase after accelerated carbonation until constant mass and % of CaO in binder on crushed cement pastes after one year of water-curing (in right).
GGBS(62%) and 45% for CEM I FA(30%). Figure 7B shows the correlation between weight increase and the percentage of CaO initially present and there is a linear relation between these two. Consequently, CaO of binder is the only parameter depending of the weight increase after an accelerated carbonation until constant mass.

Table 2: % of weight increase after accelerated carbonation until mass constant in chamber (HR 65% et 1.5% CO₂) on crushed cement pastes after one year of water-curing.

<table>
<thead>
<tr>
<th></th>
<th>CEM I</th>
<th>CEM I MK(10%)</th>
<th>CEM I MK(25%)</th>
<th>CEM I FA(30%)</th>
<th>CEM III GGBS(62%)</th>
<th>CEM III GGBS(82%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight increase (in %)</td>
<td>19</td>
<td>17</td>
<td>15</td>
<td>13</td>
<td>15</td>
<td>15</td>
</tr>
</tbody>
</table>

3.7 Resistance to coupled aggressions

In the real environmental conditions, concretes are attacked by multiple aggressive agents in the same time. Therefore, it is interesting to make some tests with a coupled aggression, especially they are few in the literature [15,18-19]. $D_{app,Cl}$ are measured after a partially carbonation of cement pastes cylindrical samples and then compared to reference ones (see figure 9). After carbonation, $D_{app,Cl}$ increases for all cement pastes (except CEM III GGBS(62%)). The highest increases are for CEM I MK(25%), for CEM I FA(30%) and for CEM III GGBS(82%) (a factor 4). For CEM III GGBS(82%) cement paste, cracks are observed after carbonation on the exposed surface and can explain this result on $D_{app,Cl}$. These cracks are not observed on the other concretes. In contrast, carbonation of cement materials participate to close porosity by formation of CaCO₃ [3] and therefore $D_{app,Cl}$ of carbonated cement materials should be decrease. As seen in section 3.3, interactions between chloride and cementitious matrix are influent on $D_{app,Cl}$. Since the results presented in figure 9, it seems they decrease which is confirmed by results obtained on chloride binding isotherms presented in [15], explaining the increase of $D_{app,Cl}$.

![Figure 9](image-url)
Finally, the same experiments were made with a contact solution containing both chloride and sulphate (see figure 10). When the contact solution contains sulphate, $D_{\text{app,Cl}}$ increases for all cement pastes (comparing black and red bars in figure 10). No visible cracks are observed on cement pastes cylindrical samples. $D_{\text{app,Cl}}$ of CEM I MK(25%) cement paste is the lowest. It is probably due to a finer poral network of CEM I MK(25%) (see figure 3B) but also to his higher capacity for bind chloride (see figure 3B) and obviously sulphate too. Chloride and sulphate seem to compete to interact with cementitious matrix (see figure 11). When the cement pastes are partially carbonated before contact with chloride and sulphate solution (see figure 10), there are the same results obtained in figure 9, including no difference for CEM III GGBS(62%). There is almost no chloride binding in carbonated cement paste (see figure 11) which contribute to increase $D_{\text{app,Cl}}$.

![Figure 10](image-url)

**Figure 10:** Chloride apparent coefficients obtained by profile method on cement paste after 365 days of water-curing. In black, cylindrical sample are put in contact with NaCl solution (0.5 M NaCl) during 3 months. In red, cylindrical sample are put in contact with NaCl solution with sulphate (0.5 M NaCl + 0.06 M Na$_2$SO$_4$) during 3 months. In green, cylindrical sample are partially carbonated in accelerated conditions (AC) (2 months, 1.5 % CO$_2$, 65% RH and $T=20^\circ$C) after 15 days of drying ($60^\circ$C), then they are put in contact with NaCl solution with sulphate (0.5 M NaCl + 0.06 M Na$_2$SO$_4$) during 3 months.
Figure 11: Chloride binding isotherms obtained by profile method on CEM I MK(10%) after 365 days of water-curing. In black, cylindrical sample are put in contact with NaCl solution (0.5 M NaCl) during 3 months. In red, cylindrical sample are put in contact with NaCl solution with sulphate (0.5 M NaCl + 0.06 M Na$_2$SO$_4$) during 3 months. In green, cylindrical sample are partially carbonated in accelerated conditions (AC) (2 months, 1.5 % CO$_2$, 65% RH and T=20°C) after 15 days of drying (60°C), then they are put in contact with NaCl solution with sulphate (0.5 M NaCl + 0.06 M Na$_2$SO$_4$) during 3 months.

12 CONCLUSIONS

In terms of durability properties, SCM concretes exhibit advantages compared to CEM I concretes. A decrease in porosity is recorded and the apparent coefficient of chloride ion diffusion. Chloride ingress depends also on the type of SCM used and the age of cement materials. However, the lower Portlandite content of SCM concretes makes them more sensitive to carbonation for a same exposure condition (RH, T°C, % of CO$_2$ and time).

Further investigation of the microstructure yields a better understanding of these advantages and disadvantages concerning carbonation, chloride and sulphate ingress. The finer pore network for all SCM cement materials and the largest capacity to bind chlorides for MK ones improve $D_{app,Cl}$ compared to CEM I materials one. However, $D_{app,Cl}$ increases if the material is partially carbonated before chloride diffusion. The influence of sulphate on chloride ingress has been demonstrated. Chloride and sulphate compete to bind with cementitious matrix and $D_{app,Cl}$ increases for all cement materials in this case.

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REFERENCES


A STUDY ON AN INDICATOR FOR ENVIRONMENTAL IMPACTS OF CEMENT INDUSTRY

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Key words: Cement, Environmental Impact, Indicator, Waste, By-product

Abstract. An indicator appropriate to evaluate the environmental impacts of cement industry was investigated in this study. Although there are already many indicators for environmental impacts developed all over the world, a comparatively simplified evaluation method was discussed to easily calculate the environmental impacts. Especially as an indicator for resource recycling, recycling rates were focused on in this study. The value of this indicator was compared to the value obtained from a conventional LCA method. As a result, the values of the indicator for resource recycling are in good agreement with the values obtained from the conventional LCA method. For this indicator, the use of normal cement provided a higher value than the use of blended cement. It was suggested that the indicator for resource recycling which is calculated using recycling rates could be effective in the evaluation of environmental impacts of cement manufacturing.

1 INTRODUCTION

It is well known that a large amount of CO₂ is released in cement manufacturing due to decarbonation of limestone and fossil fuel combustion. In Japan, the CO₂ emission for the cement industry accounts for 3 to 4% of the domestic total CO₂ emission. Statistically, in fiscal year 2012, 1,276 million tons of CO₂ (1,343 million tons CO₂-eq. of greenhouse gases) were emitted domestically [1], while the CO₂ emission in cement industry in Japan was about 42 million tons. As a method for environmental impact reduction of cement, therefore, blended cement is often used nowadays.

On the other hand, cement manufacturing significantly contributes to resource recycling. About 28.5 million tons of industrial wastes and by-products, which include blast furnace slag, coal ash, dirt and sludge, by-product gypsum, construction soil, ash (dust), nonferrous slag, wood chips, foundry sand, steel slag, waste plastic, waste oil, waste activated clay, reclaimed oil, used tire, meat and bone meal, waste coal and so on, were recycled in cement manufacturing in fiscal year 2012 as alternative raw materials and fuels as shown in Table 1.
Table 1 Wastes and by-products recycled in cement manufacturing in Japan in 2012 [2]

<table>
<thead>
<tr>
<th>Kind</th>
<th>Amount (thousand tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blast furnace slag</td>
<td>8,485</td>
</tr>
<tr>
<td>Coal ash</td>
<td>6,870</td>
</tr>
<tr>
<td>Sewage sludge</td>
<td>2,987</td>
</tr>
<tr>
<td>By-product gypsum</td>
<td>2,286</td>
</tr>
<tr>
<td>Waste soil from construction</td>
<td>2,011</td>
</tr>
<tr>
<td>Cinder, soot, dust</td>
<td>1,505</td>
</tr>
<tr>
<td>Nonferrous slag</td>
<td>724</td>
</tr>
<tr>
<td>Wood chips</td>
<td>633</td>
</tr>
<tr>
<td>Foundry sand</td>
<td>492</td>
</tr>
<tr>
<td>Waste plastic</td>
<td>432</td>
</tr>
<tr>
<td>Steel slag</td>
<td>410</td>
</tr>
<tr>
<td>Waste oil</td>
<td>273</td>
</tr>
<tr>
<td>Waste white clay</td>
<td>253</td>
</tr>
<tr>
<td>Recycled oil</td>
<td>189</td>
</tr>
<tr>
<td>Waste tire</td>
<td>71</td>
</tr>
<tr>
<td>Meat and bone meal</td>
<td>65</td>
</tr>
<tr>
<td>Coal mining waste</td>
<td>0</td>
</tr>
<tr>
<td>Others</td>
<td>835</td>
</tr>
<tr>
<td>Total</td>
<td>28,523</td>
</tr>
</tbody>
</table>

This value corresponds to about 10% of domestic total reused amount.

Japan is a very small island country that has limited general and industrial waste disposal sites. Effective utilization of resources and resource recycling are very important as well as reduction of CO₂ emission for creating a sustainable society at least in Japan. Although cement manufacturing has a heavy load on the environment in terms of CO₂ emission, we also pay attention to significant contribution to resource recycling. From this point of view, the use of blended cement instead of normal cement reduces the recycling of wastes in cement manufacturing. It can be said that the environmental impacts of cement manufacturing should be evaluated from the viewpoints of not only CO₂ emission but also resource recycling.

Based on the above background, an indicator appropriate to evaluate the environmental impacts of cement industry was investigated in this study. There are already many indicators for environmental impacts developed all over the world. In this study, however, a comparatively simplified evaluation method was discussed to easily calculate the environmental impacts.

2 INDICATORS FOR RESOURCE RECYCLING

The Concrete Committee of the Japan Society of Civil Engineers organized the Subcommittee on Development of ‘Environment-Harmonized Concrete Material Science and Engineering’ which proposed an environmental harmonization index to evaluate ‘minimal
impact on the environment.” This index involves the factors “$I_G$” for CO$_2$ emission index, “$I_W$” for resource recycling index and “$I_L$” for local material utilization index. The factor $I_W$ is expressed as follows:

$$I_W = \frac{U_r - C_w}{U_v + U_r}$$ \hspace{1cm} (1)

where $U_r$ is the amount of recyclable resources used, $C_w$ is the amount of waste generation and $U_v$ is the amount of virgin resource used; when $U_r < C_w$, $I_W=0$, and where the recyclable resources include both wastes and by-products.

As shown in Equation (1), this $I_W$ does not distinguish between waste recycling and by-product recycling. It is generally thought, however, that the waste recycling is more difficult than the by-product recycling. The difficulty of recycling could be expressed by the recycling rate of resources. Thus, the following equations were considered as indicator for resource recycling.

$$I_{W1} = \frac{\sum\{10(1 - R_k)\} w_k - C_w}{U_v + U_r}$$ \hspace{1cm} (2)

$$I_{W2} = \frac{\sum\{11/(10R_k + 1) - 1\} w_k - C_w}{U_v + U_r}$$ \hspace{1cm} (3)

$$I_{W3} = \frac{\sum\{10 - R_k/(1 - 0.9R_k)\} w_k - C_w}{U_v + U_r}$$ \hspace{1cm} (4)

where $R_k$ is recycling rate for a recyclable resource of $k$ (as a decimal) and $w_k$ is the amount of the recyclable resource $k$ used, and where $U_r$ equals $\sum w_k$. In each equation, only the weighing factor of $w_k$ is different. Each weighing factor can be expressed in a different manner. The relationship between $R_k$ and the weighing factor can be shown as Figure 1.

![Figure 1 Relationship between the recycling rate $R_k$ and the weighing factor of $w_k$](image)

**3 EVALUATION OF INDICATORS**

The recycling rate for wastes and by-products that were used in the Japanese cement industry in both 1996 and 2011 is shown in Table 2 [3]. The values from close years to 1996 were used for 1996, where data of recycling rate were lacking.
Table 2 Recycling rates of wastes and by-products used in this study [3]

<table>
<thead>
<tr>
<th>Kind</th>
<th>Recycling rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Year 1996</td>
</tr>
<tr>
<td>Blast furnace slag</td>
<td>100</td>
</tr>
<tr>
<td>Fly ash</td>
<td>100</td>
</tr>
<tr>
<td>Coal ash</td>
<td>70</td>
</tr>
<tr>
<td>Sewage sludge</td>
<td>38</td>
</tr>
<tr>
<td>By-product gypsum</td>
<td>100</td>
</tr>
<tr>
<td>Waste soil from construction</td>
<td>32</td>
</tr>
<tr>
<td>Cinder, soot, dust</td>
<td>2</td>
</tr>
<tr>
<td>Nonferrous slag</td>
<td>79</td>
</tr>
<tr>
<td>Wood chips</td>
<td>22</td>
</tr>
<tr>
<td>Waste plastic</td>
<td>24</td>
</tr>
<tr>
<td>Steel slag</td>
<td>91</td>
</tr>
<tr>
<td>Waste oil</td>
<td>30</td>
</tr>
<tr>
<td>Waste tire</td>
<td>91</td>
</tr>
</tbody>
</table>

Using these recycling rates for wastes and by-products, the indicators for resource recycling shown in Equations (2) to (4) were evaluated in regard to portland cement (PC), portland blast furnace slag cement (BB) and portland fly ash cement (FB). The blast furnace slag and fly ash content rates to BB and FB were assumed to be 42.6 % and 15.5 %, respectively while simulating the Type B cement shown in the Japan Industrial Standard (JIS) R 5211 for portland blast furnace slag cement and R 5213 for portland fly ash cement.

The amount of recyclable resources used ($w_k$) for each of the wastes and by-products in the BB and FB were set by multiplying the $w_k$ value for PC by the content rate of portland cement in BB and FB (that is $(100 – 42.6) %$ and $(100 – 15.5) %$). Based on this assumption, the amount of recyclable resources used in each cement in Japan in both 1996 and 2011 is summarized in Table 3 [3]. In this table, since the recycling rates of foundry sand and waste activated clay have not existed, these materials were combined into the category ‘Others’.

Using the values in Tables 2 and 3, the evaluation results of the indicators for resource recycling represented in Equations (2) to (4) are shown in Figures 2 and 3, where it is assumed that the recycling rates of others are 37 % in 1996 and 52 % in 2011 which are the average recycling rates in each year in Japan [3]. In Figure 2, the results for PC obtained from the recycling rate and the amount of wastes and by-products used in 1996 (PC1996, 1996 basis), the recycling rate in 1996 and the amount of wastes and by-products used in 2011 (PC2011, 1996 basis), and the recycling rate and the amount of wastes and by-products used in 2011 (PC2011, 2011 basis) are compared. In Figure 3, the results for PC, BB and FB obtained from the recycling rate and the amount of wastes and by-products used in both 1996 and 2011 are compared.

From Figure 2, the value of the indicator largely increases from 1996 to 2011 in any case when the same recycling rate (1996 basis) is used, since the amount of wastes and by-products recycled increases significantly from 1996 to 2011. Meanwhile, when the recycling rate for each year is used, the increase in the value of the indicator from 1996 to 2011 is not so
Table 3 Wastes and by-products recycled in each cement in Japan in 1996 and 2011 [3]

<table>
<thead>
<tr>
<th>Kind</th>
<th>Year 1996</th>
<th></th>
<th></th>
<th>Year 2011</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Blast furnace slag</td>
<td>0.0</td>
<td>426.0</td>
<td>0.0</td>
<td>0.0</td>
<td>426.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Fly ash</td>
<td>0.0</td>
<td>0.0</td>
<td>155.0</td>
<td>0.0</td>
<td>0.0</td>
<td>155.0</td>
</tr>
<tr>
<td>Coal ash</td>
<td>34.6</td>
<td>19.9</td>
<td>29.3</td>
<td>115.8</td>
<td>66.5</td>
<td>97.8</td>
</tr>
<tr>
<td>Sewage sludge</td>
<td>9.6</td>
<td>5.5</td>
<td>8.1</td>
<td>46.4</td>
<td>26.7</td>
<td>27.6</td>
</tr>
<tr>
<td>By-product gypsum</td>
<td>26.2</td>
<td>15.0</td>
<td>22.1</td>
<td>37.5</td>
<td>21.5</td>
<td>31.7</td>
</tr>
<tr>
<td>Waste soil from construction</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>33.8</td>
<td>19.4</td>
<td>28.6</td>
</tr>
<tr>
<td>Cinder, soot, dust</td>
<td>4.6</td>
<td>2.6</td>
<td>3.9</td>
<td>24.2</td>
<td>13.9</td>
<td>20.5</td>
</tr>
<tr>
<td>Nonferrous slag</td>
<td>14.8</td>
<td>8.5</td>
<td>12.5</td>
<td>11.7</td>
<td>6.7</td>
<td>9.9</td>
</tr>
<tr>
<td>Wood chips</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>10.2</td>
<td>5.8</td>
<td>8.6</td>
</tr>
<tr>
<td>Waste plastic</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>7.6</td>
<td>4.4</td>
<td>6.4</td>
</tr>
<tr>
<td>Steel slag</td>
<td>12.9</td>
<td>7.4</td>
<td>10.9</td>
<td>7.7</td>
<td>4.4</td>
<td>6.5</td>
</tr>
<tr>
<td>Waste oil</td>
<td>1.3</td>
<td>0.8</td>
<td>1.1</td>
<td>4.6</td>
<td>2.6</td>
<td>3.9</td>
</tr>
<tr>
<td>Waste tire</td>
<td>2.7</td>
<td>1.5</td>
<td>2.3</td>
<td>1.3</td>
<td>0.7</td>
<td>1.1</td>
</tr>
<tr>
<td>Others</td>
<td>28.4</td>
<td>16.3</td>
<td>24.0</td>
<td>28.4</td>
<td>16.3</td>
<td>24.0</td>
</tr>
<tr>
<td>Total</td>
<td>135.1</td>
<td>503.6</td>
<td>269.2</td>
<td>329.3</td>
<td>615.0</td>
<td>433.2</td>
</tr>
<tr>
<td>Virgin materials (total)</td>
<td>1,520.9</td>
<td>873.0</td>
<td>1,285.2</td>
<td>1,326.8</td>
<td>761.6</td>
<td>1,121.1</td>
</tr>
</tbody>
</table>

large in any case. This is because the recycling rate itself gains from 1996 to 2011. Especially the change in the value of $I_{w2}$ is small. It shows that the reduction of the weighing factor associated with the increase of the recycling rate largely affects the result.

From Figure 3, the value of the indicator for PC is the largest, followed by FB and BB. It can be said that portland cement makes a larger contribution in terms of resource recycling than portland blast furnace slag cement and portland fly ash cement. The difference in the value of the indicator between 1996 and 2011 becomes clear when the indicator of $I_{w3}$ is used, which is attributed to the difference of the relationship between the recycling rate and the weighing factor of $w_k$.

4 COMPARISON BETWEEN PROPOSED AND CONVENTIONAL INDICATORS

The evaluation results of the indicator for resource recycling were compared with the results of a conventional LCA method. As a conventional LCA method, LIME (Life-cycle impact assessment method based on endpoint modeling) that has been developed in Japan was adopted in this study because this method is suitable for Japan’s industrial, economical and geographic conditions.

The estimation results for PC, BB and FB [4] are shown in Figure 4 compared to the values for $I_{w1}$, $I_{w2}$ and $I_{w3}$. The integrated values estimated by LIME show negative values, which means a contribution to the environment due to the recycling of wastes and by-products is greater than a burden to the environment due to CO$_2$ emission and so on. It is also found that the values of the indicator for resource recycling are in good agreement with the values
Figure 2 Evaluation results of each indicator for portland cement

Figure 3 Evaluation results of each indicator for each cement
obtained by LIME.

It was suggested, therefore, that the indicator for resource recycling which is calculated using recycling rates and the amount of wastes and by-products recycled could be effective in the evaluation of environmental impacts of cement manufacturing.

5 CONCLUSIONS

- As an indicator appropriate to evaluate the environmental impacts of cement manufacturing, an indicator for resource recycling which is calculated using recycling rate and the amount of wastes and by-products recycled was proposed.
- It was suggested that the indicator for resource recycling could be effective in evaluating the environmental impacts of cement manufacturing from the results compared with a conventional LCA method.

REFERENCES

CAN A GENERAL STRUCTURAL CODE FOR BOTH NEW AND EXISTING CONCRETE STRUCTURES ENHANCE THE WAY WE APPROACH SUSTAINABILITY FOR EXISTING STRUCTURES?

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Key words: fib Model Code, sustainability, existing concrete structures, structural assessment, durability, extension of life, adaptation

Summary: The paper discusses the adaptation and evaluation of existing structures and considers the development of a general structural code to deal with both new and existing concrete structures following an integrated life cycle perspective and service life design approach, promoting a holistic treatment of defined performance requirements, incorporating consideration of structural safety, serviceability, durability and sustainability.

1: INTRODUCTION

Discussions have been underway in fib (Fédération Internationale du Béton) about advancing the fib Model Code for Concrete Structures 2010 (MC2010) [1] and the desire to extend the current fib code provisions for matters relating to existing concrete structures.

These diverse discussions include an international workshop hosted by TNO in The Hague, the Netherlands on 30 June 2015. Invited speakers included representatives from different fib Commissions and other organisations who brought perspectives from across the globe. Their presentations stimulated vigorous discussion on many topics including:

- fundamental principles and reliability concepts for new and existing structures
- models and material characterisation for existing structures
- inspection, maintenance and retrofitting of existing structures
- evaluation, decision-making and management of existing structures, and
- ways of defining and evaluating sustainability of new and existing concrete structures.

The workshop outcomes set down ideas for possible ways forward and some aspirations for linking and addressing the requirements for new and existing structures. The findings were considered by the fib Presidium which formed a fib MC2020 Core Group to work on possible ways to take the fib MC2020 project forward. The debate has considered factors relevant to the preparation of a single merged general code that fully integrates the provisions for the design of new concrete structures and matters relating to existing concrete structures, including situations where new structural members are incorporated as parts of existing structures.

Such a general structural code would need to be able to deal effectively with both new and existing concrete structures and to follow an integrated life cycle perspective and service life
design approach, which promotes a holistic treatment of defined performance requirements, incorporating consideration of structural safety, serviceability, durability and sustainability, establishing a link to wider issues such as the through-life management, cost, environmental and societal impacts of concrete structures. Examples of the desired improvements include procedures for verifying what constitutes the most sustainable engineering solution, especially with regard to the range of circumstances encountered with existing structures. Amongst other goals, the fib MC2020 project provides the opportunity to further develop and enhance the framework to be used for establishing the ‘sustainability’ of different options for managing or making an intervention to extend the useful life of a concrete structure or building.

Thus, summarising, such a general structural code should have integrated provisions for both the design of new structures and all the activities associated with the assessment, the through-life management of existing concrete structures and performing any necessary interventions.

2: SUSTAINABLE DECISION MAKING

Classically sustainable decision making has been based upon consideration of three factors, that is: environmental, economic and social aspects of the entity being considered, without there being any assigned hierarchical difference between these three factors. Typically the life-cycle assessments made of the sustainability and of the carbon footprint of a constructed asset have assumed that the functionality of the structural elements of the building or element of infrastructure concerned are assured such that these will not require any maintenance, repair or replacement under normal circumstances, such that they will not have significant operational energy requirements nor produce notable emissions within the study period being considered.

![Figure 1: Some facets of sustainable construction including functional requirements (Matthews [2])]({{<imagename>}})

However, this is not necessarily the case for existing structures and, as part of the assessments made of through-life sustainability impacts or of the through-life carbon footprint of such structures, consideration needs to be given to the functional performance requirements
and how well an existing structure, which may have experienced some degree of deterioration or damage during its life, is meeting these requirements. Furthermore due to revised performance requirements or changed circumstances, such as those associated with increased vehicle weights or climate change effects, the magnitude of the applied loads that need to be resisted may have increased during the life of the structure under consideration or may be expected to increase in the future. Figure 1 illustrates various facets of sustainable construction including those relating to functional requirements.

3: POTENTIAL BENEFITS OF ADAPTATION OF EXISTING BUILDINGS

Re-use / adaptation of existing buildings and other constructed assets offers a potentially sustainable way to extend their useful life, with some adaptations and changes in use being quite innovative, as the examples presented in Figure 2 show.

![Figure 2: Right: Adaptation of a water tower structure in Jaesbergborg, Denmark into 40 flats
Left: Conversion of a grain silo in Oslo, Norway into student housing (courtesy P Hajek [3])](image)

The re-use / adaptation of existing buildings and extension of life of other constructed assets brings a variety of environmental, economic and social benefits relative to their reconstruction, which potentially includes:

- a reduction in demolition works and the transport of demolished material from the site
- less new construction work
- reduced use of primary material and energy sources that would be required for a new development, as well as savings in the transport of these materials
- lower impact on the surrounding area (less noise, dust and other emissions)
- reduction of excavation works including the transport of excavated material
- lower first and operational cost, reduced environmental impacts and social disruption
Figure 3 shows illustratively the potential beneficial reductions in the operational energy performance of existing buildings of various ages following upgrading and adaptation to meet contemporary requirements relative to their embodied energy (ie. that associated with their original construction and through life-care to date), together with the indicative additional energy investment that would be required to achieve the desired improvements. A similar diagram could be prepared for the associated embodied and operational environmental impacts.

However, prior to embarking upon such improvement works it is necessary to establish the suitability of the existing building for these works and, sometimes critically, its prospective future durability, through-life care needs and whether particular works are required to achieve the desired extension of life (or possibly even continued service in some extreme cases).

4: THROUGH-LIFE CARE & EXTENSION OF LIFE OF CONSTRUCTED ASSETS

Figure 4 which is taken from Chapter 9 of fib MC2010[1] presents an idealised representation of the through-life performance of a structure which requires a remedial intervention to allow it to meet its intended design service life / extend its useful life.

In the case of a new build structure, the first condition control activities (inspection, survey, testing and assessment) would be performed after completion of construction, with the Birth Certificate document being issued if the structure passes the evaluation. During the post-construction lifetime, knowledge of the performance of the structure is accumulated by means of through-life condition surveys and / or monitoring.

Evaluation of the performance records against the performance criteria established during design (or as subsequently revised) supports decision-making as to whether there is a need for a physical intervention or a revision of the through-life care plan for the structure. After carrying out the (required) intervention, the condition control procedure is performed and a Rebirth Certificate for the structure is issued, assuming the structure passes the evaluation.
**Figure 4**: Through-life asset management, assessment, evaluation & intervention (Figure 9.3-1 fib MC2010 [1])

**Notes to Figure 4:**

**Condition assessment & evaluation** – refer Chapter 9 of fib MC2010 [1]

<table>
<thead>
<tr>
<th>Stage 1</th>
<th>Details about the design concepts and execution of the original structure along with the expectations for its performance, including material specifications and information upon the quality on execution, to allow the Birth Certificate document to be prepared.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 2</td>
<td>Gathers information upon / monitors through-life environmental and loading influences, as well as performance post construction and prior to a preventive or remedial intervention.</td>
</tr>
<tr>
<td>Stage 3</td>
<td>As Stage 1 but for preventive or remedial intervention(s), plus the definition of appropriate success criteria upon which to evaluate the subsequent performance of the intervention to allow the Re-birth Certificate document to be prepared.</td>
</tr>
<tr>
<td>Stage 4</td>
<td>Monitoring of through-life performance post-intervention, using defined performance indicators and environmental parameters and related factors establishing the context within which this data should be evaluated against the performance / success criteria defined in Stage 3 (and recorded in the Re-birth Certificate document, which forms part of the Service-Life File).</td>
</tr>
</tbody>
</table>
During the post-intervention lifetime the knowledge of the performance of the (possibly adapted or upgraded) structure is accumulated by means of through-life condition surveys and / or monitoring.

Figure 4 uses the terms ‘Birth Certificate’ and ‘Re-birth Certificate’, with these being defined as follows.

**Birth Certificate:** A document, report or technical file (depending on the size and complexity of the structure concerned) containing engineering information formally defining the form and the condition of the structure after construction.

The document / report should provide specific details on parameters important to the durability and service life of the concrete structure concerned (e.g. cover to reinforcement, concrete permeability, environmental conditions, quality of workmanship achieved etc) and the basis upon which future knowledge of through-life performance should be recorded. This framework provides a means of comparing actual behaviour / performance with that anticipated at the time of design of the structure. The document / report facilitates ongoing (through-life) evaluation of the service life which is likely to be achieved by the structure under the environmental and loading conditions appertaining.

It is relevant to note that the issue of a Birth Certificate could be used to provide formal recognition that a newly built structure satisfies the specified performance requirements at the time of its completion, and has been formally accepted. If the structure does not meet these requirements it would not be considered to be completed. On this basis there could potentially be a need for some forms of intervention before the "Birth Certificate" is issued for a structure (NB. This could equally apply in the case of the “Re-birth Certificate” defined below).

**Re-birth Certificate:** Similar to the Birth Certificate for a structure, but relates to the information and circumstances associated with a project for the repair / remediation / refurbishment of the structure or a part thereof to extend its anticipated service life.

Chapter 9: Conservation in *fib MC2010*\(^{[1]}\) gives further details of these concepts, which are also additionally developed in a paper by Matthews et al\(^{[4]}\) that provides supplementary explanation and background to what is contained in *fib MC2010*. The cited paper is part of a special series of publications relating to *fib Model Code 2010* which provide further details and insights into the background to and application of its recommendations.

Included in this special series of publications is a paper by Sakai\(^{[5]}\) concerning the inclusion and treatment of sustainability in *fib Model Code 2010*. Amongst other matters, Sakai discusses the wider relationships between structural safety, sustainability and the environmental burdens which arise from selecting different levels of structural safety. The central issue in these considerations is the level of safety sought. This is a particularly difficult decision to make in respect of low probability but extremely high consequence events (such as extreme earthquakes and the incidents that may subsequently arise as a result of them). If too high a level of safety is adopted, the conservatism of this decision, and the associated design and assessment connotations that this has, will have adverse economic, environmental and social implications. On the other hand if a low level of structural safety is adopted, the resulting structure will be efficient for the loads and circumstances that it will typically experience. It would also be efficient in terms of its cost, the resources used and the environmental impact it produces. However, should the actions exceed those assumed, it would have a decreased level of safety.
Thus again consideration of functionality of the constructed asset in conjunction with economic, environmental and social factors is central to making a holistic evaluation of its sustainability. Providing an appropriate level of robustness for exceptional events, and resilience for subsequent recovery after such events, is clearly important. Central considerations in this will be human safety (and potentially environmental protection depending upon the nature of the building or constructed asset), social disruption, recovery and the environmental burden that would arise from subsequent repair / reconstruction activities.

Recognising that one of the central principles of fib MC2010 is its adoption of the concept of performance-based design and assessment taking account (amongst others) of structural safety (ultimate limit state, robustness, etc), serviceability, sustainability and durability over a defined service life; such performance requirements need generally accepted verification procedures that can be used to demonstrate whether the requirements have been satisfied or not. The current sustainability provisions of fib MC2010 only present a framework and do not include detailed procedures for verification, which leaves an important challenge for fib and the wider engineering community. Knowing this ISO/TC71/SC8 has been working over recent years on the development of environmental standards for the assessment of concrete and concrete structures[6].

The concept of fib MC2020 provides an important opportunity to make further progress towards an holistic approach to the evaluation of sustainability of both new and existing concrete structure which takes account of functionality (structural safety and related factors) of the constructed asset in conjunction with economic, environmental and social factors over the anticipated length of the residual life of the asset, which would potentially be based on consideration of a number of scenarios defining future asset usage, management and through-life care.

5: STRUCTURAL ASSESSMENT & EVALUATION OF EXISTING STRUCTURES

The diverse portfolio of existing structures poses an important sustainability challenge to contemporary engineering practice. In this regard, structural engineers are faced with the critical task of assessing the actual safety level of existing structures and what the possibilities are for extending their useful life in accordance with sustainability principles, while taking account of social and economic constraints.

To discharge their task structural engineers seek to utilise currently available guidance, but this is often of limited assistance. If structural engineers turn to contemporary structural codes conceived for the design of new structures with current materials, this can lead to an excessively conservative outcome with associated negative environmental, social and economic consequences. This arises because the conservative verification procedures given in design codes are not appropriate to the circumstances relating to existing structures and can incorrectly indicate that the structures concerned do not satisfy the relevant performance limit states, which are therefore judged to be unsafe or unsatisfactory, when in fact they are not. This can result in a large amount of unnecessary intervention work being undertaken for the upgrading of the structures or (worse still) their demolition and reconstruction, with significant negative environmental, social and economic consequences.

It is clear that for existing structures there is a need to establish ‘new principles and new verification rules’ which go beyond the current scope of the design codes conceived for new
structures. The process of extending and upgrading the current provisions to create new ‘future’ structural codes will require a revised way of treating uncertainties. For new structures the treatment of uncertainty is essentially based on information gained from past experience, however, for existing structures it is essential to acquire information about the specific structure concerned, although general knowledge of the typical properties of materials previously used may be valuable during the initial stages of the assessment.

These details required will typically include knowledge of construction process and the use of the structure, the alterations and interventions which have been made, as well as the deterioration processes which may be active and how much deterioration has occurred and where, along with details of the possible misuse and / or damage caused to the structure. Gathering suitable information will typically involve the desk study of available records and information about past performance, but will commonly require survey inspections, testing, investigations and possibly monitoring over an extended period.

For existing structures it is necessary to consider two types of uncertainty where:

- Type 1 uncertainties are related to the inherent natural variability which is encountered, these have an aleatoric or random character and the scatter that is observed can be measured and described in objective terms; and
- Type 2 uncertainties which deal with model and statistical uncertainties that need to be addressed, they can be classified as epistemic uncertainties and are related to a lack of appropriate knowledge, with their treatment being based upon intuition, expert opinion or engineering judgement, or a combination of all of these.

For practical engineering problems a Bayesian approach is used giving the same weight to the two types of uncertainties; a likelihood of occurrence interpretation is used for the random uncertainties, whereas a degree of belief interpretation is used for the epistemic uncertainties.

The random uncertainties are related to mechanical properties of the materials, to the actions and to the model uncertainties introduced in description of material properties and actions; they can be modelled by means of continuous random variables, defined by their Probability Density Function (PDF), Cumulative Distribution Function (CDF) and Coefficient of Variation (COV).

The epistemic uncertainties are mainly related to the lack of structure knowledge and to the possible choice between alternative resistance models; they can be modelled as discrete random variables, described by their PDF or by means of event tree, which is more appropriate when there are a limited number of variables.

An important issue for existing structures is the selection of an appropriate target reliability level, which may be modified relative to the values assumed for new structures, with consideration being given to economic, social and sustainability considerations. In this regard:

- Economic considerations need to recognise that it will be more expensive to achieve a given (higher) level of safety in existing structures, compared with new structures. Furthermore design rules for new structures are very often conservative because the cost of achieving a higher degree of safety during design and construction is typically very small. Achieving a higher level of safety in an existing structure will typically involve considerable expenditure, so the cost of achieving an incremental improvement in safety in an existing structure will be great because of the significant costs of the upgrading works relative to those for a new structures during design.
- Sustainability aspects are related to the need to reduce waste and to implement recycling, as well as to reduce energy consumption and emissions from the works.
• Social aspects need to consider factors such as the disruption caused when interventions are made upon existing structures, as there will be a need to displace the occupants, and the important limitations which arise when dealing with heritage structures. These are considerations which do not affect new structures.

Finally the modification of the target reliability values can be made with consideration being taken of the:
• current design code values and contemporary practice
• economic optimisation criteria (based upon the minimum expected overall cost)
• types and importance of the structures concerned
• possible failure consequences, and
• social and economic criteria

However, human safety must always be considered at the same level of risk accepted for new construction, namely a maximum probability of an individual becoming a victim of structural failure of 1/100,000 per year.

The choice of target reliability values may also need to consider low probability but extremely high consequence events (such as extreme earthquakes and the incidents that may subsequently arise as a result of them), as discussed previously.

Another important aspect with existing structures is the structural analysis undertaken, and the structural model used should be able to reflect the actual condition of the structure. When necessary, it is important that appropriate deterioration models be used for the prediction of actual and future evolution of structural behaviour with time, which implies an ability to quantify the deterioration mechanisms acting and the corresponding uncertainties. Structural performance can be analysed by means of linear elastic analysis, linear elastic analysis with limited redistribution, plastic analysis and non-linear analysis; with the selection of the type of analysis being based on structural type, failure consequence class, validity of the models used for new structures and on the availability of new verification models.

It is important to emphasise that if the minimum requirements for the validity of resistance models used for new structures are not fulfilled and that new models able to describe the actual structural behaviour and the deterioration processes are not available, only non-linear analysis or design by testing or their combination should be used in the assessment process.

In regard to the safety format to be used for the verification of existing structures, it is important to recognise that the partial factor format can be used in conjunction with linear elastic analysis, linear elastic analysis with limited redistribution and plastic analyses. However, for non-linear analysis a full probabilistic analysis or global resistance factor analysis should be used.

6: fib MODEL CODE 2020

The concept of fib Model Code 2020 has evolved as a basis for advancing the current fib Model Code for Concrete Structures (fib Model Code 2010) and as providing a means of addressing the many aspirations that have emerged in the associated technical discussions.

The numerous aspirational goals that have been identified for fib MC2020, include that it should ideally:
• be a merged structural code dealing with both new and existing concrete structures
• be an operational model code that is oriented towards practical needs
include worldwide knowledge with respect to materials and structural behaviour
recognise the needs of engineering communities in different regions of the world
take an integrated life-cycle perspective
give a holistic treatment to structural safety, serviceability, durability & sustainability
define fundamental principles and a safety philosophy based on reliability concepts
use the performance based concept to remove specific constraints for novel types of concrete and reinforcing materials
provide fully integrated provisions based on generalized models and implementation of the level of approximation approach applicable in both the design of new structures and all the activities associated with the assessment
utilise refined mechanical response models, based on rational physical behaviours to replace where possible those based on empirical relationships with limited validity, thereby providing a consistent basis for both design and structural assessment
allow full advantage to be taken of information that can be acquired by testing and monitoring of existing structures
address robustness and redundancy for new and existing structures
consider material degradation and/or insufficient or deficient detailing of the provided material and behaviour models
consider needs for model improvement and treatment of uncertainties in models and model parameters for existing structures and the phased) construction of interventions
give attention to new types of concrete/repair materials and so forth
give attention to through-life management aspects, and
address end-of-service-life issues such as demolition, disposal and recycling

Thus there are profound challenges in various aspects of the proposed work. Important related considerations include not only involving all the fib Commissions, so that it is possible to draw upon the full range of expertise and knowledge held in the fib family, but also engaging with the wider engineering community, gathering a team of active authors and expert contributors from all over the world, as well as seeking improved more effective ways of working and adopting the most effective means of delivering the MC2020 project outputs.

Sustainability evaluation for concrete structures is a complex topic. It will require a framework for the verification of sustainability that is compatible with the through-life performance-based design of new concrete structures and the assessment/evaluation of existing ones that takes account of functionality (structural safety and related factors) of the constructed asset in conjunction with economic, environmental and social factors. Amongst other requirements, the needs include:

- That assessment is undertaken on whole building/complete constructed asset basis rather than simply being based upon the quantity of materials used
- An assessment methodology that takes account of the available sets of sustainable indicators (following CEN, ISO standards as appropriate)
- Appropriate input data – which for existing structures would be based on the actual state of the structure, its estimated remaining working life and performance level
- Definition of strategies and tactics for the through-life care and management of the asset
- Ways to evaluate alternative options for managing the remaining working life of structure, ideally taking account of the implications of current deterioration and damage on functionality and a prognosis of their future development
• Ways of taking account of predictions of the future lifetime of the concrete structure for potential intervention / through-life care options, together with associated implications for the functionality / performance of the asset
• Ways of evaluating the effectiveness of intervention and care options versus the alternative of demolition and the construction of a new structure

While the evaluation of sustainability of new concrete structures and the assessment / evaluation of existing ones could follow the general principles of the performance-based design approach identified in *fib* Model Code 2010, the verification of sustainability might potentially require various measures based upon whole building / complete constructed asset performance evaluated using metrics based upon some form of functional units / performance indicator.

Mueller[7] has suggested that fundamentally sustainability might be thought about in terms of the factors which it is desirable to maximise and those which it is desirable to minimise - as indicated in the following (simplified) tabulation.

<table>
<thead>
<tr>
<th>Maximise</th>
<th>Performance of structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Increase service life by:</td>
<td>Increase performance by using:</td>
</tr>
<tr>
<td>• adequate service life design</td>
<td>• building materials with increased strength and enhanced durability</td>
</tr>
<tr>
<td>• monitoring and structural inspection</td>
<td>• materials with additional functionality</td>
</tr>
<tr>
<td>• adequate rehabilitation work</td>
<td>Improving resilience</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minimise</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Use materials with reduced environmental impact, e.g. composite cements</td>
<td></td>
</tr>
<tr>
<td>Development of concretes with reduced binder and / or cement clinker content</td>
<td></td>
</tr>
<tr>
<td>Development of environmentally friendly production and building techniques</td>
<td></td>
</tr>
</tbody>
</table>

Mueller[7] also recognises the need to consider sustainability in terms of environmental impacts considered in conjunction with implications for performance (functionality) and durability, proposing the following (simplified) functional / performance based metric for sustainability which takes account of these factors, while (knowingly) neglecting other aspects of the life cycle such as the end of life phase (dismantling, demolition and reuse), as well as social and political considerations.

Sustainability = Lifetime (Service Life) x Performance / Environmental impact

Previous attempts at developing metrics for buildings based on functional units have typical been expressed in terms of environmental impact per unit area of useable floor space or gross floor area (or similar) for a given study period. The environmental impacts have been expressed in various ways depending on how the environmental impacts are measured (eg. multiple environmental indicators, which all have different units of measurement, or in terms of a normalised single value metric such as Ecopoints, as used in the BRE / BREEAM system). In simple terms these approaches are akin to an environmental impact / m², which is similar to the long used simple financial metric of cost / m². However, as the previous approaches relate to a simpler situation than that now being contemplated, careful consideration will be required to establish appropriate metrics for the new circumstances.
7: CONCLUDING REMARKS

The next generation of fib Model Code (fib MC2020) will need to introduce a framework for the verification of sustainability for the design of new concrete structures and the assessment / evaluation of existing ones that takes account of functionality (structural safety and related factors) of the constructed asset in conjunction with economic, environmental and social factors to make an holistic evaluation of its sustainability. This will need to be informed by a wide ranging and international debate on achieving an appropriate balance between these factors, with participation extending beyond fib to include other global and national organisations.

In regard to existing concrete buildings and other constructed assets, adaptation, upgrading and other forms of intervention / through-life management provide potentially sustainable ways of extending their useful life. A critical aspect of these activities is the assessment / evaluation of existing concrete structures. However, it is important to recognise that undue conservatism can lead to structures being condemned as unfit or requiring extensive and unnecessary packages of remedial / upgrade works to extend their useful life with associated adverse implications for the financial cost of the works and their environmental and social impacts. Accordingly it is essential that better procedures are developed for these tasks. The wide ranging aspirations of the fib MC2020 project could provide the catalyst for their development.

8: ACKNOWLEDGEMENTS

The authors wish to acknowledge the valuable contributions made by the participants at the international workshop in The Hague, the Netherlands on 30 June 2015, along with the valued efforts of TNO colleagues who hosted that meeting, and our other fib colleagues forming the fib MC2020 Core Group.

9: REFERENCES


ENGINEERING THE WAY FOR SUSTAINABILITY

ICCS16

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Key words: concrete, sustainability, durability, environment, cements, aggregates, fibres

Abstract. Engineers play an extraordinary role in understanding and in application of sustainability rules. We should be able to develop and show the most reasonable way to keep the right proportions of human needs and nature acceptance. Sustainability is considered as a necessary effort to find a reasonable balance among safety, serviceability, robustness, durability, environmental protection as well as social and economic needs. We have to find innovative solutions from the point of view of structural engineering including new materials as well as new structural concepts that extend current practice for supporting sustainability. Increased durability (as well as increased service life and increased maintenance cycles) provide one of the best solutions to support sustainability. We are going towards a better acceptance and better understanding for sustainability as one of the key aspects in the material production as well as in the design, construction and use.

1 INTRODUCTION

Most of the international associations keep sustainability as a flagship on their list of activities emphasizing its importance. It resulted in the joint effort of like JCI, ACI, fib and RILEM to organize the 1st ICCS2013 conference in Tokyo [1]. The preliminary activities from fib have been already published in series of fib bulletins [2]-[6].

There are already many books and articles to support understanding of the topic [7]-[11]. In March 2014 a declaration was prepared (called Takamatsu Declaration) at the end of the International Workshop on Concrete Sustainability in Takamatsu by the participants to drive the world towards greater sustainability. We meet again in Madrid for the 2nd ICCS2016 to review the present state-of-art and develop further steps.

Future structural materials should follow optimal selection of material compositions and
construction processes with wider use of blended and ternary cements and wider use of recycled aggregates as well as alternative binders. Use of slag and fly ash in blended cements is not only for environmental purposes but may have also advantageous like improved sulphate resistance, lower shrinkage, reduced probability of AAR etc. Wider acceptance of natural fibres as kind of tensile reinforcement is also of importance.

Concrete became a widely used material (second after sweat water) (Figure 1). Blaszczyński and Król [12] considers it as good and unfortunately bad information. They say: ‘Good because of fact, that thanks to concrete we are able to build solid and sustainable structures making our life easier and better. Bad because making a concrete is connected with huge energy cost and even bigger emission of greenhouse gases’.

![Protect the environment – Limit CO₂ – Create future (Chateau la Coste, France)](image)

**Figure 1:** Protect the environment – Limit CO₂ – Create future (Chateau la Coste, France)

Durability is a key factor in reaching sustainability. Hooton and Bickley [13] distinguish: many ways to reduce the carbon footprint of concrete including reduction in its Portland cement clinker content by methods including: (a) optimization of total aggregate gradations, (b) use of water-reducing admixtures, (c) intergrinding clinker with limestone, and (d) use of supplementary cementing materials (SCMs). However, the most effective way to improve sustainability of concrete structures is by making them last longer through design for durability, and by minimization of construction defects. In almost every case, durable concretes will include all of the above listed aspects, but from a design approach, the emphasis needs to be on durability. Durability design includes more than the selection of concrete materials and mix proportions. It also requires that construction detailing, temperature control, adequate compaction, protection of fresh concrete, and curing be detailed in the specification and that inspection and testing be carried out to ensure that the specifications are being followed. The performance requirements need to be stated explicitly, and the objectives made clear. In addition to discussing various aspects of concrete durability, this contribution will discuss the merits of performance and objective-based specifications as well as the processes required to make them work.

## 2 CONCRETE – EVOLUTION IN BINDING SYSTEMS

### 2.1 Portland cement

High performance is often coupled by high durability. Excellent examples are the UHPC bridges providing a very dense material structure by using high cement content and special method of production (Figures 2 and 3).
2.2 Blended cements: from binary systems to ternary or quaternary blended cements

The production of cement clinker causes 5–8% of the global man made CO₂ emissions. The reduction of the CO₂ emissions is an important aspect of the cement research. High number of studies involved materials, which are suitable to replace the clinker partially. Materials suitable for cement substitution are known as supplementary cementitious materials (SCMs) (blast furnace slag, fly ash, etc.). The first blended cements consisted of Portland cement clinker partially replaced by one type of SCM. These are the binary systems or binary blended cements, e.g. slag cement [14][15][16][17].

According to several researchers, ternary blends made of Portland-cement, silica fume and fly ash offer significant advantages over binary blends and even greater enhancements comparing with the ordinary Portland cement [18][19].

Fly ash or blast-furnace slag are known to show synergistic effects with limestone in ternary systems [20][21].

While ternary composite cements are well investigated there is a lack of knowledge regarding quaternary composite cements. The latter include three SCMs besides OPC with blast-furnace slag, limestone and Si-rich fly ash (type V according to EN 197-1) being the most commonly used materials. In terms of phase composition quaternary systems have similar composition to already existing composite cements based on OPC, including blast-furnace slag or type V fly ash. Such systems might therefore be promising cement compositions [22][23].

In the study on quaternary cement blends of Schöler et al. [23] found, that at the ages of 2 and 7 days all strength values are located in the same range independent from the composition of the systems. At the age of 28 days the highest values are reached with 5 wt.% additional limestone while both, higher limestone content or limestone absence lead to lower compressive strength (Figure 4). Thus, the presence of low quantities of limestone increases compressive strength as observed in OPC or OPC-FA blends.
Figure 4: Compressive strength at 2, 7 and 28 days of hydration for systems containing different ratios of fly ash and limestone as well as 20 wt.% or 30 wt.% blast furnace slag [23].

In order to balance the requirements for higher-performance cements and sustainable development, the industry is progressively resorting to blended cements (mostly binary systems) including additions of industrial by-products, such as fly ash, slag, and silica fume. These environmentally friendly binary cements usually enhance concrete durability in hostile environments. They are often associated with shortcomings, however, such as the need to moist-cure concrete for longer time, low early-age strengths, increased demand for costly chemical admixtures, cracking tendency due to plastic shrinkage, and durability problems due to de-icing salts. There is growing interest in ternary and quaternary cements that could be optimized to achieve a synergistic effect allowing component ingredients to compensate for their mutual shortcomings, thus providing advantages over binary systems [19].

Heat development in thick (mass) concrete elements influenced by the cement type
Following experiment intended to model the hydration heat development and (ΔT) in a thick concrete wall influenced by the selection of cement type and placing of concrete in layers. Selected cement types were: CEM I 42.5 N and CEM III 32.5/B N-LH/SR. Amount of cement was 350 kg/m³, w/c=0.48, aggregates 0-4 mm: 844 kg/m³, 4-8 mm: 375 kg/m³, 8-16 mm: 656 kg/m³, Glenium C323: 0.4%. Sizes of concrete block were: 1.00×1.00×1.00 m. Casting was carried out in layers of 30 cm with a top layer of 10 cm. Specimen was covered with 5 cm insulation on all sides also on the top. Thermal couplers were positioned in the concrete block before casting.

Measurements indicated lower maximal hydration temperature as well as delayed appearance of maximal temperature using CEM III/B-S 32.5 N cement compared to CEM I 42.5 N (Figure 5).
3 CONCRETE – AGGREGATE

3.1 Aggregate: sustainable use of primary sources

Efforts are needed to limit primary resources as aggregates for concrete. We can do it when necessary and unavoidable, but we have to analyse opportunities for using recycled aggregates any time when possible.

3.2 Aggregate: use of recycled concrete

After demolishing group of buildings or large constructions mountains of recycled aggregates might be available (Figure 6). Different materials must be separated and classified.

Extensive research was directed for possible uses of crashed as aggregate to concrete [24]. Classification, way of mixing, allowable percentages of recycled fractions, behaviour as fresh and hardened concrete are under consideration.

Figure 5: Development of hydration heat in 1 m thick concrete element with different cements: CEM I 42.5 N or CEM III B 32.5 N-LH/SR

Figure 6: Aggregates from demolition of an industrial building
3.3 Aggregate: use of other waste materials

PET powder as fine aggregate in concrete
There is an increasing amount of polyethylene terephthalate (PET) polymer to recycle. Research was directed to study and compare the changes in strength, hydrotechnical and thermal properties of mortar, containing waste PET particles as by-product generated during the recycling process [25]. Different sieve fractions of PET particles are shown in Figure 7. Figure 8 indicate the Molecular structure and resin identification code of polyethylene terephthalate. Figure 9 shows the scanning electron microscopic picture of PET powder’s morphology used in the experiments (M: 1000). Figure 10 is the photo of ring test to measure crack sensitivity.

Figure 7: PET (polyethylene terephthalate) particles of 4, 2, or 1 mm sizes [26]

Figure 8: Molecular structure and resin identification code of polyethylene terephthalate

Figure 9: Morphology of PET powder used in the experiments, SEM picture, M: 1000 [25]

Figure 10: Ring test to measure crack sensitivity [25]

In this research cement and cement+lime mortar mixtures were studied with addition of PET powder particles. The fine-grained plastic aggregate was used to substitute the natural fine aggregate (quartz sand) in different volume fraction. 5-5 mixes were made varying the
quantity of PET. In addition to a reference mix without PET, mixes with 1, 2, 4 and 8 % of PET by volume were prepared. In these blends the amount of the cement, lime and the water-binder ratio were the same, however, the quantity of PET and the amount of superplastisizer were changed. During the studies fresh mortar tests (flow table test, density measurement), and hardened mortar tests (density, flexural and compressive strength, immersion and capillary water absorption test, fire resistance test, cracking measurement, thermal conductivity measurement, sorption isotherm curve definition, water vapour resistance, heat capacity measurements) were performed.

Results indicated that the consistency of fresh mortar containing PET aggregates has the same or lower class, while the density decreased. As expected, due to the increasing addition of PET powder both the flexural and compressive strengths decreased and simultaneously the water absorption increased, as well as the early age shrinkage cracking tendency improved. In the thermal tests the best result were obtained for the maximum amount of PET powder.

**Application of limestone quarry waste in self-consolidating concrete** [26]

Aim of a research was to design self-consolidating concrete containing high amount limestone by utilization of *limestone quarry wastes* for aesthetic-architectural purposes (e.g. facades, street furniture). The mining and processing limestone result a large amount of mixed fraction debris and quarry waste limestone slurry (powder), which is considered as mining waste and requires deposition and further treatment. Reuse of these materials may reduce the environmental impacts. The commercially available direct ground limestone powder is well suited additional material for self-consolidating concretes. Quarry waste limestone slurry could replace the other types of limestone powder products in concrete decreasing the material and energy consumption and this way the cost of the concrete. For thin, slender structural elements is beneficial if the concrete can be compacted into every corner of a formwork, purely by means of its own weight without vibration. Using self-consolidating concrete these requirements can be achieved.

Seven mixes with different particle size distributions were designed and examined during the research. Three mixtures were made with crushed limestone aggregate of different particle size distributions; another three mixtures with quartz aggregate were mixed as reference and one more with mixed quartz-limestone aggregate, respectively. Direct ground limestone powder was applied as filler, except in recipe of mixed aggregate, where quarry waste powder was utilized. The type and amount of cement, superplasticizer and plastic fibres were the same in all mixes. In the research different test methods were used to determine the fresh concrete properties (e.g. slump flow test, T₃₀ time, J-ring test (Figure 11), storage life time test) as well as the hardened properties of concretes (compressive strength, flexural tensile strength, modulus of elasticity, abrasion resistance, freeze-thaw resistance, chloride migration).
The specific surface of limestone slurry was close to the specific surface of the white cement.

Specific surface of fines in this research:

- CEM I 52,5 N (white): 417 m²/kg
- Limestone powder (commercial): 343 m²/kg
- Limestone slurry (quarry waste): 407 m²/kg

All mixtures resulted self-consolidating, self-compacting concrete. Samples made with limestone aggregate reached a lower level of compressive and flexural tensile strength as well as elastic moduli than concrete made with quartz aggregate. As it was expected concrete containing limestone aggregate is not considered abrasion resistant and in terms of freeze-thaw resistance also performed worse than concretes containing quartz aggregate. These concretes without quartz aggregate are not suggested for purposes which need de-icing salt in winter. With the proposed concrete mixture (combination of mixed quartz-limestone aggregate with waste limestone slurry) the purpose of containing the largest possible amount of limestone can be achieved.

**Recycled glass for production of lightweight aggregates**

A process has been developed in Hungary to produce lightweight aggregates from 100% recycled glass. The name of the product is Geofil (Figure 12). Advantage of this product is not only the opportunity for 100% recycling, but also to provide the possibility to recycle municipal or hazardous glass wastes, too. Laboratory tests indicated favourable mechanical behaviour. The product is lightweight artificial gravel with a diameter of 1 to 25 mm providing primarily heat and sound insulating properties (Fig. 12). It has good bonding capability if embedded in gypsum, cement or resin matrix. The product has three main types called Geofil A, B and C. Type A is used for thermal insulation, type B is used for structural LWAC with thermal insulating property and type C is used for structural LWAC [27].
Use of Geofil aggregates has a contribution not only to recycling but also to reduce transportation cost by using it as lightweight aggregate.

4 CONCRETE – FIBRES

4.1 Reuse of fibers (steel fibers from waste automotive tires)

In order to support sustainability of concrete structures research was directed to use of recycled steel fibres (RSF) originating from used car tyres [28]. Test results indicated successful use of recycled steel fibres with following conclusion. Fibres recycled from used car tyres can provide FRC with similar properties as commercial fibres in terms of tensile flexural test results. A balanced situation, in between favouring bond conditions while maintaining adequate workability was found, and a limitation of the aspect ratio to 150, or even less, seemed to be necessary. Longer fibres tended to ball. Addition of fibres improved cracking. Crack width control has been improved by increasing tension stiffening and reducing transfer length.

4.2 Zero footprint fibers – natural fibers in concrete (sisal, palm, hemp, flax, etc.)

One of the challenges for fibre reinforced concrete is the wider use of natural fibres, like hemp fibres. Most of the natural fibres have relatively high tensile strength, however, their long term alkaline resistance should be also provided [29]. An experimental study was directed to measurements on the hemp fibres with two different coatings as well as on concrete specimens with these hemp fibres. This study includes details on fibre handling, fibre tensile force-displacement behaviour and flexural behaviour of FRC containing linseed oil or varnish protected braid of hemp fibres or unprotected braid of hemp fibres. In order to simulate the alkaline environment of cement matrix the strands were immersed in Lawrence solution [30].

In present paper the braids of hemp fibres cut into 50 mm long pieces themselves as well as FRC specimens with braids of hemp fibres were studied. Both unprotected and protected braids were tested. Linseed oil or polymer dispersion (varnish) was used as protective agent to increase the alkali resistivity of the hemp fibres. Catalyst was used with the linseed oil to accelerate the polymerization reaction of the unsaturated oil. First we have tested the tensile force-displacement behaviour of the unembedded braid of fibre (initial state). In order to simulate the alkaline environment of cement matrix the strands were immersed in Lawrence solution with following proportions: 0.48 g/l Ca(OH)$_2$, 3.45 g/l KOH and 0.88 g/l NaOH (pH=12.9). The Lawrence solution is an estimation of the pore water composition of a fully hydrated cement paste. The braids of fibres were immersed for 30 days under temperature of 55 °C. The test aimed to simulate a time elapse of ca. 15 years [4]. After storage in Lawrence solution we also tested the braid of fibre tensile force-displacement with 10 mm/min loading rate.

Test results indicated that he loss of tensile force of protected braid of fibres was not as significant as the unprotected braid of fibres. The initial inclinations of the force-displacement curves decreased for protected braids. The inclinations of the curves decreased even more significantly for braids stored in Lawrence solution. Three-point bending tests were carried
out on notched FRC beams according to EN 14651:2005+A1:2007. The maximum crack mouth opening displacement of the FRC with protected braid of hemp fibres were higher than the FRC with unprotected braid of fibres. The residual tensile force was directly affected by the mechanical deterioration, though the mixes only differed in the protection of the braids of fibres. As natural fibres have a weak external cellular wall the surface created over it, greatly increased its resistance through the mixing process.

5 SUSTAINABILITY BASED MIX DESIGN FOR ENERGY PERFORMANCE OF CONCRETE

Worldwide, approximately 40% of primary energy use and 24% of greenhouse gas emissions are accounted by in buildings [31]. Normal weight concrete, as a material with high thermal conductivity coefficient ($\lambda=1.65 – 2.00$ W/mK [32]) primarily has a negligible role in energy efficiency of building envelope. Considering thermal bridges, on the contrary, concrete has a major effect in the overall energy performance of the building (Figure 13.). According to studies in Refs. [33] [34] even up to 30% of heat loss can occur through thermal bridges in well-insulated residential buildings. More sophisticated modelling methods demonstrate that the annual heating load in presence of thermal bridges are underestimated by 10-20% using the equivalent U-value method or the equivalent wall method [35]. Concrete with lower thermal conductivity coefficient can also contribute to heat loss savings in subterranean parts (which are usually made of concrete), floors and ceilings.

![Figure 13: Role of structural concrete in thermal bridges](image)

In energy performance calculations usually, the thermal conductivity coefficient of normal weight concretes are used as a function of the material density. However, the following examples demonstrate other design and proportioning aspects, which can improve the energy performance of residential and public buildings by reducing heat losses in building envelopes, especially through thermal bridges.

In addition to the situations mentioned above for conventional concrete elements, the following findings can contribute at design and proportioning of lightweight aggregate concrete especially for the selection of fine aggregate and binder type.
Mixes denoted CG00, CA00 and R00 were made to show the effect of aggregate type in concrete to thermal conductivity (Table 1). Different proportioning of silica fume in R00, R10 and R20 demonstrate the beneficial effect of it on the reduction of thermal conductivity. R002, R00S and R00V mixes compared to R00 illustrate the possible influence of cement type on thermal properties of concrete.

**Table 1: Mix proportions (kg/m³) to the experiments for sustainability based design**

<table>
<thead>
<tr>
<th>Mix</th>
<th>Water</th>
<th>Cement</th>
<th>Cement type</th>
<th>Silica fume</th>
<th>Sand</th>
<th>Course aggr.</th>
<th>Maximum aggregate size [mm]</th>
<th>Aggregate type</th>
</tr>
</thead>
<tbody>
<tr>
<td>CG00</td>
<td>180</td>
<td>360</td>
<td>CEM I 42.5N</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Granite</td>
</tr>
<tr>
<td>CA00</td>
<td>180</td>
<td>360</td>
<td>CEM I 42.5N</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R00</td>
<td>180</td>
<td>360</td>
<td>CEM I 42.5N</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R10</td>
<td>180</td>
<td>324</td>
<td>CEM I 42.5N</td>
<td>36</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R20</td>
<td>180</td>
<td>288</td>
<td>CEM I 42.5N</td>
<td>72</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R002</td>
<td>180</td>
<td>360</td>
<td>CEM II/A-M  (S-L) 42.5R</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R00S</td>
<td>180</td>
<td>360</td>
<td>CEM III/B  32.5 N</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
<tr>
<td>R00V</td>
<td>180</td>
<td>360</td>
<td>CEM II/A-M  (V-L) 42.5R</td>
<td>-</td>
<td>735</td>
<td>974</td>
<td>16</td>
<td>Uncrushed river bed aggregate</td>
</tr>
</tbody>
</table>

Concrete specimens of 150x150x150 mm sizes were under water for 28 days. Then cut into halves and kept in air-dried conditions at temperature of 20±2 °C and RH 45-50% for 10 days. The thermal properties were tested using ISOMET 2114 Thermal Properties Analyzer (Figure 14.) on the side of the specimen side where they were previously cut into halves. The results are given in Table 2. and illustrated in Figure 3.
Figure 14: Measuring thermal properties of the concrete

Table 2: Thermal conductivity and volume heat capacity of tested (see above) concretes

<table>
<thead>
<tr>
<th>Mix</th>
<th>Thermal Conductivity (W/mK)</th>
<th>Volume Heat Capacity (MJ/m³K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CG00</td>
<td>1.53</td>
<td>1.90</td>
</tr>
<tr>
<td>CA00</td>
<td>1.37</td>
<td>1.82</td>
</tr>
<tr>
<td>R00</td>
<td>2.99</td>
<td>1.92</td>
</tr>
<tr>
<td>R10</td>
<td>2.56</td>
<td>2.00</td>
</tr>
<tr>
<td>R20</td>
<td>2.12</td>
<td>1.82</td>
</tr>
<tr>
<td>R002</td>
<td>2.74</td>
<td>2.06</td>
</tr>
<tr>
<td>R00S</td>
<td>2.54</td>
<td>1.95</td>
</tr>
<tr>
<td>R00V</td>
<td>2.29</td>
<td>1.96</td>
</tr>
</tbody>
</table>

Figure 15: Thermal properties for different concrete mixes; parameters were: aggregate type, cement type, silica content (one column represents the average 12 measurements on 2 halves of 2 cubes to each mix and 3 points of measurements each)
Results measured on concrete specimens CG00, CA00 and R00 presented the importance in selecting the aggregate type on thermal conductivity of concrete. The use of natural uncrushed rived bed aggregate in concrete can double the heat conductivity compared to granite or andesite. That is a result of possible high content of quartzite in river bed aggregate which has thermal conductivity coefficient $\lambda > 5 \text{ W/mK}$, while andesite and granite have $\lambda = 1.8-2.7 \text{ W/mK}$. That aspect could be considered additionally LCIA (life cycle impact assessment) calculation.

By increasing the replacing level of densified silica fume from 10% to 20% in mixes R00, R10 and R20, the observed thermal conductivity reduction was 14% and 29%, respectively.

Results of R00, R002, R00S and R00V concrete mixtures, by enhanced thermal conductivity reduction, revealed another beneficial aspect of replacing Portland cement with SCM (Supplementary cementing materials). By comparing the concrete mixes using CEM I, CEM II/A-M (S-L), CEM II/A-M (V-L) or CEM III/B cements, a reduction of thermal coefficient was observed by 8%, 23% and 15%, respectively.

**Volume heat capacity**

These significant changes in thermal conductivity did not considerably influence the volume heat capacity of the concrete mixtures (blue columns in Figure 15).

6. CONCLUSIONS

Future structural materials should follow optimal selection of material compositions and construction processes with wider use of blended cements and alternative binders as well as wider use of recycled aggregates. Use of slag and fly ash in blended cements is not only for environmental purposes but may have also advantages like improved sulphate resistance, lower shrinkage, reduced probability of AAR etc. Wider acceptance of natural fibres as kind of tensile reinforcement is also of importance.

Increased durability – as well as increased service life and increased maintenance cycles – provide one of the best solutions to sustainability.

Present article intended to give an overview on practical approaches to sustainability including:

(i) binding systems: Portland cements, blended cements – from binary to ternary or quaternary blended cements;
(ii) concrete aggregates: primary sources, recycled aggregates and waste materials and (iii) fibres including reuse of fibres and zero footprint natural fibres.

A special example has been given to sustainability based mix design for energy performance of concrete concluding:

(a) the use of natural uncrushed rived bed aggregate in concrete can double the heat conductivity compared to granite or andesite. That is a result of possible high content of quartzite in river bed aggregate which has thermal conductivity coefficient $\lambda > 5 \text{ W/mK}$, while andesite and granite have $\lambda = 1.8-2.7 \text{ W/mK}$. That aspect could be considered additionally LCIA (life cycle impact assessment) calculation.
(b) By comparing the concrete mixes using CEM I, CEM II/A-M (S-L), CEM II/A-M (V-L) or CEM III/B cements, a reduction of thermal coefficient was observed by 8%, 23% and 15%, respectively.

(c) These significant changes in thermal conductivity did not considerably influence the volume heat capacity of the concrete mixtures

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GREEN CONCRETE SPECIFICATION AND ENVIRONMENTAL DECLARATIONS OF CONCRETE

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Key words: Green Specification, Environmental Labels, Environmental Declaration, Concrete, ISO

Abstract. This technical paper introduces “Green Concrete Specification” and “Environmental Declarations of Concrete” that are results of recent work in the area of sustainability by Korea Concrete Institute (KCI). While the former is a technical specification to be launched in the near future, the latter is a draft standard at present. The key concept of the Green Concrete Specification includes the “environmental management and green construction plan” submitted by the contractor to the client at the beginning of concrete construction in the following four areas: reduction of GHG emission and energy consumption, effective use of resources, environmental management of construction site and neighborhood, and water management. Environmental Declarations of Concrete is introduced in two parts: introduction of concepts of the environmental labels and declarations, and excerpts from the aforesaid draft standard. Finally an effort for standardization on environmental labels and declarations of concrete and concrete structures at international level is briefly introduced.

1 INTRODUCTION

Green building codes and standards are effective means for the mitigation of global warming as IPCC 2007 WG III report has suggested.[1] In Korea, Building Construction Standard Specification of AIK (Architectural Institute of Korea) has first adopted the concept of “environmental management and green construction” in 2013 for general building construction. Korea Concrete Institute (KCI) has expanded the contents of “environmental management and green construction” specifically for the concrete construction. This technical document called “Green Concrete Specification” is in the final stage before officially being launched, and the author introduces the outline of it in the first half of this paper. Environmental labels and declarations is a type of standard. Environmental labels and declarations of concrete backed up by a systematic support from regional and/or central authorities can be an effective tool to mitigate challenges raised by global climate change and realize more sustainable concrete industry. In the second half of this paper, a draft standard “Environmental Declarations of Concrete” recently developed by KCI is introduced. An effort for standardization on environmental labels and declarations of concrete and concrete structures at international level is also briefly described toward the end of this paper.
2 GREEN CONCRETE SPECIFICATION

2.1 General

The purpose of this new technical specification is to decrease the negative environmental impacts (such as GHG emission and waste generation), increase the positive environmental impacts (such as use of industrial byproducts), and incorporate lifecycle concept where applicable. Table 1 is the contents of “Green Concrete Specification” by KCI. The key idea is the “environmental management and green construction plan” to be submitted by the contractor to the client at the beginning of concrete construction in the following four areas:

(1) Reduction of greenhouse gas (GHG) emission and energy consumption;
(2) Effective use of resources;
(3) Environmental management of construction site and neighborhood; and
(4) Water management.

Table 1: Green Concrete Specification - Table of contents

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<th>2.1.4 Mixing water</th>
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</thead>
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<td>2.2.1 General</td>
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<td>1.5.1 General</td>
<td>2.2.3 Others</td>
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<td>2.3 Production and transportation</td>
</tr>
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<td>1.5.4 Environmental management of construction site and neighborhood</td>
<td>2.3.1 General</td>
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</tr>
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<td></td>
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<tr>
<td>2. Materials</td>
<td>3. Construction</td>
</tr>
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<td>2.1 Selection of materials</td>
<td>3.1 Concrete construction</td>
</tr>
<tr>
<td>2.1.1 General</td>
<td>3.2 Reinforcement construction</td>
</tr>
<tr>
<td>2.1.2 Cement</td>
<td>3.3 Formwork construction</td>
</tr>
<tr>
<td>2.1.3 Aggregates</td>
<td>3.4 Waste treatment</td>
</tr>
</tbody>
</table>
2.1.1 Reduction of GHG emission and energy consumption

This includes the overall energy consumption plan for the concrete construction project. The energy consumption during construction needs to be regularly monitored and recorded. Use of new/renewable energy is recommended as well as the use of the energy-efficient construction equipment and tools. While the GHG emission is directly related to energy consumption during construction, the possibility of further reduction of GHG emission is also reflected through lifecycle by the use of materials with EPD (environmental product declaration or Type III environmental declaration) and carbon footprint which are given priority for the selection of materials/products. Products produced from a factory or by a manufacturer with ISO 14001 certification are also given priority.

2.1.2 Effective use of resources

This consists of three parts: preservation of natural resources, increased use of industrial byproducts, and reduction of construction and demolition waste generation. The environmental checklist provided in the Green Concrete Specification includes listing of natural resources to be preserved (stones, plants, etc.) in the construction site as a result of consultation between the contractor and client. Utilization of existing structures such as existing pavement both for temporary and permanent use is also recommended. Utilization of industrial byproducts in concrete construction includes fly ash, blast furnace slag, and other cementitious substitutions as well as recycled aggregates. In case of demolition of existing structures, step-by-step demolition of existing structures is important and therefore required to reduce the amount of mixed wastes. The amount of construction wastes to be produced during construction needs to be estimated in the beginning. Target reutilization of the construction wastes in the field is to be set by the contractor and reported to the client. Type and amount of the construction wastes that are brought out of site need to be reported and recorded. The target values for the construction waste utilization initially set by the contractor can be later modified reflecting actual situation with consent from the client.

2.1.3 Environmental management of construction site and neighborhood

This has already been enforced by various laws in Korea. Noise and vibration need to be maintained below the specific level required by law for example. The other areas of major concern are the contamination of water, soil, and air by construction and hazardous wastes. The hazardous wastes include radioactive materials, asbestos, and used oil, which need special treatment following pertinent laws.

2.1.4 Water management

As the importance of water emerges with the threat of global climate change, the water management is an essential part of the environmental management and green construction. The consumption of water for the construction needs to be planned and recorded. The overall water use plan as well as plans for the utilization of rain water, gray water and other reclaimed water are to be included. Use of pervious surfaces such as pervious pavement or parking lots
utilizing pervious concrete is also recommended as it easily permits the infiltration of water into ground where a hard surface (such as conventional concrete or asphalt concrete pavement) does not.

2.2 Materials

Selection of the construction materials needs to consider environmental impacts during life cycle, durability, recyclability, and energy consumption during transportation. The contractor considers the use of eco-friendly materials (EPD, carbon footprint, environmental label, GR mark, etc.). Concrete mix design needs to be based on life cycle CO₂ emission analyses.

2.3 Construction

The contractor considers the adoption of licensed green technologies for construction with consent. The contractor uses the energy efficient equipment and tools with low level of noise and vibration, reduces construction wastes generation, and enhances quality control for longer service life. Amount of construction wastes generation (and demolition wastes where applicable) should be kept to a minimum by carefully establishing the construction (and/or demolition) plan both for the formwork, false work, and permanent work.

3 ENVIRONMENTAL DECLARATIONS OF CONCRETE

International Organization for Standardization (ISO) currently defines three different types of environmental labels and declarations: Type I and Type II labelling and Type III declarations.[2~5] Of three different types of environmental labels and declarations, Type I environmental labelling is probably the most often encountered one in many countries. An LCA (life cycle assessment) of a product is usually required for an environmental label which is certified by an eco-labelling body. No specific data is disclosed in association with Type I environmental labelling. Type II environmental labelling is sometimes called self-declared environmental claim. Type III environmental declaration is considered the most comprehensive and accurate one as the declarations are made as a result of an LCA. Type III environmental declarations require the third-party verification.

Environmental Declarations of Concrete, a draft standard recently developed by KCI, is in line with ISO 13315 series and an extended version of this draft is being further developed as an International Standard at present as “Environmental Labels and Declarations of Concrete and Concrete Structures.”[6-8]

3.1 Objectives

The objectives of environmental declarations of concrete are as follows:

- to provide environmental information to be used as performance criterion for concrete constituent materials, concrete, concrete products and concrete structures;
- to facilitate communication of the environmental information involved with production of concrete constituent materials, production of concrete and concrete products, execution of concrete structures, use of concrete structures including operation, maintenance and remedial activities, and end-of-life activities including
demolition, reuse/recycling and final disposal;
- to enable producers of concrete and concrete products to reduce the environmental impact of their operations and products; and
- to enable designers or constructors to reduce the environmental impact of concrete structures.

3.2 Issues for assessment of environmental performance

3.2.1 Phases in life cycle of concrete

Life cycle of concrete can be divided into four phases: production phase, execution (construction) phase, use phase, and end-of-life phase as shown in Fig. 1. The production phase can be further divided into two sub-phases: the production of concrete constituent materials (e.g. cement, additions, additives, aggregates and water) and reinforcement, and the production of concrete and concrete products.

- The environmental declarations (cradle-to-gate) can be given for each concrete constituent material in Sub-phase I-1 (e.g. cement)
- The declarations (cradle-to-gate) of ready-mixed concrete and/or concrete product in Sub-phase I-2 can be given based on information modules in Sub-phase I-1.
- The declarations (cradle-to-grave) of concrete cover all information modules in Phases I through IV.

3.2.2 Environmental aspects and impacts of concrete

The life cycle of concrete includes significant environmental aspects and impacts. Environmental aspects include the use of material resources and energy, the production of waste and the release of noise and emissions to land, water, and air for example. These environmental aspects are related to the environmental impacts (which are usually the results of environmental aspects), which can be expressed in terms of environmental impact categories as a result of LCIA as shown in Table 2.

<table>
<thead>
<tr>
<th>Environmental impact category</th>
<th>Category indicator</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global climate change</td>
<td>Global warming potential (GWP)</td>
<td>kg CO₂ eq.</td>
</tr>
<tr>
<td>Stratospheric ozone level</td>
<td>Ozone depletion potential (ODP)</td>
<td>kg CFC-11 eq.</td>
</tr>
<tr>
<td>Acidification</td>
<td>Acidification potential (AP)</td>
<td>kg SO₂ eq.</td>
</tr>
<tr>
<td>Eutrophication</td>
<td>Eutrophication potential (EP)</td>
<td>kg (PO₄)₃ eq.</td>
</tr>
<tr>
<td>Photochemical ozone creation</td>
<td>Formation potential of tropospheric ozone</td>
<td>kg C₂H₄ eq.</td>
</tr>
<tr>
<td>Natural resources use</td>
<td>Abiotic depletion potential for fossil resources</td>
<td>MJ</td>
</tr>
<tr>
<td>Fuel (natural fuel, waste material)</td>
<td>Abiotic depletion potential for non-fossil resources</td>
<td>kg antimony eq.</td>
</tr>
</tbody>
</table>
3.2.3 System boundaries and inventory data

The system boundaries and inventory data of ISO 13315-2 can be followed.[7]

3.2.4 Scenarios

When needed, scenarios should be appropriately established to evaluate the environmental performance of concrete specific to application. As shown in Fig. 1, the scenarios are needed for the execution and use phases and also for the end-of-life phase. Service life of concrete can be determined based on principles and procedures of ISO 16204 considering durability aspects of concrete.[9]

3.3 Program for environmental declaration

3.3.1 Definition of product categories

When concrete constituent materials, reinforcement, ready-mixed concrete, concrete products and structures have similar functions and applications, the product categories can be defined.

3.3.2 Application of LCA methodology

The quantified environmental information in Type III environmental declaration shall be
based on results from one or more life cycle assessment in accordance with the ISO 14040.[10]

3.3.3 Development of product category rules (PCR)

When applicable, adoption of readily available PCR documents in the same product category in the appropriate market area should be first considered to develop a PCR for a product category. The PCR document shall include the goal and scope of one or more LCA-based information for the product category. The PCR shall also determine the life cycle stages and information modules to be included. The PCR shall include impact categories, the inventory parameters, and the environmental impact category indicators.

3.3.4 Comparability

Comparability using Type III environmental declarations shall be carried out only at the structural level. This comparison requires a scenario and the PCR shall describe the rules and requirements for that scenario.

3.4 Requirements for environmental declaration

Quantitative data shall be reported in appropriate and consistent units of measurement as prescribed by PCR. Qualitative data, where provided, shall be comparable. Type III environmental declarations in a product category shall follow the format and include the parameters as identified in the PCR provided by the program operator including data from LCA, LCI or information modules, and additional environmental information.

3.5 Verification

Verification of a Type III environmental declaration shall include PCR review, independent verification of data, and Type III environmental declaration.

4. CONCLUDING REMARKS

4.1 Green Concrete Specification

“Green Concrete Specification” developed by KCI was introduced. The key idea involves the “environmental management and green construction plan” that consists of four areas as described. Data for the “environmental management and green construction plan” may consist of the following different cases.

(Case 1) Specific quantities can be suggested as the reduction target:

- energy consumption during concrete construction;
- CO₂ emission reduction by adopting specific concrete mix design;
- generation of construction/demolition wastes by type; and
- water use.

(Case 2) Number of application can be suggested:
- use of products with EPDs, carbon footprints, GR marks, and environmental labels;
- measures taken in the field for noise and vibration control; and
- measures taken in the field to prevent contamination of water, air, and soil.

(Case 3) Cases that do not belong to (Case 1) and (Case 2):
- preservation of natural resources and
- others

It is noted that the technical difficulties are expected for the actual preparation and execution of the “environmental management and green construction plan” by the users of this specification especially in the beginning. It is believed to be important to pursue the suggested concept of the green construction plan while the proposed “Green Concrete Specification” needs to be updated and modified based on initial and continuous feedback from the actual applications in the field.

4.2 Environmental declarations of concrete

Environmental labels and declarations is a rapidly growing concept and constitute an important part of a group of environmental standards in the area of sustainable concrete construction. A draft standard on “Environmental Declarations of Concrete” developed by KCI was introduced. Some technical difficulties that need to be overcome to effectively implement this standard may include the following:

- Functional unit: a functional unit for a ready-mixed concrete, for example, can be conveniently defined as m\(^3\) so that the environmental impact, such as CO\(_2\) emission, can be summed up as CO\(_2\)-eq per 1m\(^3\) of ready-mixed concrete. When it is desired to extend the idea to a concrete structure, however, it becomes a more complex matter to define a functional unit. The functional unit should be kg or m\(^3\) in general, while it can be m\(^2\) for concrete structures with large surface/volume ratio such as pavement.
- Service life prediction: concept of durability should be utilized to effectively provide parameters necessary to determine service life of concrete.
- Scenarios: a full LCA of concrete requires scenarios for the construction, use, and end of life stages.
- Recycled aggregates: it is important in the concrete construction to utilize recycled aggregates to reduce the amount of natural aggregate consumption. Production of good quality recycled aggregates consumes large amount of energy and also produces GHGs. Therefore there is environmental advantages and disadvantages at the same time. How to assign proper weight for positive and negative impacts for recycled aggregates?

REFERENCES

NEW ROUTE TO SYNTHESIZE BIOBASED PCE SUPERPLASTICIZER

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Key words: superplasticizer, PCE, biobased, life cycle assessment, renewable

Abstract. In this article a new route to synthesize biobased PCE superplasticizer is presented. Based on the raw material mass, this PCE superplasticizer contains more than 80% biobased carbon. Additionally, a Life Cycle Assessment (LCA) comparison with conventional crude oil based PCE was done and confirm the reduced carbon footprint and primary energy demand. In addition to clinker reduction, biobased superplasticizers can help further reduce the carbon footprint, and improve the sustainability of concrete in the future.
INTRODUCTION

Polycarboxylate ether (PCE) superplasticizer are nowadays widely used as concrete admixtures to improve concrete workability during placing and durability after hardening. In addition, PCE can be used to reduce the environmental impact of concrete, mainly by enabling concrete mix designs with reduced amounts of cement and / or increased use of secondary cementitious materials such as slag, fly ash or limestone. However, up to now, all high performing PCE superplasticizer are based on monomers derived from crude oil. The limited availability of fossil resources and the general need in all industries to reduce carbon emissions have accelerated the development of biobased chemicals which possess a low carbon footprint [1].

In addition to natural polymers, the fast development of biomass conversion in bio refineries has also extended the number of available base chemicals within the recent years [2,3]. Within concrete production, the use of natural polymers like carbohydrates, cellulose derivatives or lignosulfonate has a long history. However, biobased PCE superplasticizer have not been available up to now. This paper introduces the use of biobased PCE derived from molasses, which is a by-product of the refining of sugarcane.

RESEARCH SIGNIFICANCE

The goal of this work is to provide high performing PCE superplasticizer, based on renewable raw materials. PCE are based on carboxylic acids and polyethylene oxide derivatives. Biobased polyethylene glycol derivatives (X-PEG), needed for the production of biobased PCE, were developed and used for synthesis of standard PCE. The performance of the resulting biobased PCE were compared with the corresponding PCE synthesized with crude oil based X-PEG derivatives. The amount of biobased carbon was calculated based on the chemical composition and the carbon footprint and primary energy demand of an admixture, was calculated in comparison to an admixture based on conventionally produced PCE.

EXPERIMENTAL PART

Polymers: Comparative samples of two different polycarboxylate ether were prepared by reaction of carboxylic acids and polyethylene glycol derivatives (X-PEG), according to a method described in literature [4,5]. The X-PEG was either based on conventional crude oil or synthesized from biobased ethylene oxide. Two sets of PCE polymers, with different polymer structures, and therefore different performance, were synthesized. Polymer PC-1, a polymer with extended workability retention, possess 1.5 carboxylic units per side chain and a side chain length of 1000; whereas Polymer PC-2 a typical strong water reducer, possess 2.0 carboxylic units per side chain and a side chain length of 2000. Polymers, synthesized from biobased X-PEG are named as PC-1B and PC-2B respectively. The polymers based on conventional raw materials are named as PC-1C and PC-2C. Quality of the biobased PCE was assessed based on polymer analysis and mortar performance of the resulting PCE polymers.
Biobased carbon content and primary energy demand was calculated based on the known PCE formulation and the known amount of biobased carbon of each raw material according to suppliers’ information. Life Cycle Assessment (LCA) is a standardized method to assess and compare the inputs, outputs and potential environmental impacts of products and services over their life cycle. This LCA was performed according to ISO 14044 (ISO 2006) and EN 15804 (CEN/TC 350 2012). LCA was performed as cradle to gate assessment for 1 kg concrete admixture, which investigates the potential environmental impact of a product from raw material acquisition and processing to manufacturing and packaging.

To perform the LCA, an inventory analysis step was necessary, i.e., data collection and calculation procedures to quantify relevant inputs and outputs of the product system. The LCA was modelled using the GaBi 6.3 Software. Product data collection was carried out by the manufacturer and corresponds to measured or qualified data, including formulations of the products, transportation, packaging and production. The products were then modelled in the LCA software based on this collected data and on datasets from the commercial databases such as ELCD (European Reference Life Cycle Database) and ECoinvent. The packaging considered was an Intermediate Bulk Container (IBC). The production data originates from an admixture production site in Germany.

The impact assessment phase associates inventory data with environmental impact categories (calculated with the CML 2001 method). There are several impact categories which can be determined, however, for the scope of this paper the analysis focuses on the “Global Warming Potential” (GWP 100 years), also known as carbon footprint, which is measured in kg of carbon dioxide (CO₂) equivalents for a 100-year time horizon. GWP data for the biobased X-PEG were provided by the supplier.

**Materials & Methods**

Biobased X-PEG, was co-developed with an Indian, backward integrated ethoxylate supplier. All other raw materials are commercial standard grades.

Mortar flow test: A mortar is prepared by mixing 750 g of cement (CEM I 42.5 N), 2140 g of fine aggregate and limestone filler, with water containing the superplasticizer at a water cement ratio of 0.44. Polymer solution (40%) of the respective polymers was used with constant dosage of 0.9% wet weight by wt. cement. Cement and sand were blended in a Hobart mixer for one minute. The water, containing the defined amount of polymer is added and the mortar is mixed for 3 minutes. Flow table spread is measured according to DIN EN 1015-(3). Flow spread was measured initially and after 30 and 60 minutes. Before each measurement the mortar was remixed for 30 seconds. Samples for strength development were prepared according to DIN EN 196-1 and compressive strength was measured after 1, 7 and 28 days storage at 90% rel. humidity and 20°C.
EXPERIMENTAL RESULTS AND DISCUSSION

Polymers

For the synthesis of biobased PCE, X-PEG based on renewable resources was used. The biobased X-PEG is synthesized by a unique process, using molasses, a by-product from sugar production, as feedstock. Molasses is transformed into ethanol by fermentation. Key step of the synthesis of biobased ethylene oxide (EO), is the conversion of ethanol into ethylene. Ethylene is then converted to EO, analog to EO produced from crude oil based ethylene. The biobased EO was used for the synthesis of X-PEG derivatives, and in turn for the synthesis of PCE. A schematic diagram summarizing the synthetic route for both types of polymers, the oil based and the biobased one, is shown in Figure 1.

Figure 1 – Schematic diagram for synthesis of biobased and crude oil based PCE

![Schematic diagram for synthesis of biobased and crude oil based PCE](image)

Table 1: Chemical composition and analytical data of the synthesized polymers

<table>
<thead>
<tr>
<th>Polymer</th>
<th>X-PEG source</th>
<th>C/E ratio</th>
<th>Conversion</th>
<th>Viscosity (40% solution)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC-1C</td>
<td>crude oil based</td>
<td>1.5</td>
<td>90.7 %</td>
<td>75 mPas</td>
</tr>
<tr>
<td>PC-1B</td>
<td>biobased</td>
<td>1.5</td>
<td>90.8 %</td>
<td>73 mPas</td>
</tr>
<tr>
<td>PC-2C</td>
<td>crude oil based</td>
<td>2.0</td>
<td>91.8 %</td>
<td>168 mPas</td>
</tr>
<tr>
<td>PC-2B</td>
<td>biobased</td>
<td>2.0</td>
<td>89.2 %</td>
<td>174 mPas</td>
</tr>
</tbody>
</table>

The chemical composition and analytical data of the synthesized polymers are summarized in Table 1. The PCE performance was tested in mortar flow experiments using crude oil based PCE as reference. The results of the mortar performance tests are shown in Table 2 and 3 and Figure 1 and 2.
Table 2 – Flow table spread results of all Polymers

<table>
<thead>
<tr>
<th>Polymer</th>
<th>Flow table spread mm (in)</th>
<th>Density g/cm³</th>
<th>Air content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC-1C</td>
<td>158          236          222          2.398   2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC-1B</td>
<td>160          235          223          2.396   2.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC-2C</td>
<td>193          163          128          2.400   1.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC-2B</td>
<td>198          164          125          2.403   1.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3 – Compressive strength of mortar specimens, of all Polymers

<table>
<thead>
<tr>
<th>Polymer</th>
<th>Compressive strength of mortar specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 day MPa</td>
</tr>
<tr>
<td>PC-1C</td>
<td>30.2</td>
</tr>
<tr>
<td>PC-1B</td>
<td>30.7</td>
</tr>
<tr>
<td>PC-2C</td>
<td>28.4</td>
</tr>
<tr>
<td>PC-2B</td>
<td>28.9</td>
</tr>
</tbody>
</table>

Figure 2 – Flow table spread results of Polymer PC-1B and PC-1C
It can be clearly seen that neither the analytical results nor the polymer performance test in mortar or strength measurements, show significant differences between the conventional and the biobased polycarboxylate superplasticizer. Based on these results it can be concluded that the production of PCE based on X-PEG from renewable resources is possible without compromising polymer performance.

**Biobased Carbon Content**

Biobased carbon content of PC-1B and PC-2B was calculated based on the known PCE composition and the known biobased carbon content of the raw materials. The carboxylic acid of all polymers was based on conventional crude oil based monomers, and contain therefore no biobased carbon. The X-PEG macromonomers of the biobased PCE, PC-1B and PC-2B are based on ethylene oxide synthesized in a multiple step process from molasses as shown in Figure 1. Apart from the end group (X-), all carbon atoms of the X-PEG side chains are based on biobased carbon. As the side chains are by far the biggest weight fraction of PCE plasticizer, the majority of the carbon atoms are biobased. The calculated biobased carbon contents of PC-1B and PC-2B are 83% and 89%, respectively. This indicates that by the use of X-PEG side chains, biobased PCE superplasticizer contain more than 80% biobased carbon, independent of their performance.

**Carbon footprint and primary energy demand**

LCA was performed for a typical PCE based admixture. The admixture with 40% solid content contains PC-2 as main component and minor amounts of other additives like an appropriate defoaming system and biocide.
In order to evaluate the potential reduction in the carbon footprint and primary energy demand of an admixture, based on biobased superplasticizer, GWP data for the biobased X-PEG were introduced in an concrete admixture model. Figure 4 shows the cradle to gate GWP comparison for 1 kg concrete admixture containing oil-based PEG vs biobased PEG, based on PC-2. The raw materials are responsible for most of the impact; while the fraction caused by production and packaging is of less importance. By using biobased PCE as raw material of the model admixture, the Global Warming Potential of 0.93 kg CO2-eq. per kg admixture can be reduced by 9% or 0.08 kg CO2-eq. /r kg. The primary energy demand of 28.3 MJ/kg admixture can be reduced by 36% or 10.3 MJ/kg.

**SUMMARY AND CONCLUSIONS**

It was shown that the biobased X-PEG is suitable for the synthesis of PCE superplasticizer without compromising performance. Even when carboxylic acid raw materials are based on conventional crude oil based chemistry, the resulting PCE contains more than 80% of biobased carbon. The use of biobased X-PEG incorporated into a PCE-based concrete admixture indicates that the global warming potential can be reduced by 9% or 80kg CO2 per ton and the primary energy demand can be reduced by 36% or 10.3 MJ per kilogram of admixture. In summary, the use of PCE, based on renewable raw materials, can help to improve the sustainability of concrete admixtures.
ACKNOWLEDGMENTS

The authors wish to express their gratitude and sincere appreciation to India Glycols Limited for providing X-PEG samples and LCA data.

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OVERVIEW OF RESOURCE CONSERVATION AND CLOSED-LOOP RECYCLING IN CONCRETE TOWARD SUSTAINABILITY

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Key words: Resource, Recycle, Technology, Closed-loop, Standard, Guideline

Summary. Providing excellent performance as a structural material, concrete has long been deemed essential for modern civilization and recognized as a material that will continue to maintain and support the development of human society. The sheer amount of concrete in use and in stock compared with other building materials brings up the issue of the enormous amount of waste generated when concrete is disposed of. Besides, resources for good quality aggregate are beginning to be depleted at a high speed. Concrete has conventionally been regarded as being difficult to recycle. The construction industry has addressed these problems and carried out research and development regarding the recycling of concrete. The development history of concrete recycling is summarized in a paper from the viewpoints of the properties and the application of recycled concrete aggregate. The paper categorizes recycling technologies according to the quality of recycled concrete aggregate into several types. Advanced techniques avoid down cycling and recover recycled concrete aggregate, having the same quality as natural aggregate from waste concrete, forming a closed-loop in terms of the resource circulation of concrete materials. However the problems are remained in the energy-induced environmental impact and cost increase. The time has come when establishment of a new design concept for complete recycling of concrete is definitely necessary. The principle of complete recycling is that the concrete is subject to material design to reduce waste generation and facilitate resource circulation in a closed-loop system. Development technology based on such material design is regarded as proactive technology. The materials of concrete should be used as parts of concrete during the service life of concrete and remain usable after demolition as parts of similar or other products without quality deterioration, continuing circulation in various products as the media. The paper introduces a history of development of completely recyclable concrete, with which closed-loop circulation of component materials is realized like steel and aluminum.
1 INTRODUCTION

Advanced industrial nations enjoy a convenient and affluent lifestyle through the production and consumption of various products, using a vast amount of resources extracted from the natural environment. However, a large amount of waste is generated incidental to such production and consumption. Various environmental issues are basically attributable to the fact that such vast extraction of resources, production and consumption of products, and generation of wastes have largely exceeded the capacity of the natural environment in terms of resource regeneration and waste purification. In order to achieve sustainable development, it is essential for us to acknowledge that the availability of resources and purification capacity of the natural environment are limited. It is important to keep the increasing rate of resource input lower than that of economic growth for achieving sustainable development.

While it is evident that the Earth’s resources are not infinite, they nevertheless represent a huge amount when compared with the volume that humankind uses today. However, the resources we can actually put to use are limited, and we should acknowledge that they will eventually be depleted. In other words, depletion of resources does not mean that they will totally disappear, but fundamentally means the following:

- significant reduction in availability to the next generation
- difficulty in acquiring resources as a vast amount of energy is needed to do so
- difficulty in carrying out life activities that depend on resources

In real terms, depletion of resources coincides with the following phenomena in the acquisition of a given resource:

- soaring resource price
- increase in environmental destruction through acquisition of a resource
- loss of economic advantage regarding the use of a resource

It is expected that nearly 50% of the total material input is currently accumulated every year in the form of structures. It indicates the enormous consumption of resources by the construction industry compared with other industries. Providing excellent performance as a structural material, concrete has long been deemed essential for modern civilization and recognized as a material that will continue to support the development of human society. Therefore, the sheer amount of concrete has been in use and in stock. The production of concrete is supposed to account for nearly 50% of the annual resource consumption in the construction industry. In other words, concrete accounts for nearly 25% of the total material input while other construction materials such as steel and wood are far less consumed than concrete. There is no doubt that all nations, not only developed countries, without exception, experience the same situation regarding material input for concrete production during the course of their advancement through the development of their social infrastructure. It is known that concrete is the second most consumed substance on Earth after water\(^1\). This demonstrates that concrete has an enormous impact on the creation of a society with sustainable development, not only from the aspect of the consumption of resources, but also from the aspect of the future waste generation. However, concrete has unfortunately and conventionally been regarded as being difficult to recycle. The scarcity of the residual capacity of final disposal areas, which has become increasingly serious in recent years, also alerts us to formulate a resource-recycling society. The construction industry has addressed these problems and carried out research and development regarding the recycling of concrete since the 1970s.
2 TECHNOLOGIES FOR IMPROVING RECYCLED CONCRETE AGGREGATE

2.1 Categorization of technologies for producing recycled concrete aggregate

Qualities of recycled concrete aggregate (RCA) has been classified in terms of absorption, with a high absorption meaning a large amount of cement paste adhering onto the original aggregate surfaces. It is clear that concrete made using such recycled concrete aggregate exhibits lower strength, lower modulus of elasticity, higher drying shrinkage, and lower durability. Recycled concrete with such aggregate has been a material subject to limitations in the permissible maximum design strength according to the quality of recycled aggregate with concomitant limitations in the applicable members and portions. The problem of the above recycling of concrete waste lies consisted in the intention to commercialize a product made from concrete waste that would evidently lead to concretes with lower qualities than conventional concrete. This is therefore regarded as down-cycling-type recycling in which the quality-related values of the product are lowered when distributed on the market.

Figure 1 shows three categories of methods for producing recycled concrete aggregates which include that for road subbase materials, that for low-quality recycled concrete aggregate for non-structural concrete use, and that for high-quality recycled concrete aggregate for structural concrete use. Single toggle-type jaw crushers as shown in Figure 2 are generally used for the primary crushing of demolished concrete into pieces 40 to 50mm in size regardless of the ultimate quality of recycled concrete aggregate. While the material is carried to the next process on a belt conveyor, foreign particles such as wood/plastic chips and reinforcing steel/nails are removed manually and with a magnetic separator, respectively. The materials then undergo various treatments according to their uses. Impact crushers are used for secondary and tertiary crushing when producing low- or middle-quality recycled concrete aggregate. Other equipment in practical use for producing low- or middle-quality recycled concrete aggregate includes self-propelled or vehicle-mounted jaw crushers and impact crushers that save the energy normally expended to haul the demolished concrete. While the quality of recycled concrete aggregate produced by using such equipments is improved as the number of treatment processes increases, the recovery percentage of recycled aggregate decreases with increased amounts of powder byproducts as the aggregate itself is crushed. The following innovative treatments are therefore necessary for efficient production of high-quality recycled concrete aggregate for structural concrete use in order to minimize the adhering cement paste.

2.2 Acid treatment for high quality RCA

Pre-soaking treatment with hydrochloric acid (HCl), sulfuric acid (H2SO4) and phosphoric acid (H3PO4) was conducted experimentally to reduce the amount of cement paste attached onto recycled concrete aggregate as shown in Figure 3. Accordingly it was reported that the acidic pre-treatments could effectively remove a great portion of old cement paste from recycled concrete.  

![Figure 3: Pre-soaking treatment procedures for RCA](image-url)
concrete aggregate, which showed as competitive properties as the normal aggregate, leading to marked improvements of the mechanical properties of recycled aggregate concrete (RAC), while it caused small adversarial effect to the recycled concrete aggregate.

2.3 Pulsed power discharge for high quality RCA

Pulsed power discharge was repeatedly applied to concrete waste immersed in water to produce high-quality recycled concrete aggregate as shown in Figure 4. It causes an explosive expansion of the discharge channel and generates pressure waves propagating as sonic waves in water. If they come across interfaces of different densities, i.e. aggregates and cement paste, positive and negative interferences occur. As a result, pressure and tensile stresses are generated at the interfaces. The tensile stresses primarily destroy the bond between aggregates and cement paste. From the experiments it was found that the repeated pulsed discharge treatments can produce high-quality recycled concrete aggregate and the concrete with the recycled concrete aggregate showed enough mechanical properties. The more number of discharges and the higher voltage can produce the higher-quality recycled concrete aggregate.

2.4 CO2 treatment toward high quality RCA

This paper summarizes two types of CO2 treatment conducted experimentally for improving the properties of recycled concrete aggregate.

One is the CO2 curing pretreatment which aims to densify the adhering cement paste and then strengthen the recycled concrete aggregate through a well-known carbonation reaction between CO2 and cement hydration products. Recycled concrete coarse aggregates was subjected to pure CO2 gas with the constant 10kPa pressure after vacuumed to -50kPa in an air-tight cylindrical vessel as shown in Figure 5(a).

Another is the repeated alternate immersions of recycled concrete aggregate in carbonated nanobubble water and saturated calcium hydroxide solution as shown in Figure 5(b). The carbonated nanobubble water contains many CO2 bubbles smaller than 100nm, which are not influenced by buoyancy almost at all according to the Strokes law and can survive in high concentration in water to produce calcium carbonate for a long period. The saturated calcium hydroxide solution supplies extra calcium ion for additional production of calcium carbonate.
It was reported that both treatment could reduce the pore volume in adhering cement paste leading to the decrease of water absorption and the increase of density of recycled concrete aggregate.

![Figure 5: Devices for CO2 treatment](image1)

(a) Pure CO2 gas treatment
(b) CO2 nanobubble water treatment

2.5 Scrubbing for high quality RCA

In mechanical scrubbing, demolished concrete rubbles are crushed to 40mm or less by an ordinary jaw crusher before processing and those stopped by a 5-mm sieve are processed with an eccentric tubular vertical mill shown in Figure 6 which removes adhering cement paste. And those stopped again by a 5-mm sieve after processing become a high-quality coarse recycled concrete aggregate. Fine recycled concrete aggregate is then produced similarly from the recycled concrete aggregate smaller than 5-mm. Trial runs revealed that coarse recycled concrete aggregate conformed to the requirements for natural aggregate. On the other hand, the percentage of recovery widely varied depending on the type of original aggregate, and slight difficulty was found in producing high-quality fine recycled concrete aggregate. Natural fine aggregate was therefore used when applying recycled concrete coarse aggregate produced by this method to structures in practice.

![Figure 6: Schematic diagram of the eccentric rotor mill](image2)

2.6 Heating and scrubbing for high quality RCA

In heating and scrubbing, concrete crushed beforehand into 50-mm rubbles is charged in a
vertical heating furnace with temperature around 300°C and subjected to hot air from below to make the cement paste brittle and weak. It is then scrubbed in a tube mill to separate cement paste from original aggregate. The quality of recycled concrete aggregate satisfies the requirements for natural aggregate, while the percentage of recovery of high-quality recycled concrete aggregate is sufficiently high. Also, the quality of concrete made using recycled concrete aggregate obtained by this method is virtually the same as the concrete with normal aggregate. This method has acquired certain track records in the construction of actual structures, it requires the availability of infrastructures that economically provide the heat sources necessary for heating. In addition, the superiority of this technique from the standpoint of the life-cycle load on the environment and life-cycle cost has to be socially recognized.

![Figure 7: Process of high-quality recycled concrete aggregate by heating and scrubbing](image)

3 DEVELOPMENT OF COMPLETELY RECYCLABLE CONCRETE

3.1 Concept of completely recyclable concrete

The principle of completely recyclable concrete (CRC) is that the concrete is subject to material design to reduce waste generation and facilitate resource circulation in a closed system. Development technology based on such material design is regarded as proactive technology. The materials of concrete should be used as parts of concrete during the service life of concrete and remain usable after demolition as parts of similar or other products without quality deterioration, continuing circulation in various products as the media. This is defined as a performance called resource conservability. If concrete produced with due consideration to the resource conservability at the stage of material design is applied to structures, then the components of the concrete can be completely recycled at the time of demolition. What should be done in the future is to introduce material design that permits complete recycling for at least the components of concrete, i.e., aggregate and cement materials, to ensure the material conservability in concrete as the medium and then to achieve high quality of structures.

3.2 Cement recovery-type

Cement recovery-type CRC was defined as “concrete whose materials are entirely usable after hardening as materials of cement, since all the binders, additions, and aggregate are made of cement or materials for cement”. After the most basic cement recovery-type CRC (e.g., using normal Portland cement, crushed limestone, and crushed limestone sand) was crushed,
the obtained sample was subjected to ingredient adjustment to make it the material for cement. This material was subjected to processes including calcination in an electric oven, gypsum addition, and crushing, to produce reprocessed cement. This cement had the same qualities as one available on the market and no problem was observed in fresh and mechanical properties of the concrete made using this cement. Blast-furnace slag, fly ash, etc., generated as by-products from industries other than construction have been actively reused as materials for cement and cementitious materials for concrete. Since by-products such as blast-furnace slag, copper slag and fly ash contain adequate amounts of SiO$_2$, Al$_2$O$_3$ and Fe$_2$O$_3$ that are necessary for materials of cement, concrete containing several types of these industrial by-products in combination achieves complete recyclability as clinker material after demolition without adding any other ingredients$^{12}$. This CRC with no need for ingredient adjustment contributes to the global environment from the standpoint of effective use of industrial by-products as well. Cement recovery-type CRC can formulate a semi-closed-loop circulation material flow as shown in Figure 8. Conversion from conventional concrete to CRC will substantially mitigate the environmental problem of concrete waste generation and CO$_2$ emission during cement production, while permanently preserving and storing the limestone resource in the form of structures.

3.3 Primary aggregate recovery-type CRC

Aggregate recovery-type CRC was defined as “concrete in which the aggregate surfaces are modified without reducing the mechanical properties of the concrete, in order to reduce the bond between aggregate and the matrix, thereby permitting easy recovery of original aggregate”$^{13}$. It can form a closed circulating material flow as shown in Figure 9. In order to achieve 100% circulation of concrete in a closed system, a structure needs to be a cement material supplier as well as an aggregate-supplier. By building a stock of structures keeping such an appropriate balance, all cement and aggregate can be exploited from built structures in the future. In the first aggregate recovery-type CRC, aggregate surfaces were pre-treated chemically or physically. In chemical treatment, formation of cement hydrates on the aggregate-paste interfaces is chemically restricted due to the coating by mineral oil coating, while physical treatment forms a film of a water-soluble synthetic resin emulsion to smooth the fine irregularities on the interfaces, thereby reducing the mechanical friction. Experiments were carried out to investigate the mechanical properties of concrete and the aggregate recoverability.
The surface treatment realizes high recovery of high quality recycled aggregate with little adhering paste from demolished concrete by a simple crushing technique. However, the compressive strength of concrete made with surface modified aggregate decreased. It was because surface modification causes the crack propagation zone to be predominantly formed in the weak portions at aggregate-matrix boundaries, leading to failure at a lower stress. Based on the experiments, the trade-off relationship between the mechanical properties of concrete and the aggregate recoverability still remained unresolved.

**Figure 9: Resource circulation in aggregate recovery-type CRC**

### 3.4 Advanced aggregate recovery-type CRC

An advanced aggregate recovery-type CRC was developed to ensure compatibility of the performances in a trade-off relationship between mechanical properties of the concrete and aggregate recoverability. The proposed new technology consists of two technologies as shown in Figure 10, i.e. concrete strength enhancement technology and aggregate recovery technology. The former involves, differing from conventional technologies, aggregate surface modification to increase the bonding force between the coarse aggregate and the mortar by coating a binder evenly on the surface of the coarse aggregate. Silica fume and by-product powder were contained in the binder expecting to enhance chemical and physical bonding force due to pozzolanic reaction and mechanical friction. The latter aims at recycling aggregate with low energy, which involves inclusion of dielectric material in the binder. When applied with microwave radiation, the dielectric material on the surface of the aggregate is heated and the interface between the aggregate and cement paste matrix is weakened locally and thus the separability of them is improved. As a result of experiment, the combination of the increase in the mechanical friction force and the increase in the chemical bonding force resulted in drastic increase in strength of concrete. After heated with microwave radiation with 1,800W for 90 seconds, the cylindrical specimens of φ10x20cm were roughly crushed with a jaw crusher, and subjected to a scrubbing treatment with the Los Angeles Abrasion Machine to remove cement paste. The aggregate recovery rate after heated by microwave radiation was around 93%, indicating small amount of the cement paste remained, proved a high quality of the recycled concrete aggregate. The CO₂ emission during the treatment of 1 ton of concrete rubbles is shown in Figure 11. The CO₂ emitted using the microwave heating is extremely small compared to that from the recycled concrete aggregate production process of the heating and scrubbing. Considering the extremely high quality of recycled concrete aggregate obtained by this new technology, it seems highly advantageous than conventional ones.
6 CONCLUSIONS

A recycling-oriented society is a society that continues to use resources, once they are taken from nature into the society, without returning them unless they do not represent an environmental load. In such a society, the intake of resources from nature is minimized and products and materials that cannot be recycled repeatedly are rejected. Recycling should be of high quality because recycled products are not marketable unless they are of a quality that satisfies users. Recycling should be repeatable. If a recycled product has to be dumped in a landfill after use with no chance of recycling, then the recycling is no better than producing waste of the following generation. To establish a recycling-oriented society, it is vital to materialize a new production system whereby new stock serves as resources and it is important to adopt a technology of enhancing the resource conservability of concrete and its components,
at the stage of designing the structure by incorporating the concept of upstream-process system.

REFERENCES


SUSTAINABILITY OF CONCRETE STRUCTURES IN CHANGING WORLD

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Abstract. World is changing. We are faced to increasing frequency of natural disasters due to continuously increasing global environmental changes. Earthquakes, floods, storms, hurricanes, tornados, fires, tsunamis, volcanic events, heat and cold waves, extreme dry weather etc. are more and more frequent. We are faced to increasing economical and social problems incl. terrorism. The structures for sustainable future should be better prepared for the new conditions; they should be more resilient. Concrete gradually becomes building material with high potential for expectant environmental impact reduction. This needs better knowledge about technological processes and their impacts from wide variety of sustainability aspects within entire life cycle – from acquisition of materials, through production of concrete and concrete components, construction, use, up to the demolition of concrete structure and recycling of components and materials. Already implemented realizations give clear signal that in the forthcoming era when designing and implementing concrete structures it will be necessary to take into account new requirements and criteria following from global aspects on sustainable development.

1 INTRODUCTION

Concrete is used in construction of buildings, bridges, dams, roads, tunnels – basically every contemporary construction contains concrete. The specific properties of concrete are, among others, affordability, broad availability and variability in connection with strength, reliability and perspective durability, which has made the concrete the most widely used construction material in the world and the second most used substance after water. The production of concrete in the industrialized world thus annually amounts to 1.5-3 tone per capita. In consequence of a fact that world cement production has been 12 times increased in the second half of the last century, the cement industry produces at present about 5 - 8% of global man-made CO₂ emissions – Fig. 1 [1]. More over high amount of concrete use is associated with high transport needs and demands on production and demolition processes within the entire life cycle. This all has significant impact on the environment [2].
The Figures 2 and 3 show a development of cement production in different regions of the world presented by The European Cement Association – CEMBUREAU [3]. From the graph on Fig. 2 it is evident that the production of cement is significantly growing in developing countries in Asia, Africa and CIS (Commonwealth of Independent States – some former Soviet Republics) and slightly growing in America and Oceania. However, in Europe was the production in 2014 lower in comparison to 2001. The graph on Fig. 3 shows the % fraction of world cement production in the year 2014. The dominant position had China with 56.5% of total world cement production.

Figure 1: Trends in concrete and steel production compared to population growth and its expected development up to the year 2050 [1]

Figure 2: World cement production by region – evolution 2001-2014; source CEMBUREAU [3]
As a consequence of economy growth, especially in emerging countries, the demand for construction materials, including cement and concrete, is increasing. A graph prepared by WWF [4] presented in Figure 4 shows prediction of cement production in industrialised and developing countries in the period 1990 – 2050. It is evident that the predicted global production of cement in 2050 is almost 5 times higher than in 1990, although in OECD countries the increase of cement production is expected to be around 35%. This will have a significant impact on the increasing level of global CO₂ emissions.

Current development of concrete, production technology and development of concrete constructions during last twenty years have lead to quality shift of technical parameters and
also of related environmental impacts. New types of concrete have due to mix optimization significantly better characteristics from the perspective of strength, mechanical resistance, durability and resistance to extreme loads.

The rapid development in the construction industries leads to the replacement of old existing buildings with new buildings. Consequently, the amount of demolished concrete structures is very high and is gradually increasing. This creates needs and potential for replacement of natural aggregate by recycled aggregated. The use of recycled concrete - as an aggregate for new concrete mixes, leads to saving of natural resources and helps to reduce the pressure on landfiling sites.

Considering situation shown in Figures 2, 3 and 4 it is highly important to focus on an implementation of new technologies and construction techniques especially in emerging countries and regions with the highest cement production.

2 WORLD IS CHANGING

2.1 Technology development and natural resources depletion

The accelerated development of technology during last two centuries has been associated with devastation of natural resources and with escalating pollution by emissions and wastes. During the twenty years from 1974 to 1994 the world population increased by 40% [5]. Consequently the new enormous development of the infrastructure and housing is being planned in developing countries and will start in the near future. Natural resources are, however, limited. And production of the two most common construction materials - cement and steel is increasing even faster than the population growth [1] (Fig. 1).

The extraction of natural raw materials, manufacturing of products and waste landfilling or incineration are associated with corresponding environmental impacts, including greenhouse gas (GHG) emissions. The evoked increasing entropy of the entire system does not have a reversible character. Non-renewable natural sources are irreplaceably exhaustible and regeneration ability of some renewable resources is limited.

The development of new technologies and the use of progressive materials and advanced construction solutions leading to quality improvement of buildings not only in economic, but also in the environmental and socio-cultural sense, represents a huge potential for sustainable society development. The population growth, depletion of primary materials, energy and drinking water sources, deterioration of the environment, including the consequences of global warming in the form of a higher frequency of natural disasters, necessarily lead to a more intensive search for new, environmentally suitable approaches and solutions.

2.2 Consequences

According to a WMO (World Meteorological Organisation) analysis [6] the years 2011-2015 have been the warmest five-year period on record, with many extreme weather events - especially heat waves - influenced by climate change. The probability of natural disaster is now nearly five times higher as it was in the 1970s, because of the increasing risks brought by climate change (see Fig. 5).
Some disasters – such as floods and storms – represent significantly higher threat than others. Floods and storms also cause higher economical losses; heat waves are an emerging killer. Recently the world society is faced to increasing number of natural disasters with high number of deaths [7] e.g. European heat wave (2003 – 70,000 deaths), Indian Ocean earthquake and tsunami (2004 – 280,000 deaths), the Cyclone Nagris in Myanmar (2008 – 138,000 deaths), the Haiti earthquake (2010 – 160,000 deaths), the Northern Hemisphere heat wave (2010 – 56,000 deaths), the Hurricane Sandy (2012) and others. In the same time we are faced to accelerating number of manmade disasters e.g. WTC New York terrorist attack September 11th, 2001 (almost 3,000 deaths), the Fukushima nuclear disaster (2011), terrorist Iraq Attacks (2013 – 850 deaths), the Boston Marathon bombing terrorist attack (2013) and current increasing number of terrorist attacks by Islamic state (2013 – approx. 800 killed, 2014 – approx. 2,100 killed, 2015 - more than 3,000 killed).

As a consequence of natural and manmade disasters and economy situation in particular countries a human migration is increasing. The World Bank's Migration and Remittances Factbook of 2011 [8] lists the following estimates for the year 2010: total number of immigrants: 215.8 million or 3.2% of world population. In 2013, the percentage of international migrants worldwide increased by 33% with 59% of migrants targeting developed regions. Over 54 % of people across the globe were living in urban areas in 2014 [9]. The number of people living in cities will almost double by 2050 reaching around 6.4 billion people. It will change much of the world into a global city. The European migrant crisis began in 2015 with around one million of migrants via Mediterranean Sea in this year. This ongoing process is recently (January 2016) running almost out of control.

These above-mentioned consequences have highlighted the need for communities and societies to be more resilient with respect to unexpected and continuously changing situation in the world.

Figure 5: Increasing number of disasters by hazard type since 1970; source WMO [6]
3 SUSTAINABILITY AND RESILIENCY

Application of sustainability principles in different kind of human activities became a common approach. This is supported by implementation of new sustainable oriented knowledge and other results from research, via education to the construction practice.

With respect to the general principles of sustainability, the three essential pillars should be considered in the design, construction, use and other life-cycle phases of any construction, as follows:

- environmental aspects,
- economic aspects,
- social aspects.

Development of systems for complex assessment of buildings and their implementation into construction practice is very difficult, but increasingly important process. The evaluation methodologies have to be complex, considering all relevant flows (material, energy and other) and covering the corresponding most important essential criteria. However, the admissible simplifications of the model are usually needed. Present legal legislation in many countries has not yet requested such assessment methods but progress shows that these countries, especially those supported with well economic growth, start to exhibit gradual interest. In some countries, certification of complex building quality becomes obligatory for some types of construction.

Resilience is the capacity to adapt the system to changing conditions and to maintain or regain functionality and vitality in the stage of stress or disturbance [10]. It is the capacity to bounce back after a disturbance or interruption. According to the Resilient Design Institute “resilient design is the intentional design of buildings, landscapes, communities, and regions in order to respond to natural and manmade disasters and disturbances—as well as long-term changes resulting from climate change—including sea level rise, increased frequency of heat waves, and regional drought.

Sustainable structures should be in the same time to large extent resilient. Only resilient structures can be sustainable in the changing world.

4 CONTRIBUTION OF CONCRETE STRUCTURES

Main advantages of concrete structures from the viewpoints of sustainability are: (i) thermal mass (contributing to energy savings associated with cooling and heating), (ii) acoustic properties (improving air-born sound insulation), (iii) fire resistance, (iv) long term durability, (v) structural safety, including high resistance to natural effects during exceptional effects like natural disasters comprising good resistance to earthquakes. These advantages could be significant in designing new constructions as well as in old structures reconstructed for the new use [11, 12]. With respect to specifics of concrete presented as a strong and durable material it is possible to design and construct on this material bases robust structures with high level of resiliency when faced to the exceptional natural or man-made disaster situations.
4.1 Specific advantages of concrete structures from environmental viewpoint

- Secondary materials utilization - Utilizing supplementary cementitious materials in a composition of concrete mixture (fly-ash, granulated blast furnace, microsilica) it is possible to reduce the amount of embodied energy and embodied CO₂ and SOₓ emissions.
- Recycled concrete can be utilized as aggregate substitutes in earthwork construction and up to some extent as an aggregate substitute in a new concrete production.
- Precast concrete elements in “tailor-made” manner enable waste reduction in production and also on construction site.
- Thermal mass of concrete can contribute to energy savings associated with cooling and heating.

4.2 Specific advantages of concrete structures from economy viewpoint

- Long-term durability - Concrete in comparison to other materials (timber, steel etc.) enables longer service life of buildings. Concrete structures are usually more resistant to atmospheric action, they have a good capability of withstanding wear, and they do not subject easily to degradation processes. This also results in lower operating, maintenance and demolition cost.
- Lower material cost, lower manipulation and transportation cost - Subtle concrete structures utilizing lesser amount of higher quality concrete could be cheaper, even though the unit cost of this type of concrete is higher than the unit cost of standard concrete types.
- Dismountable structures: Precast concrete structures can be designed as dismountable enabling consequential utilization of structural elements.
- Smaller thickness of peripheral structures can have a positive effect on construction economic efficiency (especially in areas with regulated size of built-up area).
- Thermal mass - Concrete structures due to their accumulative properties can in some cases contribute to decrease of operating cost for cooling and heating.

4.3 Specific advantages of concrete structures from social viewpoint

- High structural safety and reliability, higher fire resistance – This includes also high resistance to natural effects during exceptional cases of natural disasters (floods, storms, winds, hurricanes, tornados, fires, earthquakes, etc.) and terrorist attacks.
- Acoustic properties – Due to high specific weight of concrete there can by improved air-born sound insulation of structure (floors and/or walls separating different operational areas);
- Thermal mass – Concrete (material with high specific weight) can contribute to thermal stability of internal environment and consequently to energy savings.
- Maintainability - Concrete surface produced in high quality can be easily maintained, cleaned and it has long durability.
- Flexibility – Character of concrete technology enables significant design flexibility due to the possibility of forming almost any element shape limited only by structural reliability requirements.
- Healthiness - Concrete is not the source of toxic emissions or volatile organic compounds.
5 ADVANCED TECHNOLOGICAL AND STRUCTURAL PRINCIPLES

5.1 Optimization of concrete mixture

Utilizing new composite materials with significantly better physical characteristics creates realistic assumptions of achieving substantial effect from the perspective of material and energy savings. Some examples from abroad show that high performance concrete can be used for optimized shapes of RC (reinforced concrete) elements, which can be very subtle (wall thickness of 30 mm and less) due to their mechanical parameters. Nowadays, use of concretes with compressive strength around 100 MPa is not exceptional, also UHPC (UHPC – Ultra High Performance Concrete) with compressive strength over 150 MPa have already been used for some special application. These kinds of materials enable design with reduced material consumption and thus with lower environmental impacts.

Some secondary materials (waste recycling products from other processes) can be used for cement and concrete production, for production of mould elements or other components. Fly ash, slag or microsilica are used to increase strength and workability. They decrease consumption of energy intensive Portland cement due to their cementitious properties. Recycled concrete from demolition can be used as backfill replacing natural aggregate. Recycled aggregate can be used to some extent as substitute to natural aggregate in new concrete.

5.2 Shape and structural optimization; composite structures

Shape optimization can result in subtle lightened cross-sections. Their lower weight imposes lower load on supporting structures. Application of high performance concrete enables additional savings due to higher reduction of cross section dimensions. Cross section shape can be created using moulds, various types of lightening elements or by application of light-weight concrete. Mentioned techniques can lead to material savings from 30 to 70%.

The composite structures based on high performance silicates and wood represent the beneficial alternative to the timber floor structures. The timber structures have problems to achieve sufficient stiffness; the lack of mass causes troubles with acoustics, inflammability of wood limits the use from the perspective of fire safety. Combination of concrete and timber elements can lead to advantageous structural static and environmental solutions.

Textile reinforced concrete (TRC) has been developed for very thin plate or shell elements (thickness 12 to 25 mm) where the use of steel reinforcement is not applicable because of a corrosion risk due to a thin concrete cover layer. The concept of using TRC was introduced in order to reduce the thickness and thus weight of elements and associated environmental impacts. This concept is mostly used for shells with particular shapes, for strengthening of structural elements or for thin façade panels. Figure 5a shows testing of TRC plate from UHPC and two layers of 2D textile reinforcement. On the Figure 5b there is a prototype of façade panel from white TRC (13 mm thick TRC plate is fixed to steel bearing frame).
5.3 Complex structural concept for sustainable and resilient buildings

Significant advantages of subtle structural elements are material and energy savings during production, transport, manipulation and construction on building site. Accumulative properties of RC structures can significantly contribute to thermal stability improvement of buildings indoor environment. This will mainly affect buildings that need cooling in summer.

With higher demands on thermal insulation parameters of building envelopes increases also their thickness. Subtle structural elements from high performance concrete can be integrated into building envelope of energy efficient buildings with significant reduction of envelope structure and avoiding risk of thermal bridges. The envelope thickness represents restricting parameter of developer plan in regions with regulated size of built-up area. In these cases, subtle RC structures become a great advantage in the form of slender load-bearing wall or subtle RC frame. Figure 6 shows concrete structural subtle frame for construction of family house in Prague in passive energy standard.
6 CONCLUSIONS

The structures for sustainable future should be better prepared for the new situation in the changing world – they should be more resilient. Concrete gradually becomes building material with high potential for expectant environmental impact reduction. This needs better knowledge about technological processes and their impacts from wide variety of sustainability aspects within entire life cycle – from acquisition of materials, through production of concrete and concrete components, construction, use, up to demolition of concrete structure and recycling. Especially optimized concrete structures using new types of concrete in advanced technologies can significantly contribute to needed reduction of global environmental impacts. Mechanical properties of new types of concrete such as high compressive strength, durability, water tightness etc. create conditions for designing subtle structures that leads to saving up to 70% of material in comparison with ordinary concrete, and consequently to reduction of embodied CO\textsubscript{2} emissions. Already implemented realizations give clear signal that in the forthcoming era there will be necessary to take into account new requirements and criteria for design and construction of concrete structures following from global aspects on sustainable development and considering needs for more resilient structures better prepared for changing situation in the world.

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SWEDISH VIEW OF CONCRETE AND SUSTAINABILITY

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Key words: Concrete, Research Needs, Sustainability, Sweden, User Phase.

Abstract. Being the most frequently used man-made construction material with a history of more than 2000 years, concrete may be defined as a sustainable material per se. Correctly designed, properly constructed, and suitably maintained, a concrete structure may reach a life-span that substantially exceeds 100 years. Durability is the bases for sustainability.

The importance of sustainability has increased successively during the last 20 years. In the concrete industry, the focus has been on substituting Portland cement – that roughly stands for 5 percent of the carbon dioxide emissions globally – with various byproducts as slag, fly ash, and silica fume. However, there are many more aspects on sustainable concrete. The Swedish cement producer Cementa AB has a zero vision, i.e., there should be no CO₂ emissions in 2030. The goal will be reached by making the kiln more energy efficient, replacing fissile fuels by renewable fuels, taking the carbon uptake through carbonization into account, and by developing carbon capture and storage. We may develop new concrete mixes that either are lean with not more cement than necessary or high strength that will reduce the cross section and thus the dead load considerably.

In addition to all these measures on the material side, we must maximize the benefits of concrete during the serviceability state. The effects of energy storage capacity, brightness, and wear resistance make concrete buildings and concrete pavements competitive but by, e.g., increasing density, whiteness, and wear resistance we may save more energy for heating, cooling, illumination, and repaving. Finally, we must not forget concrete’s opportunities concerning recycling.

1 INTRODUCTION

Everyone involved in the concrete sector knows that the concrete production, especially the cement production, stands for about 5 % of the CO₂ emissions globally. Since concrete is the most used man-made material, this figure is not astonishing, but we also understand that there is a great potential for improvements. Reducing our emissions with 60 % would lead to global CO₂ reductions equaling 3 %. Reduction potentials can be identified in all phases of a
building’s or a civil engineering structure’s life, i.e., during design and production phase, user phase, and dismantling and recycling (final phase), see Figure 1. However, so far a vast majority of the research within this area has been devoted to the production phase, not least to optimization of the concrete mix and use of industrial byproducts as alternative binders. As organizer of the 2012 fib Symposium on “Concrete Structures for Sustainable Community” in Stockholm, Sweden, [1] we could easily support this statement by just categorizing and counting the number of papers. Only ten papers (or 7 %) were devoted to issues during the user phase (thermal mass, energy storage, fire protection, carbonation & carbon dioxide uptake, and recycling). Another 17 papers (12 %) were devoted to repair, renovation, and upgrading whereas 115 papers (81 %) dealt with the production phase (Table 1).

Modern buildings are normally much more well insulated and energy efficient than older ones. That means that energy used for the production phase has a relatively larger share of the total energy than previously when buildings consumed a vast majority of its energy for heating (and cooling) during the user phase. Consequently, it is logical to focus on the production phase, but this must not imply that we ignore the user phase. For concrete with a lot of advantages during the user phase – thermal mass, brightness, evenness, CO₂ uptake, and durability (Figure 1) – this is especially important.

Figure 1: Possibilities to reduce the CO₂ emissions during three main service life phases.
Table 1: Papers at the fib Stockholm 2012 symposium distributed on categories [1]

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2 OPTIMIZED CONCRETE STRUCTURE

The role of the structural engineer has since more than 100 years been to design a structure as slender as possible in order to save materials and costs. As long as the labor costs were small in comparison to material costs, the optimization of the structure was prioritized. An example is successive termination of the rebars following the declining bending moment curve. When labor costs increased during the 1960s and 1970s, all flexural reinforcement often continued along the entire beam span without cut-off. Another example when section optimization is not fully adopted is designing floor slabs in buildings. Here, the requirements on noise insulation define the concrete slab thickness despite that a thinner slab might fulfill both strength and stiffness requirements. If you would like to save materials, you ought to design the slab thickness from noise requirements and instead increase the span length as far as possibly.

Structural optimization is often connected to diversity or selection of different strength classes. Concrete skyscrapers where the concrete strength is highest in the basement columns and successively lower in columns at higher and higher stories constitute one example (Figure 2 [2]). Column-supported flat slabs where high strength concrete is used in the columns and the areas close to the column head and normal strength concrete is used in areas with minor stresses is another example [3]. A three-layer concrete pavement (Figure 3) is a third example [4]. By producing the concrete pavement with three slipform pavers instead of two it would be possible to save 27 % of the cement content while maintaining the stiffness level that is needed for distributing the traffic stresses to the subgrade.
Figure 2: Trump Hotel in Chicago, Illinois, USA. The building, completed in 2009, is 346 m high and the columns have varying concrete compressive strength from 55 to 110 MPa [2]. Photo courtesy of Mr. H. Russell.

Figure 3: Optimized concrete pavement consisting of three layers; high quality + low quality + high quality. From a structural point of view, the composite concrete pavement may be regarded as an I-beam [4].

3 OPTIMIZED CONCRETE MIX

As stated in Section 1 above, intensive research has been devoted to the optimization of the concrete mix during recent years. The use of industrial byproducts such as fly ash, ground-granulated blast-furnace slag (GGBS), and silica fume as cement substitutes is intensive in industrialized countries with access to carbon heated thermal power stations as well as iron and steel production. Today, the primary industries are responsible for the entire footprint of their products why these substitutes can be used in the concrete industry for “free”, i.e., without countable CO₂ emissions. Also limestone filler can be used as part of the cement without compromising strength. The Swedish cement producer Cementa AB has produced blended cement containing 15 % limestone filler since 1999 and recently (2013), the company
started producing blended cement containing both limestone filler (4 %) and fly ash (16 %).

A problem with fly ash, GGBS, and silica fume is that the available quantities are much too small to substitute 15, 20 or 30 % of the cement that is needed for the global concrete production. A possible solution is calcination of clay that happens to be available in the very same areas of the Globe where the most rapid construction development currently is on-going [5]. The calcination process also needs high temperature, however, not as high as the cement production. Replacing a part of the Portland cement with calcined clay will lead to substantial reduction of CO$_2$ emissions.

However, there are other savings that could be possible in both in-situ and precast concrete production. Swedish multi-story buildings often consist of steel columns and concrete slabs. In order to increase the desiccation speed enabling a rational construction process and early carpeting without causing moisture and fungi problems, concrete mixes with a rather high cement content (> 350 to 400 kg/m$^3$) are frequently used. By changing the construction sequences, it ought to be possible to reduce the cement content to 250 kg/m$^3$ that is completely sufficient for obtaining necessary strength and stiffness. The precast concrete industry is also using high cement contents not only for civil engineering structures (where they are needed for durability) but also for building elements, e.g., façade elements. The high cement content is due to a need of rapid strength development in turn necessary for high productivity. In Sweden, there have been a series of research projects at the Swedish Cement and Concrete Research Institute to develop new accelerators that could be used as alternatives to high cement contents but more research is needed.

4 MAINTENANCE

Durability is a part of sustainability. A large number of studies have been conducted in order to understand the deterioration mechanisms in reinforced concrete and improve its durability. Sweden and the other Scandinavian countries have played an important role in this research. This research has focused mainly on new concrete structures, but to some extent also on durable repairs. A topic that has been almost forgotten in the research is preventive maintenance despite that we know that fairly simple measures are able to prolong the service life substantially. Swedish as well as some other national traffic authorities state that bridges need to be cleaned at least once a year in order to remove de-icing salts and other contaminations. However, the author has not been able to find any research report that has investigated when, how frequently and how the cleaning operations should be carried out or estimated the benefits economically and environmentally [6-7]. Cleaning is not the sole preventive maintenance measure, but maybe the most important one.

5 THERMAL MASS

Concrete buildings have large thermal mass that can be utilized to diminish the temperature difference between day and night. This means that the energy needed for both heating and cooling could be reduced. Modern buildings are, however, usually well insulated independent of the used building material. Consequently, the advantage of thermal mass is less today than previously. The larger thermal mass is more important for power than energy since it cuts off both peaks and valleys.

Swedish researchers [8] have investigated the possibilities to enhance the thermal mass
(expressed as heat capacity \( c \) [J/m\(^3\)/K]) by selecting specific aggregates or cast-in phase change materials (PCM). By using magnetite (iron ore) as aggregate the heat capacity increased with 50\% enabling storage of more energy. The tests with PCM have not been fully successful. The cast-in units impair other concrete properties.

6 REDUCED ENERGY FOR LIGHTENING

Ordinary concrete has a fairly bright color, a fact that can be used in various fields. The ongoing climate change and urbanization lead to the development of so-called heat islands, i.e., large cities that have a temperature that is a couple of degrees higher than the surrounding countryside. Using concrete pavements and concrete tiles leads to increased albedo and decreased temperatures compared to asphalt pavements and black roofing. By using concrete pavements instead of asphalt pavements the energy for road lighting could be reduced with 30\%. This is even more important in tunnels due to the need of 24 hour illumination. On rural roads, the increased brightness may result in increased traffic safety.

Road lightning design methods were originally developed for asphalt pavements and were later expanded to also cover concrete pavements. Löfsjögård & Silfwerbrand [9] have shown that these methods are very approximate especially when considering wet road conditions. Modern concrete roads are also different from the ones that constituted the bases for the design method. Considering the brightness and specularity of modern concrete seems to make further energy savings possible and Löfsjögård has proposed improvements to the road lighting design method used in Sweden. Selection of bright aggregates and white cement would lead to even better result if the selection could be made without high extra costs.

7 REDUCED ENERGY FOR DRIVING ON ROADS

The road traffic causes approximately 30\% of the CO\(_2\) emissions in Sweden. Consequently, every percent savings has importance. Investigations show that a car that is driven on a concrete pavement surface uses less fuel than the very same car driven on an asphalt pavement surface. Measurements show that the saving might be slightly above 1\% for personal cars [10] and as much as 10\% for heavy vehicles [11]. The latter percentage has been questioned by Swedish researchers, but there is still a significant difference of at least 2\%. A hypothesis is that the elastic deformations, which are larger on an asphalt pavement, under the wheel implies that the car always has to overcome an uphill slope. The difference is small but has still a significant impact on the environment.

In the Stockholm area, a new highway mainly in tunnels is planned [12]. It is called the Stockholm Bypass Project and connects the traffic interchange Häggyvik in the north with Kungens Kurva in the south through a 21 km 6-lane motorway that is anticipated to carry an ADT of 140 000 vehicles. Most of the motorway will be through 18 km tunnels (mostly rock tunnels) in two separated tunnel tubes with three lanes in each (Figure 4).
The tunnel will be paved with concrete. Calculations [13] show that the fuel savings during 7.5 years will be of the same size as the entire CO₂ emissions from the cement production for the entire tunnel pavement. This is on the conservative side since the CO₂ emissions from the asphalt production for a flexible alternative have been disregarded as well as the possible energy savings for road lightening.

8 CO₂ UPTAKE

Concrete is a part of a circulation. Cement is a combination of oxides (calcium oxide, CaO, predominates) that are produced when burning limestone (mainly containing calcium carbonate, CaCO₃) at high temperatures emitting carbon dioxide (CO₂), i.e., during the calcination process. During concrete hardening (cement hydration), water is taken up and calcium silicate hydrate (C-S-H) as well as calcium hydroxide (Ca[OH]₂) are produced. Exposed to air, CO₂ is slowly taken up through carbonation. First, Ca(OH)₂ is transferred to CaCO₃. This influences the pH level in the pore solution leading to further carbonation of C-S-H. The process is very slow but in a geological perspective the cement paste turns into marly limestone and concrete to marly agglomerate [14].

In a Swedish research project [15], Andersson et al. have tried to estimate the CO₂ uptake in all existing concrete structures, including its uptake after service life. The estimation is based on both theoretical and measured values of carbonation depth. The authors concluded that 300 000 metric tons of CO₂ were taken up in Swedish concrete structures during the year 2011. That constitutes 17 % of the total CO₂ emissions from the cement production (calcination and fuel) the same year. The authors also stated that by refining the process of dismantling concrete structures, crushing the concrete, and handling the waste, these figures could be doubled.

9 RESEARCH NEEDS

This paper shows that there are several possibilities for the concrete industry and for the society to reduce the CO₂ emissions. In order to utilize more of the potential, a number of research areas and projects can be identified. The following list is not complete but contains
several important items.
- Development of new construction processes that allow moisture control for avoiding fungi problems without using high strength concrete with high cement contents.
- Development of new accelerators or other acceleration methods replacing high cement contents in the precast concrete industry.
- Investigation of optimal preventive maintenance methods and their effects on concrete structures and their durability.
- Investigations on thermal mass focusing on methods to improve the possibilities to take its beneficial effects into account.
- Development of methods to increase the brightness and albedo of concrete surfaces.
- Investigations on fuel consumption for driving on roads of various materials, including comparison between concrete and asphalt as well as efforts to find concrete materials with even better properties.
- Further investigations on CO$_2$ uptake in concrete structures and development of methods to enhance and speed up this chemical process.

10 CONCLUDING REMARKS

Sustainable concrete and sustainable concrete structures cover lot more issues than replacing parts of the Portland cement with industrial by-products. The intention with this paper is to provide an overview of possibilities during all three main service life phases, i.e., during production, user phase, and final phase including CO$_2$ uptake and recycling. The paper focuses on Swedish initiatives and Swedish research within these areas. It is important to consider the positive effects of selecting concrete such as durability, thermal mass, brightness, road surface evenness, and CO$_2$ uptake. All these properties provide advantages in comparisons with other construction materials but more important is that we may develop our concrete in order to enhance these advantages and thus reducing CO$_2$ emissions. A list of research needs is provided.

REFERENCES


A SUSTAINABILITY ASSESSMENT APPROACH BASED ON LIFE CYCLE ASSESSMENT FOR STRUCTURAL RETROFIT OF RC MEMBERS

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Summary. The rehabilitation/renovation of existing structures has progressively attracted the attention of the scientific community and government institutions. However, only few studies have been conducted so far to investigate the environmental performance and potential sustainability benefits of these techniques applied to reinforced concrete (RC) structures. To this aim, we employ a methodological framework to assess the environmental impacts of typical retrofit operations used for RC members and referred to existing buildings: i) carbon fiber and ii) steel wire wrapping and iii) steel jacketing. The corresponding environmental performances are computed by means of a Life Cycle Assessment (LCA) approach and from "cradle to gate".

1 INTRODUCTION

The renovation and rehabilitation of existing structures represent a key contributor to construction activities. In 2010, the global spend on maintenance and repair works was estimated to be about 85% of the construction sector’s total expenditure [1]. Generally, most of this rehabilitation work consists of repairing old, deteriorated structures and/or structures damaged by natural hazardous events. For reinforced concrete (RC) structures, the deterioration phenomena mainly depend on physical aging, chemical aging (e.g. corrosion), and overloads induced by harmful experiences (e.g. earthquake, fire), all of which lead to a reduction in the service life of an RC structure.

The decision-making process concerning the potential repair or demolition of a given structure should be based on economic, social, and environmental considerations, according to a sustainability outlook. In order to avoid the high costs of replacing a structure and also guarantee a given structural performance for an existing RC building, several strengthening techniques have been developed in recent decades, including external bonding steel, continuous fiber-reinforced composite laminates, jacketing, and carbon/steel/basalt/glass fabric wrapping. Significant research work has been conducted on the use of fiber reinforced polymers (FRPs) [2-4]. This is due to: their lightweight and good mechanical properties
(stiffness and strength); the fact they are corrosion-resistant; their good fatigue behavior; their easy application; and the virtually endless variety of shapes available.

However, other than the advantages offered by the strengthening techniques referred to above in terms of structural performance and rapid installation, the environmental performance/benefits of these approaches still need to be fully investigated. Indeed, very few studies have been conducted that assess the sustainability performances of these retrofit solutions [5].

In this context, the present study aims to investigate the environmental footprint related to the application of three retrofit techniques applied to RC columns: carbon fabric wrapping (CFW), steel fabric wrapping (SFW) and steel jacketing (SJ). The main objective is to analyze and compare the environmental performance of all the above retrofit options (which are designed to guarantee the same structural performance) in order to identify the solution that has the lowest environmental impact.

2 METHODS

Given the wide set of possible scenarios and solutions, it should be emphasized that refurbishment and retrofit of existing buildings often require, as mandatory requirement, the fulfilment of several mechanical and functional requests (sometimes prescribed by national structural codes/standards), which have to be properly taken into account during the design of the retrofit itself [6-7]. Among these requirements, a selection of a set of actions should be pursued also in the light of common goals of sustainable development in the construction sector [8]. Each solution should be analyzed by using appropriate criteria (quantitatively expressed by proper indicators) considering financial, environmental, social, and structural aspects, in order to implement the optimal retrofit solution (Figure 1).

Figure 1: Schematic approach for environmental sustainability assessment of retrofit options.
By following this common road map, sustainability and structural requirements might be incorporated within the design stage of retrofit of existing structures. In other words, a comparison between the environmental performance of different retrofit options could be properly carried out by (Figure 1): Step 1, designing different retrofit configurations following diagnostics results and current/local engineering practice; Step 2, verifying that each retrofit solution satisfies some performance requirement defined at the design stages (e.g. structural properties, thermal insulation properties, space availability) while performing an LCA of the different retrofit configurations within a standardized LCA framework; and Step 3, interpreting and comparing the results according to such a framework in order to provide sustainable solution in an extended building lifetime outlook.

3 LCA OF STRENGTHENING TECHNIQUES FOR RC COLUMNS

3.1 Goal and scope definition

The environmental impacts of CFW, SFW, and SJ when applied to an RC column are assessed in a life-cycle assessment (LCA) comparative framework (ISO:14040 2006, [9]). The main hypothesis of the study is that the different strengthening solutions are designed to achieve the same structural performance in terms of the shear strength of the retrofitted RC column. In particular, this hypothesis exemplifies a typical situation when a given shear strength is required in a strengthening operation applied to an RC column (at the end of Step 1 of Figure 1). The LCA framework covers the period from cradle to gate and includes the following life cycle phases: the extraction and processing of raw materials, manufacturing, the preparation of the substrate, and the installation of the reinforcement. The other life-cycle phases such as use, maintenance, end of life, and transportation are not included in the analysis.

The details of each strengthening technique are described as follows:

- **Carbon and steel fabric wrapping**: these strengthening techniques involve adding (in the form of wrapping) a layer of composite material to an existing RC column.
- **Steel jacketing**: SJ includes the use of longitudinal and transverse reinforcement around existing columns. Longitudinal L-shaped steel ties are placed on opposite corners (over the entire length of the column). This type of strengthening improves the axial and shear strength of the column.

The functional unit chosen for the life cycle assessment is defined as "the application of different strengthening techniques on 1 m of an RC column with a given cross-section in order to achieve an increase in shear strength of 30 kN". In particular, the considered cross-section is 30 x 30 cm² in terms of its dimensions and has 4 φ 12 mm as its longitudinal reinforcement. The length of 1 m was also chosen as the reference length of the column for applying the reinforcement. The increased shear strength was calculated according to the national structural codes (NTC 2008 [10]; CNR 2004 [11]).

As depicted in Figure 2, for all the strengthening techniques, the following life-cycle phases were considered:

- **Material production**: this phase includes the raw material extraction and the manufacturing process of the materials used in each retrofit technique.
- **Application phase**: this consists of:
  1) The preparation phase (of the substrate that has to be reinforced):
     - The external surface of the RC column is thoroughly flushed with clean water to remove as much dirt, debris, and contaminants as possible.
     - The damaged or deteriorated concrete is removed.
     - The reinforcement bars are treated with anti-corrosive proper products in order to reduce the steel oxidation.
     - Where needed, a concrete layer is added to the external surface of the RC column.
  2) Installation phase:
     - A layer of primer is applied to the RC column surface.
     - The external reinforcement is applied to the RC column.

![Figure 2: System boundary of the a) CFW, b) SFW and c) SJ strengthening technique.](image)

### 3.2 Inventory Analysis

Both primary and secondary data were utilized for the life-cycle inventory analysis. Primary data were used to model steel material manufacturing (e.g. transverse and longitudinal reinforcement) and steel/carbon wrap fabric production. In particular, technical information reported in the datasheets of Mapei S.P.A. [12] and in the ReLuis 2011 guidelines [13] were used to model CFW and SFW production, while the technical data contained in [14] and in the ReLuis 2011 guidelines [12] were used to model steel material manufacturing for the SJ technique.

The secondary data were directly retrieved from the product databases available in the Simapro 7.3 LCA software package [15].

### 4 IMPACT ASSESSMENT

In the present study, the IMPACT 2002+ methodology [16] was adopted to calculate and quantify the environmental impacts of the three strengthening techniques of RC members. The LCA results are discussed in terms of damage (damage categories) and characterization assessments (mid-point categories).

#### 4.1 LCA results
Figure 3 reports the environmental results of the CFW, SFW, and SJ retrofit solutions, respectively.

In the CFW solution, the life-cycle phases investigated, i.e. material production and the application phase, were responsible for approximately the same environmental burden in almost all the damage categories. The LCA detailed analysis of the material production related to the manufacturing process of the carbon fabric revealed that the weaving process has the highest impact in this life-cycle phase: its impact is almost 39% of the total burden. In addition, the application of primer to the external surface on the RC column, accounts for 65% of the total environmental burden.

The LCA results for the SFW solution were influenced by the application phase; in fact, the environmental impact related to this life-cycle phase was almost 70% of the total burden. In particular, the installation phase with the application of primer to the external surface on the RC column has the highest environmental impact at almost 65% of the total burden.

In the SJ solution, the major environmental impact was related to the material production phase: its environmental impact accounted for 90% of the total environmental burden. In detail, the steel material production, which is related to the transverse and longitudinal reinforcement manufacturing process, has the highest environmental impact in the material production phase at almost 90% of the total burden.

![Figure 3: LCA results in terms of damage categories for a) CFW, b) SFW and c) SJ strengthening technique.](image)

Figure 4 shows the LCA comparison of the investigated strengthening techniques. As can be seen, the SJ technique has the highest environmental impact in almost all the damage and mid-point categories. In particular, in the human health, ecosystem quality, and climate change damage categories, CFW and SFW have an environmental impact that is between 20-80% lower than that of the SJ solution. The CFW technique has the highest impact in the resource damage category, whereas the SFW option is characterized by the lowest environmental impact.

It was found that the SJ solution is responsible for the highest environmental burden in terms of the material production phase. A contribution analysis (ISO:14040 2006, [9]) was conducted to evaluate which material/process influences the environmental results of the SJ solution. This analysis, which was conducted for each damage category, revealed that reinforcement production (steel material manufacturing) influences the environmental impact of the SJ solution as follows:

- **Human health category**: the LCA result is influenced by the steel materials produced in
electric and basic oxygen furnaces ("steel, converter, unalloyed" and "steel, electric, un-and low-alloyed"); this process emits a large amount of dioxins and sulfur dioxide.

- **Ecosystem quality:** the LCA results are influenced by the production of steel materials and, in particular, by steel materials produced in basic oxygen furnaces ("steel, converter, unalloyed") that emit a large amount of aluminum and zinc emissions into the air and water.

- **Climate change:** the steel materials produced in basic oxygen furnaces ("steel, converter, unalloyed") influence the LCA results in this end-point category; these materials emit a large amount of CO₂ emissions into the atmosphere.

- **Resources:** in order to produce steel ("steel, converter, unalloyed"), several non-renewable raw materials such as uranium, coal, oil, and natural gas are used.

Finally, CFW has the highest environmental impact in the resources category, as shown in Figure 4. This result is influenced by epoxy resin and carbon fiber production. The environmental impact of epoxy resin (used as impregnating resin) and carbon fibers accounts for 70% of the total burden in the material production phase, whereas the epoxy resin (used as primer) accounts for 26% of the total environmental impact in the installation phase.

![Figure 4: LCA comparison; (a) Damage assessment; (b) Characterization categories of the strengthening techniques investigated.](image)

5 CONCLUSIONS

In the present study the environmental impacts of different strengthening techniques applied to RC columns were assessed. Three techniques were examined using an LCA approach: carbon and steel fabric wrapping and steel jacketing. All of them were designed to achieve the same structural performance in terms of shear strength for a retrofitted RC column. The first objective of this research was to use a life-cycle perspective to evaluate the relative environmental impact of each alternative within a decision-making framework of retrofit design. In particular, the LCA results on the CFW solution revealed that the material production (carbon fabric manufacturing) and application phases (application of primer to the external surface of the RC column) made the same environmental contribution. In the SFW solution, the LCA results were mainly influenced by the application phase (two-component
resin used as primer), while in the SJ solution, the major environmental impact was related to the material production phase. In particular, the steel material manufacturing accounted for 90% of the total environmental burden. When the strengthening solutions were compared, the results reveal that the SJ technique has the most impact in almost all the damage and impact categories, which is mainly due to the refining and melting process for steel (material production phase). The CFW solution had the highest impact in the resources category, which was mainly due to the epoxy resin used as an impregnating system and the carbon fiber production.

As final remark of this preliminary environmental analysis, authors want to point out that these results need to be integrated with primary end of life data which are currently under collection for a comprehensive cradle to grave analysis.

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CARBON EMISSIONS CAPTURING IN CEMENT
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Key words: Carbon emissions, capturing, cement, concrete.

Abstract. The construction industry is constantly growing and demanding more materials to support its progress. Portland cement production is highly energy consuming and greenhouse gases emitting, and there are great efforts to minimize the environmental impact of the cement manufacturing.

Furthermore, the world’s population is forecasted to reach 9 billion people by 2050, and with this comes a substantial need for infrastructure and urbanization and an enormous demand for cement: a consumption of 3859 million tonnes worldwide was observed in 2012, the highest in history and it is continuously increasing.

On the other hand, international agreements for the greenhouse gas emission reductions exert pressure on the industry by demanding the reduction of CO₂ emissions by 80% prior to 2050, while the production is expected to double. Cement manufacturing produces approximately 2.4% of the global CO₂ emissions from industrial and energy sources. This means that this process and material characteristics have to change significantly in the near future if these reductions are to be achieved.

There are efforts from the cement and concrete industry on finding alternatives not only to reduce the emission of CO₂, but also to develop materials that can capture the CO₂ from the environment during its production and service life, becoming carbon-zero or even carbon-negative.

Technologies being developed range from traditional clinker materials, to alternative cements and binders, and are reviewed in this paper.

1 INTRODUCTION

The construction industry is constantly growing and demanding more materials to support its progress. Currently, Portland cement is the most widely used type of cement, and it is composed of 95% clinker. Its production is highly energy consuming and greenhouse gases emitting. For these reasons there are great efforts to decrease these two factors in order to minimize the environmental impact of the cement manufacturing.

Furthermore, the world’s population is forecasted to achieve the astonishing number of 9
billion people by 2050. This means a substantial need for infrastructure and urbanization and with this comes an enormous demand for cement: a consumption of 3859 million tonnes worldwide was expected for 2012, the highest in history and it is continuously increasing. China alone has almost doubled its consumption from 2004 to 2010, reaching an amount of 1851 million tonnes. The second-largest consumer is India, with 212 million tonnes in 2010, and in third place the United States consumed 69 million tonnes, according to the 9th International Cement Review’s Global Cement Report. So far, the consumption in Europe is decreasing over the last years regarding the economic slump: from 260 in 2007 to 190 million tonnes in 2010. Worldwide the cement and concrete industries provide jobs to 1% of the population.

On the other hand, international agreements for the greenhouse gas emission reductions exert pressure on the industry by demanding the reduction of CO\textsubscript{2} emissions by 80% prior to 2050, while the production is expected to double. Cement manufacturing produces approximately 2.4% of the global CO\textsubscript{2} emissions from industrial and energy sources. However in the Portland cement industry most of the carbon emissions (64%) are from the chemical process rather than from the fuel combustion [1], [2].

In the last years many new low-carbon cements have been introduced in the market as alternatives to OPC, such as magnesium-based binders (Novacem, Calera, Greensols, TecEco), geopolymers (E-Crete, Calix, Zeobond, HySSIL, Geo green crete), blast furnace slag (Ecocem), sulfoaluminate belite cements (Alipre, Belite – calcium sulfoaluminate-ferrite (BCSAF), Aether), phosphate binders (Ceramicrete), and CSH-binder (Celitement) [3].

\section{CO\textsubscript{2} REDUCTION}

Many efforts are being made in finding ways to decrease the use of clinker in cements by using supplementary cementitious materials (SCM), to adopt alternative fuels (use of electric energy reduces significantly the CO\textsubscript{2} emissions), and to improve the energy efficiency of the thermal process involved in the cement manufacturing. Besides, the use of high strength concrete in structures allows reducing the mass of concrete members, together with the use of concretes with low cement and water contents permitting a decrease in the cement consumption and, consequently, the carbon footprint [4].

However, these efforts can only reduce about 25\% of the emissions by 2050 (about 540-590 kg CO\textsubscript{2}/t cement, while 730 kg CO\textsubscript{2}/t cement were emitted in 2009) [5]. Another approach is the use of cements that emit less CO\textsubscript{2} during production compared to OPC. However the efficiency is not high enough to meet the targets for carbon reduction [6].

The use of oxygen combustion fuels are a good alternative as it would eliminate the need for the gas separation process, as well as plasma fusion, by assuming non-fossil electrical source, and parallel processing using CO\textsubscript{2} to make basic chemicals [7].

Looking forward to reduce the use of clinker, the adoption of supplementary cementitious materials (SCM) has been increasing over the last decades, and regulations and society accept it widely. The optimization of the binder composition not only for optimal performance and strength, but also for minimization of carbon footprint, is still un-going. Additionally, there is nowadays the use of high-SCM-substituted OPC, or cements with several main constituents, where higher amounts of SCM are being used in a way to reduce the clinker consumption. The amount of clinker and SCM used depends on materials availability regionally, regulation,
and market, and have been changing lately. Globally, the clinker factor (CF - the proportion of clinker in cement) was 0.77 in 2010, and the Cement Technology Roadmap 2009 predicts it to reach is 0.71 in long term [8]. That is, every time more SCM are being used in cement, as can be observed in Figure 1.

![Figure 1 Cement types produced by Holcim 1995-2009 [8].](image)

Intensive research is being developed aiming to find alternative cements to OPC. Geopolymers need an activation agent to solidify in a highly alkaline environment, and can be from natural sources or from waste materials (e.g. kaolin or fly ash, respectively). The characteristics, performance, and durability of the final product strongly depends on the amorphous inorganic aluminosilicates network that are formed from polycondensation reactions [8]. One of the main geopolymers is the ground-granulated blast furnace slag (GBFS), which CO₂ emissions depend on the source. Its availability is limited by iron production, and there is the inherent risk of reduced availability on the supply of these materials [6]. According to Damineli et al, the total supply of GBFS is around 13% of the global cement production [9]. GBFS requires a hydration activator (CH, CS or strong alkali) [2].

Calcium sulfoaluminate (CSA) cements, composed of varying ratios of ye’elimite (C₄AₛS), belite, C₄AF and gypsum, harden and gain strength by the formation of ettringite and C-S-H phases. Their CO₂ emissions are about 20-30% lower than OPC clinker, since the sintering temperature is around 100-150 °C lower than the needed for OPC clinker. The main advantage is the fact that these clinkers can be made in conventional cement plants, using alumina-rich clays or coal ashes as main Al source and high-sulphur fuels as main sulphur source for clinker, which can be further blended with pozzolans [4]. Nevertheless, aluminium and sulphur are not readily available globally, with limited sources which also limit the CSA cement production, not allowing this to be a complete replacement to OPC clinker [8].
3 CO₂ UTILIZATION

Carbon capture and storage (CCS) is an alternative to reduce even further the emissions in order to reach the target by 2050. There are several techniques being explored to provide efficient carbon capture in cement plants, such as amine scrubbing, calcium looping, full oxy-fuel combustion, partial oxy-fuel combustion, and direct capture [5], among others.

The most appropriated technology for retrofitting of cement manufacturing sites, since it does not require big changes in the kiln burning process, is the post-combustion CO₂ capture. Post-combustion technology can be done in several ways, especially chemical absorption by amines, potassium, etc. According to the International Energy Agency (IEA) [10], the post-combustion technology have the ability of capturing up to 77% of CO₂ exhaust gases in cement plants. However they underline that the cost of building a new cement plant with this technology would be about twice more than a traditional plant, besides the additional demand for energy to capture, process and store the CO₂ during the process.

Another more affordable option is the oxyfuel combustion, where oxygen, and not air, is used in the kilns, generating more pure CO₂ as an emitting gas. However, since the environment inside the kiln is changed, the properties of the clinker may also be changed and have to be investigated and controlled. This technology would cost around 25% more for the building of a new plant, and 25% more in the operational costs during the service life, however this cannot be applied in the retrofitting of existing cement plants, being only possible to be applied in new factories [10].

There is also the on-going development of membranes and solid absorption processes for carbon capture from cement plants, such as the calcium looping, which is a process of CO₂ capture from flue gases using CaO-based solid sorbents. The process is based on the reversibility of the reaction between CO₂ and CaO to form CaCO₃, resulting in a highly concentrated steam of CO₂ that is suitable for storage [11]. According to Dean et al [11], the CaO (mainly) resulting from the purging process (with parts of ash and calcium sulphate) could replace CaCO₃ in the cement feed, while the ash resulting could contribute for the aluminosilicates necessary for cement synthesis, usually from clay or additives.

That is, the carbon capture in cement industry is very challenging not only in the technical aspects, but also in the economical ones: today, the cost of 20-50€/t CO₂ is estimated for CO₂ capture only, not including transport, storage, and cement plant retrofitting [8].

It is widely known that fresh OPC selectively sorbs CO₂ from moist gas, and this process is widely used to cure non-reinforced OPC products such as bricks and blocks, as well as in the precast industry. This seems to be a natural way to capture CO₂ in OPC [7]. Monkman et al [12], [13] describe significant improvements in concrete blocks compressive strength when submitted to carbon injection during mixing, claiming that the carbonation may have promoted the increasing formation of C-S-H in the first 10 to 24 hour hydration. However there are concerns about the stability of the reinforcement embedded in the structural concrete due to changes in the media pH resulting from the carbonation.

3.1 Carbonated calcium silicate cements

Binders based on non-hydraulic calcium silicate have the ability to carbonate rapidly under atmospheric pressure CO₂ with some water as a catalyst (CaSiO₃+CO₂→C₃A₂O₃+SiO₂).

The advantage is that they can be produced in conventional cement kilns by reversing this
reaction, while during curing the CO₂ is recaptured. The circular gas usage in the cement plant would allow savings of more than 30% CO₂ compared to OPC. Additionally, less energy is required in the process since the temperature needed is as low as 1200°C and the clinker is softer, easier to grind than traditional OPC clinker [14].

Commercially, Solidia™ concretes use non-hydraulic binder (based on wollastonite or pseudo-wollastonite CaSiO₃ as a main binder for chemical sintering, which requires large pores from the water evaporation to allow the CO₂ entry in the volume, while the small pores keep the water to transport Ca, making it crucial to have a strict control of RH during curing process. The rate is controlled by temperature and pressure of gas CO₂ [15]. It is said that a concrete cast using 16 mass% of Solidia™ can isolate about 5% of its total mass. That means around 300kg CO₂/ton of binder [14]. Chemical properties, as well as pH, are totally different of the traditional OPC: pH of pore solution after carbonation can go below 10, making it unsafe to use traditional reinforcement as there is not passive protection to the steel bars. Mechanical properties, however, reach levels similar to OPC.

Alternatively, Calera™ is inspired by the nature, mimicking sea corals and shells formation by bio-mineralization, by capturing raw anthropogenic CO₂ mainly from power plants for the carbonation of a calcium carbonate polymorph called vaterite. Calera’s source of calcium is mainly from CaCl₂ from brines, seawater, or waste streams of chemical processes [16]. The product of the carbonation are then dissolved in an aqueous solution with high pH, where it precipitates as carbonate minerals after combination with divalent cations, resulting in a stable material that can be used either as SCC or as concrete aggregates [17].

Precipitates from cement paste with Calera calcium carbonated products were observed to be of different polymorphs such as calcite, vaterite, and an amorphous magnesium–calcium carbonate phase. Its use was not observed to alter anyhow the early age hydration of OPC, however the stabilization of ettringite by the formation of mono-carboaluminate hydrate is affected: the formation of portlandite is formed at the same rate as in normal OPC pastes while the other hydrates are formed in higher amounts. This affects the compressive strength at early ages, that becomes higher than in OPC, while at later ages the opposite is observed [17]. Besides calcium, Calera also carbonates magnesium.

### 3.2 Magnesium oxy-carbonate binders

MgO cements have the capability of sequestering substantial amounts of CO₂ during its hardening process. It is used mostly for non-structural elements as the low pH, result of the carbonation process, would depassivate the reinforcement and cause corrosion, however the durability of this material is improved by the hydration and carbonation products’ high resistance to aggressive agents. This material also allows the usage of large quantities of waste and industrial by-products, due to its low sensitivity to impurities. Finally, at the end of its lifecycle, the concrete structure can be fully recycled, where by decarbonation and dehydration processes the material turns back into MgO and can be used alone as the binder. The hardening process happens by mixing reactive MgO with water, hydrating and forming brucite (Mg(OH)₂, magnesium hydroxide), which has no strength by itself. However, by exposing it to CO₂ (considering there is enough water for reaction), hydrated magnesium carbonates (HMCs) are formed in different forms, and these are are stronger. The main forms
of HMC are nesquehonite (MgCO$_3$·3H$_2$O), hydromagnesite (4MgCO$_3$·Mg(OH)$_2$·4H$_2$O), and dypingite (4MgCO$_3$·Mg(OH)$_2$·5H$_2$O) [18].

The strength development by the formation of HMCs depends on the rate and degree of carbonation of MgO cement and are influenced by the type of MgO, with its composition and characteristics; by the concrete mix design; by the use of additives and/or admixtures; by the particle size distribution of the aggregates in the mixture, as well as the particle packing; strongly by the final porosity of the concrete paste, which will allow the CO$_2$ to penetrate; and by the curing conditions to what the concrete is submitted, such as relative humidity, temperature, transport and partial pressure of CO$_2$ [19]. The type of cure, as well as the binder characteristics, will determine the type of HMC formed and consequently the strength and carbon footprint of the structure.

When the concrete pore system allows enough CO$_2$ into the material, the hydrated magnesium can reach about 100% carbonation and forming the optimum amount of HMC. This is when the total production CO$_2$ emissions reach a near-zero levels, or even negative levels, absorbing more CO$_2$ that it emits during the manufacturing [20]. Several research have been developed aiming to evaluate the mechanical performance of reactive magnesia cement blocks [21]–[24], showing that, at 20% CO$_2$ content, blocks cured at 21°C and 98% RH achieved 70% carbonation after 28 days curing [25].

One example of this type of cement is Novacem™, a spin-out from Imperial College, London, which product absorbs more CO$_2$ than it generates during its manufacture. Its composition is a blend of magnesium oxide and hydrated magnesium carbonates, characteristic that provides the high mechanical strength.

The main issue, addressed by Novacem™, is how to produce, from basic magnesium silicates, a reactive MgO in an energy-efficient industrial process, using the reaction between supercritical CO$_2$ and magnesium silicate powdered-rocks to produce magnesite and amorphous silica for forsterite olivine (Mg$_2$SiO$_4$→2MgO+SiO$_2$) or from antigorite serpentine (3MgO.2SiO$_2$.2H$_2$O→3MgO+2SiO$_2$+2H$_2$O) [2], [14].

4 FINAL REMARKS

There are many efforts worldwide to reduce the emissions and improve the carbon capture in the cement industry. The challenge is to find suitable materials and techniques for this, and to prove their efficiency and long-term commercial potential globally, and in the volume needed.

In order to have the new technologies ready for the huge demand of the construction market it is necessary to go through rigorous testing, standardization, and public education for acceptance, specially for new unfamiliar technologies. It is necessary to assure safety and long-lasting performance.

At this stage, the new low-carbon cements cannot be used for structural purposes, especially with reinforcement. Development in this area is necessary for these technologies.
Another crucial factor is the cost of the new eco-materials, since OPC has very low cost, a competitive material should not exceed the reasonable prices set by the current material, especially until it becomes fully accepted to be produced in extra-large scale globally.

Regarding the CCS technologies, they are unlikely to be widely adopted in a short timeframe, since they do not seem to be ready to be deployed. However IEA projects that 50% of the cement plants in the Organization for Economic Cooperation and Development (OECD) countries (and at least 20% in India and China) will be equipped with CCS technologies by 2050 [10].

Even though there is a huge demand, there is also a long journey for the development of materials and technologies for a substantial reduction of carbon emissions to reach the market. On the way, any technology that promotes at least a small reduction is worth being developed and applied, as well as it is crucial that regulatory agencies, government and society support the industry for the development of the needed technologies.

REFERENCES


DESIGN FOR SAFETY IN CONSTRUCTION WORK
ICCS16

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Abstract. The concern for occupational health and safety in construction work is reflected in the many preventive measures taken. However, examples of the systematic assessment of project alternatives aimed at minimizing occupational hazards are rare.

This paper proposes a measure of occupational safety, the occupational risk index (ORI), that is based solely on the project design and resulting construction process, and is a function of the activities carried out and their specific occupational risks (probability of occurrence and most probable consequence). The proposal is illustrated with an example in which two alternatives, one precast and the other constructed in situ, are prioritized in terms of occupational safety, and certain aspects related to redesign are briefly addressed. The research is based on an analysis of the applicable legislation and interviews with experts on risk prevention.

With the ORI, occupational safety goes from having a passive influence (applications to projects that have already been designed) to an active one on the design concept itself. The ORI can thus be used as an indicator to feed multicriteria decision-analysis tools.

1 INTRODUCTION

Occupational safety is one of the most important social aspects of construction work. However, unlike other issues addressed in the decision-making stage of project management, it is usually considered only after completion of the design [1].

Tam et al. [2] applied multi-criteria methodologies to decision-making on health and safety strategies. They took the design as a starting point, but not as a concept that could be influenced to improve working conditions. Akintoye and MacLeod [3] studied risk management in construction projects. Although they did not consider the possibility of influencing the design to improve occupational conditions, they noted one aspect considered relevant in this study: the lack of familiarity of construction industry professionals with risk analysis methodologies. This notwithstanding, in an in-depth 1996 study of risk management in the construction industry, Jaselskis et al. [4] suggested that occupational risk management
expertise can be found among construction industry professionals. However, the individuals who have it are generally involved in the project management, rather than design, team.

Armengou et al. [5] applied the concept of occupational safety to the process of selecting alternatives. However, due to the lack of an occupational safety index or similar indicator, they had to assign a safety level to each alternative without the use of a tried-and-true method.

In this paper, multi-criteria analysis methodologies are combined for decision-making in project management with an assessment of the occupational risks inherent in certain construction activities. The aim is to enable the consideration of occupational safety criteria from the very start of the design process.

Some authors [1] [6] [7] have studied the safety of construction projects based on the different health and safety measures or strategies used by the contractor. The methodology described in this paper assumes that the contractor will take all the health and safety measures specified in the project design and required by law and focuses instead on the construction project’s design. It can thus be calculated prior to the contract’s award, making it possible to compare different design alternatives. Indeed, the specific objective of the methodology is to introduce the concept of occupational safety at the design level.

To this end, first, the main risks associated with different construction activities are analyzed. Next, an Occupational Risk Index (ORI) for construction work is defined, calculated as the sum total of the workload for the risk activity weighted by its relative risk. The workload for each risk activity is calculated as the total number of working hours spent on it. The relative risk of each activity is calculated based on the probability of occurrence of the risk and the likely severity of its consequences. This is followed by an example of how to obtain the ORI for two different construction alternatives. A brief discussion of different methodologies for supporting decision-making in the context of project management is also included. The discussion shows how application of the ORI would integrate occupational safety criteria into the selection of design alternatives for construction work.

2 METHODOLOGY

The methodology used in this study falls within the framework of generally accepted risk management schemes [8] [9] and consists of the following steps: establishing the context, identifying the risks, analyzing the risks, evaluating the risks, and treating the risks. Therefore, once the occupational health and safety context has been established for a given construction project, finding and grouping the relevant risks posed by the different construction activities is the next step toward developing a model. To this end, this study first analyzed relevant European and Spanish legislation, technical guides, and other supporting documents applicable to the health and safety of construction site workers. An initial round of interviews was then conducted with three panelists (health and safety experts). Finally, three additional panelists reviewed the resulting list of risks and activities. All six panelists qualified as experts according to the criteria suggested by Hallowell and Gambatese [10]. Moreover, one had extraordinary expertise, having been informed of and investigated most construction accidents occurring in Catalonia (a region of Spain with a population of 7.6 million and an area of 32,000 km²) over the last 40 years.
3 RESULTS

3.1 Main occupational risks associated with construction activities

As a result of this process, it was concluded that the main health and safety risks found in construction work and its associated activities to be included in this methodology were those included in Table 3, in addition to the following ones (Risk - Activity):

20. Same-level falls – All types of work.
21. Heat stroke, cold-related injuries and sunburn – Outdoor work under adverse weather conditions.
22. General increase in accident probability – Night work or work in reduced visibility conditions.

The following occupational diseases were also taken into consideration in the analysis:

27. Illnesses caused by asbestos – Work involving possible exposure to asbestos.
29. Silicosis – Work that produces high concentrations of silica dust.

The final methodology takes into account the risks associated with activities 1-19 in Table 3. The risks associated with 20-22 have not been included for the following reasons:

− Same-level falls – All types of work (Risk 20): Same-level falls can occur in any activity involving the movement of people and materials. They are mainly due to a lack of order and cleanliness and also to human behavioral factors and personal issues that are difficult to control. Therefore, the methodology does not consider this risk.
− Heat stroke, cold-related injuries and sunburn – Outdoor work under adverse weather conditions (Risk 21): The environmental conditions of a workplace (temperature, wind speed, humidity and radiation), along with the intensity of the work and the clothing used, can pose a risk to workers’ health known as heat stress. Heat stress can be caused by heat or cold, and certain individual characteristics can increase the risk. Because of the variability of the weather, the inability to accurately forecast it well in advance, and the influence of personal conditions on the occurrence of the risk, the methodology does not take this risk into account.
− General increase in accident probability – Night work or work in reduced visibility conditions (Risk 22): The consequences of a night accident are the same as those of the same accident during the day. However, the night accident may be more likely to occur due to the overall lack of visibility at the construction site. Even when floodlights are provided, there are dark areas where risks may not be perceived as well as in daylight conditions. As the exact increase in the probability of the occurrence of accidents due to night work is unknown, this factor cannot be assessed.

Because it is not possible to assess the likelihood of contracting an occupational disease from exposure to a single instance of construction work, occupational diseases (Risks 23-29)
were likewise excluded from the final methodology. Most of these diseases are not the result of a single exposure to the cause, but rather of prolonged exposure, and they are moreover influenced by the personal characteristics of each worker.

3.2 Definition of the occupational risk index (ORI)

All construction work has to comply with the current legislation on occupational risk prevention. However, some activities present higher risks than others. The ORI is a measure of the risk involved in a given construction project that depends on the volume and type of activities performed. It can thus be calculated once the project has been defined.

The ORI [equation (1)] is calculated as the summation of the probability of occurrence of an accident \( (P_i) \) multiplied by the severity of the most probable consequence \( (C_i) \) and by the total amount of working time to be devoted to the activity (exposure to the risk, \( E_i \)) of all the activities with risk \( (i) \) in a construction work. The higher the ORI is, the higher the risk involved in the construction work.

\[
ORI = \sum_i ORI_i = \sum_i P_i \times C_i \times E_i = \sum_i W_i \times E_i
\]  

The ORI was inspired by Fine’s method [11]. Fine used the frequency of exposition to a risk, which is appropriate for sectors with recurrent activities. However, in the present methodology, the frequency of exposure was substituted by the time of exposure to the risk, which is more appropriate to the constantly changing activities in construction.

The ORI can also be used by non-experts in health and safety. The calculation includes the person-hours spent by both direct and subcontracted employees on all risk activities. It also includes those spent on the rerouting of services or any necessary modification work carried out by utility companies, fuel companies and others, due to the execution of the work.

The following step corresponds to the risk analysis [8] [9]. In order to assign a relative importance \( (W_i) \) to each risk activity, a qualitative assessment was conducted of the most probable consequence of a potential accident, and a numerical value was assigned (Table 1). Likewise, a qualitative assessment was conducted and a numerical value assigned to the probability of an accident happening given the risk activity (Table 2). The importance of each type of risk was obtained by multiplying the accident’s consequence by its probability. Tables 1 and 2 have been adapted from Fine [11]. The consequence ratings are much higher than the probability ratings because an extremely severe risk with a low probability of occurrence should be given more weight than a probable minor injury.

<table>
<thead>
<tr>
<th>Consequences</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Catastrophe, numerous fatalities</td>
<td>100</td>
</tr>
<tr>
<td>Multiple fatalities</td>
<td>50</td>
</tr>
<tr>
<td>Fatality</td>
<td>25</td>
</tr>
<tr>
<td>Extremely serious injury (amputation, permanent disability)</td>
<td>15</td>
</tr>
<tr>
<td>Non-serious disabling injury</td>
<td>5</td>
</tr>
<tr>
<td>Minor injury (minor cuts, bruises and bumps)</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 1: Qualitative and numerical ratings for the most probable consequence of a potential accident.
Table 2: Qualitative and numerical ratings of the probability of the occurrence of an accident.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Probability</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Most likely and expected result should the hazard event occur</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Quite possible, not unusual</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Unusual sequence</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Remotely possible</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Has never happened but is conceivable</td>
<td></td>
<td>0.5</td>
</tr>
</tbody>
</table>

The assessment of the likelihood and consequences of each risk was calculated as the average of the assessments made by three occupational health and safety specialists who qualified as experts according to the criteria established by Hallowell and Gambatese [10]. Three rounds of assessments were conducted; a consensus was reached, and the deviation was bounded. Table 3 shows the resulting values (P = numerical assessment of their probability; C = numerical assessment of the consequences; and PxC = product of these values or weight), as well as the standardized weight, calculated by dividing the weight of each risk by the highest possible weight (1,000).

However, the probability of occurrence of an accident and the severity of certain consequences can vary depending on how technologically developed a given region or company is and the safety management practices. The probability assessment shown in Table 3 is a guidance value and may not apply in all cases. It is up to the local health and safety experts to determine whether these coefficients are applicable to their region and, if not, to assess the new probability and consequences. To that end, we recommend following the Delphi method as presented by Hallowell and Gambatese [10].

Table 3: Relative risk of each activity.

<table>
<thead>
<tr>
<th>Risk – Activity</th>
<th>Rating</th>
<th>Weight</th>
<th>Standardized weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Falls to lower levels—Work at heights or depths of more than 2 meters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Winch</td>
<td>1.0</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Conventional formwork</td>
<td>4.5</td>
<td>23.3</td>
<td>105.0</td>
</tr>
<tr>
<td>Self-climbing formwork for piles or dams</td>
<td>3.0</td>
<td>50.0</td>
<td>150.0</td>
</tr>
<tr>
<td>Sliding formwork</td>
<td>1.0</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Centering (during assembly and dismantling only)</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Self-launching centering (during assembly and dismantling only)</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Trestle scaffold (up to 3 meters high)</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
</tr>
<tr>
<td>Mast-climbing work platform</td>
<td>1.0</td>
<td>20.8</td>
<td>20.8</td>
</tr>
<tr>
<td>Steel-tube scaffold</td>
<td>4.5</td>
<td>21.7</td>
<td>97.5</td>
</tr>
<tr>
<td>Mobile scaffold and mobile work platforms</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Activity Description</td>
<td>s</td>
<td>a</td>
<td>e</td>
</tr>
<tr>
<td>-------------------------------------------------------------------------------------</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Hanging scaffold</td>
<td>4.5</td>
<td>31.7</td>
<td>142.5</td>
</tr>
<tr>
<td>Work platform for the maintenance or changing of the cutters on a tunnel-boring machine</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
</tr>
<tr>
<td>Nonmobile work platforms and concrete-pouring work platforms</td>
<td>3.0</td>
<td>10.0</td>
<td>30.0</td>
</tr>
<tr>
<td>Placement of concrete slabs and reinforcement-laying and concrete-pouring work on the deck of a bridge</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Hollow spaces (mainly in buildings)</td>
<td>3.0</td>
<td>25.0</td>
<td>75.0</td>
</tr>
<tr>
<td>Outside openings in facades</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Work on decks</td>
<td>3.0</td>
<td>21.7</td>
<td>65.0</td>
</tr>
<tr>
<td>Ditches</td>
<td>1.3</td>
<td>10.0</td>
<td>13.3</td>
</tr>
<tr>
<td>Cut and fill batters</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
</tr>
<tr>
<td>Shafts</td>
<td>1.0</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Work inside floating caissons</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
</tr>
<tr>
<td>Manual demolition</td>
<td>6.0</td>
<td>20.0</td>
<td>120.0</td>
</tr>
</tbody>
</table>

2 Direct or indirect electrical contact– Electrical work, work in proximity to power lines, and work with electrical equipment under wet conditions

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electrical work on overhead power lines or other unprotected live elements (work in the hazardous area)</td>
<td>3.0</td>
<td>22.5</td>
<td>67.5</td>
<td>0.068</td>
</tr>
<tr>
<td>Works in proximity to overhead power lines or other unprotected live elements</td>
<td>4.5</td>
<td>20.0</td>
<td>90.0</td>
<td>0.090</td>
</tr>
<tr>
<td>Work in proximity to live underground power lines</td>
<td>4.5</td>
<td>20.0</td>
<td>90.0</td>
<td>0.090</td>
</tr>
<tr>
<td>Work with mobile and semimobile electric concrete mixers</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
</tbody>
</table>

3 Burns caused by fire or explosion due to the rupture of a pipeline – Work close to fuel pipelines

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Work with mobile and semimobile electric concrete mixers</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
</tbody>
</table>

4 Gas inhalation – Work close to gas pipelines

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthmoving and open-cast mining</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Shafts with casing and simultaneous formwork</td>
<td>1.0</td>
<td>20.0</td>
<td>20.0</td>
<td>0.020</td>
</tr>
<tr>
<td>Shafts without casing (mainly used in mines)</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Underground work and tunneling performed with traditional methods, explosives, roadheader machines, or tunneling machines under normobaric conditions during the maintenance and changing of the cutters</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
</tbody>
</table>

5 Entrapment and subsequent suffocation due to a landslide – Earthmoving, excavation, shafts, underground work and tunnels

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle projection and accidental explosion – Blasting for excavation, shafts, underground work and tunnels</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Accidental explosion</td>
<td>1.0</td>
<td>25.0</td>
<td>25.0</td>
<td>0.025</td>
</tr>
</tbody>
</table>

6 Decompression sickness – Work under hyperbaric conditions

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintenance and changing of cutters during tunneling under hyperbaric conditions; divers in maritime construction work</td>
<td>1.0</td>
<td>15.0</td>
<td>15.0</td>
<td>0.015</td>
</tr>
</tbody>
</table>

7 Collision with or entrapment by a moving load due to its movement or detachment– Mechanical load-handling

<table>
<thead>
<tr>
<th>Activity Description</th>
<th>s</th>
<th>a</th>
<th>e</th>
<th>f</th>
</tr>
</thead>
</table>

8
<table>
<thead>
<tr>
<th>Description</th>
<th>3.0</th>
<th>21.7</th>
<th>65.0</th>
<th>0.065</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cranes and self-propelled industrial trucks</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Winches</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Placement of floating caissons</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Other means of mechanical load-handling</td>
<td>1.0</td>
<td>20.0</td>
<td>20.0</td>
<td>0.020</td>
</tr>
<tr>
<td><strong>9 Blows to upper and lower limbs – Manual load-handling</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Materials and auxiliary elements</td>
<td>6.0</td>
<td>7.0</td>
<td>42.0</td>
<td>0.042</td>
</tr>
<tr>
<td>Beams</td>
<td>6.0</td>
<td>10.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Installation of reinforcing bars</td>
<td>3.0</td>
<td>7.0</td>
<td>21.0</td>
<td>0.021</td>
</tr>
<tr>
<td><strong>10 Collision with or running over by heavy equipment or heavy goods vehicle</strong></td>
<td>3.0</td>
<td>22.5</td>
<td>67.5</td>
<td>0.068</td>
</tr>
<tr>
<td><strong>11 Cuts, blunt trauma and other injuries due to light equipment – Work with light equipment</strong></td>
<td>6.0</td>
<td>10.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Angle grinder</td>
<td>6.0</td>
<td>11.7</td>
<td>70.0</td>
<td>0.070</td>
</tr>
<tr>
<td>Manual circular saw</td>
<td>6.0</td>
<td>10.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Road cutter</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Rail cutter</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Chainsaw</td>
<td>6.0</td>
<td>10.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Ram or plate compactor</td>
<td>3.0</td>
<td>5.0</td>
<td>15.0</td>
<td>0.015</td>
</tr>
<tr>
<td>Circular or diamond table saw</td>
<td>4.5</td>
<td>15.0</td>
<td>67.5</td>
<td>0.068</td>
</tr>
<tr>
<td><strong>12 Burns – Welding</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>13 Impact injury from falling objects and projectiles – Manual, mechanical or explosive demolition; shot-hole drilling prior to the blasting of a cut slope and subsequent cleanup and field survey</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manual demolition and rubble removal</td>
<td>6.0</td>
<td>20.0</td>
<td>120.0</td>
<td>0.120</td>
</tr>
<tr>
<td>Mechanical demolition and rubble removal</td>
<td>6.0</td>
<td>7.0</td>
<td>42.0</td>
<td>0.042</td>
</tr>
<tr>
<td>Demolition by explosives and rubble removal</td>
<td>6.0</td>
<td>7.0</td>
<td>42.0</td>
<td>0.042</td>
</tr>
<tr>
<td>Shot-hole drilling prior to the blasting of a cut slope and subsequent cleanup and field survey</td>
<td>6.0</td>
<td>20.0</td>
<td>120.0</td>
<td>0.120</td>
</tr>
<tr>
<td><strong>14 Acute dust and toxin poisoning – Manual, mechanical or explosive demolition of structures or buildings in general and of hospitals, factories, slaughterhouses or any other place that may contain toxic substances in particular</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>General demolitions</td>
<td>3.0</td>
<td>5.0</td>
<td>15.0</td>
<td>0.015</td>
</tr>
<tr>
<td>Demolition of hospitals, factories, slaughterhouses or any other place that may contain toxic substances</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td><strong>15 Sufferation or poisoning in confined spaces – Work in confined spaces</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welding in confined spaces</td>
<td>3.0</td>
<td>15.0</td>
<td>45.0</td>
<td>0.045</td>
</tr>
<tr>
<td>Work in sewage systems (suffocation)</td>
<td>2.0</td>
<td>20.0</td>
<td>40.0</td>
<td>0.040</td>
</tr>
<tr>
<td>Work in sewage systems (poisoning due to inhalation or ingestion; infections)</td>
<td>3.0</td>
<td>7.0</td>
<td>21.0</td>
<td>0.021</td>
</tr>
<tr>
<td>Work in chambers, manholes and service galleries for gas, electricity, telecommunication, etc., installations</td>
<td>4.5</td>
<td>15.0</td>
<td>67.5</td>
<td>0.068</td>
</tr>
<tr>
<td>Blasting in confined spaces</td>
<td>3.0</td>
<td>20.0</td>
<td>60.0</td>
<td>0.060</td>
</tr>
<tr>
<td>Maintenance and changing of the cutters on tunnel-boring machines under normobaric conditions</td>
<td>0.5</td>
<td>20.0</td>
<td>10.0</td>
<td>0.010</td>
</tr>
<tr>
<td>Application of solvent-based paint in confined spaces</td>
<td>3.0</td>
<td>7.0</td>
<td>21.0</td>
<td>0.021</td>
</tr>
<tr>
<td><strong>16 Drowning – Work in areas at risk of flooding</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.0 20.0 60.0 0.060

17 Collision with or running over by vehicles unrelated to the construction work–Work in areas with traffic unrelated to the construction work
2.7 21.7 57.8 0.058

18 Traffic accident – Transport of elements to the construction site
Precast pieces
Concrete
Steel (structural and reinforcing bars)

3.0 10.0 30.0 0.030

19 Structural risk or macro risk–Complex operations or structures
1.0 50.0 50.0 0.050

The assessment of the probability of the occurrence of drowning was obtained from accidents occurring in the region of Catalonia and, thus, is valid for that region. For application in other regions, a new assessment should be performed, according to the hydrographic conditions.

The assessment of the probability depends on the type of work or activity. The health and safety expert should assess the probability of the occurrence of the structural accident using Table 2. The probability indicated in the table is for guidance purposes only.

Structural risk or macro risk is the risk of accident due to the failure of an auxiliary element or the structure during construction. It is caused by errors in the design, execution or management of the structure under construction rather than by a lack of health and safety measures. Some examples of structures and operations entailing structural risk are: maneuvers on launched bridges, large cranes moving heavy loads, pile construction with climbing formwork, post-stressed concrete operations, large falsework, self-launching centerings, etc. This risk includes the person-hours spent in workplaces where, were an accident involving the macro risk to occur, it would harm the workers.

3.3 Practical steps to apply the methodology

The ORI can be obtained for the different alternatives for a specific project by means of the following steps. These steps correspond to the ones presented in Ahmed et al. [9]:

1. Establishing the context: Assess whether the coefficients in Table 3 are applicable to the project. If so, Step 3 can be omitted.
2. Identifying risks: For each alternative, identify the activities within the construction project that could potentially involve each risk.
3. Analyzing risks: Local health and safety experts should assign the consequences and, especially, the probability of occurrence of each risk identified in Step 2 according to the region’s degree of technological development and the preventive measures taken.
4. Evaluating risks: For each activity determined in Step 2, ascertain the person-hours required for the project. Apply the ORI formulae [equation (1)] to each alternative.
5. Treating risks: Draw conclusions, prioritize alternatives, redesign, choose the best alternative, etc.

4 EXAMPLE OF APPLICATION

The following simple example shows how the ORI is obtained, how to interpret the results, and how to use them to redesign. The case consists of a decision between in situ and precast
solutions for three drains – OD1, OD2 and OD3 – to be used as small passages for a residential road. Figure 1(a) shows the precast solution and Figure 1(b) shows the in situ solution. The differential aspects of these solutions will be compared from the point of view of occupational safety.

The drains have a total length of 150 m, meaning that 60 precast concrete elements or 12 assembly and dismantling operations for a falsework of more than 12 meters are required. A working day is defined as 8 hours.

A team of 4 people working 10 days is needed to build the precast segments in the factory. A tenth of the time is spent on manual load-handling and a fourth of the time on mechanical load-handling. The workers in the factory do not have to work at heights of over 2 meters. The precast factory has its own concrete plant adjacent to the factory. Eight 2-hour round trips are needed to transport the steel reinforcing bars. Thirty 6-hour round trips are needed to transport the precast voussoirs by road. Three workers (1 crane operator and 2 assemblers) must assemble the 60 precast voussoirs in 6 days.

Completion of the 12 assembly and dismantling operations requires 24 days of work by 3 people plus a crane operator. A third of the 3 workers’ time is spent on scaffolding, and a sixth of their time on the falsework (during the assembly and dismantling). It is assumed that a tenth of their time will be spent on manual load-handling and a fifth on mechanical load-handling. The concrete mixer must make 70 round trips lasting 40 minutes each. Eight 2-hour round trips are needed to transport the steel reinforcing bars.

Table 4 shows the results of applying Table 3 and the equation (1) to these values. The ORI of the precast alternative is lower than the ORI of the in situ alternative, which shows that the manufactured solution is safer than the in situ one. The activities with the highest ORI for the precast alternative are the transport of precast pieces (ORI=16.20) and work in an area at risk of flooding (ORI=8.64). The first risk mainly depends on the distance from the factory to the construction site; hence, a closer precast factory should be found, if possible, in order to reduce the ORI of the alternative or the pieces should be precast on site. The second risk depends on the time spent in the area at risk of flooding. It is thus essential for most of the activities to be performed outside that area, although, in this case, that could be difficult. Another way of improving the safety for this risk would be to set up an efficient early-warning system to decrease the probability of occurrence of the accident. For the in situ
alternative, the highest ORIs were obtained for the risk of drowning and the risk of falls to lower levels.

**Table 4**: Occupational Risk Index for the two alternatives.

<table>
<thead>
<tr>
<th>Risk – Activity</th>
<th>Standardized weight</th>
<th>Precast Exposure (hours)</th>
<th>In situ Exposure (hours)</th>
<th>WxE</th>
<th>WxE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Falls to lower levels – Work at heights or depths of more than 2 meters</td>
<td>0.060</td>
<td>96</td>
<td>5.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centering (during assembly and dismantling only)</td>
<td>0.060</td>
<td>-</td>
<td>-</td>
<td>96</td>
<td>5.76</td>
</tr>
<tr>
<td>Steel tube scaffold</td>
<td>0.098</td>
<td>-</td>
<td>-</td>
<td>192</td>
<td>18.82</td>
</tr>
<tr>
<td>8 Collision with or entrapment by a moving load due to its movement or detachment – Mechanical load-handling</td>
<td>0.065</td>
<td>94</td>
<td>6.24</td>
<td>115.2</td>
<td>7.49</td>
</tr>
<tr>
<td>Crane in the construction site</td>
<td>0.065</td>
<td>80</td>
<td>5.20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Crane in the precast factory⁹</td>
<td>0.065</td>
<td>80</td>
<td>5.20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9 Blows to upper and lower limbs – Manual load-handling</td>
<td>0.042</td>
<td>32</td>
<td>1.34</td>
<td>57.6</td>
<td>2.42</td>
</tr>
<tr>
<td>Materials and auxiliary elements⁹</td>
<td>0.042</td>
<td>32</td>
<td>1.34</td>
<td>57.6</td>
<td>2.42</td>
</tr>
<tr>
<td>16 Drowning – Work in areas at risk of flooding</td>
<td>0.060</td>
<td>144</td>
<td>8.64</td>
<td>768</td>
<td>46.08</td>
</tr>
<tr>
<td>Work in wastewater systems, river beds, streams, etc.</td>
<td>0.060</td>
<td>144</td>
<td>8.64</td>
<td>768</td>
<td>46.08</td>
</tr>
<tr>
<td>18 Traffic accident – Transport of elements to the construction site</td>
<td>0.090</td>
<td>180</td>
<td>16.20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Precast pieces</td>
<td>0.090</td>
<td>180</td>
<td>16.20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.040</td>
<td>46.7</td>
<td>1.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>0.030</td>
<td>16</td>
<td>0.48</td>
<td>16</td>
<td>0.48</td>
</tr>
</tbody>
</table>

⁹The probability of occurrence of an accident in the precast factory could be lower than the probability in Table 3 assigned for construction work. This is because in factories the working conditions are more constant over time and, therefore, controlled than at a construction site, where the activities and conditions are constantly changing. Hence, the ORI value for the precast alternative obtained in the example is an upper bound. As the upper bound of the ORI value for the precast alternative is lower than the ORI value for the *in situ* alternative, the real ORI value for the precast alternative is lower than the ORI value for the *in situ* alternative ($\text{ORI}_{\text{prefab. upper bound}} < \text{ORI}_{\text{in situ}}$).

5 DISCUSSION

5.1 Occupational safety integration at the project level

When the evaluation of project alternatives takes into account the cost of construction, waste generation, energy consumption, the recyclability of materials and others, the occupational safety of each alternative should also be considered.
The alternatives are usually assessed using multi-criteria decision-analysis tools. This involves integrating all aspects affecting a decision into the construction project’s management:

- **Stakeholders:** property owner, constructor, users, the environment, neighbors, etc. This point also includes all points of view: economic, social, functional, environmental and that of future generations.
- **Project components:** materials, cost, time, risk, etc.
- ** Entire life cycle:** from planning to demolition at the end of life.

Numerous examples can be found of how this approach is applied in different fields [5] [12], although it may be less common in the social field, probably due to the false belief that the indicators are subjective or cannot be measured. The approach usually involves considering the different requirements that each alternative must meet, assigning them a relative importance, and calculating the value of each alternative according to the requirements.

Pons and Aguado [13] analyzed the alternatives for school building projects in accordance with three kinds of requirements: economic, environmental and social. The set of social requirements was assigned a relative weight of 20%, the set of environmental requirements a relative weight of 30%, and the set of economic requirements a relative weight of 50%. Within the social requirements, safety was assigned a relative weight of 65%. These weights were assigned in two seminars by experts from different organizations on the basis of current and near-future administrative priorities. The authors then assigned points to obtain the value of the occupational safety indicator for each alternative.

The ORI enables greater objectivity in the measure of the occupational safety of each alternative. Given the set of alternatives and their respective ORIs, the alternative with the lowest ORI will be the most valuable alternative with regard to occupational safety, whereas the alternative with the highest ORI will be the least valuable. Hence, while all construction projects should comply with current occupational safety requirements, those involving greater risk due to the types of activities carried out will be at a disadvantage.

6 CONCLUSION

Both construction industry professionals and academics (teachers and researchers) are sensitive to occupational safety in construction work. Classic project management tools (continuous improvement, alternatives analysis, risk management, communication management, motivation, etc.) apply to occupational safety. However, no evidence of occupational safety criteria taken into account during the design phase of projects was found. To some degree, in the construction industry, occupational safety can thus be said to be a passive rather than active concern.

In this paper, an ORI that depends solely on the occupational activities carried out as part of the construction work is presented. The ORI can be calculated during the design stage and, thus, prior to the award of the contract.

The paper also shows how to integrate the ORI in multi-criteria decision-making methods through the evaluation of alternatives. The use of the ORI as an indicator for evaluating the alternatives in a project enables the consideration of occupational safety from the start of the
design, thereby ensuring that concern for workers’ occupational safety is an active component of both the project’s design and management.

ACKNOWLEDGEMENTS

We gratefully acknowledge the financial support received from the Spanish Ministry of Economy and Competitiveness within the BIA2006-15471-C02-01 project. We would further like to express our gratitude to the Spanish Civil Engineer Association for supporting this research. Finally, we would like to thank the health and safety expert José Hernández Paterna for his invaluable contribution and the experts María Ángeles Horna and Mónica Pérez.

REFERENCES

DEVELOPMENT OF CEMENTIOUS-WOODCHIP COMPOUND PRODUCTS FOR RESILIENCE MEASURES IN DISASTER SITUATION TOWARD SUSTAINABILITY

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Key words: Cementious wood-chips Compound Products (CCP), Great East Japan Earthquake, Disaster wood-chips rubble, Mechanical property

Abstract. Today’s urbanized areas in Japan, a lot of concrete structures have been built. On the other hand, the performance conditions in a part of them have been deteriorating gradually, and the various effects of natural disasters such as big earthquake and the global warming phenomenon in urban and regional areas would be indispensable to be decreased the harmful effects immediately. The various effective resilience measures would be important to be carried out toward the near future.

Overview these conditions, the new concrete production methods to be considered the environmental effects and resilience performances would be indispensable to present in Japan. This study focused on the development of Cementious wood-chips Compound Products (CCP). The original wood-chips materials of CCP were using disaster waste wood materials released by the Great East Japan Earthquake 2011.

In the experiments, we prepared many types of CCP specimens to be applied for structural body and wall masonry unit, and evaluated the relationship between mix proportions of wood-chips and the fundamental properties. These mix proportions are different from content rates of amount of water, particle size of wood-chips and curing conditions, and we conducted these experimental evaluations in detail to the focus on strength of specimens supposing to be applied for structural body.

As these results, it has been found that the mechanical strength was related between the particle sizes of wood-chips and water cement ratio in main, and It would be expected to be increased the potential durability of CCP by optimizing the use ratio of wood-chip compositions and the storing method in structural body so as not to get a wet conditions.
1 INTRODUCTION

The Great East Japan Earthquake has happened at Tohoku area in Japan at 2011. At the site, a problem about a processing method of the rubbles of concrete or wood included in huge quantities of disaster waste was taken up intensely.

After that, in a disaster area in Tohoku area, disaster rubble was utilized aggressively for construction of a breakwater and seawall and land filling, and finally, environment impact by disposing the disaster waste and rubble was seemed to be reduced largely.

With the results of happening The Great East Japan Earthquake at 2011, over 20 million tons of disaster rubble was broken out at the moment, and over 11 million tons of Tsunami rubble was broken out along the sea shore area. Finally, it took the date of total disposal treatment more than 4 years.

On the other hands, at the time of Hanshin-Awaji Great Earthquake at 1995, over 15 million tons of disaster rubble was broken out at the moment, and these huge amount of rubble were applied as landfill of solid waste in a waterfront area or final disposal site. It cost 320 billion Japanese yen during the whole of construction, and the whole date of total construction more than 3 years was needed.

It is now, a Great Earthquake Under Tokyo Area will be predicted to occur in the near future. These are lots of old concrete building and old wooden residences in Tokyo. Therefore, when these buildings broke, it is said that the enormous amount of concrete and waste rubble would occur to some extend not to be compared with The Hanshin-Awaji Great Earthquake at 1995 and The Great East Japan Earthquake at 2011.

Moreover, some reports are noticed that 96 million tones of enormous disaster rubble will break out by occurrence a Great Earthquake Under Tokyo Area, and it's assumed that 6.4 times of enormous damage quantity against Great Hanshin-Awaji Earthquake will happen. We would have a responsibility and an important task to excuse important role to decrease the huge amount of disaster rubble as a some types of recycled resource toward the future.

This study focused on the development of Cementious wood-chips Compound Products (CCP). In the experiment, the original wood-chips materials of CCP were using disaster waste wood materials released by the Great East Japan Earthquake 2011 (Figure 1). We prepared many types of CCP specimens applied for structural body and wall masonry unit and evaluated the relationship between mix proportions of wood-chips and fundamental properties. Finally, the objectives of study are to be clear the relationship between mechanical strength and the properties of wood-chips and water cement ratio, and to develop the potential utilization of CCP by optimizing the use ratio of wood-chip compositions.

![Figure 1: The Scene of Great East Japan Earthquake (2011 Miyagi～2012 Tokyo)](image-url)
2 DEVELOPMENT OF CEMENTIOUS-WOODCHIP COMPOUND PRODUCT (CCP)

2.1 Materials of Experiments

Used materials of experiment is indicated in table 1. Various wood-chips were introduced to produce Cementious wood-chips Compound Products (CCP) in the experiment. A small chips of larch tree (KS: particle size under 3mm) and a small chips of Japanese red pine tree (AS: particle size under 3mm) have been used in a market so as to produce a woody board for using general foundation board of residences at present. KS and AS are the comparative series against Recycled disaster wood-chips materials.

The 3 kinds of Recycled disaster wood-chips were generated by using waste rubble of Eastern Japan Great Earthquake at 2011, and these were transported to the close area from Tohoku region and produced at a recycling wood factory in Tokyo area. The average size of recycled disaster wood-chips particle were divided into 3 levels among as small(RS: particle size under 3mm), middle(RM: particle size under 12mm) and Large(RL: particle size under 40mm).

2.2 Experiment Factors and Levels

An experimental factors and levels are indicated in table 2, and the experimental items and methods are indicated in table 3.

There is wood-chips masonry unit used as a product actually in the foreign area which does not occur an earthquake in main. The basic physical properties of this masonry unit were investigated a bulk density, a particle size distribution and compressive strength beforehand to research the properties of Cementious wood-chips Compound Products (CCP).

<table>
<thead>
<tr>
<th>Materials</th>
<th>Factors</th>
<th>Size of Chips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood-chips masonry unit</td>
<td>Residence wall type (W600xH300xD300mm)</td>
<td>Under 25mm</td>
</tr>
<tr>
<td>Wood chips (P)</td>
<td>RS</td>
<td>Recycled Disaster Waste Chips (S)</td>
</tr>
<tr>
<td></td>
<td>RM</td>
<td>Recycled Disaster Waste Chips (Middle Strand)</td>
</tr>
<tr>
<td></td>
<td>RL</td>
<td>Recycled Disaster Waste Chips (Large Strand)</td>
</tr>
<tr>
<td></td>
<td>KS</td>
<td>Larch Chips (Small Particle)</td>
</tr>
<tr>
<td></td>
<td>AS</td>
<td>Red Pine Chips (Small Particle)</td>
</tr>
<tr>
<td>Cement</td>
<td>C</td>
<td>Normal Portland Cement (Density: 3.16g/cm³)</td>
</tr>
<tr>
<td>AE agent</td>
<td>Ad</td>
<td>Lignin sulfonic acid compound and Polyol complex (Addition amount: C×2.5～5.0 weight%)</td>
</tr>
<tr>
<td>Thickener</td>
<td>M</td>
<td>Additives of Methyl cellulose Ether (Addition amount: C×0.5～2.0 weight%)</td>
</tr>
</tbody>
</table>

Table 1 Materials of Experiments

Image of Wood Chips Materials

| RS (≈3mm) | RM (≈12mm) | RL (≈40mm) | KS (≈3mm) | AS (≈3mm) |
CCP was produced by 3 levels of W/C ratio, 5 kinds of wood-chips and 3 levels of P/C ratio. Fundamental properties of CCP is investigated by density and void volume in a specimens. Finally, the compressive test conditions of CCP was controlled by changing environment conditions of normal type(N), wet type(W) and dry type(D) among 24 hours after curing 4 weeks packing-aired conditions.

Table 2. Experiment Factors and Levels

<table>
<thead>
<tr>
<th>Materials</th>
<th>Factor</th>
<th>Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood-chips masonry unit</td>
<td>Bending Strength</td>
<td>Placing level: High(H), Middle (M), Low(L)</td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>Curing conditions: Normal(N), Wet(W), Dry (D)</td>
</tr>
<tr>
<td>CCP</td>
<td>W/C (%)</td>
<td>40, 50, 60</td>
</tr>
<tr>
<td></td>
<td>Wood-chips(P)</td>
<td>RS, RM, RL, KS, AS</td>
</tr>
<tr>
<td></td>
<td>P/C (%)</td>
<td>10, 15, 20</td>
</tr>
<tr>
<td></td>
<td>Admixture(Ad)</td>
<td>C × 0.25%</td>
</tr>
<tr>
<td></td>
<td>Methyl cellulose Ether(M)</td>
<td>W × 0.1%</td>
</tr>
<tr>
<td></td>
<td>Environment Conditions</td>
<td>Normal (N), Wet(W), Dry (D)</td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>Normal (N), Wet(W), Dry (D)</td>
</tr>
</tbody>
</table>

![Diagram](https://example.com/diagram.png)

Table 3. Test Items and Methods

<table>
<thead>
<tr>
<th>Materials</th>
<th>Items</th>
<th>Methods and Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood-chips masonry unit</td>
<td>Particle Size Distribution</td>
<td>Extract wood-chips and dissolve cement paste, and scale the average length of particles</td>
</tr>
<tr>
<td></td>
<td>Bulk Density</td>
<td>Measure a weight of Wood-chip Block specimen (size40 × 40 × 160mm)</td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>JIS A 1108, (size φ 150 × 300mm)</td>
</tr>
<tr>
<td></td>
<td>Bending Strength</td>
<td>JIS R 5201, (size 40 × 40 × 160mm)</td>
</tr>
<tr>
<td></td>
<td>Void Volume</td>
<td>Referenced the void volume of porous concrete (size 40 × 40 × 70mm)</td>
</tr>
<tr>
<td>Chips</td>
<td>Particle Size Distribution</td>
<td>Extract wood-chips and dissolve cement paste, and scale the average length of particles</td>
</tr>
<tr>
<td></td>
<td>Density</td>
<td>JIS A 1109, Surface Drying Density and Drying Density</td>
</tr>
<tr>
<td></td>
<td>Absorption</td>
<td>JIS A 1109</td>
</tr>
<tr>
<td>CCP Specimens</td>
<td>Density</td>
<td>Environment conditions (N),(W),(D) (size φ 100 × 200mm)</td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>JIS A 1108, JIS A 1149 (size φ 100 × 200mm)</td>
</tr>
<tr>
<td></td>
<td>Elastic Modulus</td>
<td>By changing Environment conditions(N),(W),(D)</td>
</tr>
<tr>
<td></td>
<td>Void Volume</td>
<td>Referenced the void volume of porous concrete (size φ 50 × 100mm)</td>
</tr>
</tbody>
</table>
2.3 Production of Cementious wood-chips Compound Products (CCP)

Mixture proportion of Cementious wood-chips Compound Products (CCP) is indicated in table 4 and examples of CCP is indicated in figure 2.

Wood-chips masonry unit have used as building wall products actually for many years in foreign countries which does not occur earthquake issue. The basical properties of this masonry unit were controlled by purduct makers and the specifications. These product types are many varieties from the standpoint of size, thickness, strength, inserting thermal insulating material and so on. The masonry unit of the study is one of the typycal types and the weight of masonry unit is light, and so the compressive strength is set almost 2-4 N/mm².

CCP was produced by 3 levels of W/C ratio(40%,50%,60%), 5 kinds of wood-chips and 3 levels of P/C ratio(10%,20%,30%). Fresh concrete conditions of CCP was controlled by using unitary quantities of admixture and methyl cellulose ether. Finally, the compressive test conditions of CCP was controlled by normal type(N), wet type(W) and dry type(D) among 24 hours packing-aired curing conditions.

<table>
<thead>
<tr>
<th>Mark</th>
<th>W/C(%)</th>
<th>P/C(%)</th>
<th>Cement(C)</th>
<th>Water(W)</th>
<th>Wood Particle(P)</th>
<th>Ad</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>* 40</td>
<td>40</td>
<td>10</td>
<td>1274</td>
<td>510</td>
<td>127</td>
<td>C × 0.25%</td>
<td>W × 0.1%</td>
</tr>
<tr>
<td>* 50</td>
<td>50</td>
<td>15</td>
<td>1146</td>
<td>573</td>
<td>172</td>
<td></td>
<td></td>
</tr>
<tr>
<td>* 60</td>
<td>60</td>
<td>20</td>
<td>1019</td>
<td>611</td>
<td>204</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note) Mark * is normal type(N), wet type(W) and dry type(D) based on curing conditions.

2.4 Fundamental Properties of Wood-chips masonry unit

Fundamental Properties of Wood-chips masonry unit is indicated in figure 3. As for a) relationship between bulk density and void volume, the lower placing concrete in the masonry unit from H to L, the bulk density tends to increase proportionally and the void volume tends to decrease.

As for b) relationship between density and weight loss, these parameter on Drying series after getting wet was almost equal to Normal series. The water effects from taking rain would
be small in the experiment. Although as to be seen c) relationship between curing methods and mechanical properties, compressive strength and elastic modulus of Drying series after getting wet was almost equal to wet series. The water effects from taking rain would be harmful. As for d) relationship between height levels of specimen and bending strength, the lower placing concrete in the masonry unit, the higher the bending strength because of increasing the density of specimens.

Figure 3 Fundamental Properties of Wood-chips masonry unit

2.5 Fundamental Properties of Cementious wood-chips Compound Products (CCP)

Fundamental Properties of CCP is indicated in figure 4.

As for a) relationship between some particles and void volume, the bigger the particle size from RS to RL, the void volume tends to increase proportionally. In addition to the result, as for c) relationship between some particles and drying density, the bigger the particle size from RS to RL, the drying density tends to increase proportionally, and the absorption tends to decrease inversely. Although the RS series would be utilized as a raw material of CCP so that these performances of RS are almost equal to the CCP using AS and KS.

As for b) relationship between compressive strength and elastic modulus, The compressive strength of CCP was largely strengthened to 5-30N/mm² and the Elastic modulus of CCP was same tendency to be compared the wood-chips masonry unit. As already mentioned forward, the wood-chips masonry unit has a problem regarding mechanical properties mainly, so it is
important to be improved these properties by producing CCP, and the mechanical property of CCP could be estimated by the structural design code of AIJ (Architectural Institute of Japan), such as illustrated under curve line of Autoclaved lightweight concrete ($\gamma = 0.5 \text{ g/cm}^3$).

![Void Volume of Wood-tip Concrete](image)

![Compressive Strength and Elastic Modulus](image)

![Drying Density of Wood-tip aggregate](image)

![Absorption of Wood-tip aggregate](image)

**Figure 4** Fundamental Properties of CCP

### 2.6 Effects of Wet and Dry conditions of Cementious wood-chips Compound Products (CCP)

The Effects of Wet(W) and Dry(D) conditions of CCP are indicated in figure 6.

In general, Free water included in materials would be tendency to decrease several performances such as strength, deformation ability, durability and so on. The wood-chips of CCP may be also same, so the effects toward a) Density and Weight Loss Ratio and b) Compressive Strength and Elastic Modulus were observed.

As for the density and weight Loss Ratio in figure a), the density of Wet(W) series was naturally increasing than that of Normal(N) series, and after drying arrangement, it was decreasing to large extend than that of Normal (N) series. Particularly, These harmful effects against properties of CCP were seemed to be remarkable at using recycled disaster wood-chips rubble as RL. As for the compressive strength and elastic modulus in figure b), the compressive strength of Wet(W) series was almost decreasing than that of Normal (N) series, and after drying adjustment, though the strength did not recover to the extend that of
Normal(N) series. In addition, the tendency of performance decreasing of CCP was seemed to be remarkable at Elastic modulus.

<table>
<thead>
<tr>
<th>Density (g/cm³)</th>
<th>Weight Loss Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40, 50, 60</td>
<td>0.80, 1.00, 1.20, 1.40, 1.60, 1.80, 2.00</td>
</tr>
</tbody>
</table>

Figure 5 The Effects of Wetting and Drying of CCP

3. CONCLUSIONS
- This study focused on the development of Cementious wood-chips Compound Products (CCP) and the original wood-chips materials of CCP were using disaster waste wood materials released by the Great East Japan Earthquake 2011
- Wood-chips masonry unit is different the density from lower and higher part of the unit, and the mechanical properties could not expect to develop high performance.
- Mechanical strength of CCP was related between the particle sizes of wood-chips, water cement ratio and environmental conditions of wet and dry, and it is expected to be improved the potential performances by optimizing these mixture proportions so as not to keep a wet conditions.

REFERENCES
DOING MORE WITH LESS: TOPOLOGY OPTIMIZATION AS A MEANS FOR THE DESIGN OF SUSTAINABLE CONCRETE FORMS

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Key words: Architectural Design, Computational Methods, Topology Optimization

Abstract. Concrete is the single most widely used man-made material in the world. As such, it also carries the reputation of being the largest contributor to the rise in carbon dioxide (CO2) and greenhouse gas emissions. The negative reputation that concrete carries is less a matter of it being inherently a ‘bad material’ and has more to do with the sheer abundance of its use, making it an easy target for criticism. It is up to the designers of the built environment, architects and engineers, to seek more sustainable means of designing with these materials. What is needed is a design methodology that strives to do more with less.

This paper presents research exploring the potential of topology optimization as a means for the design of sustainable concrete structures. Topology optimization is a mathematical, gradient-based design procedure that can be used to determine the distribution of required material within a design domain based upon defined loads and boundary conditions while meeting a prescribed target objective, such as minimizing deflection. The concept of topology optimization has been utilized by the automotive and aerospace industry for almost thirty years now, since its early development, as problems associated with solutions meant to satisfy maximum stiffness with minimum weight are of the utmost importance.

The recent saturation of digital design tools and techniques within the architecture industry have led more and more architects to seek computationally driven, data-centric, methodologies to assist in the design process. Topology optimization is well suited as a methodology to assist in the development of form that is rooted in sound structural logic while striving for material efficiency. This paper presents case studies of recent architectural projects that utilize topology optimization for projects ranging from long-span roof structures to high-rise buildings. While not all the projects exclusively utilize reinforced concrete, these case studies are examined more for their potential in defining a methodology and workflow that seek material efficient forms through the use of topology optimization. In addition to these professional design case studies, case studies of student work from a course recently taught at the University of Oregon focuses more specifically on the issues associated with the design and production of concrete structures and highlights the opportunities and challenges of advanced manufacturing techniques as a means of realizing more sustainable concrete forms through the use of topology optimization.
1 INTRODUCTION

Concrete is the single most widely used man-made material in the world. As such, it also carries the reputation of being the largest contributor to the rise in carbon dioxide (CO₂) and greenhouse gas emissions. The negative reputation that concrete carries is less a matter of it being inherently a ‘bad material’ and has more to do with the sheer abundance of its use, making it an easy target for criticism [1]. No material is perfect. For instance, in order to achieve an equivalent performing structure in steel, one would consume upwards of twice the volume of energy, as well as cause more severe impacts on water and air pollution than to produce its concrete equivalent [2]. There are clearly no simple answers in the search for more sustainable material usage within the building industry. The strides currently being made by chemists, material scientists and manufacturers to achieve more sustainable means of production and manufacturing of their respective materials is certainly a step in the right direction, but this alone will not solve all of the problems. It is up to the designers of the built environment, architects and engineers, to also seek more sustainable means of designing with these materials. What is needed is a design methodology that strives to do more with less.

2 TOPOLOGY OPTIMIZATION

2.1 Where to place the holes

The engineer and educator Robert Le Ricolais known as the ‘father of Spatial Structures” for his exploration of long span minimal weight structures is known for his adage that “the art of structure is how and where to place the holes” [3]. This statement, as simple and elegant as the structures that Le Ricolais designed, speaks volumes to the struggle of the relationship that exists between architecture and engineering design criteria. LeRicolais, like others before and since, believed the inspiration for material efficient structural design lay in examples found in nature. His research and teaching inspired creative reinterpretations of lessons learned by nature. His research and teaching inspired creative reinterpretations of lessons learned by nature.

2.2 Background

Topology optimization is a mathematical, gradient-based design procedure that can be used to determine the distribution of required material within a design domain based upon defined loads and boundary conditions while meeting a prescribed target objective, such as maximizing stiffness or minimizing mass. Topology optimization differs from other optimization techniques, such as shape optimization, as shape optimization methods typically operate on a fixed, predefined topology [4]. Therefore, topology optimization is used to generate the initial form concepts and shape optimization can be used to fine-tune a chosen design topology (see Fig. 1). The concept of topology optimization has been utilized by the automotive and aerospace industry for almost thirty years now, since its early development, as problems associated with solutions meant to satisfy maximum stiffness with minimum weight are of the utmost importance. The process of topology optimization is sometimes referred to by various names such as the homogenization method, computational morphogenesis, and evolutionary structural optimization (ESO). There have also been and continues to be various developments and refinements of the basic method. The focus of this paper is less about the nuances of the variations of different topology optimizations methods and rather more
generally focused on an examination of its potential applicability to the design and production of material efficient structural components and systems.

![Figure 1: Three categories of structural optimization of a truss structure: (top) size optimization; (middle) shape optimization; (bottom) topology optimization. Initial design problems are shown on the left and optimal solutions are shown on the right. [4]](image)

### 2.3 New structuralism

The advancements in digital design tools, technologies and techniques have resulted in a paradigm shift in the design and production of the built environment. The relationship between architects and engineers has evolved in light of these technological changes resulting in a cultural shift in practice that has been termed ‘new structuralism’ by Rivka and Robert Oxman. The new structuralism designates the cultural turn away from formalism and towards a more sustainable model of material practice open to ecological potential. This new model favors an approach to architectural design that is motivated by a priori structural and material concepts and in which structuring is the generative basis of design. This new mode of practice challenges the traditional sequential design process, which can generally be characterized as the development of ‘form, structure and material’. A formal concept is first conceived by the architect and subsequently structured and materialized in collaboration with the engineer. This collaborative potential, with its roots in historical precedent, has been strengthened by the collaborative potential of advances in digital design technologies. This new model effectively reversed the traditional process to become one that follows the sequence of ‘material, structure, form’ [5].

### 3 CASE STUDIES IN ARCHITECTURE

#### 3.1 Akutagawa west side project

One of the first built examples utilizing topology optimization in the design and construction of a building structure is the Akutagawa West Side office building designed and
built by f-tai architects and structural engineer Hiroshi Ohmori in 2005. Ohmori utilized a variation on the basic topology optimization methodology that he developed through research done by himself and structural engineer Mutsuro Sasaki [6]. The extended evolutionary structural optimization (ESO) was utilized primarily in the design of braced frames on the south and west facades of the corner site located near the Takatsuki station in the Osaka Prefecture of Japan. These braced frames where optimized to resist the vertical load of self-weight as well as for horizontal seismic force at each floor level, see Figure 2. The frames are made of reinforced concrete and required fairly complex formwork to build. While the material efficiency achieved through the use of topology optimization can partially justify the increased complexity of the construction, it is reasonable to assume that this would have been a limiting factor on a larger scaled building.

![Figure 2: (left) Topology optimization iteration steps for the Akutagawa West Side office project; (right) view of the west facade. [6]](image)

3.2 Qatar national convention center

Another built example utilizing the extended ESO method developed by Ohmori and Sasaki is the entry roof structure for the Qatar National Convention Center (QNCC) in Doha. This project was a collaboration between architect Arata Isozaki and Sasaki. The design for the QNCC was a further development and refinement utilizing the extended ESO method based on previous collaborations between Isozaki and Sasaki for other, unsuccessful, competition entries including a design for the TAV Station in Florence, Italy [7]. The theoretical design solution developed by Isozaki and Sasaki that won the competition for the QNCC, a 36m wide x 150m long roof with support points at each end, presented some challenges with regards to its materialization and constructability. The engineering consultancy, Buro Happold, was brought on board to assist in further analyzing and optimizing the design solution for additional parameters such as rational geometry and constructability. Buro Happold’s in-house team of specialist analysts, SMART, further refined the geometry so as to allow for a system of a simplified octagonal cross section structural steel core concealed behind flat steel sheets which approximated the fluid branching support structure [8]. In effect, Buro Happold had to go to take what was originally conceived as an optimized continuous structure and simplify into an internal steel skeleton clad with non-load-bearing
steel plate that imitates the original optimization output. This means that it lost most of its initial structural logic in the process of realization, although it still formally appears “optimized”. The challenges involved in the realization of the QNCC project made it clear that structures designed utilizing this new methodology of topology optimization warranted advancements in construction processes to fully leverage the potential material efficiencies.

3.3 Unikabeton prototype pavilion

The Unikabeton (unique concrete structure) prototype pavilion is the result of a collaborative research project at Denmark’s Aarhus School of Architecture. Led by Per Dombernowsky and Asbjorn Sondergaard, Unikabeton, was developed and realized as a cross-institutional research project that set out to explore the architectural and industrial potential of linking topology optimization of concrete structures with the robotic fabrication of polystyrene formwork [9].

The research project began with initial experiments investigating the optimization of simple commonly used structural elements such as simple beams, as well as one-way and two-way slab elements. These studies allowed the team to experiment the workflow process from initial analysis and design through fabrication. For the topology optimization process the team decided to utilize software that was proven successful in the automotive and aerospace industry, but not readily used in the building industry. The teams utilized Altair’s HyperWorks suite that, in addition to general finite-element analysis, contains OptiStruct, a built-in optimization tool. OptiStruct has a proven track record in the automotive and aerospace industries, where topology optimization procedures are more common, and this gave the team confidence in the soundness of the tool. In addition to calibrating the design software, these initial small-scale topology optimization studies allowed the team to experiment with the computer numerically controlled (CNC) robotic milling of the polystyrene foam formwork. These initial design experiments revealed that not only do these optimization procedures result in highly complex structural morphologies, which were fully unpredictable due to the complexity of the calculations, but that the optimized solutions led to a reduction in material consumption between 60%-70% when compared to more conventional solutions while still satisfying comparable performance demands [10].

After these initial experiments, involving discrete structural elements, the research team set out to optimize a non-uniformly doubly curved concrete slab supported by three asymmetrically placed concrete columns. The resulting prototype structure measures 12 x 6 x 3.3 meters in size (see Figure 3). The topology optimization was carried out while also considering the fabrication requirements of the CNC milling process for the structure’s polystyrene formwork. The forms were milled from polystyrene block utilizing a large-scale CNC-milling robot at the High Technology Concrete Workshop at the Danish Institute of Technology. Traditional scaffolding and assembly techniques were utilized to support the foam formwork while reinforcement was placed. Finally, self-compacting concrete was cast into the formwork [11].

The successful optimization and fabrication of the Unikabeton prototype pavilion demonstrates the significant value of simulation-generated topology optimizations of architectural structures. The design process offered the opportunity to mimic the morphology of biological structural systems, similar to those explored by Robert Le Ricolais and others.
The new forms made possible by this technique, realized with the help of digital fabrication techniques, help blend architectural and structural considerations that result in solutions that maximize structural performance while minimizing material mass. The subtractions of superfluous material from the design helps to significantly reduce the environmental costs related to the energy saved in concrete production and transportation. Beyond just the environmental benefits, the design and fabrication processes explored in the Unikabeton project lend themselves to a large potential shift in the creation of architecture. These optimization and fabrication processes enable structures to be considered as a continuous whole, thereby eliminating the conventional classifications of systems elements such as beams, slabs and columns.

Figure 3: The doubly-curved, non-uniform, topologically optimized Unikabeton prototype pavilion. [12]

4 PERFORMATIVE PROTOTYPING

The success of the Unikabeton project inspired the author to teach an advanced technical elective course at the University of Oregon’s School of Architecture and Allied Art that built upon the research developed from the Denmark research team. What follows is an outline of the structure of that course, how topology optimization was incorporated into the development of reinforced concrete structures and discussion on the outcomes of the design research conducted over the course of the ten-week term.

4.1 Course structure

In the spring term of 2015, a design research course was offered at the University of Oregon that explored the potential of digital design and fabrication tools and techniques in the realization of more sustainable concrete structures. The course consisted of seventeen students of architecture, with a mix of both graduate and undergraduate students. The course satisfied an advanced technical elective requirement, meaning all students had to have satisfied the
prerequisite core coursework in Materials and Methods of Construction as well as Structural Behavior and Structural Design. This meant that the students had been exposed to any and all core coursework focused on reinforced concrete in the School of Architecture curriculum.

In order to consolidate and document the knowledge of current concrete technology, the course began with a two-week intensive investigation into themes focused around various aspects of reinforced concrete design and fabrication. This included themes such as: history, development and key figures in design; materials, composition and properties; fundamentals of engineering and structural behavior; production, fabrication and workmanship; and advances and innovations. Students worked in teams of three and four researching and documenting information associated with their assigned topic. Findings where presented to the entire class and compiled into a reference manual that was utilized throughout the remainder of the course. The document served as a living document, meaning that as some new information was discovered relevant to the design research, it was added to the reference.

Complex reinforced concrete forms based upon a structural logic are by no means a novel concept. Students, again working in small groups, where asked to examine the work of Pier Luigi Nervi and analyze a column from one of his various projects. Columns where analyzed for how the form responded to structural requirements. Students were also asked to research how their particular column was constructed and asked to recreate that process in the fabrication of a scale model. Plaster was used in lieu of concrete. This research and analysis was meant to start to expose students to the process of linking architecture, structure and construction considerations into the design of compelling concrete forms.

Following these two warm-up exercises and initial research, a field trip was taken to a local precast manufacturing plant in order to fully understand the current state of the industry when moving forward with the proposed performative prototypes. It was following this field trip that students were introduced to the concept of topology optimization and set out exploring its use in the design and construction of material efficient concrete structures.

4.2 Tools for topology optimization

When considering what software to utilize for this course several criteria had to be considered. Ease of use, transparency of functionality, cost, and integration with existing architectural design software were just a few of the things considered when evaluating topology optimization software. Three potential software options were explored Altair’s OptiStruct, solidThinking’s Inspire and the Millipede plug-in for Grasshopper/Rhinoceros 3D.

The use of Altair’s OptiStruct, the software used by the Unikabeton team, was quickly ruled out because of its high cost as well as its steep learning curve. The author was trained in the use of OptiStruct while a practicing structural engineer at Skidmore, Owings and Merrill in Chicago, one of the first firms in the world to utilize the software for the building industry. While certainly the most powerful of the options explored, it was feared that students would spend more time trying to understand how to use the software rather than how to apply it to the design and fabrication of the prototypes.

SolidThinking was seriously considered as an option for use in the course. Originally developed as general modeling software, solidThinking was acquired by Altair, the makers of OptiStruct in 2008. Shortly after this acquisition, Altair introduced the OptiStruct optimization solver into the functionality of solidThinking’s user-friendly modeling
environment. Marketing the functionality to product designers and architects as morphogenesis design, Altair was able to make the complex engineering technique of topology optimization more approachable to designers. After undergoing several improvements and release cycles the software is now marketed as solidThinking Inspire. Inspire was an appealing option because of the intuitive modeling interface, similar to other 3D modeling software used by architects. The process of the topology optimization is simplified and made rather intuitive as well, while still harnessing the powerful OptiStruct optimization engine behind it. The final decision to not use solidThinking Inspire was base only on the desire to avoid subjecting students to yet another piece of software.

The decision was made that students would utilize the Millipede plugin for Grasshopper, the visual programing plugin for Robert McNeel and Associate’s Rhinoceros 3D modeling software. Millipede is a structural analysis and optimization component for Grasshopper and is developed by Panagiotis Michalatos, an Assistant Professor at Harvard’s Graduate School of Design. Michalatos previously worked for the London based structural consultancy, Adams, Kara, Taylor (now AKTII). While at AKTII, Michalatos developed TopoOpt, a standalone topology optimization tool meant to be a common sandbox where architects and engineers could explore the use of topology optimization concepts in the design of architectural projects [13]. Since then, Michalatos has incorporated the TopoOpt functionality into the Millipede component for Grasshopper. Millipede allows for very fast linear elastic analysis of frame and shell elements in 3D, as well as 3D volumetric elements. All systems can be optimized using built in topology optimization methods and have their results extracted and visualized in a variety of ways. Rhinoceros 3D has become one of the most popular modeling tools for conceptual design and is used extensively in the architecture curriculum, including in the required Digital Media courses that all students must take. Because of the ubiquitous nature of Rhinoceros 3D and the students existing familiarity with the software, the Millipede plugin seemed like the most obvious choice moving forward. What it lacks with regard to some of the analytical rigor of the other two options, it makes up for with it approachable nature in an environment already familiar to the students.

4.3 Digital prototyping project

As a means to explore the potential of topology optimization in a systematic way as well as test the functionality of the Millipede, students were asked to experiment with the optimization of a series of simple spanning elements. The cases that were investigated included cases of simply supported beams with distributed and point loads, continuous two-span beams with distributed and point loads and cantilevered beams with distributed loads and a single point load. In addition to beam elements explored via 2D and 3D topology optimization studies, students also investigated cases of one-way and two-way slab elements with a multiplicity of support and load conditions. Students carefully documented their process, boundary conditions and loading patterns. The optimization results from Millipede were further refined in Rhinoceros 3D and several examples were 3D printed in order to develop physical representations of some of the more successful design solutions. While not representative of the actual final design solution from a material standpoint, the small-scale 3D printed models helped students to understand the spatial and architectural possibilities of the topology optimization process.
4.4 Physical prototyping project

The digital prototyping project exposed students to the general functionality of the Millipede and the power of topology optimization as a design tool. Building upon this experience students were asked to use these studies as a jumping off point for the development of a large-scale physical prototype to be designed and developed over several design and fabrication prototyping cycles. Working in groups of 3 to 4, students presented an initial design proposal that incorporated one of their solutions from the ‘digital prototyping’ exercise reimagined as some architectural context. These large-scale prototypes were to be refined, developed and realized in reinforce concrete (see Figure 4).

![Figure 4: Final prototype proposal examining a cantilevered beam condition.](image)

In anticipation of the need to fabricate the complex formwork for the structures they were designing, students were introduced to the CNC router in the School’s furniture workshop, most for the first time. This exposure allowed the students to understand the fabrication constraints and to be able to take these into consideration in the development of their prototypes. Most groups used polystyrene foam as the material from which to create their formwork. One group, hoping to be able to reuse their formwork opted to try to utilize medium density fiberboard (MDF), which ultimately proved unsuccessful.

The initial design prototypes were developed and realized in a quick week and a half process. This push to try to realize a built reinforced concrete prototype of their design in a short period of time resulted in, what many students believed to be, failures. These perceived failures, in fact served as a great opportunity to bring the groups, which had been working in somewhat isolation for since the development of their initial design, together and report back the various lessons learned from their experience to the entire class. The feedback from this initial prototype fed into the redesign and development of another prototype developed over the next two weeks leading to the final presentation of the course (see Figure 5).
Figure 5: Images showing the various stages of one group’s final prototyping process.

The intention of the final prototyping project was to provide for at least three design and fabrication iterations, coupled with structural load testing. Unfortunately, the learning curve associated with the topology optimization process as well as the difficulties with successfully creating forms and casting the prototypes, only allowed enough time for two iteration cycles. In addition to limited iteration cycles, the lack of a School of Engineering on campus severely limited the potential for rigorous load testing. Future iterations of this design research course plan to partner with the College of Engineering at Oregon State University, 45 miles north of Eugene, where the University of Oregon is located. This partnership will allow architecture and engineering students to collaborate on design and fabrication considerations as well as afford the architecture student access to their structural engineering lab facilities.
4.5 Feedback

Feedback on the course was fairly positive from both outside reviewers that attended the presentations of the final prototypes developed by the various design teams. Based upon course evaluations at the completion of the course, students voiced enthusiasm and excitement about the exposure to topology optimization and the opportunity for hands on prototyping with reinforced concrete. Several students expressed a desire to explore the process further in future design problems as well as explore these concepts with the other materials.

One of the clearest and most consistent criticisms had to do with student’s access and availability to advanced manufacturing technologies. While students were able to utilize the 2.5D CNC mill, it was clear that access to robotically controlled CNC mills would have increased design potentials. An additional criticism had to do with a lack of adequate workshop space in which to cast concrete. The workshop facilities students have access to are focused on woodworking and have strict rules that limit work like casting concrete. It was also clear from student feedback that at least one more prototype iteration would have resulted in a more successful final design solution.

5 CONCLUSIONS

- Topology optimization is a useful design methodology for conceptual design that brings architects and engineers together as collaborators for the design of material efficient forms.
- More research is needed to more fully understand the use of topology optimization and is applicability to the complex behavior of composite materials such as reinforced concrete.
- Further research is needed in the application of advanced manufacturing techniques in order to fully realize the continuous nature of topologically optimized forms when met with the limitations of conventional construction technology.
- The exposure of students of architecture and engineering to the concepts of topology optimization will help ensure its future integration into the building industry.

REFERENCES

DURABILITY BEHAVIOUR OF SUSTAINABLE CEMENTS EXPOSED UNDER REAL ENVIRONMENTAL CONDITIONS OF THE MEDITERRANEAN AREA

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Key words: mineral additions, microstructure, durability, corrosion

Abstract.

The use of mineral admixtures as an addition to cement is well known to reduce the pollution produced by the cement industry, making the construction industry more sustainable. It is also well known that cements including this mineral additions usually improve both the mechanical and the durability behavior of the structures built with this type of materials. However, most of the test are carried out in laboratory, either under optimal or non-optimal, but controlled, conditions. In this work cements containing ground granulated blastfurnace slag, and fly ash have been exposed from the first week to two different real environments in the Mediterranean area. This two locations represent, according to the Spanish standard for structural concrete, the most risky conditions for the durability of reinforced concrete structures: one with possibility of chloride attack, while the second one has a possibility of carbon dioxide attack. Under these conditions, the microstructure has been studied using mercury intrusion porosimetry, and some durability indicators, such as capillary suction coefficient, chloride diffusion coefficients, carbonation rates, and corrosion potential and corrosion rate. The results show that under the studied conditions the studied cements give durability properties at least as good as the ordinary Portland cement does, in real conditions. This result is important because it guarantees the durability of structures built with more sustainable materials.

1 INTRODUCTION

The Spanish standard for structural concrete EHE-08 [1] intends to promote the use of sustainable cement, including cements with active additions, as well as included an article to specifically guarantee the durability of the structures. This guarantee of the durability is specially focused on the risk of corrosion of reinforced structures.

It is well known that the cements containing fly ash or slag improve the durability of the materials [2-5]. But most of these studies are carried out in optimal laboratory conditions, and real structures are not built under those conditions. Some efforts have been done to study the influence of the environment on the properties of cements containing mineral additions [6], [7], most of them on mortars. Since the results are promising, in this paper the compatibility of durability with using sustainable cements has been studied in concrete samples that have been hardened in real locations of our environment. The locations have been selected to be representative of the most risky situations according to the EHE-08 [1],
one close to the seaside, where chlorides could come and ingress in the concrete structure, forcing pitting corrosion, and the second one in a location inside the University of Alicante, very close to a crowded highway, and close to a big roundabout, where traffic jams occur very often, being susceptible of carbonation and generalized corrosion of the reinforcements. Not only the durability properties of the bulk concrete have been studied in these two locations, but also the capacity of corrosion prevention have also been tested by electrochemical measurements. Results have been compared to samples kept in the humidity chamber of the laboratory.

2 EXPERIMENTAL SETUP

2.1 Materials

Three different cement types were used. An ordinary portland cement, with 95% of clinker, CEM I 42.5 R according to the Spanish standard [8], a cement with a high percentage of GGBFS (66-80%), CEM III-B 42.5 R according to the same standard, and finally a pozzolanic cement containing among 36-55% of pozzolans, being the main one the fly ash, CEM IV B(V) 32.5 N according to the Spanish standard. Concrete samples were prepared using the dosage shown in table 1.

<table>
<thead>
<tr>
<th>Element</th>
<th>Kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>350</td>
</tr>
<tr>
<td>Coarse aggregate 4-6</td>
<td>489.5</td>
</tr>
<tr>
<td>Coarse aggregate 6-12</td>
<td>714</td>
</tr>
<tr>
<td>Sand</td>
<td>662.75</td>
</tr>
<tr>
<td>Water</td>
<td>175</td>
</tr>
<tr>
<td>Plasticiser</td>
<td>5.25</td>
</tr>
</tbody>
</table>

Figure 1: Schematic of the samples prepared for corrosion measurements

It has to be emphasized here that the cements used were commercial cements, by two main reasons. The first one is the difficulty of mixing cement and additions in situ, in the field. In addition to this, the second reason is that the maximum amount of fly ash allowed by the Spanish standard is a 35%, lower than the fly ash content, and will contribute to the sustainability in a lower level than this cement used. As it can be seen in the nomenclature of
the cements, the Portland and Slag cements are of the same resistant class (compressive strength of 42.5 MPa at 28 days) and the only available fly-ash cement in the Spanish market is of a lower resistant class (32.5 av. compressive strength at 28 days).

Different types of samples were prepared. Those for microstructure and durability studies that were cast in cylindrical molds of 10 cm diameter and 15 cm height, from were different slides were taken. Some of them were 1 cm thick, for microstructural characterization and chloride diffusion coefficients determination, while others were 5 cm thick for capillary suction tests. The other set of samples included a steel rebar, with the geometry shown in Fig. 1. As it can be seen in the figure, a protection of 2 cm was used to prevent the fast corrosion of the edge of the steel rebar.

2.2 Exposure locations

As it was explained in the introduction two different locations, representative of the exposure classes IIb and IIIa as stated by the EHE-08 [1] were selected. Locations are shown in Fig. 2. Location A is in the Campus of the University of Alicante, close to the highway A70-A77, and a crowded roundabout where there are many traffic jams, producing high

![Figure 2: locations selected for the exposure of the prepared samples](image-url)
amounts of CO₂. Location B is in a location close to Santa Pola’s Cape, and samples were placed about 120 m from the seaside, receiving all the chlorides produced by the marine environment. Another set of samples was kept in the humidity chamber in the laboratory (optimal conditions) as a reference to study the influence of the environment.

2.3 Microstructural characterization

As it was stated in the abstract, the microstructure was characterized by means of mercury intrusion porosimetry. This technique has been widely described in the literature [9], [10]. Even though it presents many drawbacks [11] is one of the most commonly used technique. Measurements were made using an AUTOPORE IV 9500 by Micromeritics that permits the determination of the volume of pores whose accessible diameter is between 5nm and 0.9 mm. The total porosity, and the volume of pore on each decade measured were determined for each concrete. For each concrete type two samples were tested.

2.4 Durability tests

The durability of the materials containing mineral additions was tested by means of three main parameters: capillarity, chloride migration and corrosion measurements

2.4.1 Capillary suction tests

The capillary absorption test was performed according to the standard UNE 83982[12]. This test is based in the Fagerlund method to determine the capillarity of concrete. Cylindrical samples of 10 cm diameter and 5 cm height were used. According to the RILEM recommendation TC 116-PCD [13], the circumferential surface was sealed using self-adhesive tape. The selected pre-conditioning procedure was to completely dry the mortar samples for 12 hours in an oven at 105ºC and, since then until the beginning of the test, to keep them in a hermetically sealed container with silica gel during the next 12 hours. According to the standard, the samples were introduced in a container with a flat base. The container was filled with distilled water until 5±1 mm on the lateral surface and more than a 95% of the base of the sample was in contact with water. During the test, the water level was kept constant and the container was hermetically closed. Samples were weighed at different times set in the standard. For each cement type three different samples were tested

2.4.2 Chloride migration tests

The chloride diffusion coefficient was studied by means of the so called multi-regimen method [14] (standard UNE 83987). The forced chloride migration test was performed on water saturated cement pastes, according to the standard UNE 83987 [28]. The results of the test are the non-steady-state chloride diffusion coefficient, \( D_{ns} \) m²/s, and the steady-state chloride diffusion coefficient, \( D_{s} \) m²/s. In this work, only the coefficient \( D_{ns} \) has been studied, because it allows calculating the service life of structures. The experimental procedure of this test is based on monitoring the anolyte conductivity, which has been shown to be proportional to the chloride concentration of the anolyte [15]. Samples with approximately 1 cm thick were tested. The cement pastes were saturated for 24 h before the migration tests. The experimental procedure has been widely described [15], [16]. Conductivity measurements in the anolyte
chamber were taken every 12 hours.

2.4.3 Corrosion tests

The corrosion potential was measured using an AUTOLAB potentiostat/galvanostat with PGSTAT30, and a classical 3 electrode configuration, being the reference electrode a standard Ag/AgCl electrode. Once the corrosion potential was determined, potentiodynamic polarization method was used to calculate the Icorr [17], with a polarization of ±100mV around Ecorr, and at a scan rate of 0.5mV/s. The software used for the calculation of Icorr was GPES, provided with the device.

3 RESULTS AND DISCUSSION

The samples were placed on its final location one week after their preparation and the different test were made at different ages to check the evolution. As a reference some samples were kept in the humidity chamber (95% RH and 20°C) and

3.1 Microstructure

The results of the mercury intrusion porosimetry for the laboratory (reference) and locations A and B are shown in Fig. 3.

As it can be seen in the figure, the biggest porosity was shown by the samples prepared with fly ash cement especially when exposed in location A (near the highway). This result could be partly expected, because, as it was explained in the experimental section, the cement with fly ash belongs to a lower resistant class (32.5N) what usually means a more porous structure. However, the result does not follow the same behaviour in exposition B (close to
the beach). Even though both areas are close to each other, location B is much closer to the seaside, so with higher relative humidity and colder, while location B shows a lower RH and a slightly higher temperature. This asymmetric influence of RH and temperature had already been observed in slag and fly ash cements kept under non-optimal controlled conditions in the laboratory [6], [7], [18]. The ordinary Portland cement and the fly ash cement increase their porosity when they are exposed in real environments, while the slag cement improves in a real exposition under warm weather (location A). Despite the environment where samples were placed there is a pore network refinement similar or even better than for OPC in both addition cements.

So the reactions of hydration and pozzolanic are clearly affected by the environment, but since no drastic change is observed in the pore size distribution it can be predicted that no important changes will affect the use of the more sustainable cements in real exposure conditions. The previous results of the group also show an important refinement of the pore network with time, even under non-optimal conditions, especially in the sustainable cements, so this result could be only expected to improve.

Even though it is not important from the point of view of the microstructure the data of the mercury retention are show in the next table. The mercury retention is related to the tortuosity of the pore network of the concrete.

<table>
<thead>
<tr>
<th>Table 2: Mercury retention, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>CEM I</td>
</tr>
<tr>
<td>CEM III</td>
</tr>
<tr>
<td>CEM IV</td>
</tr>
</tbody>
</table>

3.2 Durability

The results of the different durability parameters determined are shown in the next points, comparing each type of cement used and each environmental conditions.

3.2.1 Capillary suction tests

Both the capillary suction coefficient and the effective porosity have been studied. The results of the capillary coefficient are shown in Table 3.

<p>| Table 3: Capillary suction coefficient \10^{-4} Kg/m^2/min^{0.5} |
|------------------------|-----------------|-----------------|</p>
<table>
<thead>
<tr>
<th>Cement Type</th>
<th>Laboratory</th>
<th>Location A</th>
<th>Location B</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I</td>
<td>16.97</td>
<td>18.83</td>
<td>19.83</td>
</tr>
<tr>
<td>CEM III</td>
<td>15.51</td>
<td>16.28</td>
<td>20.64</td>
</tr>
<tr>
<td>CEM IV</td>
<td>16.71</td>
<td>17.46</td>
<td>21.33</td>
</tr>
</tbody>
</table>

As it can be observed the lower value for the suction coefficient is for the samples kept in the humidity chamber, as a consequence of being the samples with the lower porosity,
excepting for samples of slag cement exposed in location A. This fact cannot be explained with the presented data of the porosity, and a deeper analysis shows that if well the pores with diameter below 100 nm represent about 70% of the total porosity, the percentage of retained mercury, of 59% is not the highest (max retention for slag cement, 62%, and for fly ash cement the same max retention). This means that the pore network has low tortuosity, and water can ingress in an easier way. The results for the fly ash cement are in consonance with the MIP results. However, the most important result is that each type of cement is affected by the environment, and in location A both cements with addition behaved even better than the OPC. In location B the three cements showed a worse behaviour, with loses slightly higher for cements with addition.

Table 4: Effective porosity, %

<table>
<thead>
<tr>
<th>Cement Type</th>
<th>Laboratory</th>
<th>Location A</th>
<th>Location B</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I</td>
<td>10.01</td>
<td>12.20</td>
<td>9.93</td>
</tr>
<tr>
<td>CEM III</td>
<td>11.74</td>
<td>14.40</td>
<td>12.73</td>
</tr>
<tr>
<td>CEM IV</td>
<td>12.08</td>
<td>12.14</td>
<td>12.70</td>
</tr>
</tbody>
</table>

The results of the effective porosity show the same tendency as the results for the capillary suction coefficient. The highest correspond to the slag cement (CEM III) hardened in location 4. This result could be due to the low tortuosity of the microstructure, as it was explained to justify the value of the capillary suction coefficient.

3.2.2 Chloride diffusion coefficients

As it was stated in the experimental section, the so called multi-regimen method was used to determine the chloride diffusion coefficient. It allows the calculation of both the non-steady and steady diffusion coefficients, but in this work it will only be analysed the non-steady chloride diffusion coefficient. The reason is that in a concrete structure usually there is not a constant flow of chloride ions, and the most important parameter used for the calculation of the residual life of a structure is the time before the chlorides reach the steel reinforcement, which is somehow inverse proportional to the non-steady chloride diffusion coefficient.

![Figure 4: Results of the chloride diffusion coefficient calculate at 28 days of exposure as a function of the exposure of the samples](image-url)
The results for this parameter are shown in Fig. 4. It is clear that under real exposure the value of the chloride diffusion coefficient increases. This result could be expected from the MIP results. However, the most important result is that in the real environments studied the value of the diffusion coefficient is lower for the cements containing addition, in spite of their different porosity. This fact can be attributed to two factors. First of all the higher tortuosity for the addition cements, as compared with the ordinary Portland cement, and especially in samples hardened in real environments (see Table 2). The second factor could be the higher binding ability of cements with addition (especially higher for samples with fly ash [6]). This fact was used to justify the great decrease of the value of the chloride diffusion coefficient in samples kept under controlled but non-optimal conditions [18]. That work studied the behaviour of mortar samples and after 90 days it was appreciated a huge decrease in the value of this coefficient, so also an improvement can be expected for the concrete samples in real environments.

This result is very important, since the cements that contribute to the sustainability, when used in real environments seem to be those that will behave better from the point of view of aggressive ions ingress.

3.2.3 Corrosion results

The plots of both the potential and the intensity of corrosion are depicted on Figs. 5-7 for each cement type used as a function of the environment. As it can be appreciated in the three cases the corrosion rate (icorr) of samples kept under real conditions is always lower than for samples kept in the laboratory. This fact will be due to the lower relative humidity, even for the samples kept close to the seaside. In fact, in location B, the values of the potential are the highest for every cement type, and the value of the corrosion rate the lowest.

![Figure 5: results of the corrosion measurements on CEM I reinforced concrete samples as a function of time](image-url)
Figure 6: results of the corrosion measurements on CEM III reinforced concrete samples

Figure 7: results of the corrosion measurements on CEM IV reinforced concrete samples

If the values of the corrosion rate are compared for the real locations in location B all the cement types give a very low value, meaning the samples are in no risk of corrosion region, and no important differences are appreciated among one cement type and another. In the location A the value of the corrosion rate is slightly higher for samples prepared with CEM I but the difference is not important.

So, taking into account these results the performance of the cements more sustainable is better against corrosion than the portland cement, possibly due to their finer and with bigger tortuosity microstructure, and also to the binding effect of the additions for the chlorides.
Since the EHE-08 standard considers the higher risk for reinforced concrete structures the corrosion, it can be said that the addition cement not only promote the sustainability but also give a better durability to the structures.

4 CONCLUSIONS

- The microstructure shows in general a higher total porosity for the samples of cement with addition, but they present a higher tortuosity. The results of porosimetry also show a bigger influence of the moisture in the microstructure of fly ash cement and of temperature on the slag cement.
- The capillary suction coefficient follows the same path as the porosity.
- The chloride diffusion coefficients are better for the sustainable cements in real environments compared those of portland cement. This fact could be due to a higher tortuosity or the binding capacity of those cements.
- The results of corrosion rate are much better for real environments than for samples kept in the laboratory, probably due to the higher moisture in the humidity chamber. The addition cements show in real conditions corrosion rates similar or better than portland cement.
- Under the studied conditions the cements that contribute to the sustainability could be used with no risk regarding the durability of the structures.

5 ACKNOWLEDGEMENTS

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LIFE CYCLE ASSESSMENT OF PROTECTIVE COATINGS FOR CONCRETE

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Key words: Protective coating, Life cycle assessment

Abstract. This paper presents a Life Cycle Assessment (LCA) of several different protective coatings for concrete – all conforming with EN 1504-2 including a waterborne acrylic dispersion and a newly developed waterborne polyurethane (PU) dispersion; plus a solvent dispersed, high performance acrylic coating and a solvent dispersed PU based, high performance product. This was carried out for a refurbishment scenario on 20’000 m² of concrete surface area on a structure, and which was evaluated over a period of 60 years.

LCA is a standardized method to assess and compare the inputs, outputs and potential environmental impacts of products over their life cycle. By providing a quantitative assessment of their environmental performance, it enables differentiation between products that may have similar performance, but have greater differences concerning their environmental impact, where obviously the lower this is, the better.

The analysis shows that the quality of the coatings, their durability, frequency and quantity of material required, as well as any solvent (VOC) content are the key aspects to evaluate the environmental performance of different protective coatings over a given structure’s lifespan.
1 INTRODUCTION

Concrete protection is generally carried out using protective coatings that in Europe should comply with EN 1504-2 [1]. Different types of coatings are available in the market, including water dispersed or solvent dispersed acrylic polymer based, or with a base of other polymer types such as polyurethanes (PU).

Dependent on the quality and exposure of a coating, recoating is generally required after some period of time. Based on proven experience and field results [2] [3] [4] [5], it can be assumed that a correctly applied, good quality, water dispersed acrylic coating needs to be recoated approximately every 10 years, whilst a similar solvent dispersed acrylic or PU coating can last longer and extend the period before recoating is required in order to maintain the protective performance. Nevertheless, there is a growing trend to restrict the use of solvent containing coatings due to health and safety constraints in national regulations and increasingly green building program requirements (including compliance with LEED, BREEAM, Hong Kong Green Label Scheme or Singapore Green Building Product Labelling Scheme).

EN 1504-2 details the performance requirements that a protective coating must comply with, but it does not provide any information on the impact that the use of different types of coating materials may have on the sustainability or environmental aspects.

This paper presents a Life Cycle Assessment of several different protective coatings for reinforced concrete surfaces – all conforming to EN 1504-2 and these include a water dispersed acrylic and a newly developed water dispersed PU; plus a high performance, solvent dispersed acrylic and a solvent dispersed PU based coating system.

2 LIFE CYCLE ASSESSMENT

Life Cycle Assessment (LCA) provides a method to quantify and evaluate potential environmental impact throughout a product’s life cycle (Fig. 1). It starts with the product’s raw materials, through the production phase, on to the project site; then the product’s use and end of life as either waste or into recycling. This is commonly referred to as a ‘cradle-to-grave’ approach [6].

An LCA from a construction material manufacturer can make a quantitative environmental profile within a project requirement framework, which can then help the responsible professional team to differentiate between alternative products and solutions in terms of the potential environmental impact or benefit.

This LCA of concrete protection and refurbishment systems was performed by the Global Product Sustainability Department at Sika, in accordance with International Standards ISO 14040 [6] and EN 15804 [7].
.1 Scope of the analysis

The Functional Unit taken for the LCA is the scenario of the application of these different coatings on a 20’000 m² concrete structure and evaluated over a period of 60 years. To perform the LCA, an inventory analysis step was necessary, i.e. data collection and calculation procedures to quantify the relevant inputs and outputs of each product / system.

Product data collection was carried out by the manufacturer and corresponds to measured or qualified data, including formulations of the products, transportation, packaging and production details. The systems were then modelled in the LCA software GaBi 6.3, based on this collected data and on datasets from commercial databases such as thinkstep (formerly PE International) and ecoinvent. Transport to the installation site was included. Installation process data includes energy for pre-treatment of the surface, application of the product (and estimated VOC emissions), plus waste transport and treatment. For the end-of-life waste transport and waste treatment of dismounted systems was included. The final residues of all system components are considered to be disposed of as landfill.

Figure 1: Schematics of the entire life cycle of construction products

2.2 LCA Impact Categories

The LCA was performed by Sika Services AG Global Product Sustainability Group and modelled in the GaBi 6.3 Software. Three impact categories are considered to be the most relevant for this example, as these three depict the sustainability drivers in refurbishment: carbon footprint, resource (and energy) efficiency, and pollutant emissions (also related to health and safety of the workers).

The impact assessment method used was CML 2001 – Apr. 2013 [8]. The impacts assessed were:

- Global Warming Potential (GWP) [kg CO₂-eq.] - measures the potential contribution to climate change, focusing on emissions of greenhouse gases, such as carbon dioxide (CO₂), which enhance the heat radiation absorption of the atmosphere, causing the temperature at the earth’s surface to rise.
- Cumulative Energy Demand (CED) [MJ] - accounts for the consumption of energy resources, namely the total amount of primary energy from renewable and non-renewable resources.
- Photochemical Ozone Creation Potential (POCP) [kg ethene-eq.] – measures the potential contribution to summer smog, related to ozone induced by sunlight on volatile organic compounds (VOC) and nitrous oxides (NOx)

3 PROTECTIVE COATING SCENARIOS

The performance of the different coatings is shown in Table 1 below and the scenarios are described in Tables 2 to 6.

Table 1: Protective coating EN 1504-2 performances

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Product type</th>
<th>CO₂ diffusion</th>
<th>Capillary uptake</th>
<th>H₂O vapor diffusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>Water-borne polyurethane resin</td>
<td>&gt;50 m</td>
<td>&lt;0,1 kg/(m²xh⁰.5)</td>
<td>Class I (&lt; 5m)</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>Water-borne acrylic resin</td>
<td>&gt;50 m</td>
<td>&lt;0,1 kg/(m²xh⁰.5)</td>
<td>Class I (&lt; 5m)</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>Solvent-borne polyurethane resin</td>
<td>&gt;50 m</td>
<td>&lt;0,1 kg/(m²xh⁰.5)</td>
<td>Class II (&gt; 5m &amp; &gt; 50 m)</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>Solvent-borne acrylic resin</td>
<td>&gt;50 m</td>
<td>&lt;0,1 kg/(m²xh⁰.5)</td>
<td>Class I (&lt; 5m)</td>
</tr>
</tbody>
</table>

Table 2: Protective coating scenarios

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Product type</th>
<th>Typical re-application interval</th>
<th>Cleaning agent during application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>Water-borne polyurethane resin</td>
<td>20 years</td>
<td>Water</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>Water-borne acrylic resin</td>
<td>10 years</td>
<td>Water</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>Solvent-borne polyurethane resin</td>
<td>20 years</td>
<td>Solvent</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>Solvent-borne acrylic resin</td>
<td>20 years</td>
<td>Solvent</td>
</tr>
</tbody>
</table>
### Table 3: Scenario 1 details

<table>
<thead>
<tr>
<th>Timing (year &amp; frequency)</th>
<th>Building phase</th>
<th>Action</th>
<th>Process</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year 0</td>
<td>New construction</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Primer application</td>
<td>Airless (product diluted with 15% water)</td>
<td>2’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coating application</td>
<td>Airless undiluted–2 coats</td>
<td>9’000</td>
</tr>
<tr>
<td>Every 20 years (i.e. x 2 times)</td>
<td>Maintenance</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Refresher coat</td>
<td>Airless undiluted–1 coat</td>
<td>4’500</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Total quantity</strong></td>
<td></td>
<td><strong>20’000</strong></td>
</tr>
</tbody>
</table>

### Table 4: Scenario 2 details

<table>
<thead>
<tr>
<th>Timing (year &amp; frequency)</th>
<th>Building phase</th>
<th>Action</th>
<th>Process</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year 0</td>
<td>New construction</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Primer application</td>
<td>Airless (product diluted with 15% water)</td>
<td>2’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coating application</td>
<td>Airless undiluted–2 coats</td>
<td>9’000</td>
</tr>
<tr>
<td>Every 10 years (i.e. x 5 times)</td>
<td>Maintenance</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Refresher coat</td>
<td>Airless undiluted–1 coat</td>
<td>4’500</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Total quantity</strong></td>
<td></td>
<td><strong>33’500</strong></td>
</tr>
</tbody>
</table>

### Table 5: Scenario 3 details

<table>
<thead>
<tr>
<th>Timing (year &amp; frequency)</th>
<th>Building phase</th>
<th>Action</th>
<th>Process</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year 0</td>
<td>New construction</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Primer application</td>
<td>Airless (epoxy primer)</td>
<td>4’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coating application</td>
<td>Airless undiluted–2 coats</td>
<td>16’000</td>
</tr>
<tr>
<td>Every 20 years (i.e. x 2 times)</td>
<td>Maintenance</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Refresher coat</td>
<td>Airless undiluted–1 coat</td>
<td>8’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Total quantity</strong></td>
<td></td>
<td><strong>36’000</strong></td>
</tr>
</tbody>
</table>
Table 6: Scenario 4 details

<table>
<thead>
<tr>
<th>Timing (year &amp; frequency)</th>
<th>Building phase</th>
<th>Action</th>
<th>Process</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year 0</td>
<td>New construction</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Primer application</td>
<td>Airless (product diluted with 15% thinner)</td>
<td>2’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coating application</td>
<td>Airless undiluted–2 coats</td>
<td>8’000</td>
</tr>
<tr>
<td>Every 20 years (i.e. x 2 times)</td>
<td>Maintenance</td>
<td>Surface preparation</td>
<td>Low water pressure cleaning</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Refresher coat</td>
<td>Airless undiluted–1 coat</td>
<td>4’000</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Total quantity</strong></td>
<td></td>
<td><strong>18’000</strong></td>
</tr>
</tbody>
</table>

For all scenarios, the transport to site is considered to be 300 km; surface preparation using pressure washing and the losses during application are assumed to be equal for all applications and taken as 3%.

The VOC’s released during application (including the cleaning process) are calculated as:

- Water-borne polyurethane resin: ~6%
- Water-borne acrylic resin: ~1%
- Solvent-borne polyurethane resin: ~25% (Comp. A) and ~30% (Comp. B)
- Solvent-borne epoxy primer: ~40% (Comp. A) and ~30% (Comp. B)
- Solvent-borne acrylic resin: 35%

4 LCA RESULTS

The results for the three impact categories are shown below, per construction stage.

- The Product Stage includes the ‘cradle-to-gate’ impacts of the products (e.g., raw materials, production and packaging).
- The Construction Process Stage is divided in three parts: transport to installation site, surface pre-treatment and product application.
- The End-of-Life Stage includes landfill of the products after the end of life of the structure.

4.1 Global Warming Potential

The scenario having the highest impact with regards to Global Warming Potential is Scenario 3 (solvent-borne polyurethane coating system) – refer to Figure 2. Both water dispersed coatings (Scenario 1 & 2, polyurethane and acrylic respectively) have very similar impacts, much lower than the solvent dispersed polyurethane coating (Scenario 3). The majority of the impact originates from the Product Stage of the LCA.

Surprisingly the Scenario 4 coating (solvent-borne acrylic) has the lowest impact. This is due to the reduced frequency of re-application compared to the acrylic water dispersed coating (Scenario 2). On the other hand, it also has a lower impact than Scenario 1 even if their frequency of application is similar. The reason is that production of the polymers in the
water-borne polyurethane coating has much higher impact than producing the polymers in the solvent-borne acrylic coating.

The impact equivalence comparing the different Scenarios to Scenario 1 is given in Table 7. Comparatively to Scenario 1, acrylic coatings (Scenario 2 & 4) have marginal impact (negative for the water dispersed coating and positive for the solvent-dispersed materials), while the solvent-borne polyurethane has a very significant impact, as this is equivalent to the CO$_2$ emitted by a 15 ton truck covering more than 1’200 km every year.

**Table 7: Global Warming Potential (GWP) – difference to Scenario 1**

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Tons of CO$_2$ over 60 years</th>
<th>Equivalent kms of a 15 ton truck per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 2</td>
<td>8.3</td>
<td>~150</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>65.1</td>
<td>~1’200</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>-17.1</td>
<td>~300</td>
</tr>
</tbody>
</table>

**Figure 2: Global Warming Potential (GWP) for the 4 Scenarios over a 60-year lifespan**

4.2 Cumulative Energy Demand

The Scenario having the highest impact with regards to Cumulative Energy Demand is also Scenario 3 (solvent-borne polyurethane coating system) – refer to Figure 3. Both water-borne coatings (Scenarios 1 & 2, polyurethane and acrylic respectively) have very similar impact, much lower than the solvent-borne polyurethane coating in Scenario 3. Scenario 4 (solvent based acrylic) has again the lowest impact due to the lower frequency of re-application compared to the acrylic waterborne coating (Scenario 2). Due to the fact that acrylic polymers are less impact intensive than polyurethane polymers, it also has less impact than Scenario 1, although their frequency of application is similar.
The impact equivalence comparing the different Scenarios to Scenario 1 is given in Table 8. Comparatively to Scenario 1, acrylic coatings (Scenario 2 & 4) have marginal impact (negative for the water-dispersed coating and positive for the solvent-dispersed material), while the solvent-dispersed polyurethane (Scenario 3) has a significantly higher impact, as this is equivalent to approximately 750 liters of oil being consumed every year.

Table 8: Cumulative Energy Demand CED – difference to Scenario 1

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Tera Joules over 60 years</th>
<th>Equivalent liter of oil per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 2</td>
<td>314</td>
<td>~125</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>1'898</td>
<td>~750</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>-146</td>
<td>~60</td>
</tr>
</tbody>
</table>

4.3 Photochemical Ozone Creation Potential

The Scenarios having the highest impact with regards to Photochemical Ozone Creation Potential are, as could be expected, the solvent-borne systems (Scenarios 3 & 4) due to the VOC emissions during their application – refer to Figure 4. Both water-borne coatings (Scenarios 1 & 2, polyurethane and acrylic respectively) have very low impact due to their low emissions of VOC’s.

The impact equivalence comparing the different Scenarios to Scenario 1 is given in Table 9. Comparatively to Scenario 1, the water-borne acrylic coating (Scenario 2) has a positive marginal impact, whilst the two solvent-borne systems have severe impact, as this will be equivalent to the use of approximately 300 to 500 kg of solvent containing paints every year.
Table 8: Photochemical Ozone Creation Potential POCP – difference to Scenario 1

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>kg ethene equivalent over 60 years</th>
<th>Equivalent kg of solvent paint per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 2</td>
<td>-310</td>
<td>~50</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>3'236</td>
<td>~500</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>1'988</td>
<td>~300</td>
</tr>
</tbody>
</table>

Figure 4: Photochemical Ozone Creation Potential (POCP) for the 4 Scenarios over a 60-year lifespan

5 DISCUSSION/CONCLUSION

The analysis shows that the quality of the different protective coatings, their durability, frequency of application and quantity of material required, as well as any solvent content, can all greatly influence the environmental performance over a given structure’s lifespan.

For both Global Warming Potential and Cumulative Energy Demand, the environmental impacts originate mainly in the Product Stage; coatings with a higher durability - and therefore lower frequency of recoating and less amount of inputs throughout the structure’s lifespan - perform better.

For the POCP category, the Product application stage has the most influence in the performance of the coatings due to VOC emissions during application. For this category, solvent-dispersed coatings have considerably higher impact.

By using durable, water-dispersed coatings, the frequency of applications (and thus the total material input for the structure’s lifespan) can be reduced, as they can with a durable solvent-dispersed acrylic or polyurethane coating, but without the downside of having the solvent content and all the potential risks that this entails, as well as the increasingly stricter H&S
legislation and green building program requirements.

Using an LCA approach allows differentiation of the behavior of different protective coatings with regards to their impact on the environment over their expected lifespan. This in turn allows Architects and Engineers not just to specify products with regards to their technical performance according to the European Norms, but also with regards to their environmental performance throughout the structure’s anticipated lifespan.

REFERENCES


**NOx Adsorption, Fire Resistance and CO2 Sequestration of High Performance, High Durability Concrete Containing Activated carbon**

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**Key words:** Bio-char, Activated carbon, carbon sequestration, carbon sink, fire resistance, NOx adsorption, high performance concrete, durability.

**Abstract.** The scientific community has conducted limited research to date on the use of activated carbon (or activated char) in concrete. Due to its nature, activated carbon could provide pollutant adsorbing properties and improved fire resistance of concrete. We compared properties of a C50/60 XF4, XC4, XD3, XA3 fire resisting concrete mix (with PP fibers and with activated carbon respectively) used for the construction of the viaducts of the high speed railway system in Southern Switzerland (AlpTransit project). The following properties were compared: compressive and tensile strength, elastic modulus, freeze-thaw resistance, chloride diffusion, capillary permeability, sulphates resistance, accelerated carbonation and resistance to fire (spalling depth and temperature profiles). Results have shown that the use of optimal dosages of activated carbon in high performance concrete does not affect any of its mechanical and durability properties. Results of compressive and tensile strength and elastic modulus were fully comparable with/without activated carbon. Additionally, an improved resistance to fire even in the absence of PP fibers was observed. Activated char seems to therefore offer a greener/technically promising opportunity for the production of high strength, high durability concrete for road tunnels (where increased resistance to fire is mandatory for safety issues, and pollutant adsorbing properties are desirable for environmental issues) and for civil infrastructures in general where the yearly (extremely large) volumes of concrete produced may create long term sink structures for oxides of nitrogen (NOx) adsorption. Because of its nature as a product obtained from thermal cracking of biomass in absence of oxygen, activated char also promotes the exponential increase of CO2 sequestration potential when incorporated into concrete, which is by far the most widely used construction material.

1 **INTRODUCTION**

1.1 Can concrete be a sustainable material?

Fossil fuel and cement production account for the majority of CO2 emissions on our planet. Today’s global average emissions rate from clinker manufacturing is 0.9 kg of CO2/kg of clinker (approximately 0.74 kg of CO2 of cement) [15] and the world’s concrete production in 2010 was roughly 19 Gt [6] with an estimated annual growth rate of about 10% [21].
Considering that 0.130 kg of CO₂ per kg are emitted by the final product [30], the annual emission of CO₂ for concrete production is estimated at 2.47 Gton. According to [15, 30] and [6] the reduction of CO₂ emissions from the cement (and concrete) industry can be successfully achieved by the following methods: 1. introducing innovative structural design, 2. improving durability of structures in order to extend their service life, 3. using precast elements that can be assembled/disassembled easily, 4. specifying supplementary cementitious materials (Pulverized Fuel Ash - PFA, Ground granulated blast-furnace slag – GGBS, Pozzolanas, Calcined Clays, etc.) and adapting structural/durability and quality control requirements accordingly, 5. reducing cement dosage, 6. using recycled materials, 7. utilizing the heat capacity of concrete to save energy for heating/cooling of buildings. An additional method to make concrete a sustainable material is based on the principle of making it a carbon sink by incorporating carbon based compounds in its matrix (of vegetal and animal origin) that would otherwise decompose and combine with oxygen to produce CO₂. When organic matter is thermally cracked between 400°C and 600°C in absence of oxygen by means of a pyrolysis process, the resulting product is defined as bio-char [16,17,23]. Depending on the feedstock material (wood, coconut shells, crops, animal waste, etc.) the content of pure carbon (C) in the char may vary, but it will generally be in excess of 70% of total char with the remaining % made out of ash containing (noticeably) silicon, calcium, iron and negligible amounts of alkalis (Na + K) [22]. The global yearly quantity of dead wood on our planet is estimated roughly at 67 Gt of biomass, which is about 11% of all biomass [18]. A vast amount of this dead biomass is currently burned in open air, particularly in third world or developing countries, leading to large scale (regional) phenomena of polluting haze. If, conversely, this biomass were subjected to pyrolysis, the end product would fix the carbon from the organic matter’s combustion to an “inert” compound non reactive to atmospheric oxygen to produce CO₂. It can be estimated that from 1.0 t of dead wood matter, approximately 0.3 t of bio-char can be produced. If only 1% of bio-char (by mass of concrete) were incorporated in concrete, it can be calculated that roughly 0.5 Gt of CO₂ would be sequestrated by the concrete sink (1 t of activated carbon with a C content of 70% contains 0.7 t of C or 2.52 t of equivalent CO₂ sequestered) corresponding to about 20% of the total CO₂ emissions generated by the cement industry. The incorporation of bio-char in cementitious materials is a relatively novel concept and recent studies focused mainly on either assessing the optimum dosage of bio-char in concrete so as not to negatively affect its mechanical properties [20] when concrete is used as a carbon sink, or [23] on the effects of bio-char incorporation in plasters for the control of humidity. Other recent studies [7,8] aimed to determine the NO₅ adsorbing properties of concrete manufactured with activated carbon. Activated carbon is thermally-treated-in-pareence-of-water-vapor char deriving from wood biomass to produce a high specific surface (B.E.T \geq 800 \text{ m}^2/\text{g}) and highly porous structure with noticeable fluid adsorbing and absorbing properties. The basic scheme of activated carbon production is given in Figure 1 below.
The end product (activated carbon) can be either in powder form or pellets and is widely used in agriculture, in pollutants removal technologies, in filtering processes, air purification systems and in the food and pharmaceutical industry amongst others.

1.2 Scope of this research

The objective of this research is to define the beneficial properties of activated carbon for concrete in terms of its adsorbing/absorbing properties, keeping in mind the long term (stable) storage potential of concrete structures for the carbon contained in the char. Of particular interest is the potential water vapor adsorption properties of the activated carbon and its noticeable NOx adsorbing properties as already determined by other researchers [7,8]. Water vapor adsorption by char particles could in fact dissipate some of the internal vapor pressure developed during exposure of concrete to fire which causes disruptive spalling, a major issue for both structural and human safety during and immediately after fires in tunnels. If concrete containing activated carbon were exposed to a NOx (nitrogen di-oxide: NO2 and nitrogen oxide: NO) polluted environment of a road tunnel, it would not only improve the fire resistance of the concrete lining, but it would provide the reduction of hazardous gases such as NO2 produced by vehicles.

Water vapor adsorption onto activated carbon involves primary and secondary adsorptive sites. The primary adsorptive sites consist of oxygenated surface compounds capable of forming hydrogen bonds with water molecules. The secondary adsorptive sites are the previously adsorbed water molecules, which can form hydrogen bonds with the adsorbing water molecules. It is the combination of the primary and secondary sites that determines the change in adsorption capacity with the change in water vapor pressure [24]. Recent research has shown that concrete containing activated carbon can provide absorption of NO2 over the concentration and rate ranges relevant for the remediation of atmospheric pollution in tunnels and parking garages [8]. Possible reactions between NOx and calcium hydroxide to form nitrite and nitrate salts have been used to explain the phenomenon [8]. NO2 is a highly reactive and hazardous gas of the NOx group, which tends to concentrate in road tunnels [11,12,14]. In bright sunlight and at temperatures above 21°C photolysis of NO2 produces ozone (O3). NO, another very unstable gas of the NOx group, can in turn react with oxygen (O2 and O3) to form NO2. During rush hour in metropolitan tunnels, the concentration of NO2 can be several orders of magnitude greater than the concentration limit set by the EPA in
1.3 Mechanisms of the spalling of concrete exposed to fire

Spalling mechanism is defined as the breaking up of layers (pieces) of concrete from the surface of a concrete member when it is exposed to high and rapidly rising temperatures such as those encountered in fires [9]. Several factors affect the explosive spalling of concrete [9]: heating rate and profile, section size and shape, moisture content, permeability, age, strength, compressive stress before and during heating, restraint to thermal expansion, aggregate type and size, cracking, and reinforcement. Spalling is recognized as a coupled thermo-hydral process associated with the transmission of mass (water in liquid and vapor phases plus air). As temperature increases, water is partly evaporated and generates a pressure in the porous network. The pressure gradient being the major driving force for mass transfer. Vapor and air are partially evacuated to the heated surface, but they also migrate to the center of the element, where vapor condenses as soon as the thermodynamic conditions are satisfied. As a result, a quasi-saturated level is formed (the so called “moisture clog”). After some time the concrete element contains (inward from the heated surface) a dry and dehydrated zone, a drying and dehydrating zone and a quasi semi-saturated layer that acts as an impermeable wall for gases. The pressure peak is located at this saturated level. The lower the permeability of the material, the sooner the moisture clog is generated and the closer it is to the surface. Likewise, a higher the pressure and the pressure gradient also occurs [2]. There is not general consensus in existing literature as to whether it is the internal vapor pressure developed during fire exposure that dominates and causes the breaking of the matrix, or if it is the biaxial high compressive stresses generated by the restrained thermal expansion which cause failure and explosive spalling regardless of the moisture content and pore pressure. Regardless of the mechanism causing spalling, the conventional approach to the mitigation of this phenomenon is to incorporate polypropylene (PP) fibers in the concrete matrix [1]. The fibers melt at about 160°-170 °C thus creating channels and microcracks in which the water and water vapor can be transported before developing high pore pressures. As a result of the melting of PP fibers, an increased permeability is observed. Fire tests have shown that the permeability of concrete with 1.5 kg/m³ PP fibers can be up to four times greater than the permeability of concrete without fibers [9].

2 EXPERIMENTAL INVESTIGATIONS

We compared properties of a C50/60 XF4, XC4, XD3, XA3 (according to EN 206-1 standard) fire resisting concrete mix (with PP fibers) used for the construction of the viaducts of the high speed railway system in Switzerland (AlpTransit project). A reference mix was cast with and without activated carbon. When activated carbon was used, PP fibers were omitted from the mix. The following properties were compared with varying dosages of activated carbon (Table 1): compressive (cube) strength and tensile strength, elastic modulus, freeze-thaw resistance, chloride diffusion, capillary permeability, sulphates resistance, and accelerated carbonation. Additionally, fire resistance (spalling depth and temperature profiles) and NOx adsorption tests were carried out on concrete samples containing 1.06% of activated
carbon by mass of concrete. The activated carbon for this research (pellets with nominal Ø 3 mm) was provided by Messrs. Gamatec SA utilizing a drum pyrolysis reactor based on the patented ATTTEROM® technology, using a double stage process: pyrolysis to produce the char, coupled with thermal activation to increase char porosity and specific surface. Table 1 presents the composition of the reference concrete mix (containing PP fibers and pulverized fuel ash, type II according to EN 450-1/2) and the patented comparative mix containing activated carbon and a highly reactive siliceous pozzolanic (HRSP) compound having specific surface B.E.T of 15 m²/g and SG of 2.63.

Table 1: Composition of the reference mix and comparative mix

<table>
<thead>
<tr>
<th>Component</th>
<th>Dosage [kg/m³]</th>
<th>% By mass of concrete*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reference mix</td>
<td>Comparative mix</td>
</tr>
<tr>
<td>CEM II / A-D 52.5R</td>
<td>350 kg/m³</td>
<td>16.5%</td>
</tr>
<tr>
<td>Pulverized Fuel Ash</td>
<td>40 kg/m³</td>
<td>--</td>
</tr>
<tr>
<td>HRSP</td>
<td>--</td>
<td>1.27%</td>
</tr>
<tr>
<td>Activated Carbon (AC)</td>
<td>--</td>
<td>0.48% /1.06% /1.43%</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>4.9 kg/m³</td>
<td>0.33%</td>
</tr>
<tr>
<td>Air Entrained</td>
<td>0.14 kg/m³</td>
<td>0.01%</td>
</tr>
<tr>
<td>PP fibers</td>
<td>2.0 kg/m³</td>
<td>--</td>
</tr>
<tr>
<td>Silceous-Calcareous CA - Ø max 32 mm</td>
<td>270 kg/m³</td>
<td>3.45%</td>
</tr>
<tr>
<td>Silceous-Calcareous CA - Ø max 16 mm</td>
<td>684 kg/m³</td>
<td>32.9%</td>
</tr>
<tr>
<td>Silceous-Calcareous CA - Ø max 8 mm</td>
<td>576 kg/m³</td>
<td>18.5%</td>
</tr>
<tr>
<td>Silceous-Calcareous FA - Ø max 4 mm</td>
<td>268 kg/m³</td>
<td>13.6%</td>
</tr>
<tr>
<td>Entrained air</td>
<td>4%</td>
<td>8%</td>
</tr>
<tr>
<td>Free water – W/C</td>
<td>165 kg/m³</td>
<td>5.9% -</td>
</tr>
<tr>
<td></td>
<td>0.41</td>
<td>0.33</td>
</tr>
</tbody>
</table>

*explicit composition of comparative mix is undisclosed at present stage

All tests conducted in this research were carried out by an ISO/CEI 17025 accredited laboratory (IMM SA) according to relevant Swiss / European standards, with the exception of the NOx adsorption test and fire resistance test carried out by accredited European laboratories (Tera Environment, France and EFECTIS, Netherland respectively) according to ad-hoc developed procedures (for NOx adsorption) [8] and testing recommendations/fire curves [27, 28].

2.1 Mechanical and physical properties of the reference mix and comparative mix

Table 2 below summarizes the results of the mechanical and physical tests along with the reference standards utilized for the tests and the acceptance criteria (where available) for Switzerland based on [29].
Table 2: Summary of mechanical and durability properties of the reference mix and comparative mix

<table>
<thead>
<tr>
<th>Concrete requirements</th>
<th>Comparative mix</th>
<th>Reference mix</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>C50/60 XF4, XD3, XC4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Parameter</strong></td>
<td>0.48% AC AC</td>
<td>1.06% AC AC</td>
<td></td>
</tr>
<tr>
<td>Elastic Modulus (SIA 262/1 - G)</td>
<td>41800</td>
<td>41300</td>
<td>41900</td>
</tr>
<tr>
<td>[MPa]</td>
<td>40400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural Strength (SN EN 12390-5)</td>
<td>5.0</td>
<td>5.4</td>
<td>5.4</td>
</tr>
<tr>
<td>[MPa]</td>
<td>4.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shrinkage (364-day) (SIA262/1-F)</td>
<td>419</td>
<td>452</td>
<td>458</td>
</tr>
<tr>
<td>[με]</td>
<td>423</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freeze - Thaw Resistance (SIA262/1-C)</td>
<td>200</td>
<td>70</td>
<td>80</td>
</tr>
<tr>
<td>[g/m²]</td>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance to Chlorides Penetration (SIA 262/1 - B)</td>
<td>2.3·10⁻¹²</td>
<td>2.2·10⁻¹²</td>
<td>1.7·10⁻¹²</td>
</tr>
<tr>
<td>[m²/s]</td>
<td>3·10⁻¹²</td>
<td></td>
<td>&lt;10⁻¹²</td>
</tr>
<tr>
<td>Absorption by Capillarity (SIA 262/1-A)</td>
<td>6.8</td>
<td>4.5</td>
<td>4.2</td>
</tr>
<tr>
<td>[g/(m²·h)]</td>
<td>5.7</td>
<td></td>
<td>&lt;10</td>
</tr>
<tr>
<td>Resistance to sulphates (SIA 262/1-D)</td>
<td>0.36</td>
<td>0.31</td>
<td>0.46</td>
</tr>
<tr>
<td>[%]</td>
<td>0.32</td>
<td></td>
<td>≤1.2</td>
</tr>
<tr>
<td>7-day Compressive Strength (SN EN 12390-3) –</td>
<td>52.3</td>
<td>51.3</td>
<td>53.0</td>
</tr>
<tr>
<td>[MPa]</td>
<td>50.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-day Compressive Strength (SN EN 12390-3) –</td>
<td>71.8</td>
<td>65.8</td>
<td>61.8</td>
</tr>
<tr>
<td>[MPa]</td>
<td>65.3</td>
<td></td>
<td>≥64</td>
</tr>
<tr>
<td>Coefficient of carbonation (SIA 262/1-I) -</td>
<td>--</td>
<td>1.38</td>
<td>--</td>
</tr>
<tr>
<td>[mm/√year]</td>
<td>--</td>
<td></td>
<td>≤4.5</td>
</tr>
</tbody>
</table>

2.2 Results from fire tests

A 0.6x0.6x0.4 m³ unreinforced, unstressed slab cast with the comparative mix containing 1.06% of activated carbon by mass of concrete was subjected to a fire test following the RABT-ZTV (railways) fire curve (Figure 2). This highly used curve simulates a 50 m³ petrol tanker fire with a fire load of 300 MW lasting for 120 minutes [28]. The main test parameters are presented in Table 3 while Figure 2 shows the actual fire curve applied (T in °C on the vertical axis) and two photographs of the sample, prior and post testing. Typical discoloration due to dehydration of the cement matrix’s hydrate phases is clearly visible after the fire test. Figure 3 presents the temperature profiles at different depths and time.
Figure 2: Temperature applied for the fire test and appearance of the sample before and after testing

Table 3: Results of the fire test with RABT - ZTV, railways fire curve

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value/Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age (days)</td>
<td>35</td>
</tr>
<tr>
<td>Moisture content (%)</td>
<td>3.9</td>
</tr>
<tr>
<td>T @ 60 mm depth @ 45 minutes (°C)</td>
<td>128</td>
</tr>
<tr>
<td>Maximum/average spalling depth (24 points of measurement) (mm)</td>
<td>22 / 8</td>
</tr>
<tr>
<td>Maximum crack width (mm)</td>
<td>0.7</td>
</tr>
</tbody>
</table>
2.3 NO$_x$ adsorption test

Adsorption tests were carried out following the methodology developed by [8] according to the scheme of Figure 4.
The results from the NO\textsubscript{x} adsorption test carried out for 24 hours on a 150 x 150 x 40 mm\textsuperscript{3} sample of concrete with/without activated carbon are reported in Figure 5 below.

![Graph showing adsorption curves](image)

**Figure 5:** Adsorption curves for the concrete without activated carbon (left) and containing 1.06% activated carbon by weight of concrete (right) (courtesy of Tera environment)

Tables 4 and 5 document the upstream and downstream concentrations of NO\textsubscript{x} in the test chamber when the reference concrete and the activated carbon mix were introduced respectively. The inlet NO\textsubscript{2} concentration is about 2.5 times the threshold limit for human health of 100 ppb set by the EPA in 2010 [14].

<table>
<thead>
<tr>
<th>Table 4: NO\textsubscript{x} adsorption test by the reference concrete mix</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Average upstream concentration</strong></td>
</tr>
<tr>
<td>NO\textsubscript{2} (ppb)*</td>
</tr>
<tr>
<td>NO (ppb)</td>
</tr>
</tbody>
</table>

*1ppb NO\textsubscript{2} = 1.88 µg/m\textsuperscript{3}*

<table>
<thead>
<tr>
<th>Table 5: NO\textsubscript{x} adsorption test by the comparative concrete mix with 1.06% activated carbon</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Average upstream concentration</strong></td>
</tr>
<tr>
<td>NO\textsubscript{2} (ppb)*</td>
</tr>
<tr>
<td>NO (ppb)</td>
</tr>
</tbody>
</table>

*1ppb NO\textsubscript{2} = 1.88 µg/m\textsuperscript{3}*

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3 DISCUSSION OF RESULTS

3.1 Mechanical and physical tests

From the data presented in Table 2, the addition of activated carbon has no detrimental effect on any physical or durability property up to a dosage of 1.43% by mass of concrete. All results are fully comparable within the known reproducibility (R) values for the testing procedures, and therefore observed differences are not due to an effective difference in performance but rather to the intrinsic variability of test results. Of particular interest are the high freeze-thaw resistance, the very low permeability to chlorides and the very low coefficient of carbonation that indicate the potential of very high durability properties for the concrete containing activated carbon. The samples containing 1.43% of activated carbon by mass of concrete gave the lowest 28-day compressive strength results, yet still exceeded 60 MPa.

3.2 Fire tests

It’s a well known and documented fact that concrete without PP fibers is subject to explosive and severe disruptive spalling, even when mixes are supposedly designed to resist fire per se without any additional passive fire protection [1,2,3,4,5,13,25]. In this circumstance the reinforcing bars are suddenly exposed to very high temperatures because of the lack of protection by the concrete cover, and they may rapidly yield and fail. It is therefore a desired performance of concrete exposed to fire to minimize explosive spalling by limiting the temperature at the depth of the rebars (usually 60-70 mm for structures designed to resist fire) [9]. The results obtained in the present research which show a very limited spalling depth for a PP-free high performance/high strength concrete mix, are therefore very promising in the attempt to define a sustainable material with improved performance and extended service life. The general consensus that high performance concrete has a higher spalling tendency than low strength concrete, because of its lower permeability, does not seem to have been confirmed by the present study, indeed. Possible reasons are: high entrained air content (8%) beneficial in dissipating internal pressure, very low water/cement ratio and consequently very low moisture content (3.9% only at 35 days), and presence of activated carbon as a possible pressure relief compound in replacement of traditional PP fibers.

3.3 NOx Adsorption test

Figure 5 clearly demonstrates the influence of the concrete sample containing activated carbon in decreasing the concentration of NO2 gas from the inlet to the outlet of the test chamber. These results confirm those published by [8] and prove, despite a limited control period of only 24 hours, that activated carbon may effectively improve the pollutant adsorption properties of structural concrete.

4 CONCLUSIONS

- Activated carbon as produced from thermal cracking of biomass has a high specific surface and porous structure with noticeable fluid (gas and liquid)
adsorbing/absorbing properties.
- Addition of activated carbon to high performance concrete in optimal dosages has no effect on its performance, both in the fresh and hardened state.
- The fire resistance of a patented high performance concrete mix using calcareous/siliceous aggregates, free water/cement ratio of 0.33, 8% of entrained air, with an highly reactive siliceous pozzolanic compound and 1.06% activated carbon by mass of concrete, had very limited spalling in a fire test conducted with the RABT-ZTV (railways) fire curve.
- The NO₃ adsorbing properties of concrete containing 1.06% activated carbon by mass of concrete have been investigated, showing the effect of such a conglomerate in dramatically decreasing the concentration of NO₂ in the test chamber.
- Because concrete is the most widely used material on the planet and because activated carbon is inert in respect of atmospheric oxygen, incorporation of 1% of this material in concrete could perceivably lower the carbon footprint of concrete, leading to a market of tradable carbon credits on the order of tens of billion of dollars in the next decades [19].
- Our ongoing research in this field is presently targeting the fire resistance (spalling depth and temperature at the reinforcement) of reinforced concrete with activated carbon under restrained conditions, and long term and larger scale NOₓ adsorption tests to calculate a steady state coefficient of adsorption of air pollutants for the design of a pollution-adsorbing concrete lining for road tunnels.

REFERENCES


[22] https://www.ecn.nl/phyllis2/Browse/Standard/ECN-Phyllis#biochar


PARAMETRIC ANALYSES ON SUSTAINABILITY INDICATORS FOR DESIGN, EXECUTION AND MAINTENANCE OF CONCRETE STRUCTURES
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Key words: Sustainability, CO2 emission, Cost, Structural Performance, Safety Margin

Summary. Sustainability is defined as a concept including environmental, economic, and social aspects. It is likely to be taken into account in concrete work from the viewpoints of resources and energy consumption, initial construction and/or lifecycle costs, durability, and environmental impact. During the lifecycle of a concrete structure, sustainability is generally considered with one or a few of these viewpoints, but it is not so easy to find the best solution among alternatives because no comprehensive sustainability indicator exists. This paper examines the sensitivity of design parameters such as the structure factor for structural capacity on cost and CO2 emission of reinforced concrete beams. Dimensions of reinforced concrete beams, quantity of reinforcing bars, materials strengths, and environmental conditions are varied as parameters for design calculations as well as the structure factor. For each set of those parameters, initial construction cost and CO2 emission are calculated. As a result, based on the parametric analyses, the structure factor does not have a considerable impact on the CO2 emission and the cost. From this, it will be necessary to think about the degree of structural robustness for structural capacity in terms of a comprehensive sustainability indicator from the viewpoint of sustainability.

1 INTRODUCTION

Sustainability is defined as a concept that is taking into account environmental, economic, and social aspects, and is one of the key issues in a construction sector to be well considered in the 21st century. Sustainability is likely to be taken into account in construction work from the viewpoints of resources and energy consumption, initial construction and/or lifecycle cost, durability, and environmental impact [1].

CO2 emissions in construction account for about 30% of all emissions in Japan [2]. The most used construction material is cement concrete. Since the cement production emits large amounts of CO2, it is clear that the effects of CO2 emissions from this industry can no longer be ignored. In Japan, efforts to reduce greenhouse gas emissions have been promoted in various fields with the purpose of realizing a 3.8% reduction by FY 2020 compared to FY 2005 emissions [2].
Some design, execution, and maintenance codes recently have introduced the framework of the sustainability; for example, short but rather important specifications can be found in the fib Model Code 2010 [3]. During the lifecycle of a concrete structure, sustainability is generally considered with one or a few of these viewpoints, but it is not so easy to find the best solution among alternatives because no comprehensive sustainability indicator exists. For example, when the margin of safety is taken more, more resources and energy may be needed for construction and higher construction cost will be consequences. In other words, the sufficient balance among each sustainability indicator should be achieved.

The current JSCE Standard Specifications for Concrete Structures [4] instructs that structural planning must consider the influence of the construction of concrete structures on the environment, including on nature and society. However, it does not specify any performance requirements regarding the environment, nor does it make any clear statements regarding the reduction of environmental loads. When it comes to safety, performance requirements have been set, and verification is done to check whether the designed cross-sections and structures satisfy the performance requirements by using Equation (1), where \( S_d \) is the structural response, \( R_d \) is the resisting capacity, and \( \gamma_i \) is the structure factor.

\[
\gamma_i \frac{S_d}{R_d} \leq 1.0
\]  

The structure factor, \( \gamma_i \) is generally set at 1.0, and values exceeding 1.0 have almost never been discussed. It has been considered that the minimum safety requirement for a structure is satisfied when the structure factor is set at 1.0, and economic efficiency has been prioritized. Therefore, in the final verification of safety, the margin of safety that is obtained by using the structure factor exceeding 1.0 has not been considered. However, the structure factor relates directly to structural robustness and gives the final safety margins to structures against unforeseen failures. The Great East Japan Earthquake of 2011 provided us with a chance to reconsider the essential problems regarding the sustainability of infrastructure. The possibility of unexpectedly great tsunami loads acting on structures was clarified, and this raised the new issue of reconsidering how great the safety margin should be. Vigorous discussions continue how such unforeseen events should be taken into account for determining design actions and safety margins. Generally, greater safety margins incur increased costs and environmental loads. This is the collision between the social aspect of sustainability, which corresponds to the ample margin for safety, and the economic and environmental aspects of sustainability, which correspond to small costs and environmental loads. Therefore, essentially, the decision on the final safety margin should be made after examining the influence of the margin on the costs and environmental loads.

This study, as a simplest case for examination, chose the bending and shearing of reinforced concrete (RC) beams as structural parameters, quantified the relationship between safety margin as one of the social aspects and CO\(_2\) emissions as one of the environmental aspects and the costs as one of the economic aspects, and analyzed the influence of the safety margin on the other two sustainability indicators. Dimensions of RC beams, quantity of reinforcing bars, materials strengths, environmental conditions, and design service life are varied as parameters for design calculations as well as the structure factor. For each set of those parameters, initial construction cost, life-cycle cost including expecting restoration cost, CO\(_2\) emission are calculated. As a result, based on the parametric analyses, the structure factor does not have a considerable impact on the CO\(_2\) emission and the cost.
2 METHOD FOR SUSTAINABILITY EVALUATION

2.1 Outline

The procedure used in assessing sustainability indicators in this study is shown in Fig. 1. The details of the sustainability evaluation are explained below.

2.2 Structure type and environmental conditions

A simply supported RC beam was focused on. Structural verification was done on the RC beams whose spans \( (L) \) and rectangular cross-sectional dimensions such as height \( (h) \), width \( (b) \), and effective depth \( (d) \) were varied. For the calculation, a concentrated load of 200 kN was applied at the center of the span.

Chloride ion concentrations at the concrete surface \( (C_0) \), which was chosen as an index for the environmental condition where the beam was located, were changed in 2 levels: \( C_0 \) in Case 1 was set at 1.5 kg/m\(^3\), which was to simulate the normal outdoor environment of airborne chloride ions. That in Case 2 was set at 4.5 kg/m\(^3\), which simulates an environment in which steel members readily corrode.

The effective depth was changed based on the necessary concrete cover to the steel reinforcement embedded in the concrete so that corrosion of the reinforcement does not occur during the service life of the structure. The design service life was set at 50 years. The Fick’s second law of diffusion was used to determine the concrete cover. As the result, the necessary concrete covers in Case 1 were calculated less than 40 mm, but were set at 40 mm because that value is specified as the minimum value in the Standard Specifications for Concrete Structures [4]. The concrete covers in Case 2 were determined as the calculated minimum value that satisfies the durability, which varies by the materials and amount of reinforcement.

Table 1 summarizes the design parameters of each case.

![Flow of sustainability evaluation](image-url)

**Figure 1**: Flow of sustainability evaluation
Table 1: Design parameters of the studied RC beams

<table>
<thead>
<tr>
<th>Case</th>
<th>$C_0$ (kg/m$^3$)</th>
<th>L (m)</th>
<th>h (m)</th>
<th>b (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>1.5</td>
<td>5</td>
<td>0.5</td>
<td>0.35</td>
</tr>
<tr>
<td>1-2</td>
<td>1.5</td>
<td>10</td>
<td>0.6</td>
<td>0.45</td>
</tr>
<tr>
<td>1-3</td>
<td>1.5</td>
<td>15</td>
<td>0.75</td>
<td>0.5</td>
</tr>
<tr>
<td>1-4</td>
<td>1.5</td>
<td>20</td>
<td>0.75</td>
<td>0.6</td>
</tr>
<tr>
<td>2-1</td>
<td>4.5</td>
<td>5</td>
<td>0.5</td>
<td>0.35</td>
</tr>
<tr>
<td>2-2</td>
<td>4.5</td>
<td>10</td>
<td>0.6</td>
<td>0.45</td>
</tr>
<tr>
<td>2-3</td>
<td>4.5</td>
<td>15</td>
<td>0.75</td>
<td>0.5</td>
</tr>
<tr>
<td>2-4</td>
<td>4.5</td>
<td>20</td>
<td>0.75</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table 2: Mix proportions of concrete

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of cement</th>
<th>Comp. strength (MPa)</th>
<th>W</th>
<th>C</th>
<th>S</th>
<th>G</th>
<th>Ad</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC50</td>
<td>Ordinary Portland cement</td>
<td>30</td>
<td>157</td>
<td>328</td>
<td>783</td>
<td>1071</td>
<td>0.82</td>
</tr>
<tr>
<td>OPC40</td>
<td>Ordinary Portland cement</td>
<td>40</td>
<td>162</td>
<td>411</td>
<td>688</td>
<td>1081</td>
<td>1.03</td>
</tr>
<tr>
<td>BB50</td>
<td>Slag cement type B</td>
<td>30</td>
<td>156</td>
<td>332</td>
<td>764</td>
<td>1076</td>
<td>0.83</td>
</tr>
<tr>
<td>BB40</td>
<td>Slag cement type B</td>
<td>40</td>
<td>161</td>
<td>419</td>
<td>672</td>
<td>1081</td>
<td>1.05</td>
</tr>
<tr>
<td>FA50</td>
<td>Fly ash cement type A</td>
<td>30</td>
<td>149</td>
<td>290</td>
<td>840</td>
<td>1065</td>
<td>2.90</td>
</tr>
<tr>
<td>FA40</td>
<td>Fly ash cement type A</td>
<td>40</td>
<td>155</td>
<td>379</td>
<td>735</td>
<td>1081</td>
<td>3.79</td>
</tr>
</tbody>
</table>

2.3 Mix proportion of concrete

Table 2 presents the mix proportions of concrete examined. The mix proportions were set based on the proportions of commonly used ready-mixed concrete. Three types of cement were used such as ordinary Portland cement, blast furnace slag cement type B, and fly ash cement type A. Two levels of water-to-cement ratio were examined: about 50% and 40%. The nominal strengths of the ready-mixed concrete (30 MPa and 40 MPa) were used as the characteristic values of compressive strength.

2.4 Verification of safety in bending and shearing

The resulting bending moment and the resulting shear forces were calculated, which was indicated as $S_d$ in Equation (1) based on the general beam theory. The corresponding ultimate bending capacity and shear capacity that were indicated as $R_u$, were also calculated as per the standard verification methods for a linear RC structural member [4].

By increasing the structure factor $\gamma_i$ in Equation (1), the sectional capacity increases; as a result of the increase in sectional capacity, the design will have a cross section with a greater safety margin. Hereinafter, the safety margin expressed in the structure factor is called “the margin”. In this study, the margin was changed with 0.1 increments in between 1.0 and 1.5, and the influences of the margin on CO$_2$ emissions and initial construction cost were examined. The minimum amounts of tensile reinforcement and shear reinforcement were determined based on the structure factor $\gamma_i$ for the above examination.

2.5 Evaluation of environmental impact

An inventory analysis was done for CO$_2$ emissions. That analysis used data based on the
current the JSCE Recommendations [5]. The CO₂ emissions intensities of the component materials used in this examination are listed in Table 3. CO₂ emissions were determined by using these data and considering the amount of materials, which differ by the margin.

2.6 Evaluation of economic aspect

To evaluate the economic aspect, the costs of materials necessary for constructing the RC beam were assumed (hereinafter: “the cost”). The unit costs of ready-mixed concrete and the steel reinforcement (JIS SD 345 – yield stress of 345 MPa) are listed in Table 4. The data on unit costs were those for Sapporo as of August 2015. The amount of reinforcement and the volume of concrete necessary for the different margins vary. The costs for different margins were determined by multiplying the necessary amount and volume by each unit cost, and the costs for the two materials were summed.

<table>
<thead>
<tr>
<th>Material</th>
<th>CO₂ (kg-CO₂/t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary Portland cement</td>
<td>766.6</td>
</tr>
<tr>
<td>Slag cement type B</td>
<td>458.7</td>
</tr>
<tr>
<td>Fly ash cement type A</td>
<td>624.0</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.9</td>
</tr>
<tr>
<td>Sand</td>
<td>3.7</td>
</tr>
<tr>
<td>Chemical admixture</td>
<td>123.0</td>
</tr>
<tr>
<td>Steel reinforcement</td>
<td>767.4</td>
</tr>
</tbody>
</table>

**Table 3: CO₂ emissions when producing materials**

<table>
<thead>
<tr>
<th>Material</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC50</td>
<td>13 300 Yen/m³</td>
</tr>
<tr>
<td>OPC40</td>
<td>14 750 Yen/m³</td>
</tr>
<tr>
<td>BB50</td>
<td>13 500 Yen/m³</td>
</tr>
<tr>
<td>BB40</td>
<td>15 150 Yen/m³</td>
</tr>
<tr>
<td>FA50</td>
<td>13 300 Yen/m³</td>
</tr>
<tr>
<td>FA40</td>
<td>14 750 Yen/m³</td>
</tr>
<tr>
<td>Steel reinforcement</td>
<td>65 Yen/kg</td>
</tr>
</tbody>
</table>

**Table 4: Costs of concrete and steel reinforcement**

3 INFLUENCE OF STRUCTURAL PARAMETERS ON SUSTAINABILITY INDICATORS

3.1 Influence of the beam span

Figure 2 shows the rate of change in CO₂ emissions and the cost when the margin; that is the structure factor, was changed in the range from 1.0 to 1.5 for Cases 1-1 to 1-4. In the figure, to examine the influence of the span (i.e., the scale of the structure) on the two indicators, only the results using concrete with OPC50 are shown.

The influence of increase in the margin on CO₂ emissions and the cost is the greatest in Case 1-1 (L=5m). When the margin is set at 1.1, the CO₂ emissions increase by 2.3% and the cost increases by 3.4%. When the margin is set at 1.5, the CO₂ emissions and the cost increase
by 12.1% and 17.8%, respectively. In Case 1-3 ($L=15m$), the influence of the margin on the increases in CO$_2$ emissions and the cost is the smallest. For a margin of 1.1, the CO$_2$ emissions and the cost increase by 1.5% and 2.7%, respectively. When the margin is set at 1.5, the CO$_2$ emissions and the cost increase by only 7.9% and 13.7%, respectively. If we assume that about a 10% increase in CO$_2$ emissions is permissible for a certain increase in the margin, then the margin can be set at 1.4 in Case 1-1, in which the influence of the margin on CO$_2$ emissions is the greatest, and the margin can be set at 1.5 in Case 1-3, in which the influence of the margin on CO$_2$ emissions is the smallest. If we assume that about a 10% increase in the cost is permissible for a certain increase in the margin, then the margin can be set at 1.3 in Case 1-1, in which the influence of the margin on the cost is the greatest, and the margin can be set at 1.4 in Case 1-3, in which the influence of the margin on the cost is the smallest. The above examination clarifies that the influence of the span (i.e., the size of the structure and the dimensions of its members) should be considered when the margin is determined in designing.

(a) Rate of change in CO$_2$ emission  
(b) Rate of change in cost

Figure 2: Influence of beam span on CO$_2$ emission and the cost

3.2 Influence of cement type

Figure 3 shows the rate of change in CO$_2$ emissions and the cost when the margin is changed in a range from 1.0 to 1.5 for each mix proportion in Case 1-2.

The influence of increase in the margin is the greatest for the concrete that uses BB50. When the margin is set at 1.1, the CO$_2$ emissions increase by 3.0%, and when the margin is set at 1.5, the CO$_2$ emissions increase by 16%. The influence of increase in the margin is the smallest for the concrete that uses OPC40. When the margin is set at 1.1, the CO$_2$ emissions increase by only 1.6%, and when the margin is set at 1.5, the CO$_2$ emissions increase by only 8.4%. The influence of increase in the margin on the cost is the greatest for the concretes that use OPC50 and that use FA50. When the margin is set at 1.1, the cost increases by 3.1%, and when the margin is set at 1.5, the cost increases by 15.9%. The influence of increase in the margin on the cost is the smallest for the concrete that uses BB40. When the margin is set at 1.1, the cost increases by only 2.7%, and when the margin is set at 1.5, the cost increases by only 14.0%. If we assume that about a 10% increase in CO$_2$ emissions is permissible for a certain increase in the margin, then the margin can be set at 1.3 for the cases using BB50, in which the influence of the margin on CO$_2$ emissions is the greatest, and the margin can be set at 1.5 for the cases using OPC40, in which the influence of the margin on CO$_2$ emissions is
the smallest. If we assume that about a 10% increase in the cost is permissible for a certain increase in the margin, then the margin can be set at about 1.25 for the cases using BB40, in which the influence of the margin on the cost is the greatest, and the margin can be set at 1.3 for the cases using OPC50 and FA50, in which the influence of the margin on the cost is the smallest. From the above examination, it was found that the setting of the margin is also influenced by the cement type.

Figure 3: Influence of the type of cement on CO$_2$ emission and the cost

Figure 4: Influence of effective depth on CO$_2$ emission and the cost

3.3 Influence of the effective depth

Figure 4 shows the rate of change in CO$_2$ emissions and the cost when the margin is changed in a range from 1.0 to 1.5 for Cases 1-1 and 2-1. To examine the influence of the effective depth (concrete cover thickness), which affects the durability of the structure, the results of examination for cases in which only OPC50 is used are shown. When Cases 1-1 and 2-1 are compared, the influence of increase in the margin on the increases in CO$_2$ emissions and the cost are greater in Case 2-1 than those in Case 1-1. When the margin is set at 1.1 in Case 2-1, the CO$_2$ emissions and the cost increase by 2.8% and 4.0%, respectively. When the margin is set at 1.5, the CO$_2$ emissions and the cost increase by 13.5% and 19.6%, respectively. In Case 1-1, when the margin is set at 1.1, the CO$_2$ emissions and the cost
increase by 2.3% and 3.4%, respectively. When the margin is set at 1.5, the CO₂ emissions and the cost increase by only 12.1% and 17.8%, respectively.

If we assume that about a 10% increase in CO₂ emissions is permissible for a certain increase in the margin, then the margin can be set at about 1.35 in Case 2-1, in which the influence of the margin on CO₂ emissions is greater, and the margin can be set at 1.4 in Case 1-1, in which the influence of the margin on CO₂ emissions is smaller. If we assume that about a 10% increase in the cost is permissible for a certain increase in the margin, then the margin can be set at about 1.25 in Case 2-1, in which the influence of the margin on the cost is greater, and the margin can be set at 1.3 in Case 1-1, in which the influence of the margin on the cost is smaller.

From the above, it is found that the effective depth (concrete cover thickness) also has an influence on the setting of the margin.

4 CONCLUSIONS

This study examined the influence of span, cement type, and effective depth of concrete beams on increases in CO₂ emissions and the cost that accompany increases in the margin (structure factor). The following main conclusions were drawn in this study:

1. It was clarified that the setting of margin was influenced by span, cement type, and effective depth. The influence of each of these three parameters was able to be quantified.
2. When a 10% increase in the cost as a result of an increase in the margin is assumed to be permissible, the margin can be set at between 1.25 and 1.40.
3. When a 10% increase in CO₂ emissions as a result of an increase in the margin is assumed to be permissible, the margin can be set at between 1.35 and 1.50.

Based on the above results and from the viewpoint of sustainability, it is necessary to consider the balance between the margin, which is a safety indicator, and other sustainability indicators when designing RC structures in the future. The authors will make more detailed examinations on this theme.

ACKNOWLEDGMENTS

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REFERENCES

SELF – COMPACTING CONCRETE CO₂ UPTAKE
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Key words: Self – Compacting Concrete, Carbonation, CO₂ uptake, FTIR

Abstract. Among a number of the factors related to sustainability of concrete, the carbon footprint is one of the most discussed. Thus a lot of attention is paid to the opportunities of CO₂e emission reduction in global carbon footprint calculation. One of such possibilities is assuming the CO₂ sequestration in process of concrete carbonation. In the paper the results of three SCC mixtures carbonation, after 56, 112 and 168 days in two types of sample form, described in literature are presented. One type of form was a typical cubic sample and the other was a crushed concrete. A grain size of a crushed concrete was chosen to imitate a concrete rumble after a concrete demolish. Controlled conditions of 1% concentration of CO₂, 21°C temperature and 60% RH were kept in carbonation chamber in test period. After test in carbonation chamber samples were investigated with two test methods – a basic pH indicator and the Fourier Transformation Infrared Spectroscopy (FTIR) test. A compare of result from different test method in three test period are discussed in paper.

1 INTRODUCTION

As a sustainable development have become a significant factor, the concrete technology faced a challenging problem. The cement amount, that minimum content has been determined to provide concrete strength or to satisfy the requirement of given exposure class, on the other hand cause a higher carbon footprint, has a negative impact on the environment. Particularly the problem concerned to some special concrete types such as Self – Compacting Concrete (SCC), where cement content is higher, than in ordinary concretes. The solution to this
problem was development of a low – binder content SCC. According to Wallevick et al. [1], in this aspect Self – Compacting Concrete can be classified due to its binder content as it has been presented in Table 1.

<table>
<thead>
<tr>
<th>SCC type</th>
<th>Powder content (cement and binders)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rich SCC</td>
<td>≥ 550 kg / m³</td>
</tr>
<tr>
<td>Regular powder content SCC</td>
<td>450 – 550 kg/m³</td>
</tr>
<tr>
<td>Lean SCC</td>
<td>380 – 450 kg/m³</td>
</tr>
<tr>
<td>Green SCC</td>
<td>315 – 380 kg/m³</td>
</tr>
<tr>
<td>ECO SCC</td>
<td>≤ 315 kg / m³</td>
</tr>
</tbody>
</table>

Such a new approach to the problem was possible to develop a new type of a concrete admixtures, that allowed to highly reduce amount of water, but also to stabilize the concrete mixture.

However the powder content of the mixture, cannot be the indicator of concrete sustainability. One of the possibilities how to express concrete sustainability is the comparison of the carbon footprint. It is a sum of greenhouse gas emissions and removals in a product, expressed as CO₂ equivalents (CO₂e) and based on a life cycle assessment (LCA) [2]. In such analysis, it should be taken into consideration all stages of the life cycle of the product, including raw material acquisition, production, use and the end – of – life stage. As the analysis of concrete carbon footprint of the first stages (A1 – A3, which are raw materials to manufacturing) is well established, the other stages, especially end – of – life, requires an analysis of a various processes that occurs in concrete. One of them is a CO₂ absorption, that occurs in process of concrete carbonation. In stage of use (B1 – B5) scale of the process is insignificant, as it concerns in most cases only to exposed concrete surfaces. However in end – of – life stage, when concrete is demolished to a concrete rubble, process accelerates, as a exposed surface increases, what described Lagerblad [3].

The paper presents the results of the research on carbon dioxide uptake of three SCC mixtures in the process of carbonation measured with FITR spectroscopy analysis. Samples in the form of cubes and crushed concrete were placed in carbonation chamber for 56, 112 and 168 days under a controlled conditions. The study aims to evaluate a total carbon footprint based on applied test methods.

2 SELF – COMPACTING CONCRETE MIXTURES

As a carbon footprint depends on a number of factors – not only on a cement content, the Green SCC and its reference regular SCC described in literature [4] was compared to an ordinary architectural SCC applied and developed by Skanska S.A. The class of tested
concrete was the same and equals C30/37. Detail composition of concrete mixtures is presented in Table 2.

**Table 2: Self – Compacting Concrete mixtures**

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>Green SCC</th>
<th>Regular SCC</th>
<th>Skanska SCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete class</td>
<td>-</td>
<td>C30/37</td>
<td>C30/37</td>
</tr>
<tr>
<td>Cement kg/m³</td>
<td>285</td>
<td>340</td>
<td>400</td>
</tr>
<tr>
<td>Fly ash kg/m³</td>
<td>100</td>
<td>190</td>
<td>100</td>
</tr>
<tr>
<td>Sand kg/m³</td>
<td>700</td>
<td>652</td>
<td>744</td>
</tr>
<tr>
<td>Gravel kg/m³</td>
<td>1060</td>
<td>985</td>
<td>873</td>
</tr>
<tr>
<td>Admixtures % C.m.</td>
<td>2.24</td>
<td>3.15</td>
<td>1.00</td>
</tr>
<tr>
<td>Water kg/m³</td>
<td>175</td>
<td>160</td>
<td>185</td>
</tr>
</tbody>
</table>

% C.m. – percentage cement mass

Although a difference between Green SCC and Skanska SCC in cement and powder (cement and binder) content is evident, a type of applied cement causes a significant input on a total carbon footprint, at cradle to gate stage. Due to application of a blast furnace cement (CEM III/A 32,5) in Skanska SCC, total carbon footprint can be even lower than in Green SCC, what was described in [5] and is presented in Table 3.

**Table 3: Calculation of CO2 emission coefficient at cradle to gate stage**

<table>
<thead>
<tr>
<th>Components of the concrete mix</th>
<th>Green SCC</th>
<th>Regular SCC</th>
<th>Skanska SCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRODUCTION OF MATERIALS (kg CO₂e/m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement</td>
<td>265.06</td>
<td>316.21</td>
<td>231.2</td>
</tr>
<tr>
<td>Aggregates</td>
<td>3.27</td>
<td>3.04</td>
<td>3.00</td>
</tr>
<tr>
<td>Other components</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Total</td>
<td>268.38</td>
<td>319.30</td>
<td>234.25</td>
</tr>
<tr>
<td>TRANSPORTATION OF MATERIALS (kg CO₂e/m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement</td>
<td>9.78</td>
<td>11.66</td>
<td>13.72</td>
</tr>
<tr>
<td>Aggregates</td>
<td>30.19</td>
<td>28.08</td>
<td>27.74</td>
</tr>
<tr>
<td>Other components</td>
<td>2.29</td>
<td>3.73</td>
<td>2.34</td>
</tr>
<tr>
<td>Total</td>
<td>42.25</td>
<td>43.47</td>
<td>43.80</td>
</tr>
<tr>
<td>PRODUCTION OF CONCRETE (kg CO₂e/m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Electricity</td>
<td>2.08</td>
<td>2.08</td>
<td>2.08</td>
</tr>
<tr>
<td>Diesel fuel</td>
<td>0.83</td>
<td>0.83</td>
<td>0.83</td>
</tr>
<tr>
<td>Fuel oil</td>
<td>0.98</td>
<td>0.98</td>
<td>0.98</td>
</tr>
<tr>
<td>Total</td>
<td>3.89</td>
<td>3.89</td>
<td>3.89</td>
</tr>
<tr>
<td>TOTAL EMISSION COEFFICIENT (kg CO₂e/m³)</td>
<td>314.51</td>
<td>366.65</td>
<td>281.94</td>
</tr>
</tbody>
</table>
The amounts of emissions coefficients of Green SCC and regular SCC were taken from [3] and emissions coefficients of Skanska SCC were calculated according to [6]. Although presented results might be surprising, a total carbon footprint analysis requires the sum of emission coefficients of entire life cycle, also the end – of – life stage. At this stage, as it has been stated earlier, next to calculation of the emissions, also a carbon dioxide uptake has to be considered.

3 CO₂ UPTAKE RESEARCH

In research concrete samples were prepared according to recipes presented in Table 3. Achieved rheological properties of mixtures are presented in Table 4.

<table>
<thead>
<tr>
<th>Test method</th>
<th>Green SCC</th>
<th>Regular SCC</th>
<th>Skanska SCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump - flow mm</td>
<td>680/670</td>
<td>770/780</td>
<td>750/740</td>
</tr>
<tr>
<td>T₅₀ s</td>
<td>4,1</td>
<td>3,4</td>
<td>3,4</td>
</tr>
<tr>
<td>V – funnel s</td>
<td>5,1</td>
<td>4,9</td>
<td>3,7</td>
</tr>
</tbody>
</table>

Although a mixture properties varies from those described in [4], considering the purpose of the research and different manufacturer of admixtures and sources of aggregates, preceding differences are acceptable. After 90 days of curing in laboratory conditions, samples were divided into two sample types – normal cubic and crushed concrete rubble, which was reflecting concrete after demolition. Crushed concrete samples were made from a cylindrical samples broken with a testing machine and a hammer drill. Size of crushed concrete ranged from 8 – 31,5 mm, as it has been presented in Figure 1.

![Crushed concrete samples](image)
Next, all three series in both samples type were placed in to the carbonation chamber under controlled conditions for a 56 days, 112 days and 168 days, as it has been presented in Figure 2.

![Figure 2: Carbonation chamber](image)

Controlled conditions, in accordance with PN-EN 13295[6], were as follows: 1% concentration of CO$_2$, 21°C temperature and 60% RH. The carbonation chamber had a perforated trays, that were spinning around during whole investigation time. It ensured uniform conditions to all the samples placed in the chamber.

3.1 Carbonation depth measured with a pH indicator

According to [6], after designated period, samples were put out from the chamber and carbonation depth with a pH indicator was measured. After 56 days to examine a carbonation depth, apart form the phenolphthalein 10% solution, the Rainbow Test was applied. However a tint of the sample with the Rainbow Test was less pronounced than the results of the phenolphthalein 10% solution. Therefore only a phenolphthalein solution was used in the remainder of the study. Achieved results of carbonation depth after 56, 112 and 168 days are presented in Table 5 and Figure 3.
Table 5: Carbonation depth measured with a pH indicator

<table>
<thead>
<tr>
<th>Sample</th>
<th>56 days Rainbow Test</th>
<th>56 days phenolphthalein</th>
<th>112 days phenolphthalein</th>
<th>168 days phenolphthalein</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Green SCC</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubic sample</td>
<td>5 – 12 mm</td>
<td>8 – 12 mm</td>
<td>10 – 16 mm</td>
<td>17 – 20 mm</td>
</tr>
<tr>
<td>Crushed sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
</tr>
<tr>
<td><strong>Regular SCC</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubic sample</td>
<td>5 – 6 mm</td>
<td>4 – 6 mm</td>
<td>7 – 10 mm</td>
<td>9 – 12 mm</td>
</tr>
<tr>
<td>Crushed sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
</tr>
<tr>
<td><strong>Skanska SCC</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cubic sample</td>
<td>4 – 6 mm</td>
<td>6 – 8 mm</td>
<td>7 – 12 mm</td>
<td>8 – 14 mm</td>
</tr>
<tr>
<td>Crushed sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
<td>Whole profile of sample</td>
</tr>
</tbody>
</table>

Figure 3: Carbonation depth measured with a pH indicator
In all cases, irrespectively of the series and duration of carbonation, carbonation depth of the reference samples was 0 mm. Reference samples were kept next to carbonation chamber in conditions described in [6].

3.2 FTIR analysis

After determining the depth of carbonation process with a pH indicators, samples were tested with a Fourier Transformation Infrared Spectroscopy (FTIR).

This method bases on a measure of radiation intensity as a function of a wavelength. In a sample treated with infrared wave, each group resonates in its characteristic absorption frequencies. The frequency of the absorbed radiation matches the transition energy of the bond or group that vibrates. In theory with the FITR spectrum, it is possible to record a fingerprint of the bonding in molecules. FTIR analysis is a very powerful tool.

Nevertheless analysis of the concrete, which is a heterogeneous material, might be difficult. Investigated material was collected from the following surfaces; 0 – 5 mm, 5 – 10 mm, 10 – 15 mm, 15 – 20 mm, 20 – 25 mm for the cubic samples and from the surface and the middle of the grain’s crushed sample. The spectrometer used in the research, the Nicolet iS10, examined material in form of pastilles, made in press with a KBr matrix. Proportions of the pastilles was 0,027 g of examined concrete material and 0,150 g of KBr. Considering a number of the concrete’s constituents, it is essential to examine concrete matrix, where the process of the carbonation occurs. In literature there are number of publications [7,8], where described collection method of tested material was drilling the sample to designated depth or grinding whole surface. In test condition, when amount of tested material is vestigial and taken into account the fact, that in most of the cases, concrete volume consists in approximately 70%, of the aggregate, the chance to examine proper material is very low. Therefore an innovative test method was applied. From the sample fracture, tested material was grinded manually with a hard graver from the examined sample layer and sifted manually, to separate particles of sand from examined material. Comparative analysis of tested material with a spectrums from the device’s library have confirmed suitability of adopted test method. Achieved results are presented in Figure 4, Figure 5 and Figure 6. Also a reference sample were investigated to determine reference level of spectrum. For each test a background spectrum was measured.
Figure 4: FTIR transmission spectra of the Green SCC cubic and crushed concrete samples
Figure 5: FTIR transmission spectra of the Regular SCC cubic and crushed concrete samples
Figure 6: FTIR transmission spectra of the Skanska SCC cubic and crushed concrete samples
4 RESULTS AND CONCLUSION

As it has been described in literature [8, 9], the carbonation is represented by the transformation of the C═O bonds of CO₂ to C–O bonds in CaCO₃. Such a bond is represented in the wave number range of 1 410 – 1 510 cm⁻¹. In achieved results the above mentioned peak representing presence of CaCO₃ is evident. Assuming the peak of the reference sample, where a carbonation did not detected using the pH indicator, peaks below represents the progress of the carbonation. Although peaks below the reference samples exhibit a progress of carbonation process, differences are not that evident, as in researches described in literature [7,8] (in research material was collected from the 5 mm thick layers). It can be assumed that are in accordance with the results achieved with a carbonation depth measure with a pH indicator. Samples of crushed concrete exhibit the uniform carbonation through whole of the sample thickness. It was proved by the results of pH indicator (out of color range) and spectrum’s peaks at the same value for samples taken from the surface and middle of sample’s profile. Range of carbonation zone in adopted grain size of crushed concrete, confirms, that in the second life stage process of CO₂ uptake is crucial and has to be taken into account in total carbon footprint of the concrete, as it has been presented in [9, 10]. However the quantitative analysis of the FITR test of complex carbon footprint analysis needs a further study.

REFERENCES

STRENGTH DEVELOPMENT OF CONCRETE:
BALANCING PRODUCTION REQUIREMENTS
AND ECOLOGICAL IMPACT

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Key words: Concrete, mix design, cement replacing materials, early age strength, environmental impact

Abstract. The effective production of concrete structures requires adequate control of strength development in order to realise the scheduled production cycles. Demoulding of elements can take place only when sufficient strength is gained and the production cycle has to be maintained with seasonal changes of temperature. The use of Portland Cement promotes high early age strengths, but comes with a relative high impact on the environment since decarbonation and a high energy demand come along with cement production. Supplementary cementitious materials have been widely applied to improve the sustainability of concrete but the rate of early age strength development often is compromised to some degree.

An experimental study was executed with the aim to maintain a similar strength level at early age but lowering the content of Portland clinker in concrete. Parameters of the study were the replacement level of Portland Cement, the curing temperature and the use of strength accelerator. At a comparable workability level, specimens were produced of which the compressive and flexural strengths were determined at different ages after casting. The Dutch CUR tool ‘Green Concrete 3.2’ was used to determine the environmental impact of the mixtures. The results show that concrete can have a much lower impact on the environment without compromising on the production conditions. Quantifying the trade-off between the use of Portland Cement and other mixture components and adding heat in the process is important information in order to balance production requirements and the ecological impact of concrete structures.

1 INTRODUCTION

The advantages of concrete are freedom of shape, possibilities to integrate other functions and components, to build structures with limited maintenance costs, ease of use and very high
durability. A significant reduction of the environmental impact can convince owners to choose concrete rather than other building materials. Ecological aspects like the protection of resources, material recycling and a long technical life-time of structures are important for our society; owners and contractors are aware of not only the benefits related to the environment by producing environment-friendly structures but many include environmental targets in their business strategy. With the production of Ordinary Portland Cement (OPC) comes high CO2-emissions. A very effective way to reduce the environmental impact of concrete is to replace OPC by alternative binders. For example, the hydraulic activity of ground granulated blast furnace slag (GGBS) was already known in 1862 [1]. In the Netherlands, GGBS-cement is a common cement type, which has been successfully applied in many large-scale infrastructure projects. In other countries, such as Norway, fly ash (FA) is a more common additive in concrete. Considerable volumes of OPC have already been replaced in concrete by supplementary materials, but the effect is compensated by the still rising demand for concrete worldwide. In order to improve the sustainability of products and structures made with concrete, alternative solutions have to be developed. Such solutions are also required from the economic point of view in order not to lose market share to other materials. General agreement has to be achieved concerning the assessment method and quantification of the environmental impact; an example of impact indicator is the Environmental Product Declaration (EPD). The development of such ‘instruments’ requires a coordinated and cooperative approach of different countries.

Production efficiency is a distinct characteristic and requirement of the precast industry and many in-situ cast concrete structures. Due to its chemical composition and hydraulic reaction, the use of OPC ensures relatively high early age strengths. In order to compensate for a lower strength at early age caused by clinker replacement, an appropriate curing regime and/or a hardening accelerator have been applied. This study was executed in order to determine the potential for a reduction in environmental footprint by cement replacement without compromising on the (early age) strength development. The energy-efficiency of concrete was quantified with the parameter ‘relative strength cost’ that relates the environmental impact and the compressive strength of concrete. The discussion on the environmental impact of concrete based on only the mixture composition might seem isolated not taking into account the total life cycle of a structure but it indicates the potential for an optimization on the material level.

2 ENVIRONMENTAL IMPACT QUANTIFICATION

Large differences can be identified worldwide with regard to the methods applied and progress in quantification of the environmental impact of the construction sector and the recognition thereof. In the future, it probably will be common practice to include such approaches in tenders and contracts. A life cycle analysis has to consider many aspects. In order to compare buildings or concrete structures it is necessary to weight different aspects with regard to environmental impact and to express them in the same unit: costs in Euro being an obvious choice. According to the Dutch law ‘Bouwbesluit’ the depletion of raw materials and emission of greenhouse gases has to be determined for new buildings and renovation
projects. A national database has been established in the Netherlands [2], which can be applied to quantify the environmental impact of infrastructures. In addition, the CUR-tool ‘Green Concrete’ [3] was developed by SGS Intron for CUR Commission B-88 and it is the base to produce EPD’s. The tool aims at users who want to determine the environmental impact of structures, structural elements and products made with concrete. It is also a tool to optimize concrete and concrete structures with regard to the environmental impact. The user chooses building materials and processes from a database; with own data, the database can be extended. For the calculation of the environmental cost parameter MKI (Dutch: Milieu Kosten Indikator) eleven environmental impact categories from LCA data in a building product EPD are taken into account with conversion factors that reflect their relative effect. Table 1 lists the eleven parameter and accompanying conversion factors. The outcome of MKI-calculations are costs in Euro/unit. The MKI is a factor already taken into account in the Netherlands for the tender of community works as well as for office buildings.

Table 1: Eleven environmental impact categories and MKI conversion factors

<table>
<thead>
<tr>
<th>Nr.</th>
<th>Impact category</th>
<th>Abbreviation</th>
<th>Unit</th>
<th>Conversion factor [Euro/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Abiotic Depletion, fuels</td>
<td>ADP</td>
<td>kg Sb eq</td>
<td>0.16</td>
</tr>
<tr>
<td>2</td>
<td>Abiotic Depletion, minerals</td>
<td>ADP</td>
<td>kg Sb eq</td>
<td>0.16</td>
</tr>
<tr>
<td>3</td>
<td>Acidifying Pollutants</td>
<td>AP</td>
<td>kg SO2 eq</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>Eutrophication Potential</td>
<td>EP</td>
<td>kg PO4 eq</td>
<td>9</td>
</tr>
<tr>
<td>5</td>
<td>Freshwater Aquatic Eco-Toxicity Potential</td>
<td>FAETP</td>
<td>kg 1,4-Dichlorobenzene eq</td>
<td>0.03</td>
</tr>
<tr>
<td>6</td>
<td>Global Warming Potential, 100 years</td>
<td>GWP 100 Y</td>
<td>kg CO2 eq</td>
<td>0.05</td>
</tr>
<tr>
<td>7</td>
<td>Human Toxicity</td>
<td>HTP</td>
<td>kg 1,4-Dichlorobenzene eq</td>
<td>0.09</td>
</tr>
<tr>
<td>8</td>
<td>Marine Aquatic Eco-Toxicity Potential</td>
<td>MAETP</td>
<td>kg 1,4-Dichlorobenzene eq</td>
<td>0.0001</td>
</tr>
<tr>
<td>9</td>
<td>Ozone Depletion Potential</td>
<td>ODP</td>
<td>kg CFC11 eq</td>
<td>30</td>
</tr>
<tr>
<td>10</td>
<td>Photochemical Ozone Creation Potential</td>
<td>POCP</td>
<td>kg Ethylene eq</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>Terrestrial Eco-Toxicity Potential</td>
<td>TETP</td>
<td>kg 1,4-Dichlorobenzene eq</td>
<td>0.06</td>
</tr>
</tbody>
</table>

The Global Warming Potential (GWP) is the environmental impact category often referred to as the carbon footprint. Table 2 shows the weight of CO2 produced per unit of concrete component according to [3].

Table 2: Conversion factors GWP and assumed transport distances for concrete components

<table>
<thead>
<tr>
<th>Component</th>
<th>Abbreviation</th>
<th>Type</th>
<th>Reference in database</th>
<th>Distance</th>
<th>GWP [kg CO2 eq]</th>
<th>Per unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I 52.5 R</td>
<td>CEM I</td>
<td>Binder</td>
<td>SBK CEM I-NL</td>
<td>186 (T)</td>
<td>8.2E-1</td>
<td>kg</td>
</tr>
<tr>
<td>CEM III 52.5</td>
<td>CEM III</td>
<td>Binder</td>
<td>CEM III-A NL</td>
<td>186 (T)</td>
<td>4.4E-1</td>
<td>kg</td>
</tr>
<tr>
<td>GGBS, Orcem</td>
<td>GGBS</td>
<td>Binder</td>
<td>SBK Hoogovenslakke</td>
<td>150 (T)</td>
<td>1.9E-2</td>
<td>kg</td>
</tr>
<tr>
<td>Fly ash</td>
<td>FA</td>
<td>Binder</td>
<td>Poederkoovlieges c2</td>
<td>150 (T)</td>
<td>3.3E-3</td>
<td>kg</td>
</tr>
<tr>
<td>Limestone, powder</td>
<td>LS</td>
<td>Binder</td>
<td>Kalksteenmeel (BE)</td>
<td>150 (T)</td>
<td>2.2E-2</td>
<td>kg</td>
</tr>
<tr>
<td>Limestone, gravel</td>
<td>Aggregate</td>
<td>Kalksteen (BE)</td>
<td>230 (S)</td>
<td>2.3E-3</td>
<td>kg</td>
<td></td>
</tr>
<tr>
<td>River sand</td>
<td>Aggregate</td>
<td>Zand (D)</td>
<td>200 (S)</td>
<td>3.8E-3</td>
<td>kg</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>Water</td>
<td>Leidingwater</td>
<td>0</td>
<td>3.4E-4</td>
<td>kg</td>
<td></td>
</tr>
<tr>
<td>Accelerator</td>
<td>ACC</td>
<td>Admixture</td>
<td>Plastificerder</td>
<td>150 (T)</td>
<td>3.9E-1</td>
<td>kg</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>SUP</td>
<td>Admixture</td>
<td>Superplastificerder</td>
<td>150 (T)</td>
<td>7.2E-1</td>
<td>kg</td>
</tr>
<tr>
<td>Ship (S)</td>
<td>Transport</td>
<td>Binnenvaartschip</td>
<td></td>
<td>4.6E-2</td>
<td>km</td>
<td></td>
</tr>
<tr>
<td>Truck (T)</td>
<td>Transport</td>
<td>Truck, empty retour</td>
<td></td>
<td>1.3E-1</td>
<td>km</td>
<td></td>
</tr>
</tbody>
</table>

The applied very fine OPC CEM I 52.5 R 7000 requires more grinding to reach the higher fineness compared to the reference Portland cement CEM I 52.5 R. No detailed information
was available with regard to the production and an overall 15% increase in CO₂-emissions was accounted for this binder type for additional grinding.

3 EXPERIMENTAL SET-UP

The experimental study, described hereafter, was a part of a larger program [4] and consisted of producing and testing fifteen mortars, which had a comparable flowability and which were tested at different ages in the hardened state (flexural and compressive strengths). The study was based on a 100% Portland Cement (CEM I 52.5 R) reference mixture. The applied OPC has a high early strength and it is often applied in prefabrication for example to produce prefabricated prestressed elements. Table 3 shows the composition of the binder in mortar; OPC cement was replaced by weight. The water-cement ratio of the reference mixture (100% OPC) was 0.45; the same water dosage was applied for all mixtures (the water-binder ratio always was 0.45). The volume percentage of the sand in mortar amounts 48.5 Vol.-%. In order to enhance the early age strength development, four hardening accelerators were selected and tested: BASF Master X-seed 100 (B), Sika Rapid C-100 (S), Mapefast CF/L (M) and Demula ACCEL IF (D); the dosage was determined based on the product sheets and was fixed at 80% of the optimal prescribed dosage, equal to 0.32, 2.40, 1.58 and 2.40 kg for each 100 kg binder material, respectively.

Table 3: Binder composition in mortar [Weight-% of binder]

<table>
<thead>
<tr>
<th>Nr.</th>
<th>CEM I</th>
<th>CEM I+</th>
<th>CEM III</th>
<th>Slag</th>
<th>LS</th>
<th>FA</th>
<th>ACC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td>B</td>
</tr>
<tr>
<td>5</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>6</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td>M</td>
</tr>
<tr>
<td>7</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td>D</td>
</tr>
<tr>
<td>8</td>
<td>70</td>
<td>85</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>9</td>
<td>28.6</td>
<td>71.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>10</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>70</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>85</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>13</td>
<td>15</td>
<td></td>
<td></td>
<td>85</td>
<td></td>
<td></td>
<td>B</td>
</tr>
<tr>
<td>14</td>
<td>20</td>
<td></td>
<td></td>
<td>75</td>
<td></td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>15</td>
<td>15</td>
<td></td>
<td></td>
<td>85</td>
<td></td>
<td></td>
<td>S</td>
</tr>
</tbody>
</table>

The mineral composition of the binders (with the exception of the limestone powder) is given in Table 4. The dosage of superplasticizer (BASF: Glenium 51) of the mortars was adjusted in order to obtain a flow spread of 250±20 mm. Since several mixtures were also tested as concretes, a similar paste consistency assured that the test results are not significantly affected by differences in workability. Two types of Portland Cement were tested: CEM I 52.5 R HES (Holcim) and CEM I 52.5 R 7000 (Heidelberg); the latter cement was much finer. Besides, limestone powder OMYA Betocarb and fly ash class F were applied.
Table 4: Composition of binders and cement replacing materials [Weight-%]

<table>
<thead>
<tr>
<th>Component [weight-%]</th>
<th>CEM I 52.5 R HES</th>
<th>CEM I 52.5 R 7000</th>
<th>CEM III/A 52.5 N</th>
<th>GGBS Orcem</th>
<th>Fly ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>62.1</td>
<td>65.5</td>
<td>53.6</td>
<td>38.6</td>
<td>3.0</td>
</tr>
<tr>
<td>SiO₂</td>
<td>17.3</td>
<td>22.6</td>
<td>26.3</td>
<td>29.3</td>
<td>54.2</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.5</td>
<td>3.9</td>
<td>7.0</td>
<td>11.6</td>
<td>23.5</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.8</td>
<td>1.4</td>
<td>1.6</td>
<td>1.5</td>
<td>7.9</td>
</tr>
<tr>
<td>MgO</td>
<td>0.8</td>
<td>0.8</td>
<td>-</td>
<td>8.0</td>
<td>1.9</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.4</td>
<td>0.2</td>
<td>-</td>
<td>0.2</td>
<td>1.1</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.7</td>
<td>0.7</td>
<td>-</td>
<td>0.5</td>
<td>3.4</td>
</tr>
<tr>
<td>Na₂O-equivalent</td>
<td>3.4</td>
<td>3.4</td>
<td>3.6</td>
<td>0.02</td>
<td>0.9</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>0.03</td>
<td>&lt;0.1</td>
<td>0.07</td>
<td>0.007</td>
<td>0.003</td>
</tr>
<tr>
<td>MnO₅O₇</td>
<td>3.4</td>
<td>3.4</td>
<td>3.6</td>
<td>0.02</td>
<td>0.9</td>
</tr>
<tr>
<td>S²</td>
<td>0.4</td>
<td>0.7</td>
<td>0.2</td>
<td>0.8</td>
<td>1.3</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The mortars were prepared with a 5 litres Hobart mixer according to the following procedure: Binder materials and water were added first in the bowl; the mixing starts at a low speed of 145 rounds per minute (rpm) for 60 s. Afterwards, sand is added steadily during the next 30 s and mixing continues for 30 s at a mixing speed of 145 rpm. Then, the mixer was stopped for 90 s, during the first 30 s, the mortar adhering to the wall is re-added to the cement paste by making use of a scraper. Then a rest period of 60 s was kept. During this rest period, the superplasticizer is added to the cement paste. At the end, the paste is again mixed at a speed of 145 rpm for a period of 90 s.

Directly after mixing the flow spread was determined according to NBN EN 12350-8 [5] (dimensions of cone: height: 60 mm; upper/lower diameters: 70/100 mm). The flow test was executed on a smooth wooden plate (formwork ‘betonplex’), which was moistened just before filling the cone. When the target flow of 250±20 mm was not reached, an extra amount of superplasticizer was added to the cement paste. Then, the paste was remixed for 60 s after which a rest period of 60 s is applied before conducting the flow test again. This step was repeated until the flow spread was within the acceptance range; the maximum number of remixing steps was two. With the required flow spread, test specimens were cast. A mould consists of three prisms (height/width: 40 mm; length: 160 mm). The mortar was agitated for 60 s by making use of a jolting apparatus. The temperature effect was studied by curing at three different levels (20/35/50°C). The mortars that were cured at room temperature were stored in a room with an average relative humidity of 93±5% and a temperature of 20±2 °C. Heat treatment during the first 18 hours was executed by storing moulds in a container filled with water for a steam curing cycle. The moulds were placed on a sieve plate above the water level. The water was gradually heated at heating rates equal to 20 °C/h and 24 °C/h for temperatures of 35 °C and 50 °C, respectively. After 18 hours of heat treatment and until testing at 28 days, the prisms were cured in a climate room with a temperature of 20±2 °C and a relative humidity of 93±5%. Flexural and compressive strengths of the prisms were determined according to NBN EN 196-1 [6]; the applied testing machine was a ‘Walter+baiag’ compression machine. After the execution of the flexural test (three-point bending) on
prisms the specimen broke into two parts which were both subjected afterwards to a compression test. The compressive and flexural strengths of the mortars were determined after 18 hours of curing and for an age of 28 days.

4 RESULTS AND DISCUSSION

4.1 Mechanical testing

The compressive strengths of 15 mortars in function of the curing temperature are shown in Figures 1 and 2. The highest strength level at 28 days was obtained for all curing temperatures with a mixture of the fine Portland cement (CEM I+), the slag cement and S-accelerator (M12); with the same combination the highest compressive strength was obtained after 18 hours and for a curing temperature of 50°C. The lowest strength level at 28 days for all curing temperatures was found for mixtures produced with a 30% replacement by fly ash and with or without an accelerator (M10&M11); for M10 (20°C) no experimental values could be determined after 18 hours, since the strength was very low. With the exception of Mixtures M3 and M13, the differences in strength after 28 days for the same mixture but different curing temperatures were moderate; M3 and M13 showed relatively high differences in compressive strengths obtaining the highest strengths for curing temperatures of 20°C. At 28 days, the compressive strength was often slightly higher for the 20°C curing cycle, a result that can have consequences and has to be taken into account for the mix design when applying the same mixture in different curing regimes.

![Figure 1: Comparison of compressive strengths (28 days after casting)](image-url)
Focussing on the early age strength (Figure 2), higher strengths (compared to M1) were obtained with Mixture M2 for 20°C (10% limestone powder replacement), replacement by finer cement (M8&M12) and the replacement of OPC with CEM III/GGBS and use of S-accelerator (M9&M15). Especially, the last combination (M15) realised a significant reduction in environmental footprint. Limestone powder has a beneficial influence on the C₃S-hydration during the first 15 hours [7,8]. The dilution effect caused by the lower cement content in limestone powder containing pastes is overruled by the filler action and the additional nucleation sites generated with the limestone powder addition. Technical and economic benefits were realised with M2, but it was only a moderate benefit with regard to the environmental impact. In case of a 15% limestone powder replacement, the dilution effect could have compensated the early age compressive strength benefit, which was the reason that the finer cement type CEM I 52.5R 7000 having a fineness of 740 m²/kg was applied for M8. The replacement of CEM I by GGBS (30% replacement level) reduced the early age strength independent of the curing temperature (M3); the addition of an accelerator (M4-M7) reduced the difference compared to the reference mixture. The largest reduction of the difference was obtained with the S-accelerator (M5) for all curing temperatures; at 20°C a comparable strength level with the reference mixture was obtained. Due to the higher activation energy of slag-blended mortars, the influence of heat curing on their strength development is more pronounced at 50°C compared to Portland Cement containing mortars. As a consequence, the difference in strength at higher temperatures is less pronounced. Mixtures M9 and M15 can be considered most optimal when taking into account environmental impact, economic aspects and level of compressive strength. Since Mixture M15 realises a higher cement replacement (35% slag content compared to 30% in case of Mixture M9) it is considered to be preferable. Heat curing also showed a pronounced effect on the strength of the reference mixture; the difference between 35°C and 50°C was small. The binder composition of concrete affects the strength development in the curing range of 20°C to 50°C hereby providing possibilities for
the optimization of the curing regime. The largest strength difference for the three curing regimes was obtained for Mixture M12 combining the effects of accelerator, OPC fineness and higher activation energy.

Table 5 shows the test results with regard to flexural strengths after 18 hours of curing and 28 days after casting. For both compressive and flexural strengths similar trends were observed (the results that were higher than the flexural strength of M1 are bold and underlined). Mixture M12 performed very good with regard to compressive and flexural strengths at different ages and with changing curing conditions. The most optimal mixture (M15) with regard to performance, economy and environmental impact containing 35% of slag by a replacement with CEM III/A 52.5 N had early age compressive strengths at 18 hours of +24%, -15% and +1% for 20°C, 35°C and 50°C and flexural strengths at 18 hours of +29%, +8% and -9% for 20°C, 35°C and 50°C compared to the reference mixture M1, respectively. The flexural strengths of M15 at 28 days were lower for all curing temperatures compared to M1. The results in Table 5 show that the flexural strength at 28 days is less critical for mortars containing supplementary binders compared to 18 hours of curing - more mixtures reached at least the same strength level.

Table 5: Flexural strengths 18 hours and 28 hours after casting

<table>
<thead>
<tr>
<th>T-curing</th>
<th>20 °C</th>
<th>35 °C</th>
<th>50 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture</td>
<td>18 h</td>
<td>28d</td>
<td>18 h</td>
</tr>
<tr>
<td>M1</td>
<td>3.40</td>
<td>6.82</td>
<td>5.38</td>
</tr>
<tr>
<td>M2</td>
<td>3.85</td>
<td>5.88</td>
<td>5.04</td>
</tr>
<tr>
<td>M3</td>
<td>2.04</td>
<td>8.15</td>
<td>4.52</td>
</tr>
<tr>
<td>M4</td>
<td>2.95</td>
<td>8.13</td>
<td>5.48</td>
</tr>
<tr>
<td>M5</td>
<td>3.84</td>
<td>8.63</td>
<td>5.27</td>
</tr>
<tr>
<td>M6</td>
<td>3.29</td>
<td>7.40</td>
<td>4.84</td>
</tr>
<tr>
<td>M7</td>
<td>2.61</td>
<td>5.85</td>
<td>4.50</td>
</tr>
<tr>
<td>M8</td>
<td>4.91</td>
<td>6.71</td>
<td>4.97</td>
</tr>
<tr>
<td>M9</td>
<td>4.57</td>
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<td>4.99</td>
</tr>
<tr>
<td>M10</td>
<td>1.58</td>
<td>4.91</td>
<td>4.19</td>
</tr>
<tr>
<td>M11</td>
<td>2.62</td>
<td>4.25</td>
<td>4.39</td>
</tr>
<tr>
<td>M12</td>
<td>4.27</td>
<td>7.11</td>
<td>5.82</td>
</tr>
<tr>
<td>M13</td>
<td>2.77</td>
<td>7.02</td>
<td>6.31</td>
</tr>
<tr>
<td>M14</td>
<td>2.60</td>
<td>7.42</td>
<td>5.38</td>
</tr>
<tr>
<td>M15</td>
<td>4.38</td>
<td>4.58</td>
<td>5.81</td>
</tr>
</tbody>
</table>

4.2 Environmental impact

The environmental impact was assessed by making use of the CUR-tool ‘Green Concrete 3.2’. The level of CO₂-emission as well as the environmental cost index MKI were determined. The total MKI score and CO₂-emmissions of the mixtures are composed of three individual contributions: production (use of components), transport of components and demolition of elements; transport of prefabricated elements and the service phase are not taken into account for the calculations. The energy required for the curing of the mixtures was not considered in the calculations and the strengths were compared for the same curing regime. As Table 2 shows, the largest reduction in Global Warming Potential can be achieved
with the replacement of Portland clinker since the emission value as well as the dosage in concrete are high. GGBS and FA have comparable and low emission levels. For the interpretation of the results it has to be considered that there is not (yet) a general consensus about the exact conversion values resulting in an uncertainty with regard to the calculation of MKI.

The Global Warming Potential of the reference mortar (M1) is 586 kgCO₂/m³mortar; the MKI is 45.4 Euro/m³mortar. Relative to the reference mortar, the most CO₂-reduction was realised with Mixtures M13&M14 (58% CO₂ of M1; 42% reduction) and M12&M15 (59% CO₂ of M1). Mixture M15 has a comparable cement replacement level, but the environmental cost index score is slightly higher due to the higher required dosage of the S-accelerator compared to the B-accelerator. The relative share of each component to the total MKI is shown in Table 6. The highest reduction in MKI (Table 6) was again achieved for Mixtures M13&M14 (66% of MKI of M1), whereas this were 69% relative to the MKI of M1 for M15. Differences in compositions are only reflected in a varying MKI in terms of production. For M15, the MKI related to the transport and the demolishment contribute 11% and 2% of the total MKI, respectively.

### Table 6: Relative contribution to the environmental cost index score MKI of the production, transport of raw materials and demolishing phase

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Production</th>
<th>Transport</th>
<th>Demolishment</th>
<th>Total</th>
<th>% to ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>41.26</td>
<td>3.42</td>
<td>0.68</td>
<td>45.36</td>
<td>100.00</td>
</tr>
<tr>
<td>M2</td>
<td>37.19</td>
<td>3.36</td>
<td>0.68</td>
<td>41.23</td>
<td>90.89</td>
</tr>
<tr>
<td>M3</td>
<td>29.78</td>
<td>3.28</td>
<td>0.67</td>
<td>33.73</td>
<td>74.37</td>
</tr>
<tr>
<td>M4</td>
<td>30.01</td>
<td>3.28</td>
<td>0.67</td>
<td>33.97</td>
<td>74.88</td>
</tr>
<tr>
<td>M5</td>
<td>31.22</td>
<td>3.28</td>
<td>0.68</td>
<td>35.18</td>
<td>77.55</td>
</tr>
<tr>
<td>M6</td>
<td>30.73</td>
<td>3.28</td>
<td>0.67</td>
<td>34.69</td>
<td>76.48</td>
</tr>
<tr>
<td>M7</td>
<td>31.22</td>
<td>3.28</td>
<td>0.68</td>
<td>35.18</td>
<td>77.55</td>
</tr>
<tr>
<td>M8</td>
<td>38.75</td>
<td>3.34</td>
<td>0.67</td>
<td>42.76</td>
<td>94.28</td>
</tr>
<tr>
<td>M9</td>
<td>29.68</td>
<td>3.39</td>
<td>0.67</td>
<td>33.75</td>
<td>74.40</td>
</tr>
<tr>
<td>M10</td>
<td>28.32</td>
<td>3.20</td>
<td>0.66</td>
<td>32.18</td>
<td>70.94</td>
</tr>
<tr>
<td>M11</td>
<td>29.71</td>
<td>3.20</td>
<td>0.66</td>
<td>33.57</td>
<td>74.00</td>
</tr>
<tr>
<td>M12</td>
<td>27.77</td>
<td>3.43</td>
<td>0.67</td>
<td>31.87</td>
<td>70.27</td>
</tr>
<tr>
<td>M13</td>
<td>25.96</td>
<td>3.39</td>
<td>0.67</td>
<td>30.02</td>
<td>66.19</td>
</tr>
<tr>
<td>M14</td>
<td>25.81</td>
<td>3.37</td>
<td>0.67</td>
<td>29.85</td>
<td>65.81</td>
</tr>
<tr>
<td>M15</td>
<td>27.25</td>
<td>3.43</td>
<td>0.67</td>
<td>31.35</td>
<td>69.11</td>
</tr>
</tbody>
</table>

As the strengths of the mixtures were not the same, in the following the ‘relative strength cost’ (the normalized environmental impact-to-strength ratio) is discussed. Since the early age strength level is especially important for prefabrication and the replacement of clinkers has a pronounced effect on this aspect, both environmental indicators are related hereafter to the 18 hours compressive strength. Figure 3 summarizes the results with regard to the ratio Global Warming Potential to early age compressive strength (Figure 4: ratio MKI to early age compressive strength). Significant differences are obtained; Mixtures M8, M9 and M12-M15 had the lowest Global Warming Potential and MKI. In several cases the ratio relative to the reference mixtures was below 50%. The relative difference between GWP and MKI was small, indicating the pronounced effect of GWP has on the environment.
In practice, concrete contains coarse aggregates and less cement paste than mortar. For the present study the paste volume was assumed to be 30.5 Vol.-% compared to the 51.5 Vol.-% with which the mortars were prepared. Due to the higher cement paste volume of a cubic meter mortar, MKIs and CO₂-emissions of mortars are higher compared to the equivalent concretes. With the assumed paste volume of 30.5 Vol.-% one cubic meter of the reference
concrete contains 397 kg CEM I 52.5 R, whereas one cubic meter of reference mortar contains 668 kg CEM I 52.5 R. The effect of the mixture composition on MKI and GWP caused by the production of one cubic meter of concrete and mortar for Mixture M1&M15 are depicted in Figure 5.

![Figure 5: Global Warming Potential (a, left) and MKI score (b, right) per cubic meter of the reference concrete/mortar (M1) and the slag concrete/mortar (M15)](image)

5 CONCLUSIONS

In an experimental study fifteen mortars with different binder compositions were tested with regard to their compressive and flexural strengths at different ages. The environmental impact was assessed and related to the compressive strength as a performance indicator. Based on the study the following conclusions can be drawn:

- The replacement of cement by fly ash decreased the compressive strength at 18 hours and 28 days. The compressive strength of a Portland cement-limestone powder combination was similar at both ages for a limestone replacement of 10%, the highest strength compared to the reference was obtained at 20°C.

- In order to realise a large reduction in environmental impact, ground granulated blast-furnace slag was tested as a cement replacing material. The addition of the hardening accelerator SIKA Rapid C-100 (containing of C-S-H nanoparticles) (partially) compensated the loss in early age strength. Due to the addition of a hardening accelerator, up to 35% of Portland cement can be replaced by blast-furnace slag at similar strength levels for different curing regimes.

- Mortars were produced at similar strength levels compared to a reference mixture containing only Portland Cement with a reduction of about 40% in Global Warming Potential and a 30% decrease of the environmental impact parameter MKI.
The accumulation of different environmental impact parameters in a single number is a valuable method for the optimization of concrete.

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Sustainability and Human Habitat

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Abstract. The idea of sustainability calls into question the way the human being inhabits the earth. This article is a conceptual contribution on how to make the human habitat more sustainable. Taken the Heidegger’s conception of “dwelling” as starting point, a new form of understanding the organization of the human space is proposed. The crisis of human housing responds partly to the notion of space, which is seen as a “place” (locus) and its organization – the urbanization - is resolved in terms of “localization”. Many challenges of the city organization come from reducing the organization of human space to the organization of physical space. One of them is given by the fact that physical space cannot be shared, can only be divided. Another challenge is that physical space is finite. This leads to the conflict with the human aim to development and growth, and opens the typical dilemma in approaching sustainability, between “preservation” and “development”. The aim of this article is to propose a new view for the organization of human dwelling, which overcomes the physic-based conception of space. In doing so, the meaning of human “habitat” as “space for developing capabilities” (habitus) will be recovered, and the implications for the organization of the human habitat considered. The human action is not only “consumer” of resources. Human life is a process of improvement of possibilities as well. The human house is the city (polis). However, the city is above all a practical and social space constituted in a temporal process of learning and capabilities development. This habitat as capabilities-based space can be shared and its development is infinite so that a way is opened for overcoming the logic of competence and the conflict of sustainability with development. Finally, three focal points of the urbanization of human habitat are proposed.

1 INTRODUCTION
The idea of sustainability calls into question the way the human being inhabits the earth. This article is a conceptual contribution on how to make human habitat more sustainable. In his well-known lecture delivered in 1951, Martin Heidegger associated the habitat problem with a more detailed reflection on the meaning of the human “dwelling”\(^1\). The organisation of human space is today in crisis and the human habitat crisis is connected to a human dwelling crisis\(^2\). Animals have no problems with their dwellings, yet humans do\(^3\). Homelessness is characteristic of modernity\(^4\).

Nietzsche was one of the first thinkers to question the existence of a place for humans. In *Also sprach Zarathustra* he states: *Zarathustra, you know well how eagerly I am seeking my dwelling; I am seeking my home and this search is eating me up. Where is my home?*\(^5\). Humans seem to be a species whose freedom makes them vulnerable to abandonment, and to being without a dwelling, who do not find a home where they feel comfortable, and are located in a space with which they appear to be in conflict at all times. It seems that we have no answer to Nietzsche's question: we do not know where man’s home is.

Using Heidegger’s conception of “dwelling” as a starting point, a new form of understanding the organisation of human space is proposed. It will be shown that the human dwelling crisis is partly due to the notion of space, which is understood as the organisation of a “physical” space. Human space is seen as a “place” (locus) and its organisation, or urban planning, is resolved in terms of “locating”\(^6\). Many challenges in organising cities are the result of reducing the organisation of human space to the organisation of physical space. One of these challenges is the fact that physical space cannot be shared, but can only be divided. This means that the organisation of human life is determined by the logic of competition (my place must not be occupied by others)\(^7\) and by the logic of a cost-benefit analysis\(^8\). Another challenge is that physical space is finite. This leads to a conflict with the human aim of development and growth\(^9\), and opens the typical dilemma in approaching sustainability, between “preservation”\(^10\) and enhancement of human capabilities\(^11\).

This article will focus on two key concepts, space and dwelling, and two fundamental hypotheses will be analysed:

1. The cause of the human habitat crisis lies in the idea of space with which it has come about. It will be shown that such crisis is partly in response to the notion of space, which is understood as the organisation of a “physical” space.

2. Due to the idea of space, no connection has been made between the organisation of space, or building, and human dwelling, or living; indeed it appears to be more of a conflict.

Within urban planning thinking, many factors have obviously influenced the traditional arrangement of these two aspects: the idea of space and the space-habitat connection. However, although it seems to be merely incidental, one of those which has had the most influence on practical urban planning arrangements has been that town planning has traditionally been carried out by local authorities. Although this may seem to be an inconsequential fact, it has had two consequences which have however had a determining influence on the way of thinking about human spaces. Firstly, the planning process has increasingly been reduced to organisation of physical space. Secondly, no attempt has been
made to adjust the organisation of space to the special nature of human and social behaviour, but rather there has been an attempt to influence social behaviours using the instrument available, that is, by means of “intervention” in the physical space. This has led to an instrumental relationship between the organisation of space and life, which has caused many problems, including degradation of the habitat and unsustainability.

2 HABITAT AS TERRITORY

The first point to be analysed is the focus on space and the physical place. From the Greek polis to the contemporary metropolis, one of the most typical features of the city is concentration in space. The city has traditionally been considered a grouping of persons who are concentrated in a defined physical space. According to the dominant way of thinking, this space is seen as a place. This is a sector of land on which buildings and other facilities are located, and on which people carry out activities; these people are more dependent on and connected to each other than people from other places. The city takes shape as a settlement surrounded by undeveloped space and physically separated from other settlements. The city unites and, at the same time, separates physically.

When seen in this way, as a limited unitary place, the city plays a primary fundamental role: it offers proximity between people. The city brings us together. Urban concentration aims to facilitate contact between individuals and groups, and take advantage of economies of scale in creating and maintaining interpersonal contact. It is also a fact that in cities there is a very close relationship between proximity, specialisation of activities and urban concentration. The growing division of labour and specialisation experienced by humans means that the need for interaction has grown. Physical proximity is useful for this increased interaction, and in turn reinforces specialisation.

However, as can be seen, and as has been discovered in various sociological analyses, there is a specific dynamic between proximity, specialisation and interaction. Firstly, it is difficult to know which factor is at the origin of this dynamic, as the relationship between these elements is circular; moreover, this circularity has proven to be explosive. There is a positive feedback circuit tending towards infinite growth: proximity facilitates specialisation, which leads to the need for more interaction, and this increases contact, etc. It should be no surprise that this circularity has converted the city of today into a place of complexity, and that the formulas for organising urban space always prove to be insufficient. Urban planners see that the solutions sought to problems rapidly lose coverage and scope, in turn proving to be problematic. Paths of communication which are designed to unite in reality become barriers between residents; the signs and rules intended to guide lead to disorientation; major ring roads and bypasses seem to be saturated as soon as they open, and what is supposed to reduce traffic congestion and travelling time ends up simply moving it to another place and becomes a trap for downtime.

Thus, we run the risk of the process of urbanisation (understood as urban concentration) degenerating into a process of frustration. The expectation of proximity and easier contacts, which leads many people to settle in the city, is frustrated by the effect of the organisation of
space itself. We believe we are closer, but essentially we find ourselves further apart. In large
cities, contact with others becomes chafing, and the need to be close a feverish “anxiety to
escape”. A symptom of this is the importance now given to the “exodus” from and “return” to
the city; and the fact that Friday evening and Monday morning are critical moments in the life
of many city dwellers.

Yet this process of territorialisation of life is not exclusive to urban living. It is the response
to a more general intellectual process: it could be called typically “modern”. This polarisation
of attention to space is characteristic of modernity; however, less attention is paid to non-
spatial organisation, connected to the temporal aspect of human existence, behaviour focused
on a purpose. The notion of the market, the basis of the modern economy, provides us with
another example of the process of territorialisation. It is one of the major and most determining
contributions of modernity, which is essentially understood in terms of space and territory.
Market liberalisation is understood as a process of creating economic spaces, in which
companies are required to conquer plots of business. National borders are replaced by
competitiveness, which is the same as saying certain territorial barriers are replaced by others.
Additionally, it can be seen that no attempt has been made to adjust the economic space, that
is, the market, to non-economic variables, like work or technology, but rather these latter
factors are managed on the basis of market laws. The strange thing is that the market today has
also been converted into an uncontrollable space.

Following the same logic, the city is seen as a place in which physical artefacts are settled
and in which the locations of activities are distributed according to geometric models. The
prevailing understanding is based on a static spatial representation. It cannot be said that
interactions which transcend mere location are ignored. Yet it is still the case that the
organisation of human habitat is focused on physical-spatial organisation, and from this basis
attempts are made to influence human behaviour positively. Interactions and human social
behaviour are being expressed using a spatial language. Nearly always, the “plans” or
intentions, are translated into “plans”, or designs, static representations of land use, and the
intensity of life is expressed in the form of density, which is the volume of activity per unit of
space.

2.1 The Paradox of Space

However, we should ask ourselves whether spatial language is appropriate for capturing
human dwelling, which occurs in a space, but which transcends a given place. Possibly what
happens is that the basic supposition of urban planning, that human life can be managed
through and on the basis of organisation of physical space, is an illusion because, as Melvin
Webber suggested, space acts as a barrier to communication. How can this be explained?

It is necessary to examine the idea of space more closely. Everyone thinks it is obvious that
material things, including of course humans, occupy a real place. In classical thinking, a
distinction is normally made between the “place” (ubi) of each thing and space. Space, which is
also real, is considered a “middle between ends”, that is, what is “between” the places
connected to each thing. It is what separates one thing from another and, therefore, is what
marks a *distance*. However, on the basis of this real space it is also possible to “conceive of” an unoccupied space, one that is infinite: an empty and completely undetermined space. There is nothing strange about the fact that in modern thinking this idea of space has come to be associated with the sphere of exercise of freedom. This freedom is in turn understood as mere independence, a sphere of pure indecisiveness. The sphere of freedom is supposedly like a large empty space, and acting freely is like organising *spaces*, establishing *places* and *situating* things in them.

However, this association between freedom and space is what truly places us in what could be called the *paradox of space*, because one of the fundamental characteristics of place, one of which almost no one needs to be convinced, is that it cannot be occupied by two things at the same time. This fact, which appears almost trivial, is however vital for the organisation of space, because physical space is converted into something *limited* precisely by the location of each thing; consequently, it means that organising physical space always involves establishing limitations.

Organising space, and “putting things in their place”, means establishing barriers, putting up fences, signs, etc.; in short, it entails restrictions on movement, as the *place* occupied by each thing cannot be shared. For a symbolic and paradigmatic example of the paradox of real space, we need look no further than the traffic light: on the one hand it makes circulation possible, and on the other hand it hinders it. It should therefore come as no surprise that spatial organisation creates incompatible “spaces of power”\textsuperscript{17}. My location is mine alone. The place I occupy cannot be occupied by another. It is understood that the idea of freedom consistent with this notion of space is the idea that “my freedom ends where the other’s freedom begins”. It is to be expected that such an idea of freedom, along with the resulting spatial organisation, may cause conflicts of interests and confrontations. The problems of implementing urban designs are connected to the ownership of spaces of power, whether physical, political, economic, etc. This power should not be understood as the capacity to act, but simply command over a territory.

It can already be seen that this way of organising space cannot be compatible with or adapted to the *real freedom* which is essential to the human being. As the anthropologists say, humans are characterised by, from a physical and biological level, the lack of any connection to *territories*, understood as spheres of control. Humans are naturally of an *undefined nature*. Humans are not connected to a territory, because they are connected to all territories. To answer Nietzsche’s question, it could be said that man does not find his home, because he is seeking a *place* and man does not have *his location* in a *place*. What makes humankind resistant to recognising any place as their *dwelling* is precisely the discontinuity between geometrically configurred physical space and their freedom. As we will attempt to demonstrate below, this freedom is not only the cause of the “housing problem”: it is also the solution.

**3 HUMAN SPACE: DWELLING AND LIVING**

We will now look at the second principle of traditional urban planning, that concerning the connection between the organisation of space and human dwelling (living). In addition to
coming from the paradox of space, the *dwelling crisis* is the result of incorrect application of the end-means concept in this relationship.

We have emphasised that it would be unfair to say that traditional urban planning approaches do not recognise the great impact of human activity on spatial conditions. In the same way, this also does not mean that human life has nothing to do with the organisation of place and physical space. What we wish to show is that the relationship between spatial organisation and human life has not been adequately resolved due to the fact that generally it is considered in functional terms: the organisation of space appears to be an *instrument (means)* for a better life. Here lies the second origin of the problems in organising human habitat.

In his 1951 lecture mentioned above, Martin Heidegger focuses precisely on this point\(^\text{18}\). The text presents a criticism of urban functionalism, insofar as it does not adequately resolve this relationship existing between the human way of living, or dwelling, and spatial organisation, what Heidegger calls “building”.

It is important to recognise that in order to inhabit or live humans must build, or organise space and, therefore, the goal or purpose of this “building” is “dwelling”. Thus, building and dwelling are respectively placed in an *end-means relationship*. Considering them from the perspective of this functional notion ensures the two activities are not confused and, in this regard, no objection can be made to their use. Not all building, for example a supermarket or airport, is recognised as accommodation, nor is it recognised as a *dwelling* for humans. The human dwelling goes beyond building activity and they are connected to each other in the same way as a means to its end. This is the reasoning found in the functionalist approach.

However, the end-means concept can also betray us and distort the essential features of the relationship between building and dwelling. This happens if building is understood as “just a means”, that is, a mere “instrument for” and a path which leads to something “separate”, which is the dwelling. According to this notion, space is capitalised on and the city is therefore understood as an *artefact*, which maintains a merely utilitarian relationship with the human dwelling. In this case, it is difficult to recognise the *built* as a place suitable for *dwelling*. It is like some restaurants, which are so functional they are not very comfortable.

Conversely, what Heidegger proposes is realising that building is *already* itself a dwelling. Logically, this means giving a new meaning to *building*, to the treatment of human spatiality, going beyond the functional and utilitarian version, which has given rise to the damaging actions to the spatial environment we have seen in many human habitats. However, at the same time, human *dwelling* itself is no longer seen as something which comes later, separated from and foreign to physical-spatial organisation. A distinction can be made between them, but they cannot be separated.

We often think of dwelling as an activity, an action, which is carried out by humans along with many other actions. They work, do business, play and, among various other actions, they dwell. But what does dwelling consist of exactly? It would be useful to try to isolate this activity from all the others. If we do so, we find it is identified simply with “being”. Dwelling is the human way of “being in the world”. Heidegger says that it is to be on the earth as a *mortal*\(^\text{19}\). We could also say that dwelling is the same as *living*. Dwelling does not name one mode of being as opposed to *another*, but rather the essential way humans are in the world\(^\text{20}\).
Thus, firstly it was suggested to us that dwelling and building are not two activities which go one after the other. Dwelling is at the same time building. It is however immediately added that what is built is, above all, human life. This allows us to take a step towards answering the question about the human dwelling place. Effectively, the location of humans is not simply the place they occupy. It is a place that they build for themselves. And what they build (in space) is, above all, their life. It is not difficult to observe that these meanings of dwelling and building have been forgotten. We have lost the sense of dwelling, because we have not been able to connect it to the human state of being, life21. The crisis of the city is a result of the fact that the process of organising human space, or building, has been disconnected from ways of living.

3.1 Habitat and Habits

The place of humans is a space for living, which they build throughout their lives and in which they must form their dwelling place. Building is not organising a physical place; it is constructing our lives. This is why it is more closely connected to what the Greeks called ethos. Ethos means the territory, the particular world or the right place for each thing. There is a physical place and there is also a biological space. Water is the “place” of the fish, the air is the “place” of the birds, etc. There is also a place for humans. However, this transcends both the physical and the biological meaning. The place of humans is not the place where they are, but the “sphere” which they create with their own behaviour. The classics understood it as a practical space, the ethical and social space22.

We are proposing a new view of the organisation of human dwelling, one which overcomes the physically-based conception of space. In doing so, the meaning of human “habitat” as “space for developing capabilities” will be recovered, and the implications for the organisation of the human habitat considered. “Habitat” comes from “habit” (habitus), which is the increase of human potential produced by actions23. Human action is not only a “consumer” of resources. Human life is also a process of improving possibilities24. Habitat is not a physical space, it is the “field of possibilities of action” which humans give themselves. The human house is the city (polis). However, the city is above all a practical and social space created through a temporal process of learning and skills development25. This idea of habitat as a skills-based space can be shared, and its development is infinite, opening the way to overcoming the logic of competition and the conflict between sustainability and development.

It is a sphere made up of behaviour, formed by actions and personal interrelationships, which trace a landscape of customs, behavioural guidelines, traditions and institutions. It is the sphere of what is done “habitually” and which, therefore, seems “familiar”. When entering it one moves from a geometric space to a human space, which is cultural, but in the etymological sense of the term, namely, in the sense that it is the result of “cultivating”, meaning “caring for”, “making grow”. It is not therefore a matter of constructing large buildings, communication arteries, etc. Rather it involves building in the sense of “erecting”, that is, productive building. Building in the sense of “caring for” has an intrinsic connection to life itself and its growth26. It is a matter of cultivating forms of behaviour with others. In short,
what is meant is that by erecting buildings, tracing streets, urban roadways, etc., it is not spaces intended for living that are being created. Above all, ways of living are being created.

4 THREE FORMS OF DWELLING

Three reference points for organising dwelling will be identified. The three chosen are as follows: the space of needs, the space for exercise of freedom and the sphere for development of knowledge and intelligence\(^27\). Each of these leads to three spheres of action: the domestic sphere, the public or political sphere and the scientific and cultural sphere. In turn, these fundamental ways of dwelling are manifested in three basic forms of dwelling: “the home”, what we could call “the streets”, which includes squares, public spaces and meeting places, in general; and finally, “the parks”, that is, spaces for teaching and collective cultivation of knowledge, science and art.

The home is the sphere of domestic activities. As such, it is the place for private life which, as H. Arendt explained, is preferably connected to the “private” and needs. Human life is, primordially, a life of needs. Humans must carry out a series of actions intended to satisfy physical, biological and personal development needs: they must protect themselves from inclement weather, feed themselves, dress themselves, rest, take care of their families, etc. The home is the place to take care of “the necessary” for life, it is a private place.

In our cities, this place has today been reduced due to the fact that survival needs, the private, are now met in the social and public sphere. This can be seen clearly if we look, for example, at finances, which after belonging to the domestic sphere for centuries, in modern times have become a public matter. Once this happens, people seek to meet their own needs not in the home but in society\(^28\). Liberalism makes it legitimate for each person to seek their own interests in public life and society. What is more, the laissez faire theory states that what moves society is private interest.

This societalisation of the private, which is, at the same time, privatisation of the public, is increasingly removing the content from domestic space. Humans meet their needs outside the home. They do not eat with their family, and barely even sleep in the same place. The consequence of this is that the domestic place is reduced, that is, the size of houses: not only the number of people living in them but also their dimensions. From the “house” there has been a move to the “flat” and from the flat to the “apartment”. The apartment is the private place of the anonymous citizen in today’s large cities, so the use of the expression “house” is now merely metaphorical.

Yet there are other reference points. Humans are not only beings with physical and biological needs. In addition to their own interests, they are interested in taking part in what belongs to everyone as beings who are equal but at the same time different. They are interested in taking part and collaborating with others in developing common assets. The public sphere is not, as has habitually been understood, the sphere for meeting one’s own interests. Rather it is accessed when going beyond the prospect of one’s own benefit in order
to serve general interests. It is a sphere of communication and collaboration in the common interest, and therefore in a strict sense it becomes a sphere of freedom.

This sphere of activities is outside the house, or home. It is in “the street”: the squares, the public places, meeting centres, association premises, etc. However, the streets of modern large cities are today a place for strolling, sometimes with no fixed destination in mind. The solitary citizen goes to the street to “hang out” or to “go shopping”. We have also lost the human and living sense of “the streets”. From this reference point, “the street”, in the sense of public spaces, meeting points, etc., is the sphere for unrestricted activity. It is not a location for stocking up or walking alone, but the place for “meeting” others. It is the place in which living becomes living together, no longer in the sense of being alongside each other, but in the deeper meaning that each person depends on the lives of others and enjoying the company of others. This is why celebrations are also held in the street.

Indeed, the forms of human life are not limited to the “cultivation” of relationships with others. Humans also “cultivate” knowledge, intelligence, science, art and religion. This activity constitutes a sphere of urban life that is still more open that the political sphere: if something belongs to everyone and it is possible to participate with everyone, it is truth and beauty. This is a more radical liberalism which, unlike ideological liberalism, consists of organising city life so that all its members take part in wise and intelligent leisure activities. This sphere of knowledge also makes up an urban place, which is different to the home and the street. It is “the park”, a place for thinking about what we do; the place to reflect on and contemplate our achievements. “Intelligent contemplation” cannot be carried out, as is sometimes thought in some cities, on the terraces of cafes. Its place is in libraries, schools, museums, universities and research and training centres.

5 CONCLUSIONS

- There is a way of thinking about dwelling which entails going beyond the functional concepts and which, on the one hand, connects with the initial approaches to human dwelling, in which it was warned that what gives it meaning is not coexistence, nor the community of place, but rather living and behaving well. On the other hand, it is also related to the most recent analyses of today’s urban dynamic.
- Today there is a need to understand the human habitat as a social system in action: abandoning the static idea of the city as a unitary and useable place, to consider of the city as a system of multiple dynamic spheres, a system in which people interact with each other through space at every moment of time, configuring multiple spaces of action. “Habitat” comes from “habit” (habitus), which is the increase of human potential produced by actions. Habitat is not a physical space, it is the “field of possibilities of action” which humans give themselves. The city is above all a practical and social space created through a temporal process of learning and skills development.
- The way of interacting established between humans is changing. In human interactions, exchanges of physical goods are being replaced by exchanges of information. It can be seen that the monocentrism of a unitary space is being replaced by sustained
polycentrism in an intense flow of information taking place through non-spatial channels. The urban settlement is being converted into an intercommunications node rather than a physical place. The new information and communication technologies are reducing the importance of physical proximity, and replacing it with communicative accessibility.

- At the same time, territorial demarcations have lost much of their significance. Contemporary agglomerations exceed administrative limitations, and do not meet the criteria of closeness and density. The criterion of differentiation between centre and periphery, which was previously explained by criteria of spatial density, is no longer a factor for distinguishing areas: many peripheral areas have assumed the typical characteristics of a central area, and the reverse. The concepts of “centre” and “periphery” have lost their spatial meaning. Today capital is not central, nor is it the most geographically extensive. The importance of cities is not determined by the surface area or population density, but by their communicative centrality. The citizen of today makes up and forms part of spheres of intercommunication, which are not defined locally. It is what Webber called “community without propinquity”. Urban space is no longer the abstract Euclidean space. When major transport and communication systems appear, space begins to be measured by time and not distance. Thus urban space can be “compressed”, “extended” or “inverted”, as in a piece of music, scenes are created and cancelled, and they accelerate or stop.

- A non-functional building-dwelling connection is recovered to open the way to escape from the paradox of the physical-technical space, and offers an opportunity for revitalisation of the organisation of habitat, making it more sustainable. While the organisation of physical space leads us down the path of imposing limitations and restrictions on actions, the organisation of practical space, and of forms of behaviour, attitudes and interpersonal relationships, expands the limitations and opens up prospects for action. While the organisation of physical space creates spheres of incompatibility; the organisation of practical space generates spheres of compatibility and communication.

- Such changes call for a perspective on the city that is more in line with a systemic approach than an analytical approach. Reflections on human space must investigate a scenario with a wider scope, which makes it possible to above all deal with spheres of action, and order them, based not on geographical reference points, but on human reference points.

- Three reference points for organising the human habitat as living space can be identified: the space of needs, the space for exercise of freedom and the sphere for development of knowledge and intelligence. Each of these leads to three spheres of action: the domestic sphere, the public and political sphere and the scientific and cultural sphere. These fundamental ways of dwelling are manifested in three basic forms of dwelling: “the home”, “the streets”, and finally, “the parks”.

REFERENCES


THE FRENCH NATIONAL PROJECT “RECYBETON”:
RECYCLING CONCRETE INTO CONCRETE

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Key words: Recycled Concrete Aggregate, Recycling Concrete Process, Recycled Concrete Structures, Sustainable Development.

Summary. France is currently producing about 20 Mt/year of demolished concrete, most of this material being used in road sub-bases or embankments. However, this flow should increase in the near future, although less and less new roads are to be built. Another 20 Mt of mixed demolition materials, a good part of it being concrete or natural rock, is also available. Therefore there is a duty, both for the society and the planet, to make the best use of this demolished concrete, in order to preserve natural aggregate resources, which are increasingly difficult to obtain, and to eradicate waste material landfills.

Based on this reality, a national project was set in 2012, gathering 47 partners, among which representatives of all construction stakeholders. Partially funded by the French Ministry of Ecology, RECYBETON encompasses five fields of activities:

- Research on material processing.
- Research on recycled materials and structures.
- Research on sustainable development.
- Standards and regulations.
- Communication and Promotion of results, among which demonstration sites.

The paper aims to present the main outputs of the project, which will produce various deliverables: a scientific book, a guide, a number of proposals to adapt standards and regulations, and, last but not least, four experimental constructions (demonstration sites) including an outside parking lot, a bridge, an outdoor space and a public building. Other demonstration sites are in project or near to be launched.
1 INTRODUCTION

Sustainable Development has become a real societal issue and the construction sector must be in line with it. In France, around 20 Mt/year of concrete waste material could be recovered by its integral recycling, which would permit:

• To limit, or even eliminate, putting a part of this waste into landfills.
• To reduce the use of natural resources which are increasingly difficult to obtain.
• To reduce material transport, as “recycled aggregate” resources often exists near construction sites, in particular in large urban centers.

This will help to reducing CO₂ emissions and energy consumption.

Currently, most of this material is being used in road sub-bases or embankments. However, this flow should increase in the near future, although less and less new roads are to be built. Therefore, there is a societal and environmental duty to make the best use of this demolished concrete.

Based on this reality, a French national project was set in 2012, gathering 47 partners, among which representatives of all construction stakeholders. An ambitious program willing with the complete recycling of concrete was prepared and two principal objectives were established:

1. Reusing all the materials obtained from demolished concrete, even the fines, as components of new concretes.
2. Recycling the fine grain size part of demolished concrete as raw material for the manufacture of cements (either to produce clinker or as an addition into a blended cement).

Besides improving recycled concrete properties and contributing to the preservation of the natural sources of aggregates, other important results are expected:

• To increase significantly the utilization rate of recycled concrete aggregates in concrete compositions.
• To fully develop the concrete recycling sector, from demolition to the production of new recycled concrete.

2 RESEARCH PROGRAM

A state of the art report made before launching the National Project identified the research needs and the barriers to implementation, thus establishing the following program (see [1], here below a synthetic content of the 6 themes):

• Theme 0 – Technical Support: Formulation of the concrete mixes and supplying of the materials needed for the researches.
• Theme 1 – Technologies and processes: Detailed investigation about crushing and sorting of demolished concretes, and cement and concrete manufacturing with recycled concrete aggregate.
• **Theme 2 – Materials and structures**: Optimization of the use of recycled concrete aggregates, variation of physical and mechanical properties of concretes containing recycled aggregates at early age and hardened, durability and fire resistance.

• **Theme 3 – Sustainable development**: Analysis of socio-economic, environmental and sanitary impacts of the development of the concrete recycling sector.

• **Theme 4 – Standard and normative aspects**: Identification of barriers and incentives for the use of recycled concrete at French, European and International levels in order to make proposals of adjustments and/or modifications of existing regulation.

• **Theme 5 – Communication and Promotion**: Showing off of the advances obtained by the National Project by the way of conferences, publications (dissemination works, scientific articles, guidance and a book) and realization of demonstration sites.

### 3 REMARKABLE OUTPUTS

After four years, important results have been obtained and the last ones will be soon achieved. Here below the most remarkable are listed according to each theme of the program. They correspond to the three first parts of the project, divided into four parts.

#### 3.1 Theme 0 – Technical support

To optimize results and be able to compare them, it was necessary for all the laboratories to be provided with:

- Homogeneous stocks of natural and recycled aggregates
- Homogeneous stocks of current cements and of cements incorporating recycled material
- A series of referent recycled concrete compositions

This continuous technical support is fundamental: the recycled concrete compositions have been validated by the Theme 0 working group. Nine concretes (6 C25/30 and 3 C35/45) with different contents of recycled concrete aggregates (R in Table 1) were then formulated. Table 1 shows the principal constituents of the nine concretes. The reference concretes, containing only natural aggregates are indicated (0R, 0R) for the couple (Sand, Gravel).
Table 1: Concretes used in French National Project RECYBETON

<table>
<thead>
<tr>
<th>Formulation No</th>
<th>Strength Class</th>
<th>Ciment</th>
<th>Addition</th>
<th>Sand</th>
<th>Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>0R</td>
<td>0R</td>
</tr>
<tr>
<td>2</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>0R</td>
<td>30R</td>
</tr>
<tr>
<td>3</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>0R</td>
<td>100R</td>
</tr>
<tr>
<td>4</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>30R</td>
<td>0R</td>
</tr>
<tr>
<td>5</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>30R</td>
<td>30R</td>
</tr>
<tr>
<td>6</td>
<td>C25/30</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>100R</td>
<td>100R</td>
</tr>
<tr>
<td>7</td>
<td>C35/45</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>0R</td>
<td>0R</td>
</tr>
<tr>
<td>8</td>
<td>C35/45</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>0R</td>
<td>100R</td>
</tr>
<tr>
<td>9</td>
<td>C35/45</td>
<td>CEM II</td>
<td>Limestone filler</td>
<td>30R</td>
<td>30R</td>
</tr>
</tbody>
</table>

These concretes, excepting some special cases, have been used for the experiments of the program and a stock of the different needed materials was also ensured.

3.2 Theme 1 – Technologies and Processes

This theme deals with three subjects: 1. Crushing and Sorting, 2. Cement incorporating materials from recycled concrete and 3. Concrete incorporating materials from recycled concrete. In what follows, the more important outputs are indicated according to these sub-themes.

3.2.1 Crushing and Sorting

The goal of this subject is to improve the technologies used to separate concrete from other construction waste materials as well as the sorting processes for the different concrete components.

A thorough analysis of the existent technologies concerning their adaptability for obtaining quality products while crushing demolished concrete was done.

A state of the art of the different techniques for separating concrete from other polluting materials (physicochemical, gravimetrical techniques) has been established.

Different sorting technologies applicable to recycled aggregates (X-ray, Infrared, by density) were studied with detail to consider their possible utilization for improving the quality of concrete recycled aggregate.

3.2.2 Cement incorporating materials from recycled concrete

The optimization of the processes for incorporating fine particles obtained by crushing concrete (fines and sand) into the manufacture of cement is the aim of this subject.

Four different recycled concrete sands, coming from different areas of the country, were thoroughly analyzed with respect to their capacities to be incorporated in the cement
composition. Even if their mineralogical composition was principally siliceous, the test showed that they can substitute clay as raw material of clinker up to 24%. The clinker thus manufactured is viable to produce cement (called CR1). An industrial production of clinker was then decided.

It was also shown that they can be correctly pounded to incorporate them as addition in blended cement, with optimum qualities as secondary components. A pilot test for producing blended cement with pounded recycled sand as addition (called CR2) was then conducted.

### 3.2.3 Concrete incorporating materials from recycled concrete

In this case, the aim is to optimize the processes for using recycled concrete aggregates in the production of new concrete, with an especial interest in the aspects of stock of recycled aggregates and mixing process.

Due to its high absorption (presence of cement paste), recycled concrete aggregates is best used at a humidity degree 1% above its measured absorption. The value of 1% has been adopted for limiting the risks of drainage and humidity gradients in the stock areas. A recommendation was then given for the researches: before mixing, it is necessary to pre-humidify (absorption + 1%) recycled concrete aggregate when stocked in a laboratory.

The influence on the rheology of concrete with recycled aggregates of different mixing technologies, different water conditions and different wearing properties in regard of micro-Deval test was also studied. The conclusions are that there is a granular reduction (cement paste separated from gravel, partially or totally, creating additional fine part), some variations in workability and some influence of a "bad" MDE, but all the parameters are controllable allowing then to prepare correct concretes with current mixers. Real concretes incorporating recycled concrete were produced for the experimental sites (see chapter 3.6).

### 3.3 Theme 2 – Materials and Structures

In this theme, five subjects are considered: 1. Recycled aggregates and fines, 2. Recycled concrete at early age and during hardening, 3. Hardened recycled concrete – Mechanical behavior, 4. Hardened recycled concrete – Durability, 5. Recycled concrete – Fire and thermal behavior. Here below, the more significant results are highlighted following these subjects.

#### 3.3.1 Recycled aggregates and fines

The goal here is to extend the use of all particle sizes obtained from crushed concrete into traditional concrete. For reaching that, it is important to verify if the standard tests for natural aggregates are applicable to recycled concrete aggregates, to qualify the kinetic of water absorption of concrete recycled sands and gravels and to analyze the temporal and geographical variability of recycled concrete aggregates.

With regard to the adaptability of standard test for natural aggregates to recycled concrete aggregates, it was noted that:

- In the case of mechanical tests: some variations are observed, correlated with the fines production of recycled aggregates.
- Considering the tests for physical and chemical characteristics: some fluctuations
concerning a few characteristics values (setting time, chloride content) were obtained and the repeatability of some tests (freeze-thaw) was not clear.

- For the density and water absorption tests: they are adapted for recycled gravels but less consistent for recycled sands.

Because the kinetic of water absorption of recycled concrete aggregates is different of that of natural aggregates, the development of an adapted test for recycled ones has been considered.

Many of the differences in the results obtained during the tests are related to the origin of the recycled concrete aggregates. That is why 13 recycling platforms and 16 recycled concrete aggregate productions from different areas of France were analyzed. It was concluded that the observed variation in some characteristics can be overcome by improving the production process, reconsidering the impurity thresholds and adopting a tighter quality control.

3.3.2 Recycled concrete at early age and during hardening

To control the changes induced by the utilization of recycled concrete aggregates during the early age and hardening of concrete is the purpose of this subject. In particular variations in the rheological aspects were studied.

When considering the influence of the couples cement/concrete admixture, it was found that until 30% of substitution of natural aggregates by recycled concrete aggregates there was no changes in the rheological properties, but for higher ratio the couples cement/super plasticizer allow a better stabilization of these properties.

Another important factor that was studied is the evolution of the concrete rheology according to the percentage of recycled concrete aggregates and its pre-saturation condition when mixing: in general, slump was less optimal but concrete flows better with saturated recycled concrete aggregates.

3.3.3 Hardened recycled concrete – Mechanical behavior

In this subject, the goal is to evaluate the changes in the mechanical properties of recycled concrete in order to consider how to improve them when it is necessary. For this purpose, several tests with concretes made with different percentages of recycled concrete aggregates were achieved. Among them we can highlight desiccation shrinkage, restrain shrinkage, compressive strength, tensile strength and concrete/reinforcing bars adherence.

In the case of desiccation shrinkage, the general trend is that cracking caused by this phenomenon is more sensible to the initial saturation ratio of recycled concrete aggregates and to its percentage of substitution in the concrete.

The restrain shrinkage was studied in laboratory and could be also observed in the real case of a demonstration site: slabs of an outside parking (see 3.6). As expected, shrinkage grows with the recycled concrete aggregate percentage in the laboratory tests. However, after more than two years of life, even the slabs containing 100% of recycled concrete sand and graves do not present any perceptible cracking in the parking lot.

The first performed compressive strength tests gave positive results when comparing reference concretes with recycled concrete with 100% of recycled concrete gravel: for C25
concretes there is only a loss of 1 MPa for the recycled concrete and of 2 MPa for the C35 recycled concrete. The trend is the same for the tensile test (splitting): a loss of 0.1 MPa for the C25 recycled concrete and of 0.2 MPa for the C35 one. The number of tests was small (3 for each formulation) to assess if this tendency is repeatable, thus more tests are been performed.

A complete test program was made for measuring the adherence between two types of reinforcing bars (HA10 and HA12) with concretes containing different ratios of recycled concrete sand and gravel (C25 and C35 with: 0R-0R, 0R-100R, 30R-30R, 100R-100R, recycled sand and gravel content respectively). The results showed that the adherence of recycled concretes is similar to that of the reference concretes.

Structural tests (beams and columns) have recently been launched.

3.3.4 Hardened recycled concrete – Durability

The aim of this subject is to determine the parameters with an impact on the durability of concretes with recycled concrete aggregates. To reach this purpose, some relevant characteristics of hardened recycled concrete have been analyzed: alkali aggregate reaction, sulphates soluble in water, transfer properties (carbonation, chlorides).

For the alkali aggregate reaction it was concluded that the modus operandi of the test has to be adapted. This adaptation is been studied.

In many cases, the measured ratio of sulphates soluble in water of recycled concrete aggregates was higher than the standard values. However the way for measuring this ratio is not adapted to recycled aggregates. It was then decided to analyze the influence of this higher measured value for these aggregates in the durability of recycled concretes.

Concerning the transfer properties of recycled concretes it was observed that the optimization of the compactness of the cement matrix in these concretes allows balancing the eventual impact of recycled concrete aggregates, which are more porous, with regard to the durability properties (carbonation, chlorides, permeability).

3.3.4 Recycled concrete – Fire and thermal behavior

To complete all the properties of recycled concretes the goal of this subject is to study its behavior at high temperatures. This is one of the last subjects of the program and at the present time only a few results are beginning to be obtained.

3.4 Theme 3 – Sustainable Development

In this theme, two important subjects are considered: 1. Social and economical aspects, 2. Environmental and health aspects. The more relevant outputs are described here below.

3.4.1 Social and economical aspects

The purpose of this part is to evaluate the economical and social impacts of recycled concrete and of the development of the concrete recycling sector.

A first prospection on the technical and economical aspects was made by analyzing two
area of production of natural and recycled aggregates: one at a regional level (French Nord Region) and another at a local level (Great Lyon). A detailed geo-localization of aggregate quarries and recycling platforms was made for these zones, thus allowing the study of the flow impact.

Using data provided by regional French organisms (CERC, Regional and Economical Cells of the Construction sector) an exhaustive survey of existent production basins of recycled aggregates was carried out. It permitted to refine the knowledge about the available resource in recycled concrete aggregate (between 17 and 25 million of tones), its distribution in the national territory, the difference between existing recycling sites and landfills near the principal cities and other interesting data which will be useful for the deployment of the concrete recycling sector.

3.4.2 Environmental and health aspects

The aim is here to determine and control the possible environmental and health impacts of recycling concrete. For this purpose, an especial interest was given to the characterization and evaluation of the environmental behavior of concretes incorporating recycled concrete aggregates.

It has been observed that there were no significant differences between the leaching behavior of concretes with recycled aggregates and those with natural aggregates, even a slight reduction on the release of copper, lead and sulfate for concretes with recycled concrete aggregate was observed.

LCA method was used to establish the environmental balance sheet of concretes with different percentages of recycled concrete aggregates. It was shown that only concretes containing 100% of recycled aggregates present degradation on the environmental markers due to their higher content on cement and admixtures, necessary for improving its mechanical properties. On the other hand, the environmental impact of concretes with ratios < 100% of recycled concrete aggregates is of the same order of magnitude of that of concretes with natural aggregates. Moreover, avoidance of landfill and preservation of resource are not fully accounted for in classical LCA methods, although these two aspects are favorable.

3.5 Theme 4 – Standard and normative aspects

This theme approaches the identification of incentives and barriers to the use of recycled aggregates at French, European and International level. One of its principal goals is to make proposals for performing a framework for the use of recycled concrete aggregates in new concretes by the way of a recommendation guide.

A complete state of the art was established for France and Europe and other countries.

At the French level, the principal barriers are the limitations in function of the recycled aggregate classification: for the strength classes, only C25/30 for sand and gravel type 2 (90% of recycled concrete) and type 3 (70% of recycled concrete) and for the exposition classes, type 1 recycled concrete (95% of recycled concrete) in different percentages (60% for X0, 30% for XC1 and XC2, and 20% for the other classes).

At the European and international level, even if some of the restrictions in France are also
retrieved in the other countries, it can also be found some most favorable measures like the acceptance of 100% of recycled gravels for some cases in Denmark, Holland, Switzerland, Japan or China (subject to complementary disposals).

Concerning the incentives, work is in progress. It has been principally detected the existence of labels classifying the use of recycled aggregates and some local initiatives as in the Geneva county in Switzerland.

3.6 Theme 5 – Communication and promotion

To show and diffuse the advances obtained by the project is the principal goal of this theme. For doing so a Web site was launched from the beginning of the project, a public conference to present work in progress was held in June 2015, a number of publications have been made (dissemination works, scientific articles, congress communications) and two redaction committees have been constituted for the redaction of a scientific book and a practical guide. Public meetings for spreading the results of the project will be scheduled but the real showcase of the project are the demonstration sites, i.e. experimental constructions where the outputs of the program are applied and at the same time assessed.

Currently, four demonstration sites with different concretes containing recycled concrete aggregates have been achieved: an outside parking lot, a bridge for bicycles as secondary work of a great public work, an outdoor space in a ready-mix concrete plant and a public building consisting in an office for library folders. Other sites are in project to be built with concrete containing recycled concrete aggregates: a public secondary scholar building and another public work (probably another bridge). In all the sites, the concrete used for the works is tested in parallel in a laboratory to determine its rheological and mechanical characteristics.

The outside parking lot was constructed with six slabs containing different ratios of recycled concrete aggregates: (0R-0R, 30R-0R, 0R-30R, 30R-30R, 0R-100R, 100R-100R, for the couples S-G). Following the indications of our project, the concretes were prepared and poured without difficult and easily placed by the operators which found these concretes very similar to current ones. Even if the laboratory test showed increasing value for restrain shrinkage in function of the ratio of recycled concrete aggregate with cracks development for the 100% formulation, any cracking or other defaults are observed in the site after more than two years of construction.

In the public work for by-passing two cities of the South of France (Nîmes and Montpellier) with the great speed train TGV some secondary works were necessary. Between them a bridge for a bicycle route for crossing a stream was chosen to test the use of a concrete with the authorized values of the standard (20% of gravel, type 1 recycled aggregate) for a C35/45 in aggressive environment because of the nearness of the Mediterranean Sea. The bridge crossing and one return wall were built with this concrete. A complete laboratory study was made with the 20% recycled concrete aggregate formulation and with another similar concrete but with 40% of recycled concrete aggregate which gave acceptable results so the decision of planning another demonstration site with this ratio of recycled aggregate was taken. After more than a year of construction, the bridge does not present observable defects.

An outdoor space consisting principally in a pedestrian path and two small walls using a
C25/30 with (30R, 30R) recycled concrete aggregate was made in the premises of a ready-mix society. The particularity of this site is that the concrete for the path was colored without difficulty and with a good finish.

The last experimental construction was a public building in a library, consisting in a room for stocking folders (estimated weight: 750 kg/m²) with 40 m² as surface area. The shallow foundations, the slab of the floor, the walls (with a door and three windows) and the slab of the green roofing were made with a concrete C25/30 with (30R, 50R) recycled concrete aggregate. The laboratory test made before the beginning of the works, when comparing with the reference concrete gave an increment of 40% for shrinkage and a reduction of 10% of compressive strength. These values were taken into account to rectify the structural calculation results. Production of ready-mix concrete, transportation, pouring and casting were made without difficulty. The first inspection after the works were finished showed some aspect defaults due to placing but not attributable to the recycled concrete. This experience consolidated the feasibility of the project of a secondary school building using concrete containing recycled concrete aggregates programmed by the same public contacting authorities (the scholar division of the Seine-et-Marne Department).

4 CONCLUSIONS

- A French National Project of Research and Development, called RECYBETON (Complete Recycling of Concrete) was launched in January 2012, with currently 47 partners, to consider the valorization of about 20 million tonnes of demolished concretes.
- Five principal themes were established to attain the two principal objectives: Reusing all the materials obtained from demolished concrete, even the fines, as components of new concretes and Recycling the fine grain size part of demolished concrete as raw material for the manufacture of cements (either to produce clinker or as an addition into a blended cement).
- The impact of existing material for crushing, sorting and mixing was analyzed showing that it is possible to use them in a correct way to obtain good quality recycled concrete aggregate and concretes.
- The possibility of using recycled concrete sand as a part of the raw material for producing clinker and its utilization as addition in blended cement was proven.
- Nowadays, after four years of work, the project has obtained many remarkable outputs. Even if they confirm many of the expected diminution on the properties of concretes with recycled aggregates, they also confirm the possibility of using these concretes by the control of the fundamental parameters governing their properties.
- The analysis of the different factors playing on the possibility of the development of the recycling concrete sector has permitted to establish the actual mesh of recycling platforms which will allow studying the flow impact of using recycled concrete aggregates.
- Excepting the concretes with 100% of recycled concrete aggregates, the other formulations do not present remarkable impacts for the environment.
- Four experimental construction (experimental sites) has proven the feasibility of making new concrete with recycled concrete aggregates.
- The project members are preparing a scientific book, a technical guide and a series of standard evolution proposals that should be delivered in 2017.

REFERENCES

CAN ARTIFICIAL RECYCLED FINE AGGREGATE TRULY REPRESENT FINE AGGREGATED FROM C&DW

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Key words: Recycled fine aggregate, mortar, construction and demolition waste, recycled concrete

Abstract. Using construction and demolition waste (C&DW) as a source for aggregates draws attention lately due to the large quantities of C&DW accumulating in the modern world. The coarse fraction of this waste was studied extensively in the past as a source for coarse aggregates in the production of new concrete. It is quite acceptable that replacing 10-20% of the virgin coarse aggregate with recycled one will have a minor effect on the properties of the new concrete. However, using the fine fraction of C&DW to replace virgin aggregate is still in question and is restricted in the standards.

Studies on recycled aggregates usually use aggregates that artificially manufactured in the lab from concrete produced under artificial conditions. The controlled conditions of concrete manufacturing enable understanding some of the impacts of the recycled aggregates but neglect the variability and complexity of the aggregates produced in real C&DW recycling plants.

The fine fraction of the waste is restricted for use in many standards and it has extensively studied in our laboratory. This paper presents part of the study aimed at comparing artificial aggregates produced from old concrete with known properties and old paste, both produced for this study, with aggregates obtained from two recycling plants. The results indicate that the properties of the aggregates and mortars prepared from these aggregates are different from the aggregates obtained in 'real life' showing that investigations on materials prepared from artificial sources are not sufficient for drawing conclusions that can be implemented in standards.

1 INTRODUCTION

Construction and demolition waste (C&DW) accumulates in large quantities worldwide. A rough estimate to these quantities is 0.5-1.0 ton/capita/year [1] with relatively large variability between countries. Recycling of this waste is, therefore, a challenge to each country. Recycled coarse aggregate has been studied extensively, and it is quite agreed that replacing up to 20% of the virgin coarse aggregate with recycled one may lead to only minor effects, if any, on the properties of the new concrete. Fine aggregate has a strong impact on the properties of concrete, mainly in its fresh state, and together with possible contaminations that may exist in recycled aggregate impose a big challenge on its recycling.

Tam and Tam [2] quote the difficulties of working with recycled aggregates as weak
interfacial zone, high porosity, poor grading, variations in quality, and highly impure.

Fine recycled aggregate are more susceptible to contamination with foreign materials than coarse fraction [3], causing negative effects on the mechanical properties of the recycled concrete, and on its durability [4].

C&DW is composed of a wide variety of elements, and the proportion of these materials is presented in different samples with great variability and heterogeneity. Tam, Gao and Tam [5] explain that this variability is due to the constant contamination of concrete waste with foreign materials and the presence of old cement mortar paste attached to the aggregates particles. According to Vieira and Molin [6] this is one of the reasons for the low usage of recycled fine aggregates in new concrete production. Rodrigues, Carvalho, Evangelista and Brito [7] found that the contaminants with negative effects (gypsum, clay) were concentrated in the fraction below 0.063 mm. Therefore, the authors suggested washing the aggregates to eliminate the fraction below 0.063 mm before using in concrete production to improve their quality.

Fine recycled aggregate was studied extensively in the past decade but many studies used artificial recycled fine material rather than fine aggregate derived from recycling plants. The artificial aggregate was manufactured from crushing concrete elements of known properties (Katz [8] and Evangelista and de Brito [9, 10]) thus its properties are well controlled but do not fully represent the properties of real C&DW from construction sites.

In this paper, the properties of concrete made from recycled fine aggregate prepared at the lab were studied and compared with those of natural crushed sand and fine aggregates extracted from recycling plants.

2 MATERIALS AND METHODS

Artificial recycled fine aggregate was manufactured by crushing in the lab old concrete prepared at water to cement ratio of 0.42 and having a compressive strength of 67 MPa at 28 days. The crushed aggregate was sieved over a 4.75 mm mesh to obtain the fine aggregate. This aggregate was denoted as RFA-C. Another type of artificial aggregate was manufactured by producing cement paste made from Portland cement (CEM I 52.5N according to EN 197) and water at water to cement ratio of 0.42. After 28 days of curing the hardened paste was crushed and sieved, yielding RFA-P. Crushed lime stone was used as a reference aggregate for this study, denoted REF.

Recycled fine aggregate from plants that process regional C&DW was collected from two plants, representing different locations in the Israel and different treatment methods. RA-3 and RA-4 were collected directly from the waste-stream after the initial stage of crushing and sieving. Aggregate RA-3 is further treated by washing and sieving yielding RA-1 and RA-2. It should be noted that the initial treatment (inspection and sorting) of the waste in the plant that produced RA-3 was much more stringent than in the other plant that produced RA-4.

Two water to cement ratios were studied: 0.4 and 0.6. Mortars were prepared at a constant ratio of cement: fine-aggregate of 1:2 wt. thus leading to the following ratios of 1:0.4:2 and 1:0.6:2 for cement:water:aggregate.

The properties of the aggregates as well as the compressive strength of the hardened mixes were tested.
3 RESULTS AND DISCUSSION

3.1 Aggregates' properties

Sieve analysis of all the tested aggregates is presented in Figure 1. It seems that both artificial recycled aggregates, RFA-C and RFA-P, and RA-4 have similar grading as the natural one whereas RA-3 contains more particles in the range of 0.15 mm to 0.30 mm. This aggregate (RA-3) was further treated by thorough washing and sieving to yield RA-1 (very fine fraction) and coarse sieving that yielded RA-2 (a coarser fraction). All the aggregates were further sieved over 4.75 mm mesh to limit the study to fine aggregate as defined in the standards (<4.75 mm).

![Figure 1: Sieve analysis of the tested aggregates.](image)

In addition to their size distribution, all the aggregates were tested for the following properties: fines (<75µm) content, absorption, specific gravity (SSD) and fineness modulus. The results are presented in Table 1. It can be seen that the aggregate derived from C&DW (RA-3 and RA-4) contain a significant amount of particles finer than 75µm (18.2% and 25.1%, respectively) compared with 12.4% in natural crushed stone. Fines content in RA-1 and RA-2 that produced from RA-3 was lower. It appears that the thorough washing of RA-1 could not remove all the fines thus 7.6% was identified in this aggregate. RA-2 that contains the coarser fraction of RA-3 was separated after regular washing and sieving and this was sufficient enough to remove most of the fine particles thus only 0.85% of fines was identified for this aggregate. The artificial aggregates prepared from old concrete or paste exhibited relatively low values of 4.4% and 7.2%, respectively. It is well agreed that the
presence of old cement paste in the recycled aggregate is responsible for the lower quality of the recycled aggregate, but it is possible to assume that the source of fines in C&DW is mostly from other sources other than old cement paste.

The absorption capacity of the recycled aggregates made from C&DW or old concrete was around 8% with two exceptions: i. low absorption values were identified in RA-1 and ii. very high absorption values (21.8%) were identified in RFA-P. It seems that most of the old paste was removed from RA-1 leaving mostly the original natural aggregate. Chemical analysis of this aggregate showed that it is made of SiO₂ coming from quartz, the natural sand used in the construction sites in vicinity to this recycling plant. The absorption value of this aggregate is slightly higher than that known for natural quartz sand from this area indicating the presence of residues of old cement paste remained despite the thorough cleaning process. Aggregate RFA-P was made from 100% old cement paste thus its high absorption value is expected.

The specific gravity of the aggregates correlates well with the absorption values, as seen in Figure 2. Higher values of specific gravity were found for the aggregates that exhibited lower absorption values and vice versa. The good correlation, including the value for the neat cement paste, indicates that the higher absorption is indeed a result of the presence of cement paste in the recycled aggregates. Representative SEM images of RA-1 and RA-2, showed small and relatively large amounts of old cement paste that still adhered to these aggregates (respectively) as seen in Figure 3.

The fineness modulus (FM) presents the trend shown also in Figure 1; RFA-C, RFA-P, RA-3 and RA-4 are quite similar in their grading (FM ≈ 3) while RA-1 and RA-2 represent the fine and coarse fraction of RA-3 (FM=1.15 and 5.37, respectively).

Table 1: properties of the tested aggregates.

<table>
<thead>
<tr>
<th></th>
<th>Crushed sand (REF)</th>
<th>RFA-C</th>
<th>RFA-P</th>
<th>RA-1</th>
<th>RA-2</th>
<th>RA-3</th>
<th>RA-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Content of material finer than 75 μm (%)</td>
<td>12.4</td>
<td>4.4</td>
<td>7.2</td>
<td>7.6</td>
<td>0.85</td>
<td>18.2</td>
<td>25.1</td>
</tr>
<tr>
<td>Absorption capacity (%)</td>
<td>2.4</td>
<td>8.9</td>
<td>21.8</td>
<td>2.5</td>
<td>6.6</td>
<td>8.7</td>
<td>7.76</td>
</tr>
<tr>
<td>Specific gravity – SSD</td>
<td>2.65</td>
<td>2.31</td>
<td>1.97</td>
<td>2.55</td>
<td>2.37</td>
<td>2.32</td>
<td>2.34</td>
</tr>
<tr>
<td>Fineness modulus (FM)</td>
<td>3.71</td>
<td>3.27</td>
<td>3.32</td>
<td>1.15</td>
<td>5.37</td>
<td>2.57</td>
<td>3.38</td>
</tr>
</tbody>
</table>
3.2 Compressive strength

Compressive strength of the mixes was tested at 7 days and 28 days. The results are presented in Figure 4. Gradual reduction in the compressive strength is seen as the quality of the recycled aggregate produced from C&DW is reduced, i.e. RA-1 with the best quality exhibited the least strength reduction and RA-4, with the lowest quality, exhibited the largest reduction. The difference between the compressive strength of mixes made from virgin aggregate or recycled aggregates was large in the low w/c mixes: a reduction of 34% (RA-1, 28 days) in mix with w/c=0.4 compared with a reduction of 20% only at w/c=0.6. It appears that the difference between the quality of the new cement paste and the quality of the
aggregates leads to this significant weakening of the mortar made with the recycled aggregates.

The two types of artificial aggregate exhibited almost the same compressive strength when used as aggregates for the production of a new mortar. At the low w/c ratio a small difference was seen between the strength of these mixes and at the high w/c the compressive strength of the mixes made with two types of artificial aggregates was the same. These results are interesting since RFA-P is made of 100% recycled cement paste while RFA-C is made from crushed concrete (though having the same original w/c ratio). It is possible that RFA-C is somewhat better than RFA-P and this minor difference could be seen only when the paste of the new mortar was made at low w/c ratio, as explained before.

At the low w/c ratio the artificial recycled aggregated exhibited strength level similar to that of RA-1 while at the higher w/c ratio it was similar to RA-2 or RA-3. Aggregated RA-2 is composed of virgin aggregates still connected by old cement paste thus influenced by the properties of the old paste, similar to RFA-C or RFA-P. Thus the high strength obtained at the low w/c ratio (similar to RA-1, a fairly clean recycled aggregate) is not yet fully clear.

![Figure 4: Compressive strength of the mixes at 7 days and 28 days.](image)

## 4 SUMMARY AND CONCLUSIONS

The properties of fine aggregates produced from old cement paste, old concrete and from construction and demolition waste were investigated, together with their effect on the properties of new mortar made from these aggregates.

It appears that the porosity of the recycled aggregates is larger than that of virgin aggregates. Thorough treatment produced aggregate with low absorption values, higher density and less
particles finer than 0.075 mm. Also, less old cement paste was seen in SEM micrographs of this aggregate.

The compressive strength of a new mortar made from these aggregates was lower than that of a reference mortar made with virgin aggregates. Lower w/c ratio increased the difference between the compressive strength of mortar made with virgin aggregate or made with recycled aggregates.

No significant difference was found between the compressive strength of mortars made with old neat concrete or old neat cement paste although the former contains, in part, virgin aggregates.

Mixes prepared with aggregates derived from the initial crushing and separation process at the recycling plants exhibited inferior strength results compared with clean aggregates made from clean concrete or paste. This indicates that additional parameters affect the properties of the new mortar in addition to the presence of old paste. This can be a result of variations in the properties of the old paste, presence of contaminants or other causes.

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FUTURE CEMENTS: RESEARCH NEEDS FOR SUSTAINABILITY AND POTENTIAL OF LC3 TECHNOLOGY

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Key words: Cement, Concrete, Environment, Durability

Abstract. Due to the enormous volumes produced the production of concrete accounts for around 5-10% of man-made CO₂ emissions. Consideration of possible routes to reduce this environmental impact indicates that the use of calcined clays in as supplementary cementitious materials in cement and concrete has by far the largest potential.

1 INTRODUCTION

We have all heard the headline figure that the production of cement and concrete accounts for 5-10% or worldwide manmade CO₂ emissions. What less of us realise is that this figure is incredibly low when we realise that concrete makes up more than half of the “stuff” produced by human beings every year. We need to appreciate that concrete is a very environmentally friendly material compare to alternatives. The large contribution of the sector presents an opportunity for improvement and we should focus our research efforts in directions likely to have the most impact.

2 OPTIONS FOR CO₂ REDUCTION

80-90% of the CO₂ emissions of concrete come from cement and most of this is accounted for by the clinker component. This suggests two strategies – reduce the CO₂ emissions of clinker or (partially) replace clinker or by other materials. As discussed in more detail in my paper [1] and by Gartner and Hirao [2] the first option has been comprehensively explored.

The energetics of clinker production in a state of the art kiln is one of the most highly efficient thermal processes existing running at up to 80% or the thermodynamic limit. Further the use of alternative fuels means that less than 20% of the energy needs to come from fossil fuels, the rest can be provided by waste materials. Thus the cement industry also provides a service; safely disposing of many otherwise problematic materials while making good use of their calorific potential.

The choice of alternative chemistries for cement is severely limited by the composition of the earth’s crust. Just 8 elements constitute 98% of this (O, Si, Fe, Al, Ca, Na, K, Mg) in fact only the CaO-SiO₂-Al₂O₃ system really needs to be considered for hydraulic binders, due to the very high solubility of hydrates based on sodium and potassium and the low mobility of iron and magnesium in alkaline solutions. In this system we find the calcium silicates which
make up Portland cement and the calcium aluminate phases CA and ye’elemite (C4A3S) which are the essential ingredients of CAC and CSA respectively. However, the production of cements with these calcium aluminate phases requires raw materials in which aluminium content significantly exceeds that of silicon. This makes means the production of these materials has a higher costs and limits them to niche applications.

By far the most successful strategy to reduce CO₂ emissions of cement production has been to replace part of the clinker in cement (or part of cement in concrete) by so called supplementary cementitious materials (SCMs). Figure 2 shows the evolution of substitution by CSI* companies over the past decades. We see that just 3 components: slag, fly ash and limestone constitute around 80% of cement substitutes. We can also see that the level of replacement is tending to stagnate in recent years. The main reason for this is the limited quantities of SCM available.

Slag is an excellent replacement for clinker: cements with 70% or more content of slag are widely used and show excellent performance, particularly regarding durability in marine and sulfate bearing environments. However, the amount of slag available worldwide is only about 5-10% of the amount of cement produced. Moreover, the distribution of slag is very uneven. A very few countries, such as Japan produce more slag than clinker, while the vast majority of countries do not produce amounts of any significance relative to cement production. It is unlikely that the supply of slag relative to cement will improve in the future. Demand for steel is increasing more slowly than that for cement and the production of steel is a highly CO₂ intensive process which drives the use of more recycled steel.

There is more fly ash available than slag, but the quality of fly ash is highly variable which means that only about 30% of fly ash is currently used in cement and concrete. While we may be able to find ways to increase this proportion, the supply of fly ash in the long term is uncertain due to pressure to reduce electricity production from coal (again one of the major contributors to CO2). In recent years we have seen the availability of fly ash in the USA fall dramatically as power stations switch to use shale gas.

The third major SCM, limestone, is available in virtually unlimited quantities, but additions of limestone alone at levels of more than around 10% lead to reduction in

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performance.

This limited availability of SCMs also highlights why the much trumpeted “clinker free” alkali activated materials (AAMs) (or geopolymers) will not provide an answer. These rely on the same SCMs as used for clinker replacement in blended cements. From a Global perspective we can consider that slag, fly ash, etc have the same potential to reduce the amount of clinker consumed either through their use in blended cements or as additions to Portland based concrete or in AAMs. The use in AAMs will on average, in fact, increase global CO$_2$ as the alkalis used for activation are not CO$_2$ neutral must be supplied.

2 OPTIONS FOR CO2 REDUCTION

In the 2009 report of the Cement Sustainability Initiative (CSI) of the World Business Council for Sustainable Development (WBCSD) the International Energy Authority (IEA) evaluated the potential of known technologies to reduce the overall CO$_2$ emission of the cement sector. Figure 1, taken from [3] shows the results for low and high demand scenarios for the growth in demand for cement. The blue portion shows the potential from the more widespread use of waste fuels, the red the potential contribution from the installation of more energy efficient plant and the rather thin green portion to potential from SCMs, which is limited by the relatively small quantities available. It was recommended that the remaining grey area, to reach the target of 18% reduction in cement sector CO2 emissions, should come from carbon capture and storage [4]. In the past few year demonstration projects have evaluated the possibility of this technology. Leaving aside technical issues, it is clear that even in the best case scenario in 5-10 years’ time CCS will be very expensive increasing the cost of clinker production 3-4 times! It is not realistic to suppose that such an expensive option will be adopted in developing countries in preference to investment in the development of infrastructure, where the supply of low cost building materials is essential.

So where can we go. By far the most viable route to further CO$_2$ reductions would be to find new sources of SCMs. There is much research published on the potential of agricultural ashes, but these are very limited in amount and the seasonality and geographical distribution mean that even the use of these limited quantities in concrete is usually not economically viable.
The only materials potentially available in the quantities required are calcined clays. The use of calcined clays as pozolanic materials has a long history. In the 1970s they were widely used in India before the use of fly ash became established. In Brazil they have been used for more than 50 years and currently account for a considerable proportion of the SCMs used in this country. While the use and durability of calcined clays as pozzolans is well established from a technical standpoint they are usually of marginal economic viability due to the need to calcine the clay. This situation has changed radically by the discovery [5] that the coupled substitution of calcined clays with limestone allows materials to be produced with good properties at very high levels of clinker replace. This is what we call LC3 technology (Limestone calcined clay cements). LC3 materials with only 50% clinker can develop the same or higher compressive strength as the reference Portland material from as little at 7 days.

The study of Antoni showed the potential of such materials with relatively pure calcined clays – metakaolin. Such materials are not of interest from an economic standpoint as the typical cost is 3 time that of Portland cement. However, even clay with relatively modest kaolin content, down to around 40% can show good performance. Figure 2 illustrates for a large number of clays that the single most important parameter is the kaolin content.

![Figure 2. Strength of Mortars according to European norm, for LC3 50 mixes with clays of different kaolinite contents](image)

3. INDUSTRIAL TRIALS OF LC3 CEMENT

LC3 materials are not just something that works well in the lab. Large scale industrial trials in Cuba and Indian have shown that these can be produced easily with existing technology. In 2013 110 tonnes of clay were calcined and 130 tonnes of LC3 cement were produced in Cuba in an old wet process kiln [7]. While the process was far from optimal the cements made with these materials were used successfully by unskilled workers as a one to...
one replacement to make cement blocks and concrete and used in construction, Figure 3. In India a first trial was made using static calcination. The materials produced could again be used as one to one replacement and the materials produced to build a demonstration house Figure 3 [8]. A second production of around 100 tonnes was made in a rotary kiln in January 2014, this was used for research purposes and for autoclaved aerated blocks used to build an annex building in the Swiss Embassy compound in New Delhi.

![Figure 3. Precast elements being made from LC3 cement in Cuba, left and demonstration house built from LC3 materials in India, right.](image)

**4. DURABILITY**

In the introduction of any new materials the key question is durability. Consequently we are undertaking numerous studies to verify the good performance of LC3 concretes. This includes both laboratory studies and field exposure. In Cuba 4 exposure sites have been established and the first results are described in another paper at this conference [9]. In India, reinforced concrete columns have been cast and erected at several sites in the different climatic zones of India.

First results from the field and lab indicate that LC3 materials have very good resistance to chloride penetration.
5. SUSTAINABILITY

Both in India and Cuba Detailed analyses have been made of the relative costs and CO3 savings possible with LC3 materials. In both countries it was demonstrated that LC3 saves about 30% CO2 relative to OPC and between 10 and 20% relative to the blended cement currently being used in those countries (fly ash in India and natural pozzolan in Cuba).

In terms of the CSI roadmap study shown earlier, the availability of calcined clays is virtually unlimited and would allow a reduction of the clinker factor well beyond the level of 71% by 2050 imposed by the projected availability of SCMs. For example, if the average clinker factor could be reduced to 60% through the use of calcined clays in addition to the SCMs currently used, the additional annual saving of CO2 would be 300 million tonnes – equivalent to the total annual emissions of a medium size industrialized country such as France.

6 CONCLUSIONS

The following conclusions are outlined:

- The realistic options for further reduction of CO2 emissions related to the production of cement and concrete are limited.
- LC3, the coupled substitution of clinker by calcined clay and limestone is probably the most an important additional technology to lower CO2 emissions and save resources
- Suitably clays widely available worldwide often as wastes
- Can be produced materials can be produced with existing technology
- Field trial have shown they are very robust – both in production and use
• The first indications show that the materials have good durability particularly in marine environments.

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SUSTAINABILITY APPLIED TO PREFABRICATION

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Key words: Sustainability, precast concrete structures, MCDM, MIVES

Abstract: Precast concrete elements are factory made products that permits to industrialize the construction industry and, therefore, to shift work from temporary construction sites to modern permanent facilities. Factory production entails rational and efficient manufacturing processes, skilled workers, systematization of repetitive tasks, and lower labor costs as a consequence of automated production. Automation is gradually being implemented in factories and is already in place in areas such as the preparation of reinforcing steel, mould assembly, concrete casting, and surface finishing on architectural concrete. And other stages in the processes that permits to increase the efficiency and productivity are sure to be followed. As there is a fib Bulletin under preparation in Commission 6 Prefabrication, by the Task Group 6.3 Sustainability, in which both authors are members, the conclusions of this document will be presented including a proposal for an evaluation model based on MIVES method that can be applied to assess the sustainability of precast concrete structures.

1 INTRODUCTION

As prefabrication makes optimal use of materials, its potential for savings is much greater than in cast-in-situ construction. Structural performance and durability are also enhanced through design, modern manufacturing equipment and carefully planned working procedures.

The environmental burden of prefabrication is mainly the burden caused by the raw materials of concrete (especially production of cement and steel). Usually, the environmental burden of precast concrete is less than that of other concrete construction because of the reduced use of materials in comparison with on-site construction [1-2].

In addition, thermal inertia of heavy materials is well known for both in warm and cold climates. Most people have experienced the comfort of coming into a comparatively cool stone building on a hot day in a warm climate. In precast structures, several systems have been developed using this characteristic.

Moreover, the use of new technologies like self-consolidating concrete (SCC) can significantly reduce the noise and vibration in the production process. The use of high-performance concrete (HPC) enables design and production of more reliable and more durable structures with optimized shape. The potential for savings in structural material consumption and consequently natural resources is evident [3].

However, the authors have not identified methods that allow assessing the global sustainability index associated to precast concrete systems with which the stakeholders can compare alternatives coupling objectively economic, social and environmental requirements and minimizing, at the same time, the degree of subjectivity in the decision process.

To this end, the MIVES method (from the Spanish, Integrated Value Model for Sustainability Assessment) is used in this research as a decision-making method to assess the sustainability of precast concrete structures/systems.

In this regard, the MIVES method is based on the use of value functions [4]. So far, MIVES has
already been used for industrial buildings [5-7], underground infrastructures [8], hydraulic structures [9], wind towers [10], sewerage systems [11], post-disaster site and housing selection [12-13] and construction projects [14-15]. It should be highlighted that in the current Spanish Structural Concrete Code [16] MIVES method is proposed for assessing the sustainability of concrete structures [17]. Finally, it must be added that the MIVES method has even been expanded to include the uncertainties involved in the process of analysis [18].

The main goal of this research contribution is to present the work in progress that is being carried out by the fib TG 6.3 Sustainability. Part of this task group is focused on proposing a model based on MIVES to assess the sustainability index of precast concrete elements/systems. The resulting fib bulletin will complement other bulletins on environmental aspects related to concrete structures [1; 19-20].

3 THE ESSENTIALS OF THE MIVES METHOD

3.1. Application stages

The assessment of the sustainability index by using the MIVES method should be carried out following these steps:

- Define the problem to be solved and the decisions to be made.
- Produce a basic diagram of the decision model, establishing all those aspects that will be part of a requirements tree that may include qualitative and quantitative variables.
- Establish mathematical functions to convert the qualitative and quantitative variables into a set of variables with the same units and scales.
- Define the importance or relative weight of each of the aspects to be taken into account in the assessment.
- Define the various design alternatives that could be considered to solve the previously identified problem.
- Evaluate and assess those alternatives by using the previously created model.
- Make the right decisions and choose the most appropriate alternative.

In this particular case, the problem to be solved (stage A of MIVES) consists in assessing the sustainability index of a precast concrete structure or product to be posteriorly compared with other possible alternatives.

The requirements tree (stage B of MIVES) consist in a hierarchical diagram (Fig.1) in which the various characteristics of the product or processes to be evaluated are organized, normally at three levels: indicators, sub-criteria, and criteria. At the final level, the specific requirements are defined and the previous levels (criteria and indicators) are included in order to desegregate the requirements; this permitting: (1) having a global view of the problem; (2) organizing the ideas and (3) facilitating the comprehension of the model to any stakeholder involved in the decision process.

Afterwards, mathematical elements from the general multi-criteria decision theory are used [21-23] to formalize a method to convert the different criteria magnitudes and units into a common, non-dimensional, unit that will be called value (stage C of MIVES). In this sense, it should be noticed that this method accounts for both qualitative and quantitative variables related with the indicators.

In any multi-criteria decision problem, the decision maker have to choose between a group of alternatives [22], this being discrete or continuous. In the field of the precast concrete technology, it is possible to enumerate all the existing alternatives to give response to a specific problem. Thus, when the preferences \(x\) are known with respect to a set of design alternatives \(\{x\}\), a value function \(V:P \rightarrow \mathbb{R}\) can be fixed such that \(P_x \succ P_x\) so that \(V(P_x) > V(P_x')\), \(P\) being equal to a set of criteria to be evaluated for alternative \(x\). The problem consists of generating a non-dimensional value function \(V(P_x)\) that reflects the preferences of the decision maker for each alternative while integrating all the criteria \(P_x = (P_{1,x}; P_{2,x}; \ldots; P_{N,x})\).
The solution is a function $V$ consisting of the sum of $N$ value functions $V_i$ corresponding to the $N$ criteria which comply with $V_i : P \rightarrow R$ so $P_{ix} \geq P'_{ix} \iff V_i(P_{ix}) > V_i(P'_{ix})$. For the case of problems structured in the form of a requirements tree, the resulting Sustainability Index (SI) can be assessed by using the Eq. 1.

$$SI = V(P_x) = \sum_{i=1}^{N} \alpha_i \cdot \beta_i \cdot \gamma_i \cdot V_i(P_{ix})$$  \hspace{1cm} (1)

In Eq. 1, $V(P_x)$ measures the degree of sustainability (value) of the alternative $x$ evaluated with respect to various criteria $P_x=(P_{1x};P_{2x};\ldots;P_{N,x})$ considered. $\alpha_i$ and $\beta_i$ are the weights of the criteria and sub-criteria to which each criterion $i$ belongs; $\gamma_i$ being the weights of the different indicators. $V_i(P_{ix})$ are the value functions used to measure the degree of sustainability of the alternative $x$ with respect to a given criterion $i$. Finally, $N$ is the total number of criteria considered in the assessment. Weights $\alpha_i$, $\beta_i$, and $\gamma_i$ represent the preference, respectively, of certain indicator ($\gamma_i$), sub-criterion ($\beta_i$) and criterion ($\alpha_i$).

The main objectives of the $V_i$ functions are:

- To homogenize the criteria units. In this regard, it is also highly recommended to delimit the values that these functions can generate. In this way, all the criteria have one single scale of assessment, normally between 0 and 1. These values represent the minimum and maximum degree of sustainability, respectively.
- To make it possible to weigh the $V_i$ functions by weights $\alpha_i$, $\beta_i$, and $\gamma_i$. It also makes it easier to obtain these weights ($\alpha_i$, $\beta_i$, and $\gamma_i$) since it will only be necessary to establish the relative priority of certain criteria, sub-criteria, or indicator with respect to other ones, regardless of whether some may present different scales of quantification.

Once the value functions have been defined, it is necessary to calculate weights $\alpha_i$, $\beta_i$, and $\gamma_i$ for each branch of the requirements tree (stage D of MIVES). To this end, numerical values established by experts in the field are used. First, the weights of each criterion ($\alpha_i$) are calculated. Then, within each criterion, weights $\beta_i$ for the several sub-criteria are calculated, and finally, the same process is done for each sub-criterion to obtain the indicator weights $\gamma_i$. 

**Figure 1**: General requirements tree.
The initial trees are often excessively complex, or discrepancies occur among the experts, or, simply, it is desirable to carry out an organized process to avoid difficulties in establishing the weights. In these situations, the analytic hierarchy process (AHP) [24-25] may be used. Afterward, to compensate for possible subjective bias because of the use of semantic labels in AHP, a subsequent process of analyzing, comparing and, in case of being necessary, modifying the resulting weights is recommended.

The various alternatives \( x \) are defined in the following stage (stage E). After that, these alternatives are evaluated (stage F), and the sustainability index associated with each of them is calculated by using Eq. 1.

### 3.2. Definition of the value functions

Defining the value function requires measuring preference or the degree of satisfaction produced by a certain alternative. Each measurement variable may be given in different units; therefore, it is necessary to standardize these into units of value or satisfaction, which is basically what the value function does. The method proposed a scale for which 0.0 reflects minimum satisfaction \( (P_{\text{min}}) \) and 1.0 reflects maximum satisfaction \( (P_{\text{max}}) \).

To determine the satisfaction value for an indicator, the MIVES model outlines a procedure consisting of:

- The tendency (increase or decrease) of the value function.
- The points corresponding to \( P_{\text{min}} \) and \( P_{\text{max}} \).
- The shape of the value functions (linear, concave, convex, S-shaped).
- The mathematical expression of the value function.

The general expression of the value function \( V_i \) used in MIVES to assess the satisfaction of the stakeholders for each indicator responds to the Eq. 2.

\[
V_i = K_i \cdot \left[ 1 - e^{-m_i \frac{P_{i,x} - P_{i,\text{min}}}{n_i}^A_i} \right]^{-1}
\]  

(2)

In Eq. 3, variable \( K_i \) is a factor that ensures that the value function will remain within the range of 0.0-1.0 and that the best response is associated with a value equal to 1.0:

\[
K_i = \frac{1}{1 - e^{-m_i \frac{P_{i,\text{max}} - P_{i,\text{min}}}{n_i}^A_i}}
\]  

(3)

In both Eq. 2 and Eq. 3:

- \( P_{i,\text{max}} \) and \( P_{i,\text{min}} \) are the maximum and minimum values of the indicator assessed.
- \( P_{i,x} \) is the score of alternative \( x \) that is under assessment, with respect to indicator \( i \) under consideration, which is between \( P_{i,\text{min}} \) and \( P_{i,\text{max}} \). This score generates a value that is equal to \( V_i(P_{i,x}) \), which has to be calculated.
- \( A_i \) is the shape factor that defines whether the curve is concave \( (A_i < 1) \), a straight line \( (A_i \approx 1) \) or whether it is convex or S-shaped \( (A_i > 1) \).
- \( n_i \) is the value used, if \( A_i > 1 \), to build convex or S-shaped curves.
- \( m_i \) defines the value of the ordinate for point \( n_i \) in the former case where \( A_i > 1 \).

The geometry of the functions \( V_i \) allows establishing greater or lesser exigency when complying with the requisites needed to satisfy a given criterion. For example, the convex functions experience a great increase in value for scores that are close to the minimum value, and the increase in value diminishes as the score approaches the maximum. This type of function is used when one wishes to encourage compliance with minimum requirements. That may be the case, for instance, with sufficiently exacting standards in which mere compliance is highly satisfactory. Another instance...
may be when the aim is to reward the use of new technologies, and their implementation is seen as very positive (even when it is a partial or a minor one), with a view to encouraging better practices. This is particularly the case, for instance, of using recycled aggregate.

It can be seen that the shape of the function depends on the values that the parameters $A_i$, $n_i$ and $m_i$. The interpretation of these parameters facilitates the understanding and the use of Eq. 2. Table 1 gives characteristic values of these parameters for the definition of value functions. This parameters may vary according to the preferences of the decision maker.

<table>
<thead>
<tr>
<th>Function</th>
<th>$n_i$</th>
<th>$m_i$</th>
<th>$A_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>$n_i \approx P_{i,\text{min}}$</td>
<td>$\approx 0.0$</td>
<td>$\approx 1.0$</td>
</tr>
<tr>
<td>Convex</td>
<td>$P_{i,\text{min}} + \frac{P_{i,\text{max}} - P_{i,\text{min}}}{2} &lt; n_i &lt; P_{i,\text{min}}$</td>
<td>$&lt; 0.5$</td>
<td>$&gt; 1.0$</td>
</tr>
<tr>
<td>Concave</td>
<td>$P_{i,\text{min}} &lt; n_i &lt; P_{i,\text{min}} + \frac{P_{i,\text{max}} - P_{i,\text{min}}}{2}$</td>
<td>$&gt; 0.5$</td>
<td>$&lt; 1.0$</td>
</tr>
<tr>
<td>S-shaped</td>
<td>$P_{i,\text{min}} + \frac{P_{i,\text{max}} - P_{i,\text{min}}}{5} &lt; n_i &lt; P_{i,\text{min}} + \left(\frac{P_{i,\text{max}} - P_{i,\text{min}}}{2}\right)$</td>
<td>$0.2 - 0.8$</td>
<td>$&gt; 1.0$</td>
</tr>
</tbody>
</table>

When the shape of the value function for an indicator is unclear, this may be defined by a working group. In these cases, several value functions (discrete or continuous) may be defined according to the members of the group. Therefore, a family of functions is obtained as can be seen in Fig. 2.

![Figure 2: Value function generated by a working group composed of different decision makers.](image)

The simplest way to solve these differences consist in taking the mean of the different values (after excluding extreme cases). The parameters $A_i$, $A_i$, $n_i$ and $m_i$ can then be estimated through a minimum squares approach. It is also possible to work with a range of values in such a way that two values correspond to each y-value (the mean and the standard deviation). This would require a statistical approach in the subsequent decision process.
4 REQUIREMENTS TREE PROPOSED BY THE FIB TASK GROUP 6.3 FOR THE SUSTAINABILITY ASSESSMENT

The requirements tree proposed by the members of the fib TG 6.3 Sustainability of Precast Structures is presented in Table 2. This tree has been established after several meetings of the subcommittee and this is meant to assess the sustainability index of structural concrete systems or its elements.

Table 1: Requirements tree proposed by the fib TG 6.3 for the sustainability assessment of structural concrete elements.

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Criteria</th>
<th>Indicator</th>
<th>Units</th>
<th>Value function</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1 Economic (λ_{R1} = 40%)</td>
<td>C1 Total Costs (λ_{C1} = 50%)</td>
<td>I1 Total costs (λ_{I1} = 100%)</td>
<td>€</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>C2 Quality (λ_{C2} = 10%)</td>
<td>I2 Non quality costs (λ_{I2} = 100%)</td>
<td>Attr.</td>
<td>DL</td>
</tr>
<tr>
<td></td>
<td>C3 Dismantling (λ_{C3} = 10%)</td>
<td>I3 Dismantling costs (λ_{I3} = 100%)</td>
<td>€</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>C4 Service Life (λ_{C4} = 50%)</td>
<td>I4 Maintenance (λ_{I4} = 50%)</td>
<td>€</td>
<td>IS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I5 Resilience (λ_{I5} = 50%)</td>
<td>€</td>
<td>IS</td>
</tr>
<tr>
<td>R2 Environmental (λ_{R2} = 45%)</td>
<td>C5 Materials consumption (λ_{C5} = 55%)</td>
<td>I6 Cement (λ_{I6} = 20%)</td>
<td>Tn</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I7 Aggregates (λ_{I7} = 20%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I8 Steel (λ_{I8} = 30%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I9 Water (λ_{I9} = 10%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I10 Plastics and others (λ_{I10} = 10%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I11 Reused Material (λ_{I11} = 10%)</td>
<td>Tn</td>
<td>IS</td>
</tr>
<tr>
<td></td>
<td>C6 Emissions (λ_{C6} = 55%)</td>
<td>I12 CO₂ emissions (λ_{I12} = 60%)</td>
<td>TnCO₂eq</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I13 Total waste (λ_{I13} = 40%)</td>
<td>Tn</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C7 Energy consumption (λ_{C7} = 10%)</td>
<td>I14 Embodied Energy (λ_{I14} = 33%)</td>
<td>MWh</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I15 Construction Energy (λ_{I15} = 33%)</td>
<td>MWh</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I16 Service Energy (λ_{I16} = 33%)</td>
<td>MWh</td>
<td></td>
</tr>
<tr>
<td>R3 Social (λ_{R3} = 15%)</td>
<td>C8 Interaction with third parties (λ_{C8} = 50%)</td>
<td>I17 Comfort. Thermal, air, noise (λ_{I17} = 50%)</td>
<td>Attr.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I18 Noise pollution. Construction (λ_{I18} = 10%)</td>
<td>Db</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I19 Particles pollution. Construction (λ_{I19} = 30%)</td>
<td>Tn</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I20 Traffic disturbances. Construction (λ_{I20} = 10%)</td>
<td>Attr.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C9 Risks (λ_{C9} = 50%)</td>
<td>I21 Health and Safety. Production (λ_{I21} = 33%)</td>
<td>Attr.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I22 Health and Safety. Construction (λ_{I22} = 33%)</td>
<td>Attr.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I23 User’s Safety (λ_{I23} = 33%)</td>
<td>Attr.</td>
<td>DS</td>
</tr>
</tbody>
</table>

DS: decreasing S-shape; IS: Increasing S-shape; DL: decreasing linear.

As it can be noticed, the requirements tree consists of 23 indicators (I) groups in 9 criteria (C). The number and type of indicators result from the different meetings carried out by the members of the committee; these indicators have been considered as those representatives to dealt with the sustainability assessment of construction systems or structural elements (in particular precast and in-situ constructed structural concrete elements) covering from the extraction of the materials to the dismantling operations.

The economic requirement (R1) is represented by four criteria related to costs: total (C1), quality (C2), dismantling (C3) and service life (C4) costs. Five indicators have been included; among these, the resilience, as beneficial property, is taken into account. The environmental requirement (R2) gathers three criteria: material consumption (C5), emissions (C6) and energy consumption (C7) and eleven indicators. Finally, the social requirement (R3) consist of two criteria: interaction with third parties (C8) and risks (C9).

The experts’ panel have fixed and defined these indicators aiming at guaranteeing the independence of these and avoiding potential overlapping. In this regard, more indicators could have been included; nevertheless, the addition of extra indicators (with weight below 5%) can: (1) difficult the application of the method (more indicators to be assessed); (2) potential overlapping
between the indicators and (3) lead to lose the general view of the problem. Thus, indicators with less than 5% of weight have been disregarded.

Finally, the distribution of weights proposed for this initial requirements’ tree has been fixed by using the direct assignment method; that is: the experts agreed the weights of each indicator based on their own experience. Needless to say that this distribution can be adjusted to other stakeholders’ preferences in order to take into account other economic scenarios or environmental and social sensitivities. In this regard, the next step to be carried out by the fib TG 6.3 is to send this requirements tree to the members of other fib commissions so that other distributions can be assumed. This process would help to guarantee the representativeness of the sustainability assessment method proposed.

The bulletin includes two real application examples with the aim of illustrating the whole sustainability assessment process using MIVES. Besides, other examples published in the research literature are cited in the references section of the bulletin.

4 CONCLUSIONS

In this research contribution, a method for assessing the sustainability index of construction system/elements based on the MIVES method has been presented. This consists in part of the work in progress carried out by the members of the fib TG 6.3 Sustainability of Precast Structures. The method allows:

- Assessing the sustainability of construction systems and, in particular, precast and in-situ structural concrete elements.
- Minimizing the subjectivity in the decision-making while considering the three main requirements of the sustainability: economic, environmental and social.

There is still work to be performed in order to calibrate the method and define all the components; however, to the authors’ knowledge, this would be the first sustainability assessment method available for precast concrete structures.

REFERENCES


SUSTAINABILITY ASSESSMENT OF CONCRETE WITH RECYCLED CONCRETE AGGREGATES

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Key words: concrete, sustainability, LCA, recycled aggregate, low CO₂

Summary. Concrete is the most widely used construction material. It causes a substantial environmental impact, derived from CO₂ emission, mainly during Portland cement manufacture because of the calcination of the limestone and the fuel consumption. There are several strategies for reducing the amount of Portland cement, e.g. using alternative clinkers, like alkali-activated cements or belite cements, or partial replacement of Portland by secondary cementitious materials, like fly ash, natural pozzolans and/or limestone.

Aggregates, as the largest component of concrete, can also have a significant effect on the environmental cost of the concrete mixture. The choice of aggregate influences a wide range of sustainability attributes and as with most aspects of sustainability there are frequently tradeoffs between one choice and another. Aggregate materials of some kind are usually available locally and making best use of this can be preferable for reducing transport related energy use and carbon emissions as well as keeping expenditures in the local economy.

In many countries recycled concrete aggregates (RCA) have been proven to be practical for low-strength concretes and to a limited extent for some structural grade concrete. When structures made of concrete are demolished or renovated, concrete recycling is an increasingly common method of utilizing the rubble. Using RCA for new concrete reduces the demand for virgin aggregate conserving natural resources, while minimizing the waste stream by diverting demolished material from landfill. However each case should be individually evaluated and transport and recycling process (sorting, crushing and sieving) must be taken into account.

This paper presents a review of the general strategies for reducing concrete environmental impact and a real comparative LCA evaluation of two types of aggregate for precast concrete elements, one natural and one recycled.
1 INTRODUCTION

About 25 billion tons of concrete are produced each year in the world, making concrete the most widely used construction material [1]. Concrete demand and production is expected to increase 2.5 times between 2005 and 2050 [2]. The largest share of this growth will take place in China, India and other countries on the Asian continent [3]. Concrete causes a substantial environmental impact, mainly during cement manufacture, because of such huge quantities [4].

Making one ton of cement requires about 2 tons of raw material (limestone and shale), consumes about 4 GJ of energy in electricity, produces 0.73–0.99 t CO₂/t clinker, about 3 kg of NOₓ, an air contaminant that contributes to ground-level smog, and about 0.4 kg of PM₁₀ – an airborne particulate matter that is harmful to the respiratory tract when inhaled [5]. The cement industry accounts for approximately 5 percent of current anthropogenic carbon dioxide (CO₂) emissions world-wide [4, 5]. A major difference between the cement industry and most other industries is that fuel consumption is not the dominant driver of CO₂ emissions. More than 50 percent of the CO₂ released during cement manufacture, approximately 540 kg CO₂/t clinker [5], is from calcination in which limestone (CaCO₃) is transformed into lime (CaO).

There is a diverse audience of decision makers and manufacturers who are interested in understanding and lowering the environmental impact of concrete and other building materials. Generic Life Cycle Assessment method and specific issues regarding concrete evaluation have been reviewed for this research paper in order to evaluate concretes from the environmental point of view.

2 ENVIRONMENTAL STRATEGIES

2.1 Cement

Cement manufacturing is an energy-intensive process. The cement subsector consumes approximately 12-15% of total industrial energy use. The typical electrical energy consumption of a modern cement plant is about 110–120 kWh per ton of cement. Electricity and fuel costs amount to nearly 40% of total cement manufacturing costs. Energy management is critical to the sustainability of the cement industry. The consumption of large amounts of energy leads to CO₂ and air pollutant emissions, which in turn require significant resources to manage. Energy efficiency, which includes sound plant-wide energy management practices combined with energy-efficient technologies, offers additional benefits, such as product quality improvement, increased production, and increased process efficiency. Common efficiency opportunities can be applied in most manufacturing plants regardless of the industry. Specific measures for energy efficiency in concrete manufacturing can be applied in the pneumatic conveying, cold weather concreting, humidity control, mixing, truck operation, curing, returned concrete, process water and others [6]. Over the past 20 years, the industry has increased energy efficiency by as much as 16%, and reduced GHG emissions by 10% [7].

Indirect emissions are produced by burning fossil fuels to heat the kiln. Kilns are usually heated by coal, natural gas, or oil, and the combustion of these fuels produces additional CO₂
emissions, just as they would in producing electricity. This represents around 40% of cement emissions. The electricity used to power additional plant machinery, and the final transportation of cement, represents another source of indirect emissions and account for 5-10% of the industry’s emissions.

The use of alternative fuels and raw materials is an important lever for the industry to reduce its reliance on fossil fuels such as coal and petroleum coke and thus reduce CO₂ emissions from cement production. Many of these alternative fuels are consumer waste or byproducts from other industries and recovering their energy value in cement making is a safe and proven form of energy recovery. This type of energy recovery reduces the need for fossil fuels while safely destroying wastes that would otherwise be deposited in landfills. Examples include construction and demolition wood wastes, municipal bio-solids, non-recyclable plastics and textiles, and the remaining tire fragments left over from tire recycling operations.

It is obvious that the hydration process and products are dependent on the mineral composition of Ordinary Portland Cement (OPC). Alite (C₃S) is the most important mineral phase of reactivity at early phases. However the possibility of increasing the content of Belite (C₂S) is appealing from an environmental point of view. Thus, the question that emerges is: Can the content of Belite optimized so as to have the largest content without compromising the mechanical performance? This question is indeed a hot topic in the state of the art, since recent experimental studies [8, 9, 10] and computational studies [11] have demonstrated that there is a synergy between the hydration process of Alite and Belite; namely, the heat release of mixtures of Alite and Belite exceeds the value expected from a simple compositional average of the heat release of Alite and Belite. This synergy translates into the formation denser hydration structures (with larger amounts of High Density (HD) C-S-H phases) and improved mechanical properties.

With the aim of developing new binders more “eco” than the OPCs, several alternative clinkers are currently under investigation. Alkali-activated cements (AAC) [12] typically consist of limestone, iron slag, coal fly ash or thermally activated clays, which are activated by an alkali metal solution. The binder material must contain a reasonable amount of a glassy or amorphous silica phase. AAC typically produce 95% less CO₂ emissions than a similar mass of OPC and is also beneficial from the point that it reduce the impact of other industrial by-products on the environment.

On the contrary of strategies focused on cement production – which can act only at the cement scale (substitution, alternative fuel, cement plant efficiency…), strategies that lead with the optimization of cement use can be developed also at concrete scale (use of dispersants, better packing, structural design) [13].

Although known for a long time, Belite cements (BC) have lately received great attention due to their environmental advantages in comparison to the ordinary Portland cement. The synthesis of Belite directly reduces CO₂ emissions by 33% and the amount of natural limestone needed to raw materials’ calcination. Besides, it is made at lower temperature (~600-1300°C), which reduces energy consumption [14]. However their low initial strength development has severely limited their use to few niches. Belite calcium sulfoaluminate (CSA) are surely among the most promising alternatives addressed in recent times. Belite CSAs are essentially belite rich binders with phases like yeelimite (C₄A₃S’), C₄AF and gypsum in varying ratios. The solidification of these binders is mainly based on the early formation of ettringite and the posterior densification of CSH phases. A clinker containing
yeelimite is produced at temperatures which are about 100 to 150°C lower than the sintering temperatures of Portland cement clinker. It can be made with natural materials or using high alumina fly ash as source of aluminum and sulfur. Due to the lower temperatures and the lower content of calcium, the specific CO₂ emissions of CSA cements are lower than those of Portland cement clinker [15]. However, the global availability of aluminum and sulfur sources suitable for the production of CSA cements is limited. Therefore, this group of binders can only play a subordinate role as a replacement for Portland cement clinker.

2.2 Other cementitious materials

CO₂-efficient binders (CEBs) are composed by partially replacing OPC, by one or more supplementary cementitious materials (SCMs), e.g., fly ash (an abundant industrial by-product, IBP), natural pozzolans and/or limestone etc. which display beneficial cementing behaviors. While attractive, it is challenging to replace ordinary Portland cement (OPC) by such SCMs, reliably and in large quantities (≥ 25%, by mass) due to: poor characterization of SCMs, large variability in some SCMs (e.g., fly ash), and limited supplies of quality SCMs. This results in compromised engineering properties/durability of the resultant concrete(s).

Special emphasis should be done on composing exemplar CEBs, which leverage OPC and fly ash as the major binder components, with limestone, natural pozzolans and other cements (e.g., high alumina cements) being added to tailor properties and reaction behavior. These multicomponent CEBs should be designed to display engineering performance equivalent to traditional OPC formulations, while reducing the binder-specific CO₂ impact.

2.3 Admixtures

The European Federation of Concrete Admixture Associations (EFCA), has published an eco-profile on superplasticizers [16]. It is worth noticing that the amount of CO₂ (720g/kg) emitted for the production of 1 kg of superplasticizer is only a little bit lower than the CO₂ emissions associated with the production of cement (842g/kg). The same is true for the NOₓ emissions (1.8g/kg versus 3.65g/kg). On the other hand, the amount of SOₓ emitted to manufacture the superplasticizer is significantly higher (3.6g/kg versus 0.53g/kg). However, as the amount of superplasticizer used in concrete is almost negligible when compared with its cement content, these emissions should not contribute significantly to the overall environmental impact.

2.4 Water

Water is the most consumed resource in the world. Beyond the basic needs of drinking water, personal hygiene and crop irrigation, water is used in abundance by industry, including the concrete industry. A low water-to-cement (W/C) ratio is advised especially in more demanding applications. To make sure that the water does not contain organic substances, chlorides or alkalis, drinkable water is usually applied in practice. To help reduce pressure on limited supplies of drinkable (potable) water, the concrete industry continues to address standards that allow greater use of no potable water in concrete and its production.

Four ways to use and recycle water are to reuse water on-site for repeated cycles of the same task, treat and reuse water on-site for multiple purposes, use graywater after solids have
been eliminated, and collect no potable water from such sources as rainwater, lakes, rivers, and ponds for use in construction [16].

Furthermore, mixtures with less water should be developed with new technologies to create mortar and concrete containing a minimal amount of water. As example, the incorporation of high volume of fly ash in concrete reduces the water demand.

### 2.5 Aggregates

Aggregates are the largest component of concrete (70% to 80% of the concrete volume). Research has shown that aggregate plays a substantial role in determining workability, strength, dimensional stability, and durability of the concrete. Increasing cement amounts in the mixture can increase strength up to a point when a “strength ceiling” is reached. At that point, the aggregate is the controlling factor and increasing cement content will not attain additional strength. Also, aggregates can have a significant effect on the environmental cost of the concrete mixture. The choice of aggregate influences a wide range of sustainability attributes and as with most aspects of sustainability there are frequently trade offs between one choice and another. Aggregate materials of some kind are usually available locally and making best use of this can be preferable for reducing transport related energy use and carbon emissions as well as keeping expenditures in the local economy.

Using recycled concrete aggregate (RCA) for new concrete reduces the demand for virgin aggregate conserving natural resources, while minimizing the waste stream by diverting demolished material from landfill. Recycled normal-weight and lightweight concrete can also be used for a variety of applications where virgin material (whether natural or manufactured gravel) has been traditionally used, such as aggregate base coarse under pavement and water drainage areas. When crushed concrete is reused on site, transportation costs are also avoided twice—shipping demolished material to disposal and virgin material to the site.

In many countries RCA have been proven to be practical for low-strength concretes and to a limited extent for some structural grade concrete. Alternative aggregates from selected materials and industrial by-products, on the other hand, have greater potential for use in concrete and/or as road construction materials.

### 3 LCA FOR CONCRETE

The Life Cycle Analysis (LCA) is an objective process for evaluating the environmental impacts associated to a certain system/product. A LCA tries to calculate in a rigorous and scientific manner the use of energy, water and natural resources, the emissions and releases into the air, ground and water, and the waste generation. LCAs conducted for product improvement can reveal processes, components, ingredients, and systems to target for environmental improvement.

Choices made during the planning phase of an LCA have a profound impact on the results obtained. The choice of functional unit, particularly when LCAs are conducted to compare products, is especially influential. When it comes to comparing concrete only dosages which fulfill the same function (resistant category, environmental exposure (durability), service life or life design, docility, etc.) can be compared. Another critical step in any LCA is the compilation of a credible life-cycle inventory (LCI), upon which subsequent life-cycle impact assessment can be based [19].
Current approaches also encourage problem shifting. For instance, construction is one of the largest users of energy, raw materials and water. However, energy efficiency and water conservation are prioritized on the green agenda while the key environmental impacts associated with materials use, namely, depletion of resources; and releases of solid and liquid wastes and toxic emissions to air are barely addressed.

Within the construction industry, LCA studies can be performed at many different levels. First of all, there is the cradle-to-gate approach, which only considers the impact of raw material extraction, the production of materials and product parts until the end product leaves the gate of the factory. E.g., the utilization phase is excluded when the type of construction where the concrete is used is not identified. It is assumed that concrete is inert during this phase. However, a simple cradle-to-gate analysis cannot be used when evaluating the environmental benefit of potential ‘green’ concrete types.

A cradle-to-grave approach on the other hand, can take the latter aspect into account, since the LCA looks at the material’s impact over its entire life cycle. However, in contrast with the well-known concrete production process, representative data related to the use phase and end-of-life phase for a specific concrete structure, are not always known. To get a fair picture of the environmental impact of concrete, the time-frame must be sufficiently long. However, the carbonization of concrete is not usually taken into consideration since the duration of this process is too long.

As the aim for sustainability should stimulate profound recycling and reuse, the cradle-to-cradle concept [20] is gaining importance today (Figure 1). When concrete constructions are demolished, the concrete can usually be used as filling material in road construction or as new aggregate material in concrete production. Larger pieces of concrete elements can be used as embankment material instead of being crushed. Reusing concrete leads to a reduced use of virgin natural resources such as stone and gravel [21].

Either way, ISO 14040-14044 specify a four step LCA methodology for each approach: definition of goal and scope, inventory analysis, impact analysis and interpretation [22].

3.1 Definition of functional unit and boundary limits

The definition of the functional unit (FU) is of great importance. This unit is seen as the reference unit of the product system for which the environmental impact will be calculated. When comparing the environmental impact of different concrete mix designs, a small scale FU on the material level can be appropriate. On the other hand, the use of a high strength concrete implicates that structure dimensions and thus the overall concrete amount needed can be reduced considerably. The resulting environmental benefit should be visible in the LCA output.

An efficiency indicator similar to the one proposed by [23] is seen as a first example, although it should not only be expressed in terms of a compressive strength unit, but also relate to a unit of service life. This way, additional concrete manufacturing due to replacement or repair over time, is included in the LCA [24].

Regarding the scope of the LCA, system boundaries must be described clearly using a flow diagram or process tree. For instance, transport and intermediate preparation of materials can be incorporated in different ways.
Finally, the necessary criteria regarding the quality of the data used in the LCA need to be set. The following data requirements should be addressed: time-related coverage, geographical coverage, technology coverage, precision, completeness, representativeness, consistency, reproducibility and (un)certainty of the information [22].

Figure 1: Life cycle stages, inputs and outputs for concrete life cycle assessment.

3.2 Data collection and allocation

The information obtained directly from the industries involved using detailed questionnaires results in a more reliable LCI. However, first hand data are not always provided by the companies because of confidentiality issues. As a consequence, the larger part of the LCIs is based on data from environmental reports (ERs), environmental product declarations (EPDs) and publicly available LCA related journals. Obviously, the first possibility gives the most accurate results and is the recommended method [24].

Attention needs to be paid to allocation. Whenever a system produces more than one product, the environmental impacts have to be divided over the different end products. Allocation is of particular importance when FA or BFS are in play. Whenever these materials are used as a cement replacing material, attention needs to be paid to their allocated environmental impacts. For most concrete experts, FA and BFS are considered as avoided wastes necessary for the production of coal-based electricity and steel, so their environmental load is at expense of the producer. Another references point out that the use of these by-products is sometimes necessary or highly recommended to achieve a high performance. Under this perspective, economic allocation is advised as baseline method for most allocation situations in a detailed LCA [25].

3.3 Impact analysis

This phase of LCA is aimed at evaluating the significance of potential environmental impacts based on the LCI flow results. Classical life cycle impact assessment (LCIA) consists of the following mandatory elements:
• selection of impact categories, category indicators, and characterization models;
• the classification stage, where the inventory parameters are sorted and assigned to specific impact categories; and
• impact measurement, where the categorized LCI flows are characterized, using one of many possible LCIA methodologies, into common equivalence units that are then summed to provide an overall impact category total.

In many LCAs, characterization concludes the LCIA analysis; this is also the last compulsory stage according to [22]. However, in addition to the above mandatory LCIA steps, other optional LCIA elements – normalization, grouping, and weighting – may be conducted depending on the goal and scope of the LCA study. In normalization, the results of the impact categories from the study are usually compared with the total impacts in the region of interest. Grouping consists of sorting and possibly ranking the impact categories. During weighting, the different environmental impacts are weighted relative to each other so that they can then be summed to get a single number for the total environmental impact. ISO 14044:2006 generally advises against weighting, stating that “weighting shall not be used in LCA studies intended to be used in comparative assertions intended to be disclosed to the public”.

Two methods can be distinguished:
• Problem oriented methods or classical impact assessment. Its category indicator is located right in between the LCI results and the category end points (where the environmental effect or damage occurs). A material’s impact on climate change can be expressed in kilograms CO₂ equivalents. Obviously, this is merely a quantification of an emission that contributes to the problem of climate change and not a quantification of the actual environmental damage.
• Damage oriented impact methods, much more focuses on the actual effect. They try to model the cause effect chain up to the endpoint, or the actual environmental damage, sometimes with high uncertainties. With respect to climate change, the damage on human health is quantified in terms of disability adjusted life years (DALYs). This unit counts as a measure for the Years Lived Disabled (YLD) and the Years of Life Lost (YLL) due to this damage.

3.4 Interpretation

Life Cycle Interpretation is a systematic technique to identify, quantify, check, and evaluate information from the results of the life cycle inventory and/or the life cycle impact assessment. The results from the inventory analysis and impact assessment are summarized during the interpretation phase. The outcome of the interpretation phase is a set of conclusions and recommendations for the study. According to ISO 14040:2006, the interpretation should include:
• identification of significant issues based on the results of the LCI and LCIA phases of an LCA;
• evaluation of the study considering completeness, sensitivity and consistency checks; and
• conclusions, limitations and recommendations.
Classification is the process of assigning and aggregating results from the inventory into relatively homogeneous impact categories. This process involves identifying stressors and organizing them with respect to impact on the ecosystem. For example, carbon monoxide, chlorine and methane are all stressors with the potential to impact the environment under the category of global warming. Classification includes the creation of complex stressor/impact chains, because a single pollutant can have multiple impacts, and a primary impact can result in secondary (or greater) impacts through cascading impact chain process. The general categories are: ozone depletion, acid rain potential, photochemical oxidant impact, global warming potential, controlled toxic water mass, solid mass disposal, safety risk, human health risk, ecological risk and natural resource depletion.

4 INTEGRATION OF LCA AND MULTI-CRITERIA

LCA is a powerful tool to assess environmental impacts of concrete. It does not however solve all dilemmas for the decision-maker: as LCA revolves around indicators for environmental impacts, some key impacts/values can be left out, namely social values or economic choices. Moreover, although some LCA methods offer the possibility to opt for a single environmental indicator, this is somewhat in contraction with the principle of transparency.

Multi-Criteria Analysis (MCA) appears like a complementary tool to LCA and has been used by practitioners around the world [26]. These two techniques can be combined to achieve practical and transparent objectives for the decision makers.

Valuation is assigning relative values or weights to different impacts, economic and performance measures to determine the total impact. An analytic hierarchy process (AHP) is used for this purpose, which involves a structured description of the hierarchical relationships among various decision attributes. Assigning weights is a group exercise, where experts are asked to reach on a consensus. The experts may include cost estimator, process specialist, environmental engineer, and/or ecologist/toxicologist. A research for obtaining an integrated evaluation method specifically addressed for concrete and cement is currently being developed in the framework of Superconcrete project [27].

5 COMPARATIVE LCA OF NATURAL AND RECYCLED AGGREGATE IN THE NORTH OF SPAIN

A small precast concrete elements producer decided to replace the coarse aggregates of its concrete by recycled concrete aggregates (RCA) and deliver to the market slabs and paving blocks with high rate of recycled materials in the North of Spain in 2015 [28].

Apart from the direct benefices in Green Procurement, the precast concrete producer was interested into have a realistic view of the environmental benefits of the new concrete. It was a concrete with the same formulation as the one with natural aggregates with the only difference of the 100% replacement of the natural aggregates by concrete recycled aggregates from selected slabs and blocks. For clarity purposes, the results presented in this paper are focused just in the aggregates.

The functional unit considered is 1 ton of aggregates (0-5mm). Most of the data have been supplied by the manufacturing company. Additional information has been gathered in public data bases and annual reports from transportation and energy production industries.
In the respective total energy consumption, the respective processes of the aggregates production have been included: extraction, processing and transportation for the natural aggregates and recycling treatment (crushing and sieving) and transportation for the recycled concrete aggregates. The impacts from slabs and blocks removal during urbanization works and transportation to the treatment plant have not been allocated into this analysis.

The energy consumption includes the necessary electricity and diesel for producing 1t of aggregates. Diesel consumption varies heavily depending on multiple factors: location, quality of the rocks, experience of the labor, local transportation, etc. Average diesel consumption of 0.6l/t has been estimated.

Transportation of the raw materials is a relevant factor in the economic feasibility for the manufacturing of these products. In this case, cement and aggregates are transported by 6t truck. Natural aggregates are obtained in a near quarry 2.6km from the manufacturing plant. Recycled aggregate are collected from an authorized plant for inert materials 12.5km away.

The obtained results for the main impact categories with corresponding units are shown in Fig.2. It can be observed that the natural aggregate option has lower environmental impacts for this particular case according with LCA method. This is because of the higher transportation distance for obtaining the recycled aggregate and the high availability of natural material in near quarries which minimizes the relative effect of local resources depletion.

### LCIA Results

This table shows the LCIA results of the project variants. Each selected LCIA category is displayed in the rows and the project variants in the columns. The unit is the unit of the LCIA category as defined in the LCIA method.

<table>
<thead>
<tr>
<th>Impact category</th>
<th>Natural agg.</th>
<th>Recycled agg.</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acidification potential-average Europe</td>
<td>1.06968e-2</td>
<td>1.28667e-2</td>
<td>kg SO2 eq.</td>
</tr>
<tr>
<td>Climate Change-GWP100</td>
<td>1.77149e+0</td>
<td>2.10352e+0</td>
<td>kg CO2 eq.</td>
</tr>
<tr>
<td>Depletion of abiotic resources — elements, ultimate reserves</td>
<td>1.48876e-7</td>
<td>3.24928e-7</td>
<td>kg antimony eq.</td>
</tr>
<tr>
<td>Depletion of abiotic resources — fossil fuels</td>
<td>4.37045e+1</td>
<td>8.42857e+1</td>
<td>MJ</td>
</tr>
<tr>
<td>Eutrophication — generic</td>
<td>5.97278e-4</td>
<td>7.67927e-4</td>
<td>kg PO4--- eq.</td>
</tr>
<tr>
<td>Freshwater aquatic ecotoxicity— FAETP inf</td>
<td>4.40119e-3</td>
<td>7.65882e-3</td>
<td>kg 1,4-dichlorobenzene eq.</td>
</tr>
<tr>
<td>Human toxicity – HTP inf</td>
<td>1.30581e-1</td>
<td>1.64281e-1</td>
<td>kg 1,4-dichlorobenzene eq.</td>
</tr>
<tr>
<td>Marine aquatic ecotoxicity – MAETP inf</td>
<td>1.09575e+2</td>
<td>1.61427e+2</td>
<td>kg 1,4-dichlorobenzene eq.</td>
</tr>
<tr>
<td>Ozone layer depletion – ODP steady state</td>
<td>2.52194e-7</td>
<td>2.61132e-7</td>
<td>kg CFC-11 eq.</td>
</tr>
<tr>
<td>Photochemical oxidation – high Nix</td>
<td>5.94997e-4</td>
<td>8.08468e-4</td>
<td>kg ethylene eq.</td>
</tr>
<tr>
<td>Terrestrial ecotoxicity – TETP inf</td>
<td>2.04045e-3</td>
<td>3.49395e-3</td>
<td>kg 1,4-dichlorobenzene eq.</td>
</tr>
</tbody>
</table>

**Figure 2:** Impact category values for the two evaluated options
6 CONCLUSIONS

Objective evaluation of concrete environmental sustainability is mandatory and some methods are used for this purpose. However there is a lack of consensus about the most adequate impact criteria to be considered and the methods and results are not always consistent or comparable.

Simplified indexes with updated life-cycle inventories require a minimum effort and can be useful as a first environmental evaluation of concrete formulations or OCP replacement by supplementary cementitious materials. Nevertheless accurate comparisons and new low-energy embodied cements evaluations can only be performed through a detailed review of all the materials and processes involved in the life cycle of the concrete.

LCA and Environmental Product Declaration (EPD) are the most frequently used tools for benchmarking and marketing purposes. However they not always promote the use of recycled materials and do not consider other than environmental aspects.

Local conditions (e.g. lack of spaces for landfilling, high dumping taxes, necessity of promoting materials recycling and reusing, etc.) are hardly ever taken into account in existing tools. In the case presented in this paper it has been observed that replacement of natural aggregate by recycled aggregate is not always convenient from an environmental perspective. This implies both than LCA assumptions should be reconsidered and additional aspects might be included to do a global analysis.

7 ACKNOWLEDGEMENTS

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8 REFERENCES


A FIRST APPROACH: TOWARDS A SUSTAINABLE CIVIL ENGINEERING WORKS WITH PRECAST CONCRETE SOLUTIONS

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Key words: precast concrete, industrialization, sustainability, civil engineering works

Abstract. Most of the advances carried out to set standardized methodologies to quantify the contribution to “sustainabilize” the construction are linked to buildings rather than infrastructures, and much more in particular to housing [1]. Global impact on housing are the widest and highest one, gathering the three sustainable axis: environmental (greenhouse gas emissions derived from heating or cooling to reach indoor comfort conditions), social (home is a basic need for families) and economic (it usually represents the main expense over the life of people).

Meanwhile civil engineering work has not evolved as long on this topic. Although we generally refer to greater constructions, sustainable impacts are more diffused and don’t have such a direct repercussion into the citizens and daily life.

For this reasons, there is no as many literature and investigation as in housing. It may implies a technical and promotional handicap to promote a higher use of precast concrete elements in a field governed by engineers that appreciate better their performance advantages.

This paper is intended to describe the strengths (and weaknesses) that precast concrete construction will have into the upcoming standards for civil engineering, in order to enhance their possibilities to reach a greater market share. Sustainable indicators on current draft standards will be assessed.

1 INTRODUCTION

Concepts of sustainability and sustainable development are mentioned in nearly every activity related to the use of resources, energy or the outdoor environment. But they are not new terms, having evolved significantly over the last few decades until having such an
enormous importance in many decisions to be taken nowadays, especially on those countries or economies more developed where the concerns about consequences of climate change, lack of energy or demographic growth are crucial matters.

Construction has a tremendous influence from the economic, social (as employment generator, or as a way to solve some of basic needs of people, like housing or infrastructures) and environmental (use of natural resources, energy, or damages to the environment) points of view.

Governments are more conscious about the current construction model can (and must) improve quite much in terms of potential reduction on greenhouse gases (30 – 40%), water consumption (12 – 20%), primary energy consumption (35 – 40%), raw material consumption (30 – 40%) or land occupation (20%) [2] [3] [4].

It is clear that making constructions “greener” will have a significant impact on general goals of administrations and the whole society. However, most of the construction rules qualified as sustainable are not new at all being some used in the long past when a more responsible use of the available resources was surely made, there was either not another possibility or the absence of a culture to make some architectonic excesses that we have looked recently.

But sustainable approach may also run the risk to be misunderstood due by a certain excessive use. We are living times that many construction products are directly defined as sustainable so a more moderate use of this concept should be made. By this way, we must be very careful with the interpretation and the acceptation of everything provided as sustainable, putting it on the right context. For instance, timber is always presented as the most sustainable one against other construction materials, not taking into account any factor such as the climatic, social, economic and cultural context of the location where the material is used, causing sometimes skepticism about the concept itself.

2 SUSTAINABILITY MEASUREMENT FRAMEWORK

Several procedures are already available to assess how sustainable is a building or an infrastructure. They may be divided into private methodologies and standardized models. Two principal private certification systems are BREEAM [1] which was the first building certification process developed in 1990 in the UK by the Building Research Institute; and LEED [2], developed in 1996 and operated by the U.S. Green Building Council. Both rating systems are expanded worldwide. SBTool (Canada), HQE (France) or DGNB (Germany) are other applicable certification systems. One common aspect to all of them is that they are addressed to buildings.

For the sustainable assessment of infrastructures, certification methodologies like CEEQUAL or SUNRA are remarked.

The recent proliferation of these procedures makes it difficult to make meaningful comparisons between programs, or even between environmentally-conscious construction and traditional construction. As a reaction to such numerous methods for measuring the environmental (sustainable) performance of products, the main standardizations organizations worldwide, CEN (European Committee for Standardization) and ISO (International Organization for Standardization) are now developing their own standards. ISO deals with
sustainability aspects on their committees TC207, ISO TC59 SC17 and ISO TC71SC8. While CEN through its TC 350 focuses on sustainability of construction works, having a group specifically devoted to civil engineering works issues (CEN/TC 350/WG6).

The assessment methods for environmental, social and economic performance of civil engineering works given in the standards take into account performance aspects and impacts that can be expressed with quantifiable indicators, which are measured without value judgements and which lead to a clear result for each indicator.

ISO 21931-2 [4] and EN 15643-5 [5] are the standards which set out the framework for methods of assessment of the sustainability performance of civil engineering works. Both standards are still under discussion so their approvals won’t happen until 1 or 2 years.

Moving to a product level, ISO 21930 [6] and EN 15804 [7] must be highlighted. Both standards present a similar format. They provide core product category rules (PCR) for Type III environmental declarations for any construction product and construction service, defining the parameters to be declared and the way in which they are collated and reported, the stages of a product’s life cycle that are considered in the EPD, the rules for the development of scenarios. This standards set up the basis to estimate the values corresponding up to more than 20 environmental parameters, that can be gathered into three main groups: Environmental Impact Indicators (Global Warming Potential, Ozone Depletion Potential, Acidification potential, etc.); Resource Use Indicators (use of renewable primary energy, use of non renewable primary energy, use of net fresh water, etc.); and Waste Category Indicators (hazardous and non hazardous waste disposed, Radioactive waste disposed). The assessment of social and economic performances at product level is not yet covered by European standards, at least through European scale.

And in particular, the European Standardization Committee for precast concrete products CEN/TC 229 has just initiated the works to define a specific product category rules (PCR) guidance for the development of Type III environmental declarations for precast concrete products according to EN 15804.

It is also important to remark the following fact. Against the approach followed by most of construction materials which only declare their environmental parameters just up to the end of production process, no taking into account the rest of impacts (which is called as cradle to gate option), precast concrete products environmental declarations will be likely based on the cradle to grave option, showing the users all the impacts along the whole life cycle, including also the deconstruction and reuse of the elements for another construction.

3 ROLE OF THE PRECAST CONCRETE SOLUTIONS TO INCREASE THE SUSTAINABILITY OF THE CIVIL ENGINEERING WORKS

There is increasing demand for construction options that will contribute to achieving sustainable development so the precast industry is becoming more resource efficient and environmentally aware and how its products can contribute to achieving greener construction.

The concept of civil engineering works includes a wide range of projects, in which the precast concrete elements play each time a more relevant role:
Table 1: Precast concrete elements for civil engineering works, following the clause 5 of ISO 21931-2 [3]

<table>
<thead>
<tr>
<th>Civil engineering works fields</th>
<th>Infrastructures</th>
<th>Precast concrete elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Industrial processes infrastructures</td>
<td>Power generation plants</td>
<td>Walls or any other precast concrete structure</td>
</tr>
<tr>
<td>Utilities for conveying electricity, gas, water, etc.</td>
<td>Pipes, manholes and inspection chambers, curb inlets and catch basins, box culverts</td>
<td></td>
</tr>
<tr>
<td>Water and other treatment systems</td>
<td>Wastewater and septic tanks, grease interceptors</td>
<td></td>
</tr>
<tr>
<td>Energy generation and supply</td>
<td>Masts for wind turbines, electrical lines, telecommunication lines, supports for lighting, etc.</td>
<td></td>
</tr>
<tr>
<td>Linear infrastructures</td>
<td>Bridges</td>
<td>Deck elements, girders, piers, abutments</td>
</tr>
<tr>
<td>Footbridges</td>
<td>Any element is prefabricable</td>
<td></td>
</tr>
<tr>
<td>Highways</td>
<td>Safety barriers, paving slabs, sound walls</td>
<td></td>
</tr>
<tr>
<td>Railways</td>
<td>Sleepers, slab tracks</td>
<td></td>
</tr>
<tr>
<td>Tunnels</td>
<td>Vaults, segmental linings</td>
<td></td>
</tr>
<tr>
<td>Dams and other fluvial works</td>
<td>Canals</td>
<td>Precast slabs and walls</td>
</tr>
<tr>
<td>Flood defence</td>
<td>Retaining walls systems</td>
<td></td>
</tr>
<tr>
<td>Maritime works</td>
<td>Harbour</td>
<td>Slab berths, pavements</td>
</tr>
<tr>
<td>Breakwater</td>
<td>Solid blocks for levees</td>
<td></td>
</tr>
<tr>
<td>Other civil engineering works</td>
<td>Public realm works</td>
<td>Pavements, urban furniture</td>
</tr>
</tbody>
</table>

The sustainable design of buildings is different as an infrastructure has. While in the first case some requirements like fire resistance, thermal or acoustic insulation are essential, civil engineering works are moving on different criteria. Actually, there is a different distribution of the importance of the life cycle stages. While the operational phase in buildings is the main one (responsible of around the 80% of the total carbon footprint), in infrastructures the construction usually implies greater impacts rather than the service period.

Some of the assumed sustainable aspects were already intrinsic part of the manufacturing process of precast concrete products carried out for the last few decades, like the correct use of the materials or an increased use of high strength concrete, but precast concrete elements have still a big potential of growth:
Figure 1: Sleepers and segmental linings play a crucial role in the construction of railway lines and tunnels as it is currently happening in two of the biggest works at the new crossing of the Bosphorus Strait in Istanbul and the new Crossrail line in London.

Figure 2: I-section beam. One of the most common profiles to design and build the deck bridge.
Table 2: Several advantages of precast concrete elements for civil engineering works, analysed from triple dimension of sustainability

<table>
<thead>
<tr>
<th>Precast concrete characteristics</th>
<th>Environmental</th>
<th>Social</th>
<th>Economic</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Durability (extended life)</strong></td>
<td>Good long-term solution means the preservation of natural resources, reduction of impacts, energy savings and increased potential for extracting natural resources</td>
<td>Longer life of infrastructures means less disruptions for citizens</td>
<td>Initial costs are amortized along a longer period Less maintenance (fewer costs)</td>
</tr>
<tr>
<td><strong>Industrialized processes</strong></td>
<td>Lean construction: nearly no waste Dry construction: precast concrete arrives on site ready for installation</td>
<td>Increased safety: fewer labour accidents</td>
<td>Sooner refund of loans</td>
</tr>
<tr>
<td><strong>Resource efficiency</strong></td>
<td>Reduction of the consumption of natural resources by using waste materials in products (ex. recycled aggregates from concrete wastes) Partial elimination of a global problem</td>
<td>Increased use of better materials (high-resistance/performance concrete, prestressed steel) means an optimized ratio materials/cost</td>
<td></td>
</tr>
<tr>
<td>Increased use of self-compacting concrete (SCC)</td>
<td>Reduces its electricity consumption No vibrations makes the factory much quieter and safer</td>
<td>Reduction of losses by sick leaves</td>
<td></td>
</tr>
<tr>
<td>New raw materials (as fibers)</td>
<td>Increase the characteristics (i.e. mechanical) reducing the global environmental impact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Raw materials origin</td>
<td>Local supply network means travel distances are shorter and so the carbon footprint is reduced All the materials come from natural and recycled sources, mainly inorganic</td>
<td>Materials are available locally, enhancing regional economies and employment</td>
<td></td>
</tr>
<tr>
<td>Carbonation</td>
<td>Reabsorption of CO2 from the atmosphere Partial elimination of a global problem</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Photocatalysis</td>
<td>Diminish the air polluting effect by exhaust gasses (NOx, etc.) Reduction of respiratory diseases</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4 PRACTICAL CASES

Two important international companies like FCC and ACCIONA must be underlined. FCC has its own sustainability assessment methodology for civil works, called SAMCEW, which has into account the experience learnt from its own system of sustainability management for
the last years. It is based on a methodology of flexible analysis depending on the type of civil work, the location, the project characteristics or the evaluated phase, which will decide what parameters will have more impact than others. ACCIONA have carried out some interesting initiatives based on the quantification of the sustainability rate of works, becoming an international pioneer of the development of Environmental Product Declarations for infrastructures, as the railway bridge “Arroyo Valchano” [8] for the high-speed line between Madrid and Galicia which included an analysis of all materials used from cradle to gate, or the road span of the N-340 in Elche, both in Spain.

At a research level, it should be underlined the work "Cost and CO2 emission optimization of precast–prestressed concrete U-beam road bridges by a hybrid glowworm swarm algorithm" [9]. This research describes a methodology to optimize cost and CO2 emissions when designing precast–prestressed concrete road bridges with a double U-shape cross-section. To this end, a hybrid glowworm swarm optimization algorithm (SAGSO) is used to combine the synergy effect of the local search with simulated annealing (SA) and the global search with glowworm swarm optimization (GSO). The solution is defined by 40 variables, including the geometry, materials and reinforcement of the beam and the slab. Regarding the material, high strength concrete is used as well as self-compacting concrete in beams. Results provide engineers with useful guidelines to design PC precast bridges. The analysis also revealed that reducing costs by 1 Euro can save up to 1,75 kg in CO2 emissions. Finally, the parametric study indicates that optimal solutions in terms of monetary costs have quite a satisfactory environmental outcome and differ only slightly from the best possible environmental solution obtained.

![Figure 3: Construction of a maritime bridge using precast concrete box girders](image)

5 UP COMING CHALLENGES OF THE INDUSTRY

The precast concrete industry acknowledges that has not only a responsibility to improve
its performance on sustainability demands but this global approach should serve as a way to increase its competitiveness, being an ideal basis to enhance all its potential in terms of the characteristics described above, which are really appreciated by designers, owners, builders, insurers and even final users.

One of the apparent weaknesses of the concrete products is the cement, and in particular the clinker which is responsible of such a high amount of CO2 emissions. By this way, the precast industry works hard to improve the efficiency of cement use by maximizing hydration and by optimizing cement content to reduce embodied CO2. The use of other cementitious materials such as ground granulated blastfurnace slag and pulverized fuel ash is also growing. Both these additions have much lower embodied CO2 than Portland cement. [10]

Sustainability in precast factories is about improving resource efficiency, reducing waste and ensuring that standards such as ISO 14001 and EMAS are maintained. New factories incorporate major recycling systems to re-use the water or any other material waste. None of this would be possible without the sector’s highly skilled and dedicated workforce who are critical to its success and, in many cases, are the driving force behind advances in energy saving techniques, recycling initiatives and local community liaison programmes.

The “Precast Sustainability Strategy and Charter” from the British Precast Concrete Association must be underlined [11]. It was first introduced in 2007 endorsed by only 17 companies. Today, and as part of the “Raising the Bar” initiative, all full member companies of British Precast are now committed to the Precast Sustainability Charter. It is to encourage member companies of the Association to go beyond legislation and take voluntary actions to make their products and operations more sustainable. In order to meet this aim, a set of sustainability principles has been developed based on the key sustainability issues facing the precast industry. First period of analysis was from 2008 to 2012 and 12 of 14 affixed goals were achieved. In 2013, the British Precast Council approved a new set of targets by 2020 based on 2012 as the baseline year:

• Reducing overall kWh/ tonne of energy used in production by 10%
• Reducing CO2 emissions for production by 20%
• Reducing overall factory waste by 10%
• Reducing factory waste to landfill to < 0.5 kg/ tonne
• Increasing the proportion of alternative cement additions (as a % of total cement) to 25%
• Increasing the proportion of recycled/ secondary aggregates (as a % of total aggregates) to 25%
• Reducing mains water consumption by 20%
• Reduction in accident frequency of 50% between 2015 and 2020
• Increasing the tonnage, as well as production sites, covered by an EMS (e.g. ISO 14001) to 95%
• Increasing the tonnage, as well as production sites, covered by a quality system (e.g. ISO 9001) to 95%
• Increasing the tonnage, as well as production sites, covered by a Responsible Sourcing standard (e.g. BES 6001) to 95%
• Reducing the convictions for air and water emissions to zero
• Improving the capture of transport data up to 2015 (A target will be set for 2016)
• Increasing the % of employees covered by a certified management system (e.g. ISO
9001/ ISO 14001/ OHSAS 18001) to 100%

- Increasing the % of employees covered by MPA Safer by Competence training and qualifications to 100%
- Maintaining the % of relevant production sites that have community liaison activities at 100%

Other important issue to be dealt with will be the compliance of the precasters with the new essential requirement “Sustainable use of natural resources” according to the Construction Products Regulation [11]. The construction works must be designed, built and demolished in such a way that the use of natural resources is sustainable and in particular ensure the following:

- Reuse or recyclability of the construction works, their materials and parts after demolition
- Durability of the construction works
- Use of environmentally compatible raw and secondary materials in the construction works

The industry is assessing how best to present clear Life Cycle Analysis (LCA) data, potentially in the form of verified Environmental Product Declarations (EPDs) which comply with new European Standards that will ultimately feed into Building Information Modelling (BIM) systems. This will allow designers to achieve measurably low impact buildings and infrastructures based on reliable whole life data. This will also serve the manufacturers to optimize their production processes throughout a higher resource efficiency (materials, water and energy), the minimization of waste and even the innovation to use alternative sources of energy or materials.

This environmental (sustainable) concern shall become a progressive stimulus to all the enterprises. This approach is been introducing by means of the decisions taken derived from public procurement procedures as it happens in Sweden or Norway, or more recently in the region of Basque Country (Spain) which stands for the use of precast products as a way to no producing wastes.

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A STUDY INTO THE RELATIONSHIPS BETWEEN THE MECHANICAL PROPERTIES OF RECYCLED CONCRETE AGGREGATES

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Key words: Recycled Aggregate Concrete; Mechanical Properties.

Abstract. The advances in construction industry have exhausted the available natural resources and placed high impact on the surrounding environment. This has encouraged the engineers to search for more innovative materials in design. The use of Recycled Concrete Aggregate (RCA) in new concrete offers a solution to such a problem of worldwide dimensions. That is to make construction more “green” and environmentally friendly and preserve resources; hence contribute to sustainability development.

In this paper, a detailed investigation is conducted to analyze the relationships between the mechanical properties of Recycled Aggregate Concrete (RAC). Five concrete mixes were designed to study the effect of different RCA content on concrete mechanical properties: concrete made entirely with natural aggregate as control mix and four types of concrete made with natural fine and recycled coarse aggregate (30%, 50%, 75% and 100% replacement of coarse recycled aggregate). The source of RAC in this study was from the concrete specimens tested in the laboratory. Based on the results of sixty specimens carried out; the relations between compressive strength, density, splitting tensile strength, flexural strength and elastic modulus are investigated and discussed in details. A comparison is made against the suggested equations in Codes of Practice and by other researchers. It is found that such relations could be different from that of conventional concrete and new equations need to be developed.
1. INTRODUCTION

The use of Demolished Concrete Waste (DCW) in the production of new concrete is becoming increasingly important for sustainability development and a viable alternative to the consumption of natural resources. RAC is beneficial and necessary from viewpoint of environment protection and resources reservation. The work on RAC goes back to World War II and since then a lot of research has been reported in literature such as the following references [1 till 14]. In fact, none of previous research indicated that RCA is unsuitable for structural application. However, there is still insufficient information on RAC durability and on the performance of structural elements made with such material.

Lebanon, where 85% of construction demolition wastes is cementitious material which is fully recyclable [15], has not yet considered recycling. The most important reasons for not accepting construction and demolition waste as source for aggregates in structural concrete is the lack of legislation due to the fact that most of the research on RCA was conducted outside Lebanon, limiting the adoption of the findings locally because of variations in materials and quality control. To advance the use of RCA in structural applications, it is necessary to study the properties of local materials and the effect of different RAC sources and replacement levels on the mechanical properties of concrete and develop a mixture design methodology based on that.

This paper investigates the effect of different replacement levels of recycled concrete aggregates on the properties of fresh and hardened concrete. Four levels of replacement were considered: 30, 50, 75 and 100%. Results were compared with a control concrete mix prepared using only natural aggregates. Slump loss with the elapsed time was recorded for the fresh concrete. Seven and twenty-eight day compressive strength, tensile strengths, moduli of rupture and elasticity were also measured for hardened concrete. The experimental values obtained for different properties were compared with design equations proposed by ACI 318-14, EC2 and by other researchers [16, 17, 18, 19].

2. EXPERIMENTAL PROGRAM

The experimental program carried out was divided into three phases: characterization of constituent materials, concrete mixture proportioning and study of the mechanical properties of concrete.

Characterization of constituent materials

Ordinary Portland cement, natural aggregates, water and high-range water-reducing admixture were used for the different concrete mixes prepared. The RCA in this study was obtained by crushing concrete cylinders that were cast or brought to the civil engineering lab for testing their compressive strength at 28 days age, as shown in Figure 1.

After crushing, using concrete crusher seen in Figure 2, the aggregates were sieved into two sizes: medium aggregates (4.75-9.5 mm) and coarse aggregates (9.5-19 mm), as per ASTM C33 [20]. The particle size distribution of both natural coarse aggregates (NCA) and RCA were within the margins specified in ASTM C33. The test results on the physical properties of natural and recycled aggregates are given in Table 1. The acceptance criteria according to different standards are presented in Table 2.
Table 1: Properties of coarse aggregates used in present study.

<table>
<thead>
<tr>
<th>Property</th>
<th>RCA</th>
<th>NCA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(4.75 – 9.5 mm)</td>
<td>(9.5 – 19 mm)</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.416</td>
<td>2.457</td>
</tr>
<tr>
<td>Bulk density</td>
<td>1305</td>
<td>1359</td>
</tr>
<tr>
<td>Water absorption (%)</td>
<td>6.744</td>
<td>4.927</td>
</tr>
<tr>
<td>LA % of Mass Loss</td>
<td>33.37</td>
<td>22.1</td>
</tr>
</tbody>
</table>

Table 2: Acceptance regarding recycled concrete aggregates.

<table>
<thead>
<tr>
<th>Country or standard</th>
<th>9.5-19mm RCA</th>
<th>4.57-9.5mm RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Australia (AS1141.6.2, AS 1996)</td>
<td>Class 1A</td>
<td>Class 1B</td>
</tr>
<tr>
<td>Germany (DIN 4226-100) (DIN 2002)</td>
<td>Type 1</td>
<td>Type 1</td>
</tr>
<tr>
<td>Hong Kong (Works bureau of Hong Kong 2002)</td>
<td>Accepted</td>
<td>Accepted</td>
</tr>
<tr>
<td>Japan (JIS A 5021, 5022 and 5023) (JIS 2011, 2012a, b)</td>
<td>Coarse -Class M</td>
<td>Class M</td>
</tr>
<tr>
<td>Korea (KS F 2573) (KS 2002)</td>
<td>Korea (Fine)</td>
<td>Not accepted</td>
</tr>
<tr>
<td>RILEM ( 1994 )</td>
<td>Type 2</td>
<td>Type 2</td>
</tr>
<tr>
<td>Spain (EHE 2000)</td>
<td>Accepted</td>
<td>Not accepted</td>
</tr>
</tbody>
</table>

In this study only the coarse aggregates were replaced as it is not recommended to replace sand. The RCA were dried in the oven at 110±5 °C and then stocked in closed containers until the mixing time.
Mix proportions

Five concrete mixes using different recycled and control aggregates were prepared. The main variable was the percentage of RCA, in order to study its effect on the mechanical properties. The water/cement ratio was kept constant. The replacement levels of recycled aggregates were 30, 50, 75 and 100%; and the mixes were named accordingly R30, R50, R75 and R100. The control mix with 100% natural aggregates is referred to as R0. Recycled aggregates have not been soaked in water and the amount of absorbed water was calculated and added to the mixing water. The mix design is presented in Table 3.

Table 3: Mix Design

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement</th>
<th>NCA 9.5-19 mm</th>
<th>NCA 4.75-9.5 mm</th>
<th>RCA 9.5-19 mm</th>
<th>RCA 4.75-9.5 mm</th>
<th>Natural Sand</th>
<th>Free Water</th>
<th>Superplasticizer</th>
</tr>
</thead>
<tbody>
<tr>
<td>R0</td>
<td>350</td>
<td>666</td>
<td>444</td>
<td>0</td>
<td>741</td>
<td>175</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>R30</td>
<td>350</td>
<td>466.2</td>
<td>310.8</td>
<td>182.54</td>
<td>122.63</td>
<td>741</td>
<td>175</td>
<td>1.4</td>
</tr>
<tr>
<td>R50</td>
<td>350</td>
<td>333</td>
<td>222</td>
<td>304.23</td>
<td>204.38</td>
<td>741</td>
<td>175</td>
<td>1.4</td>
</tr>
<tr>
<td>R75</td>
<td>350</td>
<td>166.5</td>
<td>111</td>
<td>456.35</td>
<td>306.57</td>
<td>741</td>
<td>175</td>
<td>1.4</td>
</tr>
<tr>
<td>R100</td>
<td>350</td>
<td>0</td>
<td>0</td>
<td>608.46</td>
<td>408.76</td>
<td>741</td>
<td>175</td>
<td>1.4</td>
</tr>
</tbody>
</table>

* NCA and RCA refer to natural coarse aggregates and recycled coarse aggregates respectively.

Knaack and Kurama [21] explored three RCA mixture design methods: direct weight replacement (DWR) method, equivalent mortar replacement (EMR) method and the direct volume replacement (DVR) method. They found that these different mixture design methods result in similar compressive strength and elastic modulus of RCA concrete; however, the workability of RCA concrete maybe considerably reduced when using the equivalent mortar replacement method. In this work the DVR method was used and the results were compared with a previous work by the authors [22] where DWR is used. The comparison is not shown here. The DVR method is provided in ACI 211 [23].

Casting, curing and testing

All ingredients were fed into 0.1 m³ pan mixer. Half the quantity of coarse aggregates was first added and was dry mixed for few revolutions, followed by the natural sand, cement and the remaining coarse aggregates. Water containing high-range water-reducing admixture was added and mixed for three minutes; followed by three minutes rest then two minutes mixing time. Slump of each fresh concrete mixture was measured at 0 (initial), 30, 60 and 90 minutes after completion of mixing. Bulk density and air content of fresh concrete were also measured. Results of fresh concrete testing are presented in Table 4.

A total of 12 specimens were cast for each mix, six cylinders 150 by 300 mm for the compressive strength at ages of 7 and 28 days, three cylinders for the splitting tensile test at 28 days age and three beams 150 by 600 mm for the flexural tensile test also at the age of 28 days.
Immediately after casting, all specimens were covered with plastic sheets till next day. After demoulding, the specimens were transferred to a curing tank at constant temperature of 23±2 °C, and remained there till the time of testing.

Table 5 summarizes all test results on hardened concrete. Each result is average of three specimens’ tests. All tests are done in accordance with the relevant ASTM standards.

### Table 5: Results of hardened concrete tests

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Compressive strength $f'_c$ (MPa)</th>
<th>Density $\rho$ (kg/m³)</th>
<th>Splitting tensile strength $f_{sp}$ (MPa)</th>
<th>Flexural tensile strength $f_r$ (MPa)</th>
<th>Modulus of elasticity $E_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 days</td>
<td>28 days</td>
<td>28 days</td>
<td>28 days</td>
<td>28 days</td>
</tr>
<tr>
<td>$R0$</td>
<td>19.56</td>
<td>30.35</td>
<td>2259</td>
<td>3.05</td>
<td>5.75</td>
</tr>
<tr>
<td>$R30$</td>
<td>17.66</td>
<td>30.23</td>
<td>2249</td>
<td>3.04</td>
<td>5.85</td>
</tr>
<tr>
<td>$R50$</td>
<td>18.24</td>
<td>31.1</td>
<td>2215</td>
<td>2.8</td>
<td>5.67</td>
</tr>
<tr>
<td>$R75$</td>
<td>19.27</td>
<td>27.9</td>
<td>2175</td>
<td>3.03</td>
<td>5.4</td>
</tr>
<tr>
<td>$R100$</td>
<td>17.6</td>
<td>29.4</td>
<td>2206</td>
<td>2.78</td>
<td>5.58</td>
</tr>
</tbody>
</table>

### 3. RESULTS ANALYSIS AND DISCUSSION

#### Initial slump

To measure the workability of each mixture, slump test in accordance with ASTM C143 was conducted at different times and the results are presented in Table 4. It can be seen that the initial slump decreased with increasing the replacement levels, reaching 17% for R100. Yang et al [24] conducted tests on nine recycled aggregate concretes and a control concrete using only natural aggregates. The recycled aggregates used were classified into three different types according to their measured specific gravity and water absorption. He concluded that various mixing conditions such as w/c, water-reducing admixture ratio, and grading and volume of recycled aggregates would control the initial slump of recycled aggregate concrete.
Slump loss

To study the loss of workability with time, a proposed equation by Yang and Kim [17] was used to analyse the results. The equation suggests that the slump of fresh concrete decreases linearly with the elapsed time:

$$\frac{S_L}{(S_L)_i} = kT + 1$$  \hspace{1cm} (1)

Where $(S_L)_i$ is the initial slump measured immediately after mixing in (mm); $S_L$ is the slump of fresh concrete tested at optional time T (in minutes); and $k$ is the rate of slump loss (mm/minute). Results are presented in Figure 3.

It can be seen from the figure that the slope of slump loss increases with the increase of replacement level of recycled aggregates thus reducing the workability of fresh concrete.

Compressive strength

The development of compressive strength with time is shown in Figure 4. It can be seen that the early age strength is not much affected by the RCA replacement ratio; while for the 28 day strength, the graph suggests no significant reduction in strength for ratios up to 50%. The largest reduction in strength of 8.1% is noted for the 75% replacement ratio. Surya et al [25] concluded that the replacement of NCA with RCA did not have any adverse effect on the strength of the concrete. Yang et al [24], who studied the development of compressive strengths for RAC specimens for ages up to 91 days, concluded that the development of long-term strength is more favourable than that of natural aggregate concrete (NAC).
It is known that the compressive strength of normal concrete increases with a rising of the mass density. Xiao et al [4] studied the relationship between the compressive strength and the mix density and suggested the following equation based on linear regression model:

\[ f_{cu} = 0.069 \rho - 116.1 \]  

(2)

\( f_{cu} \) is the cube compressive strength in MPa and \( \rho \) is the mass density (kg/m\(^3\)). This equation is drawn in Figure 5 against test results, taking \( f'_c = 0.76 f_{cu} \) based on the results of reference [26]. The equation seems to underestimate \( f'_c \) as the RCA level increases.

**Splitting tensile strength**

Figure 6 presents the splitting test results. Compared to R0, R30 and R75 mixtures, a lower splitting tensile strength was recognized for R50 and R100 concrete mixtures, the reduction
was 8% and 9% respectively. Larger reduction values were reported in Ref. [27] up to 20% for R100. On the other hand, several researchers (e.g., McNeil and Kang [14]) have shown that the splitting tensile strength of RCA concrete is comparable to the conventional concrete; attributing that to the residual mortar which in limited quantities and while creating a weakened spot for compressive failure to occur, improves the tensile capacity by creating a smoother transition between mortar and aggregate. The results are compared against the following proposed equations that relate the splitting tensile strength to the compressive strength:

\[ f_{sp} = 0.56 \sqrt{f_{cu}} \]  
\[ f_{sp} = 0.53 \sqrt{f_{cu}} \]  
\[ f_{sp} = 0.24 f_{cu}^{0.65} \]

It can be seen from the figure that ACI equation gives good estimate for R0, R30 and R75 mixture while it overestimates the strength for R50 and R100 mixes. Similar results could be seen for Oluokun equation. Xiao et al proposed formula underestimated the results for all concrete mix types. It should be noted again that in Equation (5) a conversion factor of 0.76 from cylinder to the cube compressive strength has been undertaken according to Ref. [26].

Flexural tensile strength

The flexural tensile strength, also known as the modulus of rupture, of all concrete mixes are reported in Table 5 and presented in Figure 7. There is an adverse effect of the replacement of NCA with RCA on the flexural tensile strength with a maximum reduction of 6% noted for R50. The relation between compressive and flexural strengths of RAC is suggested by Xiao et al [4] and is represented by Eq. (6). ACI equation (7) is also shown.

\[ f_r = 0.75 \sqrt{f_{cu}} \]
$f_r = 0.62\sqrt{f'_c}$ \hspace{1cm} (7)

It can be inferred from Figure 7 that Xiao et al [4] equation gives better estimate of the tensile strength than ACI equation, but both underestimate the results.

![Figure 7: Experimental and predicted flexural strength of NAC and RAC mixtures](image)

**Modulus of elasticity of concrete**

The elastic modulus of NAC is found to be higher than the respective values of RAC, as shown in Table 5 and Figure 8. The elastic modulus of R30 and R50 has decreased by 18% and 16% respectively. This reduction in the modulus of elasticity associated with different RCA replacement levels suggests that these mixes are more prone to deformations than NAC. Similar results are obtained by other researchers such as Li X. [28] who recorded 45% reduction in $E_c$ for RCA 100% replacement ratio compared to 10% only in the current study. According to McNeil and Kang [14], “the main factor affecting the RCA concrete modulus of elasticity is the modulus of elasticity for the aggregate itself”. The variation between studies as to how much the modulus is reduced is attributed to the different properties of the aggregate used in each study.
There are several proposed equations to describe the relation between compressive strength and elastic modulus for RAC:

Wardeh et al [27]  
\[
E_c = 17,553 \left( \frac{f_c}{10} \right)^{0.42} 
\]  

Xiao et al. [4]  
\[
E_c = \frac{10^5}{2.8 + \frac{40.1}{f_{cu}}} 
\]  

The test results of the present study have been compared with those computed using Eqs. (8) and (9) and also compared with ACI and Eurocode Equations (10, 11) for NCA. The comparison is presented in Figure 8.

ACI-318:  
\[
E_c = 4700\sqrt{f'_{c}} 
\]  

EC2  
\[
E_c = 22000 \left( \frac{f_{cu}}{10} \right)^{0.3} 
\]  

Xiao et al, Wardeh et al and ACI equations seem to underestimate the modulus of elasticity for all concrete mixtures. Eurocode relation underestimated \( E_c \) values for R0, R50 and R100 but it gave good estimate for R75 and an overestimation for R30.

4. CONCLUSIONS

Four recycled aggregate concretes and a control concrete using only natural aggregates were tested to investigate the effect of recycled aggregate amount on fresh and hardened concrete properties. The replacement of coarse aggregate was considered only in this study with the following ratios: 30, 50, 75 and 100% in separate mixtures. The direct volume replacement method was used. The properties of concrete tested were examined and
compared with empirical equations for natural aggregate concrete recommended in some
codes of practice in addition to some relations suggested by different researchers.

The following conclusions may be drawn:
- Generally the initial slump decreased with the increase of recycled aggregate content.
The rate of slump loss was the same for R75 and R100.
- It is confirmed that the replacement ratio of coarse aggregates up to 30% has no adverse
effect on the compressive and tensile strengths of concrete. However, a reduction of
18% is recorded in the modulus of elasticity.
- The compressive strength of R50 and R100 mixtures were observed to be comparable to
that of natural aggregates concrete NAC (R0); but lower tensile strength and modulus of
elasticity were observed for both ratios.
- Mix R75 exhibited the maximum reduction in compressive strength (8%), which could
be attributed to its low density as compared with other mixtures.
- The proposed relationships between the mechanical properties of normal weight
concrete in ACI Code and Eurocode do not predict adequately the mechanical properties
of recycled aggregate concrete.
- Concrete researchers have proposed various alternative relations for the mechanical
properties of recycled aggregate concrete but seem inadequate. Further investigations on
this regard are necessary.

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A STUDY OF THE SUSTAINABILITY POTENTIAL OF CEMENT-REDUCED CONCRETE

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Key words: Cement-reduced concrete, green concrete, mix design, sustainability

Abstract. Much focus within the sustainability debate has been placed on the building industry, as the production of building materials, the erection of structures and their use is a major source of environmental impact. The production of the concrete constituent material cement alone is responsible for a substantial share of global anthropogenic carbon dioxide emissions. The principle of sustainable structural engineering is that the energy and resources consumption and emissions due to the construction and operation of a structure must be minimized. Relating to concrete structures this principle can be applied by the use of the material in the most efficient way considering its strength and durability within the service life of the structure. Green concretes with reduced cement content may provide a feasible alternative for improving concrete sustainability independently of supplementary cementitious materials. For this purpose, cement-reduced, green concrete is developed by a design process centered on packing optimization of the granular mix constituents, nearly reducing the cement content by two thirds. The results indicate that the concretes developed with the presented concrete design algorithm have a high level of technical performance, but still show deficits in workability. As the sustainability of concrete is not only a function of absolute technical performance and environmental impact, the durability of these new concretes must also be thoroughly scrutinized.

1 INTRODUCTION

The sustainability debate has been prominently ongoing in the construction industry for many years and also in particular in concrete construction. Efforts to improve the sustainability of concrete make use of fundamentally different approaches in concrete technology. Portland cement clinker has, to a large extent, been replaced by secondary cementitious raw materials such as granulated blast-furnace slag, fly ash or limestone powder, which introduces problems into controlling the properties of fresh concrete and affects its strength development. Another trend is leading towards combining several types of binders (3 or more) into one composite binder or replacing Portland cement clinker by new types of binders altogether, such as calcined clays [1] or Celitement [2]. Finally, various authors – including those of the present contribution – have presented concretes with a significantly
reduced cement or cement clinker content [3, 4, 5]. The latter will in the following be referred to as green concretes.

In a first step, the approaches and exercises mentioned above aim at minimizing the environmental impact associated with cement and concrete production. Depending on the approach chosen and the parameter examined, the environmental impact of concrete production can be reduced significantly in comparison with standard concretes not optimized from an ecological point of view. The question must currently be asked, however, whether such a reduction in environmental impact will ultimately not be made at the expense of the sustainability of the construction material or structure, since not only low environmental impact but also high performance and durability are required in particular for the construction of civil engineering structures. This aspect will be investigated in a second contribution to this conference (see [6]). In the paper at hand, the procedure for developing cement-reduced concretes will be outlined and the properties of such concretes will be presented.

The starting point in optimizing the sustainability potential of building materials and structures lies in determining their environmental impact in a standardized manner within a lifecycle assessment as described e.g. in the European standards EN ISO 14040 [7] and EN ISO 14044 [8]. In order to design ecologically improved concretes, the amount of raw materials with pronounced environmental impact must be reduced in favour of raw materials with minor impact without negatively affecting both the performance and the durability of the concrete. In contrast to other investigations found in literature (see e.g. [3, 4]), no secondary cementitious materials were used in this investigation, as the lifecycle impact data of these raw materials is questionable and the availability e.g. of fly-ash or slag is declining. Instead, pure Portland cement with high strength was used and its content in concrete reduced in a stepwise manner.

2 INVESTIGATED RAW MATERIALS

The aim of the research project was to exploit the performance qualities of each concrete raw material in an optimal fashion. Since the life cycle impact data of the low and high strength cement differ only minimally (see [6]), a Portland cement CEM I 52.5 R in accordance with [9] and a very finely ground micro-cement were selected for the development of green concrete. Hereby it was attempted to insure a good dispersion of the cement clinker particles within the granular mixture. Selected properties of the cements investigated in this paper are presented in Table 1.
Natural aggregate components from the upper Rhine valley in accordance with [14] were used in the project. Additionally, two quartz powders were added to ensure a sufficient content of fine components in the concrete with decreasing cement content. Representative life cycle impact data for the Global Warming Potential (GWP) of the cements and inert materials is presented in Table 1 and Table 2. With regard to the packing density optimization of the granular mixture, consisting of cement and aggregates, components with narrow particle size distributions were deliberately selected. An exception to this is the sand fraction 0.1/1, which has a bimodal particle size distribution. Selected properties of the mineral components used are presented in Table 2. The particle size distribution curves of the cements and inert mineral components used are shown in Fig. 1.
Table 2: Properties of inert, granular materials used (sample)

<table>
<thead>
<tr>
<th>Property</th>
<th>Dimension</th>
<th>Quartz powder 1</th>
<th>Quartz powder 2</th>
<th>Sand 0.1/1 mm</th>
<th>Sand 1/2</th>
<th>Gravel 2/8 mm</th>
<th>Gravel 8/16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density [10, 15]</td>
<td>[kg/dm³]</td>
<td>2.64</td>
<td>2.65</td>
<td>2.65</td>
<td>2.61</td>
<td>2.51</td>
<td>2.54</td>
</tr>
<tr>
<td>Water absorption [15]</td>
<td>[m.-%]</td>
<td>-</td>
<td>-</td>
<td>0.2</td>
<td>0.3</td>
<td>1.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Blaine value [11]</td>
<td>[cm²/g]</td>
<td>18.000¹</td>
<td>1448</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Granulometry and packing density</td>
<td>[-]</td>
<td>0.549</td>
<td>0.623</td>
<td>0.796</td>
<td>0.584</td>
<td>0.602</td>
<td>0.604</td>
</tr>
<tr>
<td>Max. component packing density</td>
<td>[-]</td>
<td>7</td>
<td>16</td>
<td>8</td>
<td>3</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>Number of subclasses n</td>
<td>[-]</td>
<td>12.2</td>
<td>12.2</td>
<td>4.1</td>
<td>4.1</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td>Compression index k_i</td>
<td>[-]</td>
<td>0.436</td>
<td>0.552</td>
<td>0.703</td>
<td>0.831</td>
<td>0.685</td>
<td>0.721</td>
</tr>
<tr>
<td>Packing density (sub-) classes Φ_k,i</td>
<td>[-]</td>
<td>2.34·10⁻²</td>
<td>1.02·10⁻²</td>
<td>1.06·10⁻³</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹Data supplied by producer

Figure 1: Particle size distribution curves of the cements and inert granular constituent materials used

To ensure adequate workability characteristics of the concrete, a PCE-based superplasticizer in accordance with [16] was used. The content of active substances within the liquid product amounted to 35 m.-%. The remaining water content was accounted for in the calculation of the water-cement ratio (w/c) during the mixture development. The superplasticizer was added in a relation of 1.2 m.-% of the fines content (meaning the cement and the fine quartz powder content), as is recommended for this product in [4].
3 CONCRETE MIX DEVELOPMENT

Following the basic principle in concrete technology that performance is directly linked to the w/c-ratio, a reduction in the cement content, while maintaining a constant w/c-ratio, inevitably results in a reduction of the water content of the mixture. The key challenge in the development of concretes with low cement content and therefore minimum environmental impact lies in maintaining a sufficient workability of the fresh mix at very low water contents. As in the fresh state water is needed to fill the inter-granular voids in the mix consisting of aggregates and cement particles and to lubricate the deformation of this granular system, a reduction of the water content at constant packing density would inevitably result in a loss of workability and in the formation of voids. Against this background, methods for optimizing the packing density of the granular mix constituents form a key step in the mix development process for concrete with low cement content.

In a first step, a parameter study was carried out in order to identify the influence of the composition on the packing density of the resulting mixes consisting of the raw materials detailed in Sec. 2. As a boundary condition for the performed packing calculations, the cement volume in relation to the volume of all dry constituents was set to 10, 6 and 4 vol.-%, respectively. Only one type of cement was used at a time. The volume content of all other granular constituents (i.e. inert particles) was varied to provide maximum packing density for the mix of cement and inert particles. The particle size distribution of all particles was optimized using a combination of the grain size models by Funk and Dinger [17] and the CIPM-Model proposed by Fennis [4]. For the Funk and Dinger approach, the influence of various exponents $n$ on the grain size distribution as calculated by the CIPM-Model was determined. As can be seen from Fig. 3, grain size distributions according to [17] with exponents ranging between $n = 0.37$ and 0.40 yield mixes with maximum packing density, independently of the cement content.

![Figure 2: Packing density calculated according to the CIPM-Model [4] of granular mixtures as a function of the PSD parameter $n$ [-] according to the PSD curves proposed in [17] (left) and as a function of the cement content in the dry mix (right)](image-url)
Based on a total of 1500 mix designs calculated with the CIPM, 15 mixes with maximum packing density were chosen and tested for their fresh and hardened concrete properties in the lab. The composition of selected mixes is detailed in Table 3 and is characterized by cement contents between 4 to 10 vol.-% of the dry mix, corresponding to approximately 110 kg/m³ to 265 kg/m³ in the fresh concrete.

<table>
<thead>
<tr>
<th>Raw material / characteristic value</th>
<th>Dimension</th>
<th>Concrete mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement content in dry mix [vol.-%]</td>
<td>10.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Grain size distribution [fit parameter n acc. to [17]]</td>
<td>0.37</td>
<td>0.37</td>
</tr>
<tr>
<td>Cement type [-]</td>
<td>CEM I</td>
<td>CEM I</td>
</tr>
<tr>
<td>Mixture composition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement content</td>
<td>264</td>
<td>165</td>
</tr>
<tr>
<td>Quartz powder 1</td>
<td>44</td>
<td>46</td>
</tr>
<tr>
<td>Quartz powder 2</td>
<td>22</td>
<td>70</td>
</tr>
<tr>
<td>Sand 0.1/1 (mm)</td>
<td>429</td>
<td>482</td>
</tr>
<tr>
<td>Sand 1/2 (mm) [kg/m³]</td>
<td>357</td>
<td>372</td>
</tr>
<tr>
<td>River gravel 2/8 (mm)</td>
<td>535</td>
<td>557</td>
</tr>
<tr>
<td>River gravel 8/16 (mm)</td>
<td>491</td>
<td>511</td>
</tr>
<tr>
<td>Water</td>
<td>158</td>
<td>124</td>
</tr>
<tr>
<td>Superplasticizer (PCE based)</td>
<td>3.96</td>
<td>3.37</td>
</tr>
<tr>
<td>w/c-ratio [-]</td>
<td>0.54</td>
<td>0.64</td>
</tr>
</tbody>
</table>

| Mixture properties | |
| Compressive strength f_cm,7d [18] [MPa] | 52.8 | 41.9 |
| Degree of compactability c [19] [-] | n. d. | 1.19 |
| Global warming potential (GWP)¹ [kg CO₂ / m³] | 140 | 94.3 |

¹ Calculated on the basis of representative values for global warming potential of constituent materials, see [6]

As can be seen in Fig. 3, the fresh concrete properties (i.e. the degree of compactability c) essentially are a function of the w/c-ratio applied in the mix design. By increasing the w/c-ratio, the degree of compactability c (determined according to [19]) decreases, yielding mixes with an improved workability. Despite their nearly identical grain size distribution, mixes with reduced cement content generally require a higher w/c-ratio to ensure sufficient workability. This tendency seems especially pronounced for mixes with cement contents below 6 vol.-% of the dry mix.
Figure 3: Fresh concrete compactability $c$ tested according to [19] in relation to the w/c-ratio (left) and to the packing density ratio $\phi/\phi_{G,\text{comp}}$ (right) of mixtures containing Portland cement CEM I 52.5 R (CEM I; Blaine value 5500 cm²/g) and micro-cement (μCEM; Portland cement; Blaine value 6900 cm²/g) with a cement concentration of 4, 6 and 10 % by volume in the dry mix, respectively, and fit parameters $n$ of the particle size distribution curve between 0.34 and 0.37; compactability index classes according to [20].

Further, the workability of the mixes is closely interlinked with the packing density ratio, i.e. the packing density of the mix $\phi$ related to the maximum possible packing density $\phi_{G,\text{comp}}$ of the particles (see Fig. 3, right). With a lower packing density ratio $\phi/\phi_{G,\text{comp}}$ the lubricant film between the particles increases, leading to a significant increase in workability (i.e. a reduction in compactability $c$). Hereby the actual packing density $\phi$ of the mix can be calculated from the volumes of water $V_w$ and particles $V_p$ used in the mix according to Eq. (1). The maximum possible packing density $\phi_{G,\text{comp}}$ must be calculated using the CIPM [4].

$$\phi = \frac{V_p}{(V_p + V_w)}$$

With regard to the performance of the investigated concretes in the hardened state, Fig. 3 (left) shows that all investigated concretes show an equal or significantly higher compressive strength than ordinary Portland cement concretes at equal w/c-ratio, especially at very early ages (line referring to standard concrete 7 day strength as a function of cement type and w/c-ratio; compare [21]). A pronounced increase in compressive strength was detected for concretes where ordinary Portland cement was replaced by a so-called micro-cement, i.e. a Portland cement with a significantly higher Blaine value of approximately 6900 cm²/g compared to 5500 cm²/g for the CEM I 52.5 R used in the tests (compare Table 1). Based on these findings, it can be concluded that a cement content of 4 vol.-% in the dry mix obviously generates a sufficient amount of cement gel to coat all inert raw materials and thus generates enough bond-strength between the aggregates and the cement paste to provide compressive strengths of up to 65 MPa at the age of 7 days.
Fig. 4: Compressive strength $f_{cm,cube}$ of green concrete mixtures at an age of 7 days as a function of w/c-ratio and Walz-Curves for standard concrete [21]; conversion to 150 mm cube strength and 7 day strength according to fib Model Code 2010 (left); Binder intensity related to compressive strength $f_{cm,cube}$ of green concrete mixtures at the age of 7 days with varying cement content and particle size distribution (fit parameter n) compared to literature values (28-days strength) by Fennis [4], Proske et al. [3] and Daminelli et al. [22] (right).

Fig. 4 (right) shows the binder intensity $b_i$, i.e. the amount of cement needed in 1 m$^3$ of concrete to generate 1 MPa of strength for the developed cement-reduced concretes, in comparison to standard concretes as reported on by Daminelli & al. [22] and other green concretes proposed by Proske et al. [3] and Fennis [4]. The reduction in cement content herein leads to a pronounced reduction in binder intensity, hinting toward a strongly increased sustainability potential especially of mixes with no or low durability requirements.

Fig. 4 (right) however also shows that none of the available mix design approaches for green concretes – including the one presented in this paper – succeed to de-couple the binder intensity from the strength requirements. With reducing strength requirement a somewhat pronounced increase in binder intensity is observed for all concretes. This interdependency can be explained by the fact that for low strength requirements, the cement content required according to the w/c-ratio approaches zero. However, in order to coat all inert particles with a sufficient amount of cement gel, a minimum cement content is necessary, leading to the observed increase.

Based on the results above, the following mix development scheme is proposed for the mix design of cement reduced concretes (see Fig. 5). The scheme proposed here builds up on the principles by Fennis [4].
In the first step, the desired concrete properties and the cement content have to be defined. Furthermore, the properties of the raw materials have to be experimentally determined. Based on the desired cement content and the strength requirements, the w/c-ratio can be assessed using Fig. 3 and the minimum packing density $\phi_{\text{req}}$, required to cope with this reduced water content, can be calculated using Eq. (1). In the next step, the grain size distribution of all granular constituents is optimized using a combination of the grain size models by Funk and Dinger [17] and the CIPM-Model proposed by Fennis [4], so that the packing density of the mix in the compacted state $\phi_{\text{G,comp}}$ exceeds the required packing density $\phi_{\text{req}}$. Based on the achieved packing density, the fresh concrete properties of the mix can be evaluated using Fig. 3 and the compressive strength can be predicted using Fig. 4. A detailed description of this algorithm can be found in Fennis [4] and Haist et al. [5].

4 CONCLUSIONS

In the presented work, the methodology for developing concrete mixtures with very low binder content was presented and the influence of the binder content both on the fresh and the hardened concrete properties was studied. Due to the limited availability and their questionable environmental impact data, no secondary cementitious binders were used in this research. Instead, it was investigated how much pure Portland cement is necessary to create a...
concrete with a solid matrix, without significant amounts of air-voids and pores and providing reasonable strength. As can be seen from the results, in order to ensure a constant workability at decreasing cement content, the w/c-ratio must be increased. Concretes prepared with water contents as low as required by the inter-particle void volume at maximum packing density show workability characteristics comparable to earth-moist mixtures, their workability improving significantly with decreasing solids content.

Regarding the strength of the investigated concretes, it can be concluded that also for very low cement contents the compressive strength primarily is a function of the w/c-ratio. The cement content itself does not seem to influence the compressive strength in a significant manner. Furthermore, replacing standard Portland cement by cements with significantly higher Blaine values (so-called micro-cements), the packing density of the mixes can be improved and the strength can be significantly increased at early ages.

Summarizing the results, cement-reduced concretes provide high levels of performance, but still show deficits regarding the workability of the mixes in the fresh state at very low cement contents. The durability of similar mixes has also been studied, the results of these investigations will be presented in another contribution to the given proceedings [6].

5 ACKNOWLEDGEMENTS
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Requirements and guidelines. Beuth Publishers, Berlin


APPLICABILITY OF BIOMASS PLANT WASTE TO THE DESIGN OF NEW CEMENT-BASED MATERIALS

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Keywords: biomass plant waste, reuse

Abstract

Today’s worldwide energy problem and the availability of vast amounts of biomass in certain countries have translated into an increase in the number of biomass plants designed to industrially transform agri-forest material into electric power, thereby reducing greenhouse gas emissions. This research analysed the viability of reusing the waste generated in biomass plants as raw material in the cement industry. In a preliminary stage, the waste was characterised chemically and mineralogically to determine its possible applicability to the design of new cement matrices. The physical and mechanical properties of the new cements were subsequently analysed and found to comply with the existing legislation.
1 INTRODUCTION

In today’s context of economic and social concern around environmental issues, energy efficiency and savings and heightened use of renewable sources are among the primary measures for reducing greenhouse gas emissions and complying with the provisions of the United Nations Framework Convention on Climate Change [1].

One of the sources included in Spain’s Renewable Energy Plan 2011-2020 is biomass, defined as ‘the biodegradable fraction of products, waste and residues from biological origin from agriculture (including vegetal and animal substances), forestry and related industries including fisheries and aquaculture, as well as the biodegradable fraction of industrial and municipal waste’ [1]. The use of biomass as a renewable source of electric or thermal power has intensified substantially in recent decades [2], accounting for over 4 % of the total energy consumed in the European Union. This is primarily because: i) it is an economically expedient source, generating corporate profits, creating jobs, developing the countryside and improving environmental protection, both via the capture of CO₂ and the clearing of forest underbrush; and ii) it is the most stable and the sole manageable renewable, for it is not dependent upon variable factors such as sunlight, wind or the availability of a given type of agricultural waste [3].

Along with electricity and gas, biomass-fuelled power plants produce unburnt, which at present is removed directly to landfills, reused as fertiliser or landfill as uncontrolled debris. Unlike other industrial discards, this waste is characterised by disparate chemistry and mineralogy, a problem that hobbles its reuse. Some types of unburnt must be immobilised, for they contain polluting elements with an adverse effect on soil and underground water quality [4].

In recent years, many researchers have studied the possibility of using unburnt in construction materials such as clay-based block [5, 6], concrete [2, 7], mortar [8, 9] and cement [10, 11], further to European Directive 2008/98/EC on waste, in turn based on the ‘3R” principle: reuse, recycle and reduce (the volume of waste) [12].

This study addresses the feasibility of reusing the electric power plant unburnt to design eco-efficient cements. A discussion of the determination of the chemical and mineralogical composition and pozzolanicity of the waste is followed by a description of the physical and mechanical characterisation of the new cements, conducted to ascertain their conformity to the existing legislation.

2 EXPERIMENTAL

2.1 MATERIALS

The materials used in this study included waste from a biomass-fuelled electric power plant (labelled BIO) and European standard EN 197-1 class CEM I 42.5 R cement [13].

The BIO waste shipped to the laboratory was dried to a constant weight in an oven at 100 °C and subsequently ground to the desired particle size. A new eco-efficient cement was prepared by blending the CEM I 42.5 R with 10 wt% of the waste conditioned as described. The two dry materials were blended in a shaker-mixer and stored in an air-tight container to protect it from carbonation and ambient humidity.
2.2 Methodology

A Bruker S8 TIGER XRF spectrometer was used to determine the chemical composition of pressed powder samples of the biomass plant waste and cement CEM I 42.5 R. The readings were processed with SPECTRA plus QUANT EXPRESS software.

X-ray diffraction (XRD) mineralogical studies were conducted on the materials with a Bruker D8 Advance diffractometer fitted with 1.54 Å CuKα radiation and a Lynxeye detector with a 3 mm antiscatter slit and a (0.5 %) Ni K-beta filter (i.e., Kα2 was not eliminated). Readings were taken at 20 diffraction angles ranging from 5º to 60º.

Particle morphology and size were characterised on a Bruker SEM/EDX microscope. The samples, secured to a metallic holder with a two-sided graphic adhesive, were carbon-coated to ensure conductivity and accurate signal detection.

The pozzolanic activity of the waste was found chemically using an accelerated method consisting in soaking 1 g of the material in 75 mL of a saturated lime solution (17.68 mM/L) at 40 ºC for 28 days [14]. At the specified age, the solution was vacuum-filtered on a Büchner funnel through a double layer of filter paper. The Ca ion concentration (expressed as calcium oxide or fixed lime) in the filtrate was quantified as per the (slightly modified) procedure recommended in European standard EN 196-5 [15]. The amount of lime fixed was computed as the difference in the CaO concentration between the initial saturated lime solution (17.68 mM/L) and the problem solution in contact with the waste.

The total porosity and mean pore diameter were found as prescribed in ASTM standard D 4404 [16], using a Micromeritics Autopore IV 9500 mercury porosimeter designed to operate at pressures of up to 33 000 psi (227.5 MPa) and measure pore diameters of 0.006–175 μm.

Prismatic specimens measuring 40x40x160 mm were prepared with cement containing 0% and 10 wt% of biomass waste using a water/cement ratio of 1:2 and a cement/aggregate ratio of 3:1. The drying shrinkage and normal consistency of these cement pastes were determined as laid down in European standard EN 196-3[17]. Compressive and bending (or flexural) strength of the 2- and 28-day mortars were also found, following the method described in European standard EN 196-1 [18].

3 RESULTS AND DISCUSSION

Further to the XRF findings, the chemical composition of the cement in (weight) per cent of oxides was as follows: 19.23 % SiO₂, 63.78 % CaO, 4.41 % Al₂O₃, 3.32 % SO₃, 2.60 % Fe₂O₃, 2.00 % MgO, 0.79 % Na₂O+K₂O and 0.53 % other oxides. Loss on ignition at 1000 ºC was 3.06 %.

The irregularly shaped particles comprising the waste from the biomass-fuelled power plant were densely agglutinated and measured under 100 μm (see the SEM micrograph in Figure 1). BIO chemical composition, according to the XRF analysis, was characterised by high proportions of SiO₂ (63.26 wt%), alkaline oxides (17.90 wt% Na₂O+K₂O) and calcium oxide (9.29 wt%) and smaller percentages of other oxides (1.64 wt% Al₂O₃, 2.80 wt% MgO, 1.67 wt% P₂O₅, 0.83 wt% Fe₂O₃ and 0.56 wt% SO₃). Loss on ignition at 1000 ºC was 1.93 %.
The XRD patterns showing the mineralogical composition of the cement and the BIO waste are reproduced in Figure 2. The diffractogram for the cement contained reflections typical of alite, belite, tricalcium aluminate (C₃A), ferrite (C₄AF) and gypsum, along with less intense lines for calcite. The wide amorphous hump on the pattern for the BIO waste straddling 2θ = 19°-36° with a reflection at 21.9°, in turn, may have been generated by a crystalline SiO₂ such as cristobalite.

The pozzolanicity of the biomass power plant waste (BIO) at 28 days is compared in Figure 3 to the activity of other pozzolanic materials, including standardised additions such as silica fume and fly ash and non-standardised waste: clay-based industry waste (ceramic tiles and sanitary ware) [14, 19, 20], calcined paper sludge [21] and agri-forestry waste such as elephant grass ash [10]. As the figure shows, the new material fixed lime at rates that were comparable to silica fume and sanitary ware waste, higher than ceramic tile waste and paper sludge, and much higher than fly ash, which exhibited a value approximately 4.3 times lower than the biomass waste (BIO). Elephant grass ash, in turn, fixed around 3-5 % more lime than the BIO waste.
Figure 3: Comparison of pozzolanicity in 28-day BIO samples to activity in other pozzolans

Respect to the physical properties, with a 34 mm slump and 1 mm drying shrinkage, the new cement pastes proved to be European standard EN 197-1-compliant [13].

Figure 4 shows that adding 10 wt% BIO waste to the cement induced slight declines in the compressive and bending strength (4.2 and 7.5 %, respectively) of the new mortars compared to the CEM I 42.5 R reference mortar. Nonetheless, their 2 and 28 day compressive strength was higher than the 20 and 42.5 MPa laid down in European standard EN 197-1 [13].

Total porosity and mean pore diameter for the mortars at 28 days are listed in Table 1. Note that the blended material had 11 % lower total porosity and a 9 % smaller mean pore size than the reference mortar (CEM I 42.5 R). Such pore system refinement, attributable to
both the pozzolanic activity and filler effect induced by the waste, would enhance mortar durability.

Table 1: Total porosity and mean pore size in 28-day mortars

<table>
<thead>
<tr>
<th>Property</th>
<th>CEM I 42.5 R</th>
<th>CEM I 42.5 R + 10% BIO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total porosity (vol.%)</td>
<td>14.09</td>
<td>12.52</td>
</tr>
<tr>
<td>Mean pore diameter (μm)</td>
<td>0.0861</td>
<td>0.0779</td>
</tr>
</tbody>
</table>

The addition of the biomass waste prompted no changes in the phases forming during mortar hydration. The XRD patterns (Figure 5) exhibited crystalline portlandite as well as lines for ettringite or calcium monosulfoaluminate, heavily masked by the high intensity quartz reflections. Given its scant crystallinity, no C-S-H gel was identified on these diffractograms.

Figure 5: XRD patterns for mortars at 28-days: CEM I 42.5 R and blended CEM I 42.5 R + 10 wt % BIO

4 CONCLUSIONS

The conclusions that can be drawn from the present study are set out below.

- The 28 days BIO waste exhibited high pozzolanicity that compares well with the activity reported for standardised pozzolans such as silica fume and fly ash.

- The new eco-efficient cements exhibited mechanical properties similar to the compressive and bending strength in cement I 42.5 R and proved to be European standard EN 197-1-compliant.

- Inclusion of the biomass power plant waste induced a decline in total porosity, with concomitant pore system refinement.
Adding this waste to the cement prompted no changes in the hydration products.

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REFERENCES

BIOMASS AND COAL FLY ASH AS CEMENT REPLACEMENT ON MORTAR PROPERTIES

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Key words: Biomass fly ash, fly ash, mortar, cement replacement

Abstract. Nowadays, construction sector tries to implement several options to minimize issues related to concrete sustainability. The main goal of this work was to study the effect of biomass fly ash, blended with coal fly ashes or alone, as cement replacement in mortars properties. Three replacement percentages were tested (20, 40 and 60 %wt). Fresh and hardened properties (mechanical strength along time of curing) were evaluated. Mortars with biomass fly ashes have shown lower workability than the reference one. Mortars with 20% of cement substitution presents better results for all curing times. However, in the three percentages of substitution, and for all curing periods, the best results were found for mortars composed with biomass fly ashes. This work showed that is possible to use biomass fly ashes as partial cement substituent with good results in terms of durability and quality of concrete. Its utilization at an industrial level of concrete production can decrease the energy and raw materials consumption related to cement production and allows a more sustainable option on the ash management.

1 INTRODUCTION

Concrete is the most used material on building mainly by the fact of present good mechanical and durability properties [1]. However, concrete has a significant impact on environment, since high content of natural resources and energy flows are necessary to its production. Furthermore, it is known that to produce one of the most important materials used on concrete (Portland cement) a significant emission of CO₂ is released to atmosphere, and this is related with the greenhouse gases effect and with the global warming of planet [2].

Nowadays, the environmental questions are very important in all economical sectors. The
construction sector deals with the issue of find several options that can be implemented to minimize problems related to concrete sustainability [3]. One of them, is the use of supplementary cementitious materials (usually called as pozzolan), which offer a potential reduction in global CO2 emissions [4], reduce the cost of concrete production, may enhance the workability of fresh concrete and in some cases improve the durability of concrete [1].

Coal fly ash (FA) is the most artificial pozzolan used in concrete production [1]. FA has several advantages when compared with cement, such as lower hydration heat, is a by-product of coal combustion and is cheaper than cement [2].

Countries like Portugal, which are interested in increase the energy production using renewable sources, like biomass, are now interested and deal with different issues. The use of biomass to produce energy by combustion increased in the last years, on the other way, several economical sectors use biomass as raw material to produce heat and power [4]. The increase on its use led to a significant issue related with the increase of biomass fly ash (BFA) that needs to be solved. Fly ash from biomass combustion is classified as solid waste and usually is managed by disposal in landfill. However, disposal in landfill has economic, environmental and sustainable issues [4]. In other countries, in some cases they are recycled on agricultural fields or forest, but in most cases without any form of control [5]. BFA can be used as a pozzolanic material, such as fine aggregate or as binder in cement-based materials and some studies showed good results when BFA was incorporated in concrete [6]. On the contrary to coal fly ash, biomass fly ash still need more significant researches about its usage and its commercial utilization is not yet widely reported [7].

Thereby, the main goal of this work was the production of mortars using BFA and blends of biomass and coal fly ash as substitute of cement and studied its effects on mortars properties. On fresh state workability was evaluated and on hardened state its mechanical and durability properties were determined.

2 MATERIALS AND METHODS

2.1 Coal and Biomass Fly ash

BFA was sampled in a Portuguese pulp and paper industry, which used forest residues, such as bark from eucalyptus and pine, as fuel to produce heat and power. FA was sampled from a Portuguese thermoelectric power plant. Both ashes were characterized in terms of particle size distribution, loss on ignition, chemical composition and thermal analysis.

The particle size distribution of ashes was determined, in a liquid environment, by laser diffraction using CILAS 920 equipment. The ashes were grinded in order to obtain samples with particle size lower than 90 µm for its characterization in terms of loss on ignition (LOI), chemical and thermogravimetric analysis. For the LOI and chemical composition the milled samples were previous dried at 101±1°C. LOI was determinate in an approximately 1.5 g of dried sample heated at 1100°C during 3 hours in a Carbolite furnace. The chemical composition was determined by X-Ray Fluorescence using a Panalytical Axios spectrometer. The thermal analysis was done using a STA, Netzsch 402 EP with a heating rate of 10°C/min.

2.2 Mortar Formulations

A group of sixteen formulations of mortar (Table 1) was set with the utilization of BFA as
substituent of cement and the utilization of the same ash but blended with FA. In those formulations a CEM I 42.5R (Outão, Secil) cement was used and a commercial river rolled sand 0/4 mm was used as aggregate. Three levels of cement substitution were studied: 20, 40 and 60 %wt. All mortar mixtures were made with 1 wt. part of binder (considered as the sum of cement plus fly ash): 2.5 wt. parts of aggregate and water-binder ratio (W/B) of 0.5. For each mixture three mortars were performed.

<table>
<thead>
<tr>
<th>Mortar Codes</th>
<th>% of cement substitution</th>
<th>% of FA - BFA</th>
</tr>
</thead>
<tbody>
<tr>
<td>FA0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>FA20</td>
<td>20</td>
<td>20-0</td>
</tr>
<tr>
<td>FA16-BFA4</td>
<td>20</td>
<td>16-4</td>
</tr>
<tr>
<td>FA12-BFA8</td>
<td>20</td>
<td>12-8</td>
</tr>
<tr>
<td>FA8-BFA12</td>
<td>20</td>
<td>8-12</td>
</tr>
<tr>
<td>BFA20</td>
<td>20</td>
<td>0-20</td>
</tr>
<tr>
<td>FA40</td>
<td>40</td>
<td>40-0</td>
</tr>
<tr>
<td>FA32-BFA8</td>
<td>40</td>
<td>32-8</td>
</tr>
<tr>
<td>FA24-BFA16</td>
<td>40</td>
<td>24-16</td>
</tr>
<tr>
<td>FA16-BFA24</td>
<td>40</td>
<td>16-24</td>
</tr>
<tr>
<td>BFA40</td>
<td>40</td>
<td>0-40</td>
</tr>
<tr>
<td>FA60</td>
<td>60</td>
<td>60-0</td>
</tr>
<tr>
<td>FA48-BFA12</td>
<td>60</td>
<td>48-12</td>
</tr>
<tr>
<td>FA36-BFA24</td>
<td>60</td>
<td>36-24</td>
</tr>
<tr>
<td>FA24-BFA36</td>
<td>60</td>
<td>24-36</td>
</tr>
<tr>
<td>BFA60</td>
<td>60</td>
<td>0-60</td>
</tr>
</tbody>
</table>

2.3 Fresh and hardened properties of mortars

The workability was expressed as reach spread diameter in mm and determined after 15 strokes of the flow table according to EN 1015-3:1998 [8]. The mortar specimens for the mechanical resistances and water absorption tests were prepared in a standard mixer, were placed in 40x40x160 mm³ moulds and compacted two times. The moulds were covered with plastic film to avoid the loss of water and stored in a humidity chamber (with approximately 87 % of relative humidity and 21 ºC of temperature) during 24h. After that time, the mortars were demoulded and again stored in the humidity chamber, according to [9]. The flexural strength was determined in a LLOYDS Instruments universal testing machine (with a maximum capacity of 50 kN), the compressive strength was measured in an Ele Auto Test press (with a capacity between 5 to 110 kN). The mechanical strength tests were done according to EN 1015-11:1999 [9], on three samples for each formulation, for mortars with 2, 7, 28, 90 and 180 days of curing. After 28 days of curing, mortars for water absorption analysis were tested for flexural resistance and then one part of each mortar were used to capillarity test and the other was used for the immersion test. All the mortars parts were dried at 60±5ºC until mass reached constant before water absorption tests, which were
done according to [10–13]. The coefficient of water absorption by capillarity is defined by the representation of a straight line linking the points of the measures carried out between 10 and 90 minutes and calculated based in Eq. 1 [12].

\[ C = 0.1(M1 - M0) \]  

(1)

Where,
C is the coefficient of water absorption kg/(m².min⁰.⁵)
M1 is the mass (g) of the specimen after soaking for 90 min;
M0 is the mass (g) of the specimen after soaking for 10 min.

The immersion water absorption is determined as present in Eq. 2 [13]:

\[ A_i = \frac{M_1 - M_2}{M - M_2} \times 100 \]  

(2)

Where,
A_i is the water absorption by immersion;
M1 is the mortar mass on air saturated (g);
M2 is the hydrostatic mass of the saturated mortar (g);
M3 is the dry mass of mortar (g).

The shrinkage test was done to the mortar formulations that presented the best results, in terms of mechanical resistances after 28 days of curing and for each level of cement replacement. To measure the shrinkage and weight loss three prisms with 25x25x250 mm³ for each mixture were moulded. The moulds to this experiment have a system that allows the application of stainless steel bolts in the specimen tips. The shrinkage was determined on specimens after demoulded. After that time the mortars were putted in a climatic chamber with a temperature and relative humidity control (temperature 20ºC and moisture content of 60%), where the shrinkage measuring equipment was also stored.

3 RESULTS AND DISCUSSION

3.1 Characterization of material used on mortar formulation

The particle size of the fly ashes is presented in Figure 1. Since the ashes were used as cement substituent, the particle size distribution of cement (CEMI 42.5) was also analyzed.

![Figure 1: Cumulative particle size distribution.](image-url)
The particle size distribution of FA is similar than for the cement with an average diameter of about 9 µm. However BFA presented coarser particles and its size distribution is significantly different than coal fly ash and cement. The average diameter of BFA is about 47 µm.

The obtained LOI and the chemical composition of the materials used in mortars are showed in Figure 2. As it can be seen, BFA showed the highest value for LOI (>5% dry bs), followed by FA, cement and sand. As expected, Si was the major chemical element present in sand and calcium in cement. Si was also the major chemical element (>25 %, dry bs) present in the FA, followed by Al, Fe, Ca and K. In contrast, Ca was the major chemical present in BFA (>19 % dry bs), followed by Si, Al, K, Fe, Mg. An interesting observation was the fact that only the BFA showed in its composition Cl and S and this is also due to the inorganic content of biomass [15], but also is due to the fact of these ashes have the capacity of allows the adsorption of those chemical elements, in the form HCl and SO₂, from the gases during biomass combustion [16].

In terms of minor chemical elements, the two fly ashes were enriched when compared with cement and sand. Among the minor chemical elements Ba presented the high value followed by Zn, Sr, Zr, Rb and Cu for the BFA sample. In the case of FA, Sr was the minor chemical element with higher concentration followed by Ba, Br, Zr, and Cr.

The TG and DTA signals registered for cement, FA and BFA are shown in Error! Reference source not found.. It was observed a slight decreased around of 100°C for both ashes, this loss of weight is related with the release of water adsorbed in the ash. In the BFA, a slight increase on the weight near 430°C was observed and this is related with the change of the iron phase. With increasing temperature, a significantly decrease in weight of the two fly ashes is observed in the range of 600 to 835°C, associated with an endothermic process. This weight loss must be related to the thermal decomposition of carbonates like CaCO₃. The total weight loss was approximately 2-3 % for the FA and 6-7 % for BFA and these results are similar to the value observed for LOI analysis (Figure 2).
3.2 Mortar fresh properties

The flow spread obtained results are presented in Figure 4. The values obtained for the studied formulations showed that mortars with fly ashes had low spread value than the reference mortar (FA0). However, the differences are, in general not relevant except for mortars made without FA and BFA replacement percentages higher than 40%. The incorporation of only FA did not present a significant influence in the flow spread when compared with cement. On the other side, it was observed a decrease on the flow spread value with the increased of cement substitution by BFA (alone or blended with FA) when compared with the reference mortar. The lower value was observed for the mortar with 60% of cement substitution by BFA. These results are explained by the physical characteristics, mainly the irregular shape settles on the high specific surface area. Furthermore, the presence of significant organic matter content could support the adsorption of the water molecules [6], leading to the flow spread value decrease.

3.3 Mortar hardened properties

The flexural strength values of mortars containing BFA or/and FA and of the reference mortar are shown in Figure 5.
Figure 4: Values of flow spread of mortars prepared with different fly ashes incorporations

Figure 5: Evolution of flexural strength values of mortars up to 180 days of curing
As it can be seen the values of the flexural strength for all mortars made with BFA and/or FA at 2 curing days were lower than for the reference mortar and a significant decrease was observed for all mortars with 60% of cement substitution. In mortars with 20% of cement replacement and with 2 curing days, an increase was observed with the BFA increment, being the highest value observed for mortars with 20% of cement replaced by BFA. For the other curing periods, the flexural strength values of the mortars with 20% of cement substitution (FA20 to BFA20) were slightly lower than for the reference mortar, with the exception of BFA20 at 90 days that presented a higher value than the reference, being the better values observed on BFA20 (20% of BFA). For the other cement replacement percentages, the flexural strength values were lower than the reference, being the better values once again observed for mortars with cement replaced by BFA.

The values for compressive strength of mortars are shown in Figure 6. As it can be seen, the compressive strength values of all mortars with BFA or FA and for the different curing periods showed lower values than the reference mortar, with the exception of FA16-BFA4 for 180 curing days.

![Figure 6: Evolution of compressive strength values of mortars up to 180 days of curing](image)

The values increased with the curing time and this is related with the fact of pozzolanic materials show low early strength and a higher development on strength with age [17]. Mortars with 20% of cement substitution presented the better results when compared with the reference mortar for all curing time. It was observed, that an increase on the biomass fly ash content allows an increase on compressive strength.

For the other percentages of substitution, no significant differences were observed between mortars with FA, blended and BFA only. However, in the three percentages of substitution, and for all curing periods, the best results were found for mortars composed by BFA, followed by the mortars with more content of BFA on the blends. The lower values of
compressive strength for mixtures with coal and/or biomass fly ash compared to the ones made without cement replacement can be due to its particles, which are coarser than coal fly ash and may leads to an increase on the porosity of mortars [18].

3.1.1 Water absorption

The influence of the presence of fly ashes on the water absorption by capillarity of mortars (after 28 days curing) is shown in Figure 7. Mortar with 20% of cement replacement showed lower values of water absorption for mortars with high BFA content (FA8-BFA12 and BFA20) when compared with the reference concrete (FA0).

Concrete with 40% of cement replacement by FA (FA40) exhibited higher values than the reference. However 40% of BFA led to similar capillary water absorption than for plain cement mortar (BFA40). Formulations with 60% of cement presented high values for water absorption by capillarity when compared with the reference ones, with an exception of BFA60 formulation.

Figure 8 demonstrates the values of open porosity obtained from the water absorption by immersion test for all mortar formulations. As it can be seen, all mortars exhibited higher values for water absorption when compared with the reference mortar. The presence of BFA seems to have a slight positive effect when compared to the values obtained for FA mixes.

![Figure 7: Water absorption capillarity coefficient for the different formulations](image-url)
3.1.2 Shrinkage

The shrinkage behavior of mortars along the time and with the different content of the selected fly ashes is presented in Figure 9. It was observed a higher rate of shrinkage during the first 14 days. After that period, the shrinkage values were still growing but with a lower rate. The incorporation of fly ashes reduced the shrinkage of mortar at a later age and all values were lower than for the reference mortar. Similar results were verified in [19].

The weight loss was similar for the different mortars at early ages, however after approximately 5 days differences was observed. In this case, the mortar with higher content of fly ashes and with a higher percentage of cement substitution showed the highest weight loss values. These results showed that despite the formulations with high content of fly ashes present the higher weight loss values did not present the higher values for shrinkage. This could be related with the fact of shrinkage is affected by others parameters in addition to weight loss, like porosity, mortar only with cement present a finer pore structure and this could lead to the increase on the shrinkage values (Gesoğlu et al., 2004). Other explanation for this could be related with the different types of shrinkage, the decrease on cement content on mortars with fly ash could decrease the effect of autogenous shrinkage and for that the drying shrinkage is more significant.

**Figure 8:** Values for water absorption by immersion (expressed in %) for the different mortar formulations.
9 CONCLUSIONS

Mortars with BFA exhibited slightly lower workability than the reference. It was observed a decrease on the flow spread diameter with the increased of cement substitution by BFA. Mortars with 20% of cement substitution presented better results for all curing time, in terms of mechanical strength and water absorption. However, in the three percentages of substitution and for all curing periods the best results were found for mortars composed with BFA. The replacement of cement by FA and/or BFA benefited and mitigated the total shrinkage of the mortars.

In short, this work showed that it could be possible to use biomass fly ashes as partial cement replacement with good results in terms of durability and quality of concrete. Its utilization could be important to mitigate the issues related to high volume fly ash content and can be used for the production of concrete with characteristics similar than a high volume fly ash concrete but with a better environmental performance.

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REFERENCES


DOSAGE OF ECONOMIC SELF-COMPACTING CONCRETE WITH LOW AND MEDIUM COMPRESSIVE STRENGTH

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Key words: Self-Compacting Concrete, Mix Design, Low/Medium Compressive Strength

Abstract. The use of self-compacting concrete (SCC) has been widely accepted worldwide for the production of prefabricated elements. However, the situation is very different in other cases within the construction industry such as in ready-mixed concrete and in buildings made on-site, where a reduced use of SCC is evidenced, which is mostly reserved to processes exhibiting particular complexities (e.g. filling of parts with either special geometries or heavily reinforced, inaccessible areas and/or areas with inability to vibration, repair of existing structures, underground tunnels, etc.). This is because mixtures usually exhibit high content of fines (450-600 kg/m³), which results in mixtures with higher contents of Portland cement and consequently high resistance values (40-70MPa), which are much higher than those strictly required in projects. Thus, the cost of the components that make up a SCC is comparatively high with respect to that of a conventional concrete of equal resistance. Study of SCC with mean values of compressive resistance lower than 40MPa and reduced cost is of great interest, since such resistance range is the most widely used in the production of conventional concrete. Moreover, there is few information on this subject in the scientific literature. SCC having these features could be very competitive with respect to traditional concrete, both technically and economically. Therefore, the present paper has as its main objective to present a rational method for mix design of SCC with low and medium compressive strength and low cost, based on a physical pathway while also bearing in mind environmental conditions. For this purpose all the different variables related to the available materials in a particular setting should be analyzed, the dosage itself (water/cement ratio, paste volumes) as well as the cost. These properties in fresh and hardened state as well as the SCC cost are studied. Statistical correlations are determined between the different variables and, dosage diagrams and equations are presented that enable the development of a simple dosage method. The results and costs obtained with SCC showing the benefits of using SCC from technical, economic and ecological perspectives.
1 INTRODUCTION

Self-compacting concrete (SCC) emerged in Japan in the late 80's; its advantages are many, among which are:

- extreme fluidity that leads to rapid placement and filling possibilities of formwork elements with complex shapes or small thicknesses or with congested reinforcements avoiding the presence of honeycombs and voids;
- higher efficiency of manpower and equipment, which represents reductions in costs and execution time;
- a significant contribution in the optimization of construction procedures.
- most industrialized placement in the construction site, as compaction is eliminated and, at the same time, can achieve a better quality allowing significant cost reductions by reducing construction time;
- provide greater compactness increasing the durability;
- allows a better finish which makes it very suitable for use in raw concrete;
- the permanence in time of the obtained slump, which is very important when certain placement requirements are imposed such as long periods of concrete transportation or extended concreting operations.

Worldwide, the use of self-compacting concrete (SCC) has gained wide acceptance in the precast industry; however, the situation is very different in the case of the ready mixed concrete industry. This can be seen on the statistics presented in different years by the European Ready Mixed Concrete Organization (ERMCO) in which the low use of SCC in European countries, Russia, the United States and Japan are evidence, which ranges from 1% to 2.5% [1,2]. The current use of SCC in buildings made “in situ” is mainly reduced to processes of some complexity: filling parts of special geometry or congested reinforcements, areas of difficult access for vibrators, repairing of existing structures, underground tunnels.

The reduced use of HAC in the construction industry usually responds to mixtures with high content of fines (between 450 and 600 kg/m$^3$) in order to obtain the required rheological properties to achieve self-compactability, which usually results in mixtures with high content of Portland cement, and consequently, high values of initial and final strength, much higher than those strictly required by the project. Thus, the costs of the components that constitute a SCC are higher than those of conventional concrete of equal strength.

Vilanova [3], analyzes 627 mix proportions of HAC, obtained from 138 references belonging to publications between the years 1997 and 2008 (90% of them being between 2003 and 2007). It is concluded that most HAC cited in scientific literature are those whose compressive strengths lie in the range between 42.6 to 67 MPa. This corresponds to the range usually employed for prefabricated concrete components, and some "in situ" construction programs associated to Civil Engineering, for example bridges.

Employing SCC of low / medium resistance aroused the interest of different authors who have studied SCC with low content of fines (350-380 kg/m$^3$) mainly actuating on the chemical process, in which to get mixtures with the suitable fluidity and cohesion, through the use of new generation admixtures as superplasticizers and viscosity modifying admixtures, without affecting the skeleton granular or its components as main variables [4-9]. The problem that arises from this way is the high cost that these admixtures have.
Rodríguez Viacava et al. [10] developed a conceptual approach, establishing some criteria on how to design a concrete of those characteristics following the path of physical process. Perform a dosage by physical process involves, on the one hand, the optimization of the granular skeleton to obtain workable mixtures (allowing lower dosage of superplasticizer) and better performance on durability. On the other hand, the fines content plays an important role since they are those who give cohesion and consistency to the mixture to avoid segregation and controlling exudation. In this sense, different authors obtained SCC of medium characteristic strength actuating on the fines content, using different types of additions in partial replacement of cement [11-14] but did not develop a method for dosing those SCC.

Moreover, using local waste in the production of structural concrete is very significant from an economic and environmental perspective. Rodriguez de Sensale et al [15] developed a dosage method that contemplates those aspects to obtain economic SCC with compressive strength over 40MPa with few concretes tests, but as resistance decreases the concretes tests to obtain the required results increases, so it is considered suitable to be used when compressive strength is higher than 35MPa. Hence, it is of great interest to develop a method for dosing SCC with characteristic strength values lower than 40 MPa, economical and obtained by physical process. There is few information on this subject in the scientific literature. Self-compacting concrete, of these features and with the incorporation of residual waste, could be quite competitive both from a technical and economical point of view.

Therefore, the main objective of the paper is to present the development of a method for dosing SCC with medium characteristic strength and low cost, based on the path of physical process and considering specific context conditions. Properties in fresh and hardened state and costs of the SCC obtained were studied. Statistical correlations were used to develop dosage diagrams that allowed the achievement of the main objective of the work.

2 EXPERIMENTAL INVESTIGATION

This section provides the basis for the development of a method for proportioning economical SCC to cover a range of compressive strength between 20-40MPa, satisfying fresh and mechanical properties. Concrete mixtures were tested and the results were statistically studied in order to generate dosage diagrams to simplify obtaining the materials amount required for 1m³ of SCC. Cost analysis has been done.

2.1 Materials used

Type I portland cement was used in this investigation. Rice husk ash (RHA) from a Power Plant and cement kiln dust (CKD) were used in this study for partial replacement of portland cement. Physical properties are given in Table 1. Four Aggregates, two natural fine aggregates and two coarse aggregates (named A, B, C and D) were used. The fineness modulus of fine aggregates A and B were 1.955 and 3.563, respectively; the maximum aggregate size of coarse aggregates C and D were 6.35 and 19 mm, respectively. The properties of the selected aggregates are given in Table 2. Coarse aggregate D has a shape coefficient of $\alpha = 0.23$, meeting the minimum established in the standard UNIT 1050-2005 but not being suitable for use in SCC. Because of that and in order to define the dosage
method, it was studied the use of a single coarse aggregate, C, and the mixture of two coarse aggregate.

Table 1: Physical properties of the cement and wastes used

<table>
<thead>
<tr>
<th>Property</th>
<th>Cement</th>
<th>RHA</th>
<th>CKD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>3.07</td>
<td>2.12</td>
<td>2.75</td>
</tr>
<tr>
<td>Blaine Fineness (cm²/g)</td>
<td>2490</td>
<td>8920</td>
<td>6863</td>
</tr>
<tr>
<td>Water for normal consistency (%)</td>
<td>28.9</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Initial setting time (min)</td>
<td>245</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Final setting time (min)</td>
<td>325</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mean particle size (μm)</td>
<td>-</td>
<td>12.72</td>
<td>6.297</td>
</tr>
<tr>
<td>Activity Index (%)</td>
<td>-</td>
<td>77.7</td>
<td>56.7</td>
</tr>
</tbody>
</table>

Table 2: Properties of aggregates

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Specific gravity (g/cm³)</th>
<th>Absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.66</td>
<td>0.6</td>
</tr>
<tr>
<td>B</td>
<td>2.66</td>
<td>0.6</td>
</tr>
<tr>
<td>C</td>
<td>2.59</td>
<td>0.8</td>
</tr>
<tr>
<td>D</td>
<td>2.59</td>
<td>0.8</td>
</tr>
</tbody>
</table>

For admixture selection, six superplasticizers available were studied; in order to determine the compatibility of admixtures with cement and optimize its use, Marsh cone test [16] and mini-slump test [17] were performed. Accordingly, was selected an admixture based on polycarboxylates with a solids content of 35% as had the lowest saturation point (1%), besides, is the most economical of the admixtures studied.

2.2 Optimization of concrete phases

Assuming the concrete as a two-phase material, the granular skeleton and paste are optimized previously to the mix design [15]:

2.2.1. Granular Skeleton Optimization: Consists in obtain an optimal fine aggregate / coarse aggregate ratio (AF / AG) for the different aggregates used, which is achieved when the mixture has the lowest void content (maximum compactness); this value is based on the combination of aggregate employed. For this purpose the procedure in ASTM C29/29M-2009 was followed, but without compacting.

As two fine aggregates (A and B) are employed, they were first optimized, being chosen the 20/80 mixture. Then, the mixture of fine aggregates and one coarse aggregate (C) was studied and Figure 1a) shows the results of voids content obtained. It is observed that the optimum proportion fine aggregate / coarse aggregate is one which corresponds to the ratio 80/20, which has the lowest void content.

In the same way for the coarse aggregate mixture, optimization of the two types of coarse aggregates (C and D) is performed, determining that the optimal ratio between them, C/D, is 40/60.
Based on this, mixtures of fine aggregate mixtures (A / B = 20/80) and coarse aggregate mixtures (C / D = 40/60) are studied to determine the optimal ratio between them. Figure 1b) shows the results obtained, where the optimal proportion of fine aggregate / coarse aggregate corresponds to the ratio 60/40, which has the lowest void content.

2.2.2) Paste Optimization: Through Marsh cone trial [16] cement-admixture compatibility is evaluated and the optimum dosage of the admixture or its saturation point was determined if corresponds. Each time, flow time of paste were determined at 5 minutes and 1 hour. The results obtained for the selected admixture is presented in Figure 2a. Moreover, mini-slump tests [17] were performed to determine the optimum addition/cement ratio; the results obtained for different percentages of the additions used with 1% admixture/cement are shown in Figure 2b.

2.3 Design, elaboration and study of properties and costs of concrete

Two concretes groups were formed using optimization granular skeleton described in section 2.2.1, using as fine aggregate the aggregate mixture A / B = 20/80, which are:
-Group 1 consists in concretes made with coarse aggregate C, with an optimal proportion fine aggregate / coarse aggregate of 80/20; and
-Group 2 consists in concretes made with a mixing coarse aggregate (named CD), with C/D = 40/60, and an optimal proportion fine aggregate / coarse aggregate of 60/40.

For both groups were performed substitutions of cement by CKD of 10%, 20%, 30% and 40% and by RHA of 5%, 10%, 15% and 20%, and combinations of them. The percentages of paste tested were: 36%, 38%, 40% and 42%. The water / cement ratios studied ranged between 0.40 and 1.27. The concretes were made in concrete mixer with inclined axis.

First, based on the general method for SCC [15], was defined the dosage of the concrete reference for each group without using additions (CKD or RHA). Concretes test were performed up to the self-compactability. It was found that with paste percentages of 36% and 38% was unable to obtain SCC, because of this paste percentages of 40% and 42% were used, so the method will use these percentages and the economies were obtained using high substitutions of cement. Table 3 shows some of the dosages studied and the results obtained are presented, where P is the paste content, W/C the water/cement ratio, C the Portland cement, FAgg the fine aggregate, CAgg the coarse aggregate, SP the admixture respect to cement. The nomenclature used is, for example 40CKD 15RHA which means 40% CKD and 15% RHA substitution of cement by weight.

Table 3. Mix Proportions and properties of some SCC at 28 days

<table>
<thead>
<tr>
<th>Name</th>
<th>P %</th>
<th>W/C</th>
<th>C kg/m³</th>
<th>RHA kg/m³</th>
<th>CKD kg/m³</th>
<th>FAgg kg/m³</th>
<th>CAgg kg/m³</th>
<th>SP %</th>
<th>T50 seg</th>
<th>D cm</th>
<th>L box</th>
<th>TGM %</th>
<th>T1 seg</th>
<th>f_c MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF</td>
<td>40</td>
<td>0.40</td>
<td>557</td>
<td>0</td>
<td>0</td>
<td>957</td>
<td>622</td>
<td>1.25</td>
<td>0.40</td>
<td>71</td>
<td>0.94</td>
<td>14</td>
<td>3</td>
<td>35.8</td>
</tr>
<tr>
<td>10CKD 10RHA</td>
<td>40</td>
<td>0.50</td>
<td>445.5</td>
<td>56</td>
<td>56</td>
<td>957</td>
<td>622</td>
<td>1.25</td>
<td>0.80</td>
<td>72</td>
<td>0.97</td>
<td>12</td>
<td>3</td>
<td>33.1</td>
</tr>
<tr>
<td>40CKD 15RHA</td>
<td>40</td>
<td>0.89</td>
<td>251</td>
<td>83</td>
<td>223</td>
<td>957</td>
<td>622</td>
<td>1.75</td>
<td>0.70</td>
<td>68</td>
<td>0.88</td>
<td>3</td>
<td>3</td>
<td>24.7</td>
</tr>
<tr>
<td>20CKD</td>
<td>42</td>
<td>0.50</td>
<td>467</td>
<td>0</td>
<td>117</td>
<td>916</td>
<td>611</td>
<td>2.36</td>
<td>0.50</td>
<td>74</td>
<td>0.95</td>
<td>7</td>
<td>3</td>
<td>35.9</td>
</tr>
<tr>
<td>30CKD</td>
<td>42</td>
<td>0.57</td>
<td>409</td>
<td>0</td>
<td>175</td>
<td>916</td>
<td>611</td>
<td>1.00</td>
<td>0.60</td>
<td>72</td>
<td>1</td>
<td>13</td>
<td>2.5</td>
<td>28.0</td>
</tr>
<tr>
<td>40CKD</td>
<td>42</td>
<td>0.67</td>
<td>351</td>
<td>0</td>
<td>234</td>
<td>916</td>
<td>611</td>
<td>1.00</td>
<td>1.10</td>
<td>76</td>
<td>1</td>
<td>20</td>
<td>2</td>
<td>24.1</td>
</tr>
</tbody>
</table>

In fresh state tests are performed with ranges of validity stablished by [17]: Cone Flow following UNE-EN 12350-8, finding T50 and D; L-box in accordance with UNE-EN 12350-10; resistance to segregation by GTM sieve method under UNE-EN 12350-11; and V-funnel according UNE-EN 12350-9 determining T1. Also J-ring according EN 12350-12 with the ranges established in Annex 17 [18].

In relation to the acceptance criteria for self-compacting concrete established by [17], almost all concrete met the requirements of self-compactability in fresh state, and Table 4 shows the categories reached by concrete for each group. With the J-Ring, all SCC met the acceptance parameters for the test (diameter DJ higher or equal than D-50 mm).

In relation to compressive strength results (f_c) at 28 days of age, higher values were obtained for concreted with a mixture of coarse aggregate than a single aggregate (13-23MPa and 24-36MPa, respectively), being more efficiently the aggregates mixture in these ranges of resistance. With the use of additions, range of 18-35 MPa and 24-41 MPa were achieved, respectively.
Table 4: Self-compactability criteria [17] and results according coarse aggregates used

<table>
<thead>
<tr>
<th>Test</th>
<th>Reference values (EFNARC 2005):</th>
<th>Category</th>
<th>C</th>
<th>CD</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Flowability</strong> (Slump flow)</td>
<td>Diameter D: 55 a 65cm</td>
<td>SF1</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Diameter D: 66 a 75cm</td>
<td>SF2</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>Diameter D: 76 a 85cm</td>
<td>SF3</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td><strong>Viscosidad</strong> (Slump flow and V-funnel)</td>
<td>Cone: T50 ≤ 2seg. V-funnel (T1): ≤ 8seg.</td>
<td>VS1/VF1</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>Cone: T50 &gt; 2seg. V-funnel (T1): 9 a 25seg</td>
<td>VS2/VF2</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Passing ability</strong> (L-box)</td>
<td>Passing ability: ≥ 0,80 (with 2 bars)</td>
<td>PA1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Passing ability: ≥ 0,80 (with 3 bars)</td>
<td>PA2</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Segregation resistance</strong></td>
<td>Segreg. ≤ 20%</td>
<td>SR1</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>(GTM)</td>
<td>Segreg. ≤ 15%</td>
<td>SR2</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Based on the results, statistically correlations between variables involved were studied by regression analysis and dosage diagrams were elaborated which allows to, from the compressive strength required, obtain materials amounts (cement content) and ratios (water/cement and addition/cement) that will lead to SCC dosage of medium or low resistance.

The economic viability of these concretes was studied in terms of savings in the production of one cubic meter of each mixture. Figure 3 shows results obtained with aggregate mixture and 40% of paste content.

![Figure 3: Cost saving (%) per m³, with 40% paste content in relation to the reference (REF).](image-url)
It was taken as a basis the value of a reference concrete (REF). It was observed that the saving start verified when the substitution is more than 20%, and more importantly when two additions are used.

3 DOSAGE METHOD: DEFINITION, EVALUATION AND ADJUSTEMENTS

As mentioned in Section 2, the method is valid for paste volumes of 40% and 42% and compression strength under 40 MPa. Economic results are achieved in the range employed mixing coarse aggregates and cement replacement by additions. The dosage method has three stages; the first stage is experimental, corresponding to the optimization of the concrete phases (paste and aggregates) and allows the adaptability of the method to the use of waste as additions and local aggregates; the second stage is analytic, it is based on the use of dosage diagrams and/or equations generated from correlations obtained from experimental results to reach the amount of materials needed to produce one cubic meter of concrete; the third stage is the implementation of the first trial batch and adjusting parameters to ensure the requirements of self-compactability and a medium compression strength (<40MPa).

3.1 Stage 1

Figure 4 shows the sequence of this phase coincides with [15].

![Stage 1: Optimization of concrete phases](image)

3.2 Stage 2

In this stage the amounts of materials are calculated to 1 m³ of concrete, according to the following sequence of 10 steps:

i. Define the required compressive strength (< 40 MPa), average strength of standard-cured concrete specimens at 28 days. For example 30 MPa

ii. Determine the paste volume: 40% or 42%. This is based on experimental results with the local materials used.
iii. Determine the aggregates volume: The amount of paste (P) and aggregates (A) is given by the expression: \( P + A = 1 \text{m}^3 \) of concrete, so: \( A = 1 \text{m}^3 - P \). If the paste volume is 40%, the volume fraction of aggregates is 0.60\( \text{m}^3 \).

iv. Determine the aggregates content: considering the total aggregates volume, the amount of fine and coarse aggregates are calculated using the values of specific gravity and the results of the granular skeleton optimization. From the total aggregates volume (0.60\( \text{m}^3 \)), assuming the results obtained for the aggregates used in 2.2.1, the fine and coarse aggregates volume will be 0.36\( \text{m}^3 \) and 0.24\( \text{m}^3 \), respectively. For fine aggregates A and B the batch weights are 191.52 and 766.08 kg / \( \text{m}^3 \), and for coarse aggregates B and C are 248.64 and 372.96 kg / \( \text{m}^3 \), respectively.

v. Determine the cement amount (c), the water/cement (w/c) and addition/cement (ad/c) ratios, based on steps i and ii, by two possible ways:

v.1. With the use of dosage diagrams (Figure 5), which emerged from correlations obtained from experimental results and consider the minimum consumption of cement concrete 250kg / \( \text{m}^3 \) according to UNIT 1050-2005. The grayed out areas indicate the range of validity of the method. Green dotted exemplified how to use.

![Figure 5: Dosage diagrams for SCC with 40% (a) and 42% (b) of paste content.](image)

v.2. With the use of equations that arise from correlations it can determine the cement amount (c) and the water/cement (w/c) ratio to 40% paste content (Eqs. 1 and 2, with \( R^2 \) of 0.818 and 0.838, respectively) or 42% paste content (Eqs. 3 and 4 with \( R^2 \) of 0.972 and 0.967, respectively). From mathematical resolution of these quadratic equations, two possible solutions are obtained, but adopting as valid the lower value. Equation 5, with \( R^2 = 0.999 \), determines the addition/cement (add/c) ratio for both paste volumes.

\[
\begin{align*}
    f_c &= \frac{0}{0.0001} - 0.0528(c) + 10.7895 \\
    f_c &= 0.057174(w/c)^2 - 99.176(w/c) + 67.407 \\
    f_c &= -0.00013(c)^2 - 0.20016(c) - 30.04286 \\
    f_c &= 46.142(w/c)^2 - 114.130(w/c) + 79.520 \\
    add/c &= +2.478(w/c) - 0.986
\end{align*}
\]
Using v.1 or v.2 to obtain, for example, a concrete with compressive strength (fc) of 30MPa at 28 days and 40% paste volumen is obtained a cement content of 390 kg/m³, a water/cement ratio of 0.62 and addition/cement ratio of 0.55.

vi. Determine the additions amount: based on the step v, from Eq.:

\[ Add = \frac{add}{c} \cdot c \]  

(6)

For additions mixtures, each percentage is determined from the result obtained with the minislump test (see 2.2.2). If maximum fluidity is required, the combination which present larger diameter is adopted; to further decrease the cement content higher percentages of substitution are adopted; to achieve lower segregation can use higher percentage of RHA.

vii. Determine the total water content (W_{tot}): based on the v step and from Eq.:

\[ W_{tot} = \frac{w}{c} \cdot c \]  

(7)

viii. Additive amount: is calculated according to the optimal percentage obtained in the Marsh cone test (2.2.2), taking into account the percentage of solids in the admixture. If the Marsh cone test is not performed, it is recommended to start using 1% of superplasticizer by weight of cement.

ix. Moisture corrections on aggregates: Determine the water present in aggregates (W_{agg}) and correct the quantity of aggregates

x. Mixing water amount (W_{mix}): this is calculated according to Ec. (8) where W_{SP} is the water in the admixture solution

\[ W_{mix} = W_{tot} - W_{agg} - W_{SP} \]  

(8)

3.3 Stage 3

The sequence of this stage is shown in Figure 6. The first trial batch is evaluated in fresh state according to with the requirements set out in Table 4.

According to the results obtained in the fresh state, parameters are adjusted. When a mixture does not meet the desired criteria of self-compactability, adjustments should be made in the admixture, fine aggregates, and/or the granular skeleton. When the mixture meets the criteria for self-compactability, compressive strength is evaluated and, as a result, adjustments to the water/powder ratio, the type or amount of additions [19,20] should be made. Regarding the use of additions should be considered if necessary to decrease the amount of cement to minimize costs

<table>
<thead>
<tr>
<th>Test</th>
<th>Conformity criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump-flow (D=diameter) [17]</td>
<td>(520 \leq D \text{ (cm)} \leq 90)</td>
</tr>
<tr>
<td>V-funnel V [17]</td>
<td>(7s &lt; T1 &lt; 27s)</td>
</tr>
<tr>
<td>L-box L [17]</td>
<td>(H2/H1 \geq 0.75)</td>
</tr>
<tr>
<td>Sieve segregation resistance GTM [17]</td>
<td>Segregation resistance &lt; 23%</td>
</tr>
<tr>
<td>J-ring (DJ=diameter) [18]</td>
<td>(DJ \geq D \text{ (mm)} - 50mm)</td>
</tr>
</tbody>
</table>
4 CONCLUSIONS

To obtain SCC of medium or low compressive strengths, the coarse aggregates mixture is more efficient than the use of a single coarse aggregate, also obtaining lower costs/MPa. Moreover, with increasing addition content replacing by cement, the cost/m³ of SCC are also reduced.

From the results obtained during the stages of evaluating the performance of SCC mixes, both in fresh and hardened state, it can be said that the developed method is an efficient tool for economic SCC dosing of medium and low resistances. It is a simple method that incorporates simple essay techniques based on low-cost equipment, dosing charts and correlations between the different variables involved based on experimental results.

REFERENCES


DURABILITY OF HIGH VOLUME FLY ASH CONCRETE USED IN CHANNEL REVETMENT

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Key words: Concrete, Durability, High Volume Fly Ash, Chloride Ion Diffusion, Carbonation, Frost Resistance.

Abstract. This paper presents the results of a laboratory study on durability properties of high volume fly ash concrete that is suitable for channel revetment. Chloride ion permeability, carbonation, frost-resistance and microstructure variation of concrete were concerned. Two series of mixtures with two different water/binder ratios (0.55 and 0.58 by mass) were prepared by partial replacement of cement with fly ash. The replacing dosage of fly ash varies proportions from 30 % to 60 %. The experiment results demonstrated chloride diffusion coefficient of concrete decreases with the increasing volume of fly ash. However, the resistance to carbonation and freeze-thaw durability was adverse, especially when the content of fly ash was greater than 50 %.

1 INTRODUCTION

Portland cement concrete is the most widely used man-made material in the world. However, Portland cement industry is a heavy contributor to the environmental pollution and CO2 emissions. In order to reduce green hours, fly ash is widely used in blended cement as necessary components recently [1-2]. In addition, more benefits are obtained by the utilization of fly ash, for instance, the improvement of workability, long term strength and the lower hydration heat [3-4]. Recent researches have led an increased interest in replacing cement with high volume fly ash. High volume fly ash (>30 % by volume) used can make concrete more sustainable and economical [5-6]. However, few studies have been reported to investigate the durability properties of concrete when the replacement of fly ash is over 50 %.

In this paper, the influence of different replacing dosage of fly ash will be investigated on the chloride ion diffusion, carbonation and frost resistance of concrete.

2 MATERIALS AND EXPERIMENT

2.1 Materials and mix proportions

Chinese standard 42.5 ordinary Portland cement supplied by Nanjing Conch Cement Company Limited was used in this paper. Fly ash used in this paper was from Taizhou power
plant. The chemical compositions and properties of the cement and fly ash are shown in Table 1. As the content of CaO was 13.92 %, this fly ash was high-calcium fly ash. The super-plasticizer was provided by Jiangsu research institute of building science Co., Ltd. The solid content was 42 % and the water reducing rate was about 18 %.

<table>
<thead>
<tr>
<th>Materials</th>
<th>SiO2 (%)</th>
<th>Al2O3 (%)</th>
<th>Fe2O3 (%)</th>
<th>CaO (%)</th>
<th>MgO (%)</th>
<th>SO3 (%)</th>
<th>f-CaO (%)</th>
<th>LOI (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>28.59</td>
<td>7.22</td>
<td>3.10</td>
<td>51.75</td>
<td>4.29</td>
<td>2.10</td>
<td>0.90</td>
<td>1.79</td>
</tr>
<tr>
<td>Fly ash</td>
<td>43.27</td>
<td>23.34</td>
<td>3.71</td>
<td>13.92</td>
<td>1.75</td>
<td>1.51</td>
<td>1.16</td>
<td>7.39</td>
</tr>
</tbody>
</table>

The binder content used in this paper was 290 kg/m3~310 kg/m3. The content of super-plasticizer was 1.5 %~1.7 % of binder dosage. Fine aggregate was river sand with the fineness modulus of 2.6. Gravels were crushed limestone with the size of 5-31.5 mm. Two groups of mixtures with different fly ash contents were prepared in this paper. Fly ash was used as 30 %, 40 %, 50 % and 60 % by mass of the total binder. The mix proportions of concretes were given in Table 2.

### 2.2 Experimental method

#### 2.2.1 Chloride ion diffusion test

The rapid chloride permeability test (RCPT) according to ASTM C1202 [7] was used to investigate chloride penetration resistance of various concretes by measuring the charge passed through the specimen during six hours of testing. Specimens of 100mm diameter and 50mm thickness were cast. Chloride diffusion coefficient was determined according to:

\[
Y = 2.57765 + 0.00492X
\]

Where \(Y\) was Chloride diffusion coefficient, \(X\) was the 6 hours charge passed.

#### 2.2.2 Carbonation test

The carbonation of concrete was determined by Chinese standard “Standard for test methods of long-term performance and durability of ordinary concrete” (GB/T 50082-2009) [8]. Concrete specimens were 100mm×100mm×300mm prisms. Carbonation depth was measured when carbonation age was 7, 14 and 28 days.

#### 2.2.3 Frost resistance test

Freeze-thaw cycles were applied to specimens in an automatic freeze-thaw machine according to GB/T 50082-2009 [8]. Concrete specimens were 100mm×100mm×400mm prisms. Dynamic elastic modulus and weight loss were tested after each 25 F-T cycles.
Table 2. Mix proportions of concrete.

<table>
<thead>
<tr>
<th>No.</th>
<th>W/B</th>
<th>FA ratio (%)</th>
<th>Mix proportion (kg/m³)</th>
<th>FA</th>
<th>Cement</th>
<th>Water</th>
<th>Fine aggregate</th>
<th>Gravel</th>
<th>Super-plasticizer</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-40</td>
<td>0.58</td>
<td>40</td>
<td>117 176 170</td>
<td>736</td>
<td>1173</td>
<td>4.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-60</td>
<td>0.58</td>
<td>60</td>
<td>176 117 170</td>
<td>729</td>
<td>1162</td>
<td>4.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-0</td>
<td>0.55</td>
<td>0</td>
<td>0 309 170</td>
<td>744</td>
<td>1186</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-30</td>
<td>0.55</td>
<td>30</td>
<td>62 247 170</td>
<td>737</td>
<td>1175</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-40</td>
<td>0.55</td>
<td>40</td>
<td>93 216 170</td>
<td>734</td>
<td>1169</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-50</td>
<td>0.55</td>
<td>50</td>
<td>124 185 170</td>
<td>730</td>
<td>1163</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-60</td>
<td>0.55</td>
<td>60</td>
<td>155 155 170</td>
<td>727</td>
<td>1158</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3 RESULTS AND DISCUSSION

3.1 Chloride ion diffusion

The effect of fly ash replacement on concrete’s resistance to chloride ion permeability is shown in Table 3. It can be seen that when fly ash was mixed into the concrete, the charge passed and chloride diffusion coefficient greatly reduced with an increase in replacement of fly ash. For high volume fly ash concrete curing for 28 days with W/B of 0.55 and fly ash replacement of 60 %, the maximum amplitude of charge passed and chloride diffusion coefficient is 82.4 % and 68.3 %, respectively. This has been largely attributed to morphological effect, pozzolanic effect and micro-aggregate effect of fly ash. It can also be found that the ability of the resistance to chloride ion permeability increased with the increase of the curing time and the chloride resistance of concrete with W/B of 0.55 is better than that of 0.58 due to good compactness.

Table 3. Chloride ion diffusion

<table>
<thead>
<tr>
<th>No.</th>
<th>28 days</th>
<th>90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Charge passed(C)</td>
<td>Chloride diffusion coefficient (x10⁹cm²/s)</td>
</tr>
<tr>
<td>A-40</td>
<td>1002</td>
<td>7.507</td>
</tr>
<tr>
<td>A-60</td>
<td>473</td>
<td>4.905</td>
</tr>
<tr>
<td>B-0</td>
<td>2534</td>
<td>15.045</td>
</tr>
<tr>
<td>B-30</td>
<td>1062</td>
<td>7.803</td>
</tr>
<tr>
<td>B-40</td>
<td>867</td>
<td>6.843</td>
</tr>
<tr>
<td>B-50</td>
<td>516</td>
<td>5.116</td>
</tr>
<tr>
<td>B-60</td>
<td>445</td>
<td>4.767</td>
</tr>
</tbody>
</table>
3.2 Carbonation

The carbonation depths of fly ash concrete at each mix proportion and carbonation age after curing for 28 days are showed in Table 4. It can be found that carbonation depths of fly ash concrete with W/B of 0.55 increase rapidly with the increase of fly ash. The carbonation depth at 28 days of age is 16.1 mm which is less than the general concrete cover thickness, when the dosage of fly ash is 60 %, The results indicate that the carbonation resistance of concrete incorporating fly ash is worse than that of normal concrete due to the high W/B and incompact structure.

3.3 Frost resistance

Figure 1 shows the changes in weight loss of concrete mixed fly ash with W/B of 0.55. Weight loss increases with the increase of fly ash. When the replacement of fly ash is up to 60 %, the weight loss is greatly higher than that of the other four mixtures. The maximum weight loss is about 6.71 %. Weight losses are mainly caused by the surface scaling of the concrete specimens.

<table>
<thead>
<tr>
<th>No.</th>
<th>Carbonation depth at different carbonation age (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>A-40</td>
<td>7.1</td>
</tr>
<tr>
<td>A-60</td>
<td>10.1</td>
</tr>
<tr>
<td>B-0</td>
<td>1.2</td>
</tr>
<tr>
<td>B-30</td>
<td>6.3</td>
</tr>
<tr>
<td>B-40</td>
<td>6.8</td>
</tr>
<tr>
<td>B-50</td>
<td>7.9</td>
</tr>
<tr>
<td>B-60</td>
<td>9.2</td>
</tr>
</tbody>
</table>

Changes in the dynamic modulus of elasticity of the concrete specimens are given in Figure 2. The relative dynamic modulus of elasticity is the ratio of the dynamic modulus of elasticity measured at certain freeze-thaw cycles to that measured before the freeze-thaw cycles. The effects of fly ash on relative dynamic modulus of elasticity are basically same as its impact on weight loss. With the increase of freeze-thaw cycles, relative dynamic modulus of elasticity of all mixtures shows a decreasing trend generally. Relative dynamic modulus of elasticity decreased with the increase of dosage of fly ash. When 60 % fly ash was mixed into concrete, relative dynamic modulus of elasticity is lower than 60 % after 100 cycles of freeze-thaw.
3.4 Microstructure

Environmental scanning electron microscope (ESEM) was used to investigate the microstructure variation. It can be seen in Figure 3a that cement hydration is relatively complete. Hydration products fill a large amount of porosity. The paste structure is compact. When the dosage of fly ash is 40% (see Figure 3b), hydration of cement and fly ash generates hydration products which almost filled the pores, but there still exists unhydrated fly ash particles. When the dosage of fly ash is up to 60% (see Figure 3c), hydration degree is relatively low and there exists a large amount of unhydrated fly ash particles. The microstructure of concrete is loose compared with the others.

4 CONCLUSIONS

- When fly ash is mixed into the concrete, the chloride resistance greatly increases with an increase in replacement of fly ash. Chloride resistance increases with the increase of the curing time. Besides the chloride resistance of concrete with W/B of 0.55 is better than that of 0.58.
- The carbonation resistance of fly ash concrete is worse than that of normal concrete. Carbonation depth of fly ash concrete increases rapidly when the replacement
increases. The carbonation depth at 28 days of age is 16.1 mm when the mixing amount of fly ash is 60 %. The carbonation depth of fly ash concrete is influenced by W/B.

- The frost resistance decreases with the increase of dosage of fly ash. Weight loss increases with the increase of fly ash. When the replacement of fly ash is up to 60 %, the maximum weight loss is about 6.71 %. Relative dynamic modulus of elasticity decreases with the increase of fly ash. When 60 % fly ash is mixed, relative dynamic modulus of elasticity is lower than 60 % after 100 cycles of freeze-thaw.

- Based on the results of durability properties of concretes, the optimal dosage of fly ash is between 40 % ~ 50 %. When fly ash concrete used in channel revetment engineering, it can bring great environmental and economic benefits.

5 ACKNOWLEDGMENTS

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6 REFERENCES


ECONOMICAL EFFECT ON ULTRA-HIGH PERFORMANCE CONCRETE BY USING OF COARSE AGGREGATES

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Key words: Ultra-High Performance Concrete, Sustainability, Economical Effect

Abstract. Ultra-High Performance Concrete is generally mixed with a various number of fine aggregates and steel fibres respectively polypropylene were added for stress absorbing and fire resistance. All these ingredients based on the optimization of the grain size distribution to get a very high packing density. After this optimization the ultra-high performance fibre reinforced concrete meets the target of compressive strength higher than 160 MPa. On a short view on the costs of mixing, it is observed, that the price is extremely high compared to other concretes with high strength. By looking at the cost of mixing, that type of concrete is very expensive compared with other high strength concrete if we are looking at durability and saving materials, which are not ecological worthwhile. UHPC can be in a better light than other concretes. This optimization of the ecological viewpoint has been published before by a variation of the binder and percentage of the hydraulic activated slag [1]. Because of that, the reduction of costs from an economical point of view was analyzed during this work. The new option, which will be presented, is the possibility to change the matrix itself with coarse aggregates. The usual mixtures of UHPC have an amount of 15% coarse aggregates, which leads to a non-sufficient workability. One of the steps is to use a pre-pact method. In the examination we considered the influence of coarse aggregates on the workability and sustainability. Later the reduction of the content of cost intensive materials like cement, silica fume and quartzite fume was focused. In the second step of the examination the workability of pre-pact ultra high performance concrete elements is adapted to the pre-cast segment industry. It should be shown that the addition work for the pre-pact method is lower than the costs of the saved ingredients.

1 INTRODUCTION

Ultra-High Performance Concrete called UHPC should be analyzed in the following consideration analyzed as a product with more than the modern touch of high load bearing capacity together with a high tech reinforcement of steel fibres. Most of the discussions circulate around the keywords price and workability on the construction site. The product exists nearly 20 years and since the beginning of the production UHPC was called the construction material of the future. The first problem of the sudden fatigue when the tensile strength is reached was solved with the addition of steel fibres. This addition leads to a ductile behavior during fatigue. The second problem was the price of the ingredients and at the end the sum of the costs for quality assurance and application. Nevertheless a lot of practical
examples exist, e.g. Wildbruecke in Carinthia [2], Facade of representative structures [3] and strengthening refurbishments [4]. In ordinary constructions UHPC or UHPFRC (with steel fibres) couldn’t find a general acceptance. Currently in multilevel buildings concrete with a compressive strength higher than 100 MPa is used. This products have a satisfying workability and transportability. The costs of such structures are moderate compared with the dimension and the costs of ordinary construction methods. This shows the way of optimizing the structures furthermore. One of the potential field of common using of UHPC is the pre-cast element industry. Because of that, the research team has a special focus on that production type. If we have a short period of workability and a formwork design has been optimized for this workability, a high grade of cost optimization is possible, respective by the sustainability will increase. One optimization was expected by changing the fibres from steel to polymere or stone fibres but this substitution causes a decreasing of the compressive strength [5]. Another approach is reducing the cost intensive matrix of UHPC in the matrix itself. From this point of view an alternative material is needed with the following properties:

- Compressive strength of more than 160 MPa
- Durability of the UHPC
- Workability should not be influenced
- Costs are lower than the matrix

The following pages will present a possibility study of the substitution of the UHPC matrix with basalt crushed stones with different grading curves, different contents and the experience by using this method.

2 PRINCIPE OF THE OPTIMISATION

In the first step of the model for optimizing the costs of UHPC the matrix is reduced and is replaced by an optimal distribution of ideal round spaces, see figure 1.

![Figure 1: Optimisation the possible exchangeable space](image)
The optimal distribution is not according to design an ordinary concrete. There should not be a granular structure which carry stone by stone. UHPC is the flux material and it needs space for workability. At this moment it is an iterative process during the laboratory test phase. The free spaces are replaced by hard stones, according figure 2.

![Figure 2: Substitute the free spaces with a defined hard stone grading curve](image)

The principle is simple but it is not evident which stone are usable for the replacement of the UHPC-matrix. Table 1 shows mechanical properties of natural stones. It shows that the most important stones are igneous rocks. According to the natural standard derivation and the adverse influences of pollution of other igneous rocks of the natural mineral deposit, a material should be used with a high grade of purity.

**Table 1: Example of mechanical stone properties [6]**

<table>
<thead>
<tr>
<th>Kind of Stone</th>
<th>Dry Density [g/cm³]</th>
<th>Compressive strength [MPa]</th>
<th>Young’s Modulus [10^5 N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granit, Syenite</td>
<td>2.60-2.80</td>
<td>160-240</td>
<td>3.8-7.6</td>
</tr>
<tr>
<td>Diorit, Gabbro</td>
<td>2.80-3.00</td>
<td>170-300</td>
<td>5-6</td>
</tr>
<tr>
<td>Quanzoporphy</td>
<td>2.55-2.80</td>
<td>180-300</td>
<td>2.5-6.5</td>
</tr>
<tr>
<td>Basalt, Melaphyr</td>
<td>2.90-3.05</td>
<td>250-400</td>
<td>9.6</td>
</tr>
<tr>
<td>Diabas</td>
<td>2.80-2.90</td>
<td>180-250</td>
<td></td>
</tr>
<tr>
<td>Graywacke, Quartzite,</td>
<td>2.60-2.65</td>
<td>150-300</td>
<td>5.99</td>
</tr>
<tr>
<td>quartzite Sandstone</td>
<td>2.60-2.65</td>
<td>120-200</td>
<td>1-2</td>
</tr>
<tr>
<td>Crushed gravel, Gravel</td>
<td>2.60-2.75</td>
<td>fluctuating</td>
<td>no information</td>
</tr>
<tr>
<td>Limestone, Dolomite</td>
<td>2.565-2.85</td>
<td>80-180</td>
<td>8.16</td>
</tr>
<tr>
<td>Gneiss, Granulite</td>
<td>2.65-3.00</td>
<td>160-280</td>
<td>no information</td>
</tr>
<tr>
<td>Amphibolite</td>
<td>2.70-3.10</td>
<td>170-280</td>
<td>no information</td>
</tr>
</tbody>
</table>
3 MIXTURE

3.1 Choice of stones

During the first test phase of optimizing the costs of UHPC, basalt stones from the area of Austria with the compressive strength of more than 160 MPa are used. The influence of 2 grading curves was analyzed. In the following table the properties of the used material is given.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Split 2/4</th>
<th>Split 4/8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grading curve</td>
<td>G&lt;sub&gt;90/15&lt;/sub&gt;</td>
<td>G&lt;sub&gt;90/15&lt;/sub&gt;</td>
</tr>
<tr>
<td>Dust content</td>
<td>f&lt;sub&gt;1&lt;/sub&gt;</td>
<td>f&lt;sub&gt;1&lt;/sub&gt;</td>
</tr>
<tr>
<td>Crush Resistance in</td>
<td>CS&lt;sub&gt;80&lt;/sub&gt;</td>
<td>CS&lt;sub&gt;80&lt;/sub&gt;</td>
</tr>
<tr>
<td>Compressive</td>
<td>208</td>
<td>208</td>
</tr>
<tr>
<td>strength in MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water absorption in</td>
<td>0,9</td>
<td>0,9</td>
</tr>
<tr>
<td>% by mass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density in kg/dm³</td>
<td>2,87</td>
<td>2,87</td>
</tr>
</tbody>
</table>

3.2 Mix design of the UHPC

At the beginning of the examination the target was an UHPC with defined coarse aggregates for self-compacting workability. The variation of the mixtures is shown in table 3 and 4.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Bas-1</th>
<th>Bas-2</th>
<th>Bas-3</th>
<th>Bas-4</th>
<th>Bas-5</th>
<th>Bas-6</th>
<th>Bas-7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subst. Volume in l</td>
<td>30</td>
<td>30</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>Cement</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
</tr>
<tr>
<td>Silica</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
</tr>
<tr>
<td>Quartz dust</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
</tr>
<tr>
<td>Basalt 2/4</td>
<td></td>
<td></td>
<td>297,0</td>
<td></td>
<td>297,0</td>
<td></td>
<td>445,5</td>
</tr>
<tr>
<td>Basalt 4/8</td>
<td>91,0</td>
<td>91,0</td>
<td>297,0</td>
<td></td>
<td>297,0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Quartz sand</td>
<td>600,0</td>
<td>600,0</td>
<td>599,7</td>
<td>599,7</td>
<td>599,7</td>
<td>599,7</td>
<td>467,2</td>
</tr>
<tr>
<td>Water</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
</tr>
<tr>
<td>Steel fibre 15/0,20 mm</td>
<td>78,5</td>
<td>157,0</td>
<td>78,5</td>
<td>78,5</td>
<td>157,0</td>
<td>157,0</td>
<td>78,5</td>
</tr>
</tbody>
</table>
Table 4: Mix designs of self-compacting UHPC in kg/m³ - Basalt 7 to Basalt-13

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Bas-8</th>
<th>Bas-9</th>
<th>Bas-10</th>
<th>Bas-11</th>
<th>Bas-12</th>
<th>Bas-13</th>
<th>Bas-14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subst. Volume in l</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Cement</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
<td>850,0</td>
</tr>
<tr>
<td>Silica</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
<td>143,0</td>
</tr>
<tr>
<td>Quartz dust</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
<td>245,0</td>
</tr>
<tr>
<td>Basalt 2/4</td>
<td>445,5</td>
<td>445,5</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
</tr>
<tr>
<td>Basalt 4/8</td>
<td>445,5</td>
<td>445,5</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
<td>594,0</td>
</tr>
<tr>
<td>Quartz sand</td>
<td>467,2</td>
<td>467,2</td>
<td>467,2</td>
<td>334,7</td>
<td>334,7</td>
<td>334,7</td>
<td>334,7</td>
</tr>
<tr>
<td>Water</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
<td>195,0</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
<td>20,0</td>
</tr>
<tr>
<td>Steel fibre 15/0,20 mm</td>
<td>78,5</td>
<td>157,0</td>
<td>157,0</td>
<td>78,5</td>
<td>78,5</td>
<td>157,0</td>
<td>157,0</td>
</tr>
</tbody>
</table>

During the tests it is shown another possibility for use the UHPC as pre-pact concrete. The following table shows the mix design.

Table 5: Mix designs of pre-pact UHPC in kg/m³

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Pre-1</th>
<th>Pre-2</th>
<th>Pre-3</th>
<th>Pre-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subst. Volume in l</td>
<td>520</td>
<td>345</td>
<td>261</td>
<td>261</td>
</tr>
<tr>
<td>Cement</td>
<td>405,7</td>
<td>553,8</td>
<td>627,9</td>
<td>627,9</td>
</tr>
<tr>
<td>Silica fume</td>
<td>68,3</td>
<td>93,2</td>
<td>105,6</td>
<td>105,6</td>
</tr>
<tr>
<td>Quartz dust</td>
<td>117,0</td>
<td>159,6</td>
<td>181,0</td>
<td>181,0</td>
</tr>
<tr>
<td>Basalt 4/8</td>
<td>1500</td>
<td>1000</td>
<td>750</td>
<td>750</td>
</tr>
<tr>
<td>Quartz sand</td>
<td>412,9</td>
<td>563,6</td>
<td>639,0</td>
<td>639,0</td>
</tr>
<tr>
<td>Water</td>
<td>93,1</td>
<td>127,1</td>
<td>144,0</td>
<td>144,0</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>9,5</td>
<td>13,0</td>
<td>14,8</td>
<td>14,8</td>
</tr>
</tbody>
</table>

As seen in table 5, there weren’t fibres added to the pre-mixes, because the present work is only a case study of the maximum substitution by coarse aggregates, which could have a practical significance in a pre-fabricated structural member with ordinary reinforcement.

4 WORKABILITY

4.1 Self-compacting UHPC

The mixes Bas-1 to Bas-14 are self-compacting mixtures. The proof of workability measured the maximal flow of a mortar mix of a volume Haegermann-funnel according to a slump flow test on a glass surface is a measurement of the workability.
Table 6: Results of workability

<table>
<thead>
<tr>
<th>Bas-1</th>
<th>Bas-2</th>
<th>Bas-3</th>
<th>Bas-4</th>
<th>Bas-5</th>
<th>Bas-6</th>
<th>Bas-7</th>
<th>Bas-8</th>
<th>Bas-9</th>
<th>Bas-10</th>
<th>Bas-11</th>
<th>Bas-12</th>
<th>Bas-13</th>
<th>Bas-14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bas-1</td>
<td>30</td>
<td>Bas-2</td>
<td>29</td>
<td>Bas-3</td>
<td>33</td>
<td>Bas-4</td>
<td>31</td>
<td>Bas-5</td>
<td>32</td>
<td>Bas-6</td>
<td>28</td>
<td>begin of separation</td>
<td>Bas-7</td>
</tr>
<tr>
<td>Bas-8</td>
<td>22</td>
<td>Bas-9</td>
<td>29</td>
<td>Bas-10</td>
<td>29</td>
<td>Bas-11</td>
<td>30</td>
<td>Bas-12</td>
<td>26</td>
<td>Bas-13</td>
<td>27</td>
<td>separation</td>
<td>Bas-14</td>
</tr>
</tbody>
</table>

Table 6 shows the diameter of the slump test after 2 minutes. It was difficult to produce stable mixtures with a content of steel fibres 2% by volume. It is shown in a separation when the superplasticizer added after a stiff behavior with the content of the origin. The mixtures with the basalt split 2/4 are more stable according to the amount of basalt split generally.

4.2 Pre-pact UHPC

The used pre-pact principle needs the workability of the ordinary UHPC-mortar. The usability is for pre-cast elements and because of that no steel fibres were used. The following figures show the idea behind the pre-pact principle.

![Figure 3: Principle of the pre-pact UHPC](image)

The different mixes were produced with different methods as shown in the following list.
• Pre-1 placed in 1 layer and compacted with a low amplitude vibration
• Pre-2 placed in 2 layer and no additional compacting
• Pre-3 placed in 3 layer and no additional compacting
• Pre-4 placed in 1 layer and no additional compacting

5 RESULTS

The results of the tests of the self-compacting mixtures and the pre-pact mixtures are shown as conclusion in figure 4.

![Figure 4: Results of compressive strength of the UHPC](image)

It was shown that the specimen Pre-1 was not successful because no penetration of the UHPC-mortar was observed and the mixture was not used furthermore. The other mixtures show that the substitution of UHPC-Matrix has only a small influence to the compressive strength. The most influence was observed on the workability, see table 6.

6 CONCLUSION

The conclusion has the focus on the influence of the price of UHPC, when the matrix is substituted by coarse aggregates. The technical targets have been met with all mixtures and the compressive strength is higher than 160 MPa. The workability is not given in every mixture, as shown. This is a case study that is why the mixtures are not adapted and optimized, there is a high potential for the future. The price development of the ingredients are highly rated, figure 5 shows the prices per kg.
The pure material price of UHPC with steel fibres is round about 0,80 € per liter. When basalt split is substituted to 200 l, 0,10 € can be saved. This benefit takes 13,5 %. There is a possibility to optimize the benefit till 25 %, but the production process has to be changed. In this prices are not including transportation, the mixing process and the quality assurance is not included.

7 OUTLOOK

This case study shows only the potential of substitution of UHPC matrix with coarse aggregates. The potential is much higher because it is possible to use hard stones with lower costs than basalt stones.

REFERENCES


Research Project HiPeComp on the CUAS founded by FFG

EFFECT OF INTERNAL ALKALI ACTIVATION ON LONG-TERM POZZOLANIC REACTION OF FLY ASH IN CEMENT PASTE

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††† nakarai@hiroshima-u.ac.jp, †††† kkawai@hiroshima-u.ac.jp

Key words: Ca(OH)$_2$ Content, Cement Paste, Fly Ash, Internal Alkali Activation, Porosity, Pozzolanic Reaction

Abstract. In recent years, alkali activation on fly ash particles has been suggested for accelerating their pozzolanic reaction, which enhances the durability of concrete. It was reported that the alkali activations become most effective when high temperature curing is applied, but it is limited for many applications. Therefore, alkali activation at normal temperature on fly ash cement paste 1 month after casting was investigated and it was found that the alkali activation was effective in accelerating the pozzolanic reaction at the age of 2 months. This study deals with the effects of internal alkali activation on the long-term pozzolanic reaction of fly ash up to the age of 12 months in the cement paste cured at normal temperature by examining the Ca(OH)$_2$ (CH) content, consumption of CH, and porosity. Cement pastes were prepared with a water to binder ratio of 30%. Fly ash replacement ratios were 0% (FA0) and 40% (FA40) by mass. A saturated Ca(OH)$_2$ solution was used for an internal alkali activation and water was used for comparison. The internal activation was carried out by supplying the solution naturally through an installed syringe from the age of 1 month. As a result, the injection of water or saturated Ca(OH)$_2$ solution increased the CH content in FA0 at the age of 12 months when compared with no injection. Meanwhile, the injection of saturated Ca(OH)$_2$ solution from the age of 1 month was more effective in decreasing the CH content and the volume ratio of 20-330 nm pores to the total pore in FA40 and increasing the consumption of CH than the injection of water from the age of 1 month. It indicates that the internal activation from the age of 1 month by using saturated Ca(OH)$_2$ solution accelerated the pozzolanic reaction of fly ash particles in the cement paste up to the age of 12 months more than that by using water. In addition, the internal activation from the age of 1 month reduced the total pore volume in FA0 and FA40. It can be concluded that internal alkali activation was effective in accelerating the pozzolanic reaction of fly ash in cement paste up to the age of 12 months.
INTRODUCTION

Fly ash is a by-product of combustion of pulverized coal in power plants. Fly ash is used as a supplementary cementitious material in the partial replacement of Portland cement in order to not only reduce a huge amount of carbon dioxide (CO$_2$) emission from cement manufacturing but also improve the properties and durability of the hardened concrete through its pozzolanic reaction [1, 2, 3, 4]. Pozzolanic reaction of fly ash is the reaction between its reactive silica or alumina and Ca$^{2+}$ ion or Ca(OH)$_2$ (CH) in the presence of water [5]. Hence, the reduction of CH content in the matrix appears to be able to indicate the pozzolanic reaction of fly ash [6]. Generally, the pozzolanic reaction is dependent on the characteristics of fly ash, such as the fineness of particles, the chemical composition, the active phase content and so on. Chindaprasirt et al. found that the smaller the fineness of fly ash particles, the higher the pozzolanic reaction [7]. Some external factors (water to binder ratio, fly ash replacement ratio, and the curing temperature) have been found to affect the rate of pozzolanic reaction. Poon et al. reported that the higher the fly ash replacement, the lower the rate of pozzolanic reaction of fly ash at lower water to binder ratio [8]. Hanehara et al. concluded that the higher the curing temperature or the higher the water to powder ratio, the higher the rate of pozzolanic reaction [9]. Additionally, more than 80% unreacted fly ash particles are observed in the pastes with the 45% and 55% replacement of fly ash after 90 days of curing [10], and they still remain 72.7% after 4 years in the hardened fly ash cement paste [11]. This limits the improvement in the strength and the enhancement of the durability of the concrete.

In recent years, alkali activation on fly ash particles has been suggested for accelerating their pozzolanic reaction to maximize the strength and durability of concrete [12, 13, 14, 15, 16, 17]. Alkali activation is known as chemical activation involving treatment of fly ash particles with an alkali solution directly. Calcium hydroxide (Ca(OH)$_2$) has been previously used to activate fly ash [12, 14]. Ryu et al. depicted schematically the mechanism of fly ash in the presence of alkali activator as follows: (1) due to the attack of OH$^-$ ion of alkali activator on chemically stable glassy surface layer, the corrosion at the surface of fly ash particles starts to occur, (2) the Si-O-Si and Al-O-Al are disrupted, (3) Si$^{4+}$ and Al$^{3+}$ ions are released and combine to Ca$^{2+}$ ion of alkali activator, (4) reaction products are formed [16]. This alkali activation becomes most effective when high temperature curing is applied [13, 14, 15, 16], but it is limited for many applications.

Alkali activation at normal temperature on fly ash cement paste 1 month after casting was investigated and it was found that the alkali activation was effective in accelerating the pozzolanic reaction at the age of 2 months [17]. This study deals with the effects of internal alkali activation on the pozzolanic reaction of fly ash up to the age of 12 months in the cement paste cured at normal temperature by examining the CH content, consumption of CH, and porosity.

EXPERIMENTS

2.1 Materials

Low-calcium fly ash was used as a partial replacement of high-early-strength Portland cement...
cement. Fly ash met the standard values of type II per JIS A 6201 (fly ash for concrete) and high-early-strength Portland cement ensuring the early strength of paste conformed to JIS R 5210 (Portland cement). Table 1 shows the chemical compositions and physical properties of high-early-strength Portland cement and fly ash.

Table 1: Chemical compositions and physical properties of cementitious materials

<table>
<thead>
<tr>
<th>Chemical composition and physical properties</th>
<th>Cement</th>
<th>Fly ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂ (%)</td>
<td>20.30</td>
<td>57.7</td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>2.71</td>
<td>5.43</td>
</tr>
<tr>
<td>Al₂O₃ (%)</td>
<td>4.96</td>
<td>27.54</td>
</tr>
<tr>
<td>CaO (%)</td>
<td>65.49</td>
<td>1.26</td>
</tr>
<tr>
<td>MgO (%)</td>
<td>1.21</td>
<td>1.06</td>
</tr>
<tr>
<td>SO₃ (%)</td>
<td>2.98</td>
<td>0.36</td>
</tr>
<tr>
<td>Na₂O (%)</td>
<td>0.22</td>
<td>0.44</td>
</tr>
<tr>
<td>K₂O (%)</td>
<td>0.35</td>
<td>0.76</td>
</tr>
<tr>
<td>Loss on ignition (%)</td>
<td>1.19</td>
<td>2.8</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>3.14</td>
<td>2.21</td>
</tr>
<tr>
<td>Blaine specific surface area (cm²/g)</td>
<td>4590</td>
<td>3290</td>
</tr>
</tbody>
</table>

2.2 Preparation of fly ash cement pastes

Cement pastes were prepared with a water to binder ratio of 30%. Fly ash replacement ratios were 0% (FA0) and 40% (FA40) by mass. A saturated Ca(OH)₂ solution was used for an internal alkali activation and water was used for comparison. The pastes were mixed in a mechanical mixer and were cast in 40-mm cube molds, and then the placing surfaces of specimens were sealed with aluminum tape to prevent water loss as well as carbonation. After that, a 1-ml syringe was inserted into the center of the specimen as shown in Figure 1 (a). All specimens were demolded 24 hours after casting and cured in sealed condition at 20°C.

2.3 Method of internal activation

Water and saturated Ca(OH)₂ solution were supplied through the syringes from the age of 1 month as shown in Figure 1 (b). In addition, no solution was also injected into the paste specimens for the control. Water or saturated Ca(OH)₂ solution was allowed to be imbibed naturally into the pastes over time as shown in Figure 1 (c). This was considered as the original model of internal curing. The volume of water or saturated Ca(OH)₂ solution imbibed over time was also observed for reference.
2.4 Measurement of CH content and porosity

The CH contents of the control, water-injected or saturated-Ca(OH)$_2$-injected specimens into which no solution, water or saturated Ca(OH)$_2$ solution was injected from the age of 1 month, respectively were determined by thermal gravimetry analysis (TGA) up to the age of 12 months. Sample was drilled out every 4 mm in depth and collected as powder as shown in Figure 2 (a). Each sample was soaked in ethanol for 24h to stop further hydration and dried in a vacuum desiccator for 24h before TGA. The CH content was calculated based on the ignited mass of the sample and the mass loss due to the dehydration of calcium hydroxide [17, 18].

The porosities were measured by mercury intrusion porosimetry (MIP) over a diameter range of 3 nm-300 µm at the ages of 6 and 12 months. The measurements of porosities were conducted for the specimens obtained by crushing the hardened pastes and selecting in the size range 2.5-5.0 mm from around the position of the needles as shown in Figure 2 (b). Similar to measuring CH content, each sample was also soaked in ethanol for 24h to stop further hydration and dried in a vacuum desiccator for 24h before MIP measurement.
3 RESULTS AND DISCUSSIONS

3.1 CH content

The CH contents of the control, water-injected and saturated-Ca(OH)$_2$-injected specimens at the age of 12 months are shown in Figure 3. According to the left of Figure 3, the injection of water or saturated Ca(OH)$_2$ solution increased the CH contents of FA0 specimens at all depths when compared with no injection at the age of 12 months. It is evident that the injection of water or saturated Ca(OH)$_2$ solution promoted the cement hydration in the paste without fly ash replacement (FA0) at the age of 12 months. Meanwhile, according to the right of Figure 3, the injection of water or saturated Ca(OH)$_2$ solution decreased the CH contents of FA40 specimens at all depths when compared with no injection at the age of 12 months, except for depth of 0-4 mm from the surface of specimen in the case of injection of saturated Ca(OH)$_2$ solution. This may be explained that the cement hydration at this depth range was promoted by water from the injection of saturated Ca(OH)$_2$ solution more than no injection, resulting in the higher CH content. Briefly, the injection of water or saturated Ca(OH)$_2$ solution was effective in accelerating the pozzolanic reaction in FA40 specimens at the age of 12 months because of the reduction of CH content.
Figure 3: Effect of injection of water or saturated Ca(OH)₂ solution on the CH content of FA0 (left) and FA40 (right) specimens at the depth of every 4 mm from the surface at the age of 12 months.

The CH contents of the control, water-injected or saturated-Ca(OH)₂-injected specimens at the point of the needle (i.e. depth of 16-20 mm from the surface of specimen) over time are shown in Figure 4 in order to express the effect of internal alkali activation on the cement hydration and the pozzolanic reaction over time. It appears that the injection of water promoted the cement hydration in the paste without fly ash replacement (FA0) not only at the age of 2 months [17] but also up to the age of 12 months as shown in the left of Figure 4. Meanwhile, the injection of saturated Ca(OH)₂ solution affected the cement hydration adversely in FA0 at the early ages (i.e. the ages of 2 and 4 months), but promoted the hydration at the later ages (at the age of 6 months and later) as shown in the right of Figure 4. Bui et al. explained that the negative effect of the production of CH in the cement paste at the early age is due to high concentration of saturated Ca(OH)₂ solution injected from the age of 1 month [17]. Briefly, it can be seen that the injection of water or saturated Ca(OH)₂ solution promoted the cement hydration in FA0 specimens up to the age of 12 months. It may be also expected that this injection should promote the cement hydration in FA40 specimens, resulting in the slightly higher CH content in FA40 specimens up to the age of 12 months. However, for FA40 specimens, the CH contents of the water-injected specimens were nearly the same as those of the control specimens at the ages of 2, 4, 10, and 12 months as shown in the left of Figure 4, whereas the CH contents of the saturated-Ca(OH)₂-injected specimens were smaller than those of the control specimens at the age of 2 months and slightly smaller at the ages of 4, 10, and 12 months as shown in the right of Figure 4 despite of their larger CH contents at the ages of 6 and 8 months. It indicates that the injection of water or saturated Ca(OH)₂ solution accelerated the pozzolanic reaction in FA40 specimens in addition to the cement hydration promotion, resulting in the same or slightly smaller CH contents compared with no injection up to the age of 12 months. In addition, the injection of saturated Ca(OH)₂ solution was more effective in accelerating the pozzolanic reaction in FA40 specimens than that of water from the age of 1 month due to the slight reduction of the CH contents.
3.2 Consumption of CH

The consumption of CH by the pozzolanic reaction in FA40 specimen at the point of the needle was calculated under each condition (the injection of no solution, water or saturated Ca(OH)\textsubscript{2} solution) by equation (1) [6].

\[
CH_{\text{cons.}} = CH_{PC}(\frac{c}{c+f}) - CH_{FC}
\]

where $CH_{\text{cons.}}$ is the consumption of CH by the pozzolanic reaction under each condition (%), $CH_{PC}$ is the CH content in the cement paste (FA0) under each condition (%), $CH_{FC}$ is the CH content in the fly ash cement paste (FA40) under each condition (%), $(c/(c+f))$ is the mass ratio of cement in the binder (cement + fly ash), $(c/(c+f)) = 0.6$.

The consumption of CH by the pozzolanic reaction at the point of the needle under each condition (the injection of no solution, water or saturated Ca(OH)\textsubscript{2} solution) is shown in Figure 5. It can be found that the consumptions of CH of samples in the case of injection of both water and saturated Ca(OH)\textsubscript{2} solution at the age of 12 months were larger than those of samples in the case of no injection. Additionally, the injection of saturated Ca(OH)\textsubscript{2} solution from the age of 1 month was more effective in increasing the consumption of CH at the ages of 2, 10, and 12 months than the injection of water from the age of 1 month. It confirms again that the injection of saturated Ca(OH)\textsubscript{2} solution was more effective in accelerating the pozzolanic reaction in FA40 specimens than the injection of water from the age of 1 month.

It also appears that the consumption of CH in the case of injection of water or saturated Ca(OH)\textsubscript{2} solution was nearly the same as or smaller slightly than that in the case of no injection at the ages of 4 and 6 months. It may be explained that the content of CH formed by cement in FA40 could be higher than that in FA0 when the injection of water or saturated Ca(OH)\textsubscript{2} solution was applied from the age of 1 month. As a result, the CH consumption in the case of injection of water or saturated Ca(OH)\textsubscript{2} solution from the age of 1 month was nearly the same as or smaller slightly than that in the case of no injection.

Figure 4: Effect of injection of water (left) or saturated Ca(OH)\textsubscript{2} solution (right) on the CH content at the point of the needle of FA0 and FA40 specimens over time
3.3 Porosity

Effects of the injection of water or saturated Ca(OH)$_2$ solution on the cumulative pore volume of FA0 and FA40 specimens at the ages of 6 months (left) and 12 months (right) are shown in Figures 6 and 7, respectively. It can be said that the injection of water or saturated Ca(OH)$_2$ solution reduced the total pore volumes of FA0 and FA40 specimens when compared with no injection at the ages of 6 and 12 months, except for saturated-Ca(OH)$_2$-injected specimen (FA40) at the age of 12 months. It may be attributed partially to the entrained air ranging from 50 to 200 µm in size [19].
According to Yamamoto and Kanazu, the volume of pores from 20 to 330 nm in diameter decreases relative to the total pore volume, and that of pores from 3 to 20 nm in diameter increases as the pozzolanic reaction of fly ash in the paste proceeds [20]. The volumes of 20-330 nm and 3-20 nm diameter pores of the control samples, water-injected samples, and saturated-Ca(OH)$_2$-injected samples at the ages of 6 and 12 months are shown in Figures 8 and 9, respectively. In addition, the volumes of water or saturated Ca(OH)$_2$ solution imbibed before the ages of 6 and 12 months are also shown in Figures 8 and 9, respectively for reference. As shown in Figure 8, the injection of water and saturated Ca(OH)$_2$ solution decreased the volume ratio of 20-330 nm pores to the total pore in FA40 by 51.2% and 58.4%, respectively, but not in FA0 at the age of 6 months when compared with no injection. It indicates the injection of water or saturated Ca(OH)$_2$ solution accelerated the pozzolanic reaction of fly ash in FA40 specimens at the age of 6 months.

When compared with no injection, the decrease in the volume ratio of 20-330 nm pores to the total pore of FA40 in the case of injection of water and saturated Ca(OH)$_2$ solution was 11.4% and 16.3%, respectively despite of the higher total pore volume of saturated-Ca(OH)$_2$-injected specimen (FA40) at the age of 12 months as shown in Figure 9. Moreover, the injection of saturated Ca(OH)$_2$ solution from the age of 1 month was more effective in decreasing the volume ratios of 20-330 nm pores to the total pore in FA40 than that of water at the ages of 6 and 12 months as shown in Figures 8 and 9, respectively. It indicates that internal activation from the age of 1 month by using saturated Ca(OH)$_2$ solution accelerated the pozzolanic reaction of fly ash particles in the cement paste up to the age of 12 months more than that by using water.

Figure 7: Effect of injection of water or saturated Ca(OH)$_2$ solution on the cumulative pore volume of FA40 specimens at the ages of 6 months (left) and 12 months (right)
Figure 8: Effect of injection of water or saturated Ca(OH)$_2$ solution on pore size distribution of FA0 (left) and FA40 (right) specimens at the age of 6 months.

Figure 9: Effect of injection of water or saturated Ca(OH)$_2$ solution on pore size distribution of FA0 (left) and FA40 (right) specimens at the age of 12 months.

4 CONCLUSIONS

- The injection of water or saturated Ca(OH)$_2$ solution from the age of 1 month increased the Ca(OH)$_2$ (CH) content and reduced the total pore volume of the cement.
paste without fly ash (FA0) up to the age of 12 months. It shows the cement hydration in FA0 was promoted up to the age of 12 months by the internal activation from the age of 1 month.

- The injection of water or saturated Ca(OH)\(_2\) solution from the age of 1 month decreased the CH content and increased the consumption of CH by the pozzolanic reaction of the cement paste with 40% replacement of fly ash (FA40) at the age of 12 months. Moreover, the injection of water or saturated Ca(OH)\(_2\) solution reduced the total pore volume of FA40. It shows the pozzolanic reaction in FA40 at the age of 12 months was accelerated by the internal activation from the age of 1 month.

- The injection of saturated Ca(OH)\(_2\) solution decreased the CH content and increased the consumption of CH by the pozzolanic reaction more than that of water. In addition, the injection of saturated Ca(OH)\(_2\) solution reduced the volume ratio of 20-330 nm pores to the total pore in FA40 by 16.3% compared with no injection while that of water reduced this volume ratio by 11.4% at the age of 12 months. It indicates that the internal activation from 1 month by using saturated Ca(OH)\(_2\) solution was more effective in accelerating the pozzolanic reaction of fly ash particles in the cement paste up to the age of 12 months than that by using water.

REFERENCES

EFFICIENCY FACTORS OF FLY ASH - A POWERFUL TOOL FOR MIX PROPORTIONING

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Keywords: concrete, fly ash, efficiency factors, mix proportioning

Summary Fly ash has already proved its worth as an excellent cement replacement material. The physical and chemical properties of fly ash are widely variable and hence an idea about its cementitious properties can help in assessing the amount of ash to be replaced by one part of cement to achieve equivalent strength as that of control concrete. The cementing efficiency of a pozzolan is defined as the number of parts of cement that could be replaced by one part of the pozzolan without affecting the property under consideration (mainly strength). Statistical models developed by multiple regression analysis using compressive strengths determined over a wide range of w/cm values ranging from 0.3 to 0.6 and binder contents of 300, 375 and 450 kg/m³ for predicting efficiency factors of fly ash have been presented in a previous publication of the author. The present paper is a continuation of the previous research and deals with further value addition to the previous reporting. In this paper the average values of the 28-day efficiency factors have been evaluated and their applicability on the mixture proportioning of fly ash concrete has been enumerated. The proposed values of efficiency factors if used in mix proportioning of fly ash concrete can lead to proper quantification of the ash used without merely relying on the experience and discretion of the designer. It may be expected that the findings of the present work may serve as a useful guideline for judiciously applying the concept of efficiency factors to optimize the effect of fly ash in concrete and lead to improvement in the method of mix design of fly ash concretes.
1 INTRODUCTION

With the objective of attaining sustainable construction a strong trend favoring the increased use of mineral admixtures, which are basically the waste products of industrial processes, in concrete is emerging throughout the world. Indian coals are very rich in ash content and hence ash is abundantly produced in India as a waste product in thermal power plants. It is well documented that the use of fly ash, as a partial replacement for cement results in a significant improvement in concrete properties. Efforts have been made to quantify the effect of fly ash in concrete by determining the cementing efficiency factors. The cementing efficiency of a pozzolan is defined as the number of parts of cement that could be replaced by one part of the pozzolan without affecting the property under consideration (mainly strength). According to Papadakis and Tsimas [1] the efficiency factor (k-value) is defined as the part of the pozzolanic material which can be considered as equivalent to Portland cement having the same properties as the concrete without the pozzolanic material (k=1 for Portland Cement). Quantification of the contribution of fly ash in concrete has been under study for many years and a brief review of some of the important works is presented below. Ho & Lewis [2] have observed that the k value of fly ash w.r.t. 28-day compressive strength varies over a wide range depending on the amount of fly ash added, type of cement, incorporation of chemical admixture and the particular strength level chosen. Gopalan and Haque [3] have reported that the efficiency factor depends on the quantity of fly ash in the mix. Fraay et al [4] have reported that the reaction of fly ash in concrete is only initiated after one or more weeks and during this incubation period the fly ash behaves more or less as an inert material. Hence the efficiency values of fly ash can be very low or even negative at early ages. Bijen and Selst [5] have reported that the contribution of fly ash to concrete strength is strongly dependent on the water cement ratio, type of cement and fly ash and age of concrete. Ganesh Babu and Nageswara Rao, [6] have reported that the overall efficiency factor of fly ash (k) is the combination of general efficiency factor (k_e) depending on age and an additional percentage efficiency factor (k_p) depending on replacement percentage. It has been reported that the overall efficiency factor (k = k_e + k_p) varies from 1.25 to 0.35. The authors have inferred that the efficiency of fly ash increases with decrease in w/c whereas it decreases with increase in replacement percentages. Ganesh Babu and Nageswara Rao, [7] have reported that k value has been suggested as 0.25 for replacements up to 25%, German standards recommend a value of 0.3 for replacements between 10-25%, British Code refers to a value of 0.4 for replacements up to 25%, CEB-FIP model Code proposes a value of 0.4 for replacements between 10-25%. It has also been reported that for concrete with different types of fly ash and cement (up to 28% replacement and w/c between 0.5 and 0.65) a value of 0.5 is appropriate. It has been mentioned that cementing efficiency factor of fly ash depends on the physical and chemical characteristics of fly ash and cement, mix design parameters, strength range, age, w/cm and replacement level. Hanehara et al. [8] have reported that hydration of cement is accelerated with increase in the water-cement ratio. It has been mentioned that the pozzolanic reaction of fly ash
proceed from the age of 28 to 91 days and the reaction ratio of fly ash decreases with increase in the substitution rate. It is observed from the above reporting that the efficiency of fly ash in concrete is not a constant but depends on a number of parameters like type of cement & fly ash, replacement level, age, w/cm, strength level etc. where cm is the cementitious material content. Hence efficiency of fly ash should not be considered as an intrinsic or fundamental property of the material. Since the efficiency value is not a constant one evaluation of the same requires a considerable amount of judgment and understanding on the part of the designer. However, efficiency factors can be very useful for effective utilization/quantification of fly ash in concrete. The author have been working in this direction over the last few years and the present paper reports some of the results obtained in this direction. Nowadays guidelines or standards for mix proportioning of fly ash concrete are available in a host of countries including India. Though the literature is rich in reporting on cementing efficiency of fly ash there is still a dearth of guideline as how to judiciously choose the efficiency values while proportioning fly ash mixes. The present paper is directed in this direction.

**SIGNIFICANCE OF THE PRESENT RESEARCH WORK**

The aim of concrete mix design is to achieve the desired strength at a specified age satisfying the requirements of workability, durability and economy. For control concretes guidelines for mixture proportioning are readily available in the standards and specifications of different countries. Though the process of mix design incorporating fly ash involves more variables and is more complicated, guidelines are available nowadays and in the latest version of the Indian Standard on Concrete Mix Proportioning (IS:10262:2009) [9] a guideline in the form of a numerical example has been incorporated. Though mix design is still an empirical procedure and is mostly arrived at by adopting the method of trial mixes some inherent drawbacks regarding assessment of some parameters in the IS method have been identified and based on the present research work an attempt has been made to quantify the contribution of fly ash in terms of its efficiency factors and some simple tips are proposed. The aim is not to propose a new mixture proportioning method but to suggest some improvement in the existing IS method which may result in some value addition to the existing methodology.

**MATERIALS USED**

The constituent materials used in the program were tested to comply with the relevant Indian Standards. To assure uniformity of supply, the materials were subjected to periodical control tests. The cement used was Ordinary Portland Cement, having a 28-day compressive strength of 43 MPa with a Blaine’s fineness of 325 m$^2$/kg. Fly ash, containing 60.78% SiO$_2$ (out of which reactive silica by mass was 23.1%) and having a Blaine’s fineness of 250 m$^2$/kg, from a thermal power plant in West Bengal was used. Hence the reactivity or pozzolanic effect of the
fly ash was quite low and its filler effect insignificant. Natural river sand having a Fineness
Modulus of 2.94 was used. The specific gravity and water absorption values were obtained as 2.6
and 0.58% respectively. Crushed, angular, graded coarse aggregates of nominal maximum size
20 mm were used in the investigation. The specific gravity and the water absorption of the
aggregates were 2.87 and 0.9% respectively. Potable water was employed for the mixing [10].

EXPERIMENTAL PROCEDURE

The experimental program included five sets of concrete mixtures at w/cm ratios of 0.4,
0.45, 0.5, 0.55 and 0.6 prepared by partial replacement of cement by equal weights of fly ash.
Each set had mixtures at five fly ash replacement percentages - 0 (control mix), 20, 30, 40 and
50% of the total binder content. In order to evaluate the effect of fly ash incorporation at
different binder contents three total binder contents were selected – 300, 375 and 450 kg/m³.
Thus the scheme of experimentation spanned over 5 water-binder ratios, 5 fly ash replacement
percentages at each of the three binder contents [10, 11]. For the present investigation the
proportions of the coarse and fine aggregates were determined in the laboratory by trial & error
so that the resultant aggregate mix yielded maximum packing density. This proportion has been
kept constant for all the mixes. The volume of voids in the system was calculated as (1 - Packing
Density). The paste contents of all the mixes were determined & compared with the volume of
voids within the aggregate particles. In order to fill up the voids within the aggregate particles
(so as to drive off the air voids in the concrete mix) the volume of cement paste must be larger
than the volume of voids within the aggregate particles. This was verified by actual casting
whereby mixes having paste contents less than the volume of voids resulted in unworkable mixes
and hence they were rejected. The mixes at higher binder contents and with high values of water-
binder ratios were not stable rather they exhibited high bleeding and segregation and thus were
rejected. Thus a total number of 55 mixes which were cohesive and workable were only
investigated. The amount of coarse and fine aggregates used were 42% and 58 % by mass of
total aggregates and the amounts of the different ingredients were determined by using the
absolute volume formula [12]. Thereafter only cement was replaced by fly ash (by equal
weights). Due to change in the volumes of cement and fly ash necessary changes in the masses of
the ingredients were calculated using the absolute volume method keeping all other mix
proportion parameters constant. Since the water contents of the mixtures varied over a wide
range, workability of the mixtures also varied over a wide range. The workability of the mixes
were carefully observed & measured to ensure that all the mixes were workable. To minimize
variations in workability, the compaction energy was varied for obtaining proper compaction.
Cube specimens of 150 × 150 × 150 mm were used for compressive strength determination. All
the specimens were moist cured under water at room temperature until testing. Since the curing
condition was uniform for all the specimens no adverse effect on the subsequent test results are
RESULTS AND DISCUSSIONS

 Determination of Efficiency Factor Values of Fly Ash

The 28 day strengths of 55 concrete mixes have been determined. It has been observed that with increase in fly ash contents concrete strengths have been reduced [10, 11]. In order to determine the efficiency factors of fly ash, a simple methodology has been adopted [13]. As postulated by Abrams’, there exists a hyperbolic relation between strength and w/c. Hence the relationship between loge (compressive strength) and w/c should be a linear one. A typical relationship between loge (strength) at 28 days and w/c for control concrete at binder content of 375 kg/m³ is presented in Figure 1. Using the method of least squares the equation of the best fit line for this data set has been obtained as –

\[
\log_e (S) = 5.4164 - 3.142 \times (w/c) \quad (1)
\]

where S is the strength and w/c is the water-cement ratio of the control concrete. Using this equation and the loge (strength) of fly ash concrete mix at any replacement percentage at the corresponding binder content the value of the equivalent w/c can be calculated [13]. From the value of the equivalent w/c ratio, efficiency factor has been estimated by the following expression —

\[
(w/c)_{\text{Equivalent}} = \frac{w'}{c' + kf} \quad (2)
\]

where w', c' and f are the water, cement and fly ash contents (kg/m³) respectively in the fly ash concrete, k is a parameter known as cementing efficiency factor for the fly ash. The values of the efficiency factors for all the 44 fly ash concrete mixes have been calculated in a similar way and are presented in Table 1.

Mix proportioning of Fly ash Concretes using Efficiency Factors

Bharatkumar et al. [14] have reported that the strength and workability of fly ash concrete with effective w/b \([w/(c+bf)]\) is comparable to that of conventional concrete without fly ash having same w/b ratio and water content. Constant water content yields constant workability of fresh concrete irrespective of the w/b ratio. For achieving similar strength level for control and fly ash concrete the following expression, which is similar to Equation (2), may be considered —
where, $w_1$ and $c$ are the water content and cement content in kg/m$^3$ of control concrete; $w_2$, $c_1$ and $f$ refer to water content, cement content and fly ash contents in kg/m$^3$ of fly ash concrete and $k$ is the efficiency factor of fly ash with respect to compressive strength.

For obtaining the specified workability, water contents of the control as well as fly ash concrete should be the same. For this condition

$$w_1 = w_2 \quad \text{............................ (4)}$$

After considering that the water contents in both the mixes are same, the expression (3) takes the form —

$$(c)_{\text{control concrete}} = (c_1 + kf)_{\text{fly ash concrete}} \quad \text{........................................ (5)}$$

The above Equation relates the binder contents of control and fly ash concretes. If the $k$ values at different fly ash replacement levels are known, the total binder content of the fly ash concrete, which is equivalent to the cement content of the control concrete, can be determined. A comparatively poor quality of fly ash, coarser than cement and having a low value of reactive-silica content was selected for the present research work. This was done with the apprehension that even if these values are used for estimating the efficiency factors of fly ashes conforming to IS 3812 (Part 1):2003, which should have a Blaine’s fineness of 320 m$^2$/kg and reactive silica of 20% by mass, conservative values of cement equivalence factors will be obtained. Efficiency of fly ash is dependent on the replacement percentage and decreases with increase in the replacement percentage. Efficiency values beyond 30% replacements reduce almost drastically [13]. Instead of using the actual numerical values, some form of averaging has been performed to obtain more or less representative values of the efficiency factors. Hence the values up to and beyond 30% have been added and divided by the number of observations to obtain representative values of the efficiency factors and are presented in Table 2. The average values of 28-day efficiency factors of the fly ash used in the present work have been tabulated at different ranges of water-binder ratios, binder contents and fly ash replacement percentages. It is the primarily the quality of fly ash which will determine what should be the replacement level and how much increase is needed in the total cementitious material content. If some means of assessment of the quality of fly ash is available, reasonable assessment of these parameters can be achieved instead of arbitrarily choosing these values. Such guidelines can lead to effective intelligent assessment and thereby reduce the number of trial mixes necessary to arrive at the desired mix. It is neither expected nor claimed that the efficiency values obtained for the type of ash used in the present research can cater to all types of fly ashes under all concreting conditions. However, the values can help in finding a reasonably good first hand estimate of the cement equivalence factors of fly
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ash. These can be used to determine the replacement level and increase in total binder content needed instead of arbitrarily choosing them only on the basis of experience and skill of the designer. This will reduce the number of trial mixes needed to arrive at the desired mix. A numerical example explaining the principle of mixture proportioning of fly ash concrete using efficiency factors have been included in a recent publication by the author [15]. It has been reported that the pozzolanic reaction of fly ash proceed from 28 to 91 days [8]. Hence the efficiency factors of fly ash should change at the 90 day stage with respect to its 28 day values. The 90 day compressive strengths were also determined and the corresponding values of efficiency evaluated. Table 3 presents the 90 day efficiency values which are quite higher than the 28 day values indicating significantly higher effect of fly ash at later ages with respect to 28 days.

Concrete is a friend of the environment and plays a very important role in our ecological system. It has the unique power of acting as the home and locking within itself a host of waste materials like fly ash produced due to rapid industrialization which otherwise can lead to significant environmental pollution. The present results indicate that as high as fifty percent of cement can be replaced by fly ash with significant improvement in the properties of the resultant concrete thus reiterating the fact that concrete is a sustainable material.

CONCLUSIONS

Extensive experimentation was performed over a wide range of w/cm ratios ranging from 0.4 to 0.6, binder contents varying from 300 to 450 kg/m³ and fly ash replacement percentages from 20 to 50%. On the basis of the results of the present investigation a table featuring the representative values of efficiency factors at 28 days have been furnished. The proposed values of efficiency factors if used in conjunction with the IS method can lead to proper quantification of the ash used without merely relying on the experience and discretion of the designer. It may be expected that the findings of the present work may serve as a useful guideline for judiciously applying the concept of efficiency factors to optimize the effect of fly ash in concrete and lead to improvement in the IS method of mix design of fly ash concretes.

ACKNOWLEDGEMENTS

The author is grateful to the Head, Fly Ash Unit, Department of Science and Technology, Ministry of Science and Technology, Government of India, New Delhi, India for providing necessary financial assistance to carry out this elaborate experimental work. Sincere help offered by Dr Samaresh Pan, former DST Research Fellow, is gratefully acknowledged.

REFERENCES
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Figure 1: Typical relationship between log (compressive strength) of control concrete and water-cement ratio at binder content of 375 kg/m³
Table 1: Efficiency factors of Fly ash at 28 days

<table>
<thead>
<tr>
<th>w/cm</th>
<th>20 % FA Binder (Kg/m³)</th>
<th>30 % FA Binder (Kg/m³)</th>
<th>40 % FA Binder (Kg/m³)</th>
<th>50 % FA Binder (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>300</td>
<td>375</td>
<td>450</td>
<td>300</td>
</tr>
<tr>
<td>0.4</td>
<td>0.484</td>
<td>0.630</td>
<td>0.498</td>
<td>0.643</td>
</tr>
<tr>
<td>0.45</td>
<td>0.601</td>
<td>0.658</td>
<td>0.546</td>
<td>0.513</td>
</tr>
<tr>
<td>0.5</td>
<td>0.244</td>
<td>0.831</td>
<td>0.312</td>
<td>0.575</td>
</tr>
<tr>
<td>0.55</td>
<td>-0.092</td>
<td>0.508</td>
<td>0.064</td>
<td>0.510</td>
</tr>
<tr>
<td>0.6</td>
<td>0.275</td>
<td>0.532</td>
<td>0.007</td>
<td>0.617</td>
</tr>
</tbody>
</table>

Table 2: Average values of Efficiency factors

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Up to 30 % Fly ash</th>
<th>&gt; 30 % Fly ash</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Binder (Kg/m³)</td>
<td>Binder (Kg/m³)</td>
</tr>
<tr>
<td></td>
<td>w/cm (0.5-0.6)</td>
<td>w/cm (0.4-0.5)</td>
</tr>
<tr>
<td>300</td>
<td>0.14 (25 – 38)</td>
<td>0.10 (16 – 31)</td>
</tr>
<tr>
<td>375</td>
<td>0.57 (26 – 55)</td>
<td>0.47 (19 – 43)</td>
</tr>
<tr>
<td>450</td>
<td>0.61 (31 – 56)</td>
<td>0.48 (22 – 46)</td>
</tr>
</tbody>
</table>

**Compressive strength values based on 150 mm cubes (MPa) are given in brackets

Table 3 : Efficiency factors of Fly ash at 90 days

<table>
<thead>
<tr>
<th>w/cm</th>
<th>20 % FA Binder (Kg/m³)</th>
<th>30 % FA Binder (Kg/m³)</th>
<th>40 % FA Binder (Kg/m³)</th>
<th>50 % FA Binder (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>300</td>
<td>375</td>
<td>450</td>
<td>300</td>
</tr>
<tr>
<td>0.4</td>
<td>0.519</td>
<td>0.792</td>
<td>0.562</td>
<td>0.702</td>
</tr>
<tr>
<td>0.45</td>
<td>0.934</td>
<td>1.133</td>
<td>0.710</td>
<td>0.757</td>
</tr>
<tr>
<td>0.5</td>
<td>0.552</td>
<td>0.849</td>
<td>0.613</td>
<td>0.699</td>
</tr>
<tr>
<td>0.55</td>
<td>0.527</td>
<td>1.009</td>
<td>0.646</td>
<td>0.841</td>
</tr>
<tr>
<td>0.6</td>
<td>0.390</td>
<td>0.819</td>
<td>0.461</td>
<td>0.584</td>
</tr>
</tbody>
</table>
**FUNDAMENTAL STUDY ON THE PROPERTIES OF MORTAR USING GEHLENITE CLINKER AS A FINE AGGREGATE**

**ICCS16**

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**Key words:** Gehlenite clinker, Industrial waste, Fine aggregate, Self healing, Compressive strength, Drying shrinkage, Carbonation, Water leakage test

**Abstract.** In recent years, Japan’s cement industry has been using large amounts of waste as the raw material for the production of cement. Future demand for cement in Japan, however, is expected to decrease. Therefore, to effectively utilize this waste, new use for cement clinker must be developed. This study examined the possibility of using clinker as a fine aggregate. The results show that compared to mortar containing a conventional fine aggregate, mortar containing clinker has higher compressive strength, less drying shrinkage, and lower carbonation rate.

1 INTRODUCTION

Currently, Japan’s cement industry uses a large amount of industrial waste and by-products as the raw material for the production of cement. However, the volume of waste and by-products has been a constant, but domestic demand for cement has been decreasing gradually. Therefore, to effectively utilize this waste, new uses for cement clinker must be developed.

To improve the durability of concrete structures, it is necessary to repair cracks in their early stage. If concrete had the ability to repair cracks (self healing), the durability of concrete structures could be much improved. Concrete containing clinker as aggregate, may have this ability because the un-hydrated clinker provides new cement hydrates for the cracked surface.

This study examined the properties of mortar containing clinker as a fine aggregate in order to develop new and effective uses for cement clinker.
EXPERIMENTS

2.1 Materials

Ordinary Portland cement was used as a binder (symbol: C, density: 3.15 g/cm³). Natural river sand (S, density in saturated surface-dry condition: 2.62 g/cm³) and Belite - Gehlenite clinker (GCL, absolute dry density: 3.16 g/cm³) were used as a fine aggregate. The chemical and mineral compositions of Gehlenite clinker are shown in Tables 1 and 2. The Gehlenite clinker is produced using large amounts of recycled industrial waste, such as construction waste soil, sewage sludge, waste clay and coal ash. Raw material for sintering Gehlenite clinker was prepared by blending and pulverizing a virgin material (limestone, quartzite and so on) with the industrial waste to obtain a mineral composition as shown in Table 2. Then the raw material was fed into the actual rotary kiln of Taiheiyo cement group with a production rate at 2000 tons as clinker per day. The burning temperature was around 1200 °C.

0.6 ton of waste was used for 1 ton of Gehlenite clinker. This means that the amount of waste of the Gehlenite clinker is about 2 times as much as the one of an ordinary portland cement clinker commercially produced by Japanese cement plants. The main mineral of Gehlenite clinker is belite as shown in Table 2.

The particle size distributions of this fine aggregate were within the range defined in JIS A 5005 (Figure 1) [1]. Tap water (W) was used for mixing. Poly-carboxylic acid-based high-performance air entraining and water reducing admixture (SP) and an anti-foaming agent (DF) also were used.

<table>
<thead>
<tr>
<th>Table 1: Chemical compositions of Gehlenite</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Symbol</strong></td>
</tr>
<tr>
<td>GCL</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2: Mineral compositions of Gehlenite</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Symbol</strong></td>
</tr>
<tr>
<td>C₂S</td>
</tr>
<tr>
<td>GCL</td>
</tr>
</tbody>
</table>
2.2 Mix proportions

Table 3 shows the mix proportions for the mortar. The mortar flow value of 15 times tapping was adjusted within 150±30mm by controlling the amount of SP.

<table>
<thead>
<tr>
<th>Mix name</th>
<th>GCL/S* vol. (%)</th>
<th>W/C (%)</th>
<th>Air (%)</th>
<th>Unit content ($kg/m^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>W  C  S  GCL</td>
</tr>
<tr>
<td>GCL0%</td>
<td>0</td>
<td></td>
<td></td>
<td>1464 0</td>
</tr>
<tr>
<td>GCL25%</td>
<td>25</td>
<td>55</td>
<td>2±1.5</td>
<td>1098 442</td>
</tr>
<tr>
<td>GCL50%</td>
<td>50</td>
<td></td>
<td></td>
<td>732 883</td>
</tr>
<tr>
<td>GCL75%</td>
<td>75</td>
<td></td>
<td></td>
<td>366 1325</td>
</tr>
<tr>
<td>GCL100%</td>
<td>100</td>
<td></td>
<td></td>
<td>0 1766</td>
</tr>
</tbody>
</table>

*GCL/S: Volume ratio of Geheleite clinker fine aggregate to total fine aggregate

2.3 Mixing

An Omni type mixer with a nominal capacity of 10 liters was used. The mixing time was 3 minutes.
2.4 Evaluation method

2.4.1 Fresh properties test

A mortar flow test was carried out in accordance with JIS R 5201\(^2\) and air volume measurement was carried out in accordance with JIS A 1116\(^3\).

2.4.2 Compressive strength test

A compressive strength tests were carried out in accordance with JIS A 1108\(^4\). The strength was measured at 7 and 28 days after curing in water at 20 degrees. The test specimens were \(\phi 50 \text{ mm } \times \text{h}100 \text{ mm}\) in size and number of specimens was three for each condition.

2.4.3 Drying shrinkage test

A drying shrinkage tests were carried out in accordance with JIS A 1129\(^5\). The test specimens were prismatic test specimens \(40\times40\times160 \text{ mm}\) and number of specimens was three for each condition. Measurements were started after curing for 7 days in water at 20 \(^\circ\text{C}\). The drying conditions were temperature: 20 \(^\circ\text{C}\) and relative humidity: 60%.

2.4.4 Accelerated carbonation test

An accelerated carbonation test carried out in accordance with JIS A 1153\(^6\). The test specimens were prismatic test specimens \(40\times40\times160 \text{ mm}\) and number of specimens was two for each condition. Measurements were started after curing for 28 days in water at 20 \(^\circ\text{C}\) and curing for 28 days in air at 20 \(^\circ\text{C}\) and 60% relative humidity. The accelerated carbonation conditions were temperature: 20 \(^\circ\text{C}\), relative humidity: 60 % and carbon dioxide concentration: 5%.

2.4.5 Self-healing ability test

The testing equipment is shown in Figure 2. The test specimens were \(\phi 100 \text{ mm } \times \text{h}200 \text{ mm}\) in size and number of specimens was two for each condition. The specimens were cured in water at 20 \(^\circ\text{C}\) for 28 days. The specimens were then split into 2 pieces. To simulate cracks, teflon sheets measuring 5 mm \(\times\) 5 mm on a side \(\times\) 0.3 mm in thickness were used as spacers to create gaps of 0.3 mm. The spacers were inserted at the upper and lower ends of the cross-section and sandwiched tightly by stainless steel belts as shown in Figure 2.

A synthetic resin adhesive was used to prevent water from leaking down the sides of the specimens. A PVC tube with an inner diameter of 100 mm was placed on top of the specimen as shown in Figure 2. A drainage pipe was connected to maintain a water head height of 80 mm (water pressure about 0.8 kPa).

The water used in this test was a calcium hydroxide saturated solution with pH of 12.4 to prevent the elution of Ca\(^{2+}\) on the fracture surface of the mortar\(^7\).

After pouring the water into the tube, the amount of water leakage from cracks in the specimen was measured for 3 minutes. The measurement was carried out on setting day and on days 1, 3, 7, 14, 21 and 28 days.
During these days, the specimens were stored in a calcium hydroxide saturated aqueous solution at 20 °C. The rate of decrease in the amount of water leakage was calculated using equation (1).
Masatoshi Nemoto, Hiromi Fujiwara, Masanori Maruoka, Makoto Kobayakawa, Takemi Sugawara and Takeya Kawato

Water leakage reduction rate (%) = \left(1 - \frac{Water leakage reduction rate (%)}{Initial amount of water leakage (g)}\right)

(1)

The assumed crack width was calculated using formula (2)\(^8\).

\[ q_{\text{leak}} = \frac{C_w \times b \times \Delta P \times w^3}{12 \times \mu \times L} \]

(2)

- \(q_{\text{leak}}\): Water leakage from the cracks (mm\(^3\)/s)
- \(C_w\): Correction factor in consideration of the irregularities of the crack
- \(b\): Length of the crack (m)
- \(\Delta P\): Pressure difference (N/m\(^2\))
- \(\mu\): Viscosity coefficient of water (Ns/m\(^2\))
- \(L\): Waterway length (m)
- \(w\): Crack width (mm)

3 TEST RESULTS AND DISCUSSION

3.1 Fresh properties

Figure 3 shows the results of mortar flow test. Figure 4 shows the air content of the mortar. Figure 5 shows the addition rate for SP.

As the GCL replacement ratio increase, in order to obtain a mortar flow value within the target range, it is necessary to increase the volume of SP. When the GCL/S was 100%, it was impossible to obtain a value within the target range, although the volume of SP was 4%, the upper limit recommended by the manufacturer. Photo 1 shows the state of the mortar (GCL/S = 100%). This is due to the shape of the GCL. The fine aggregate GCL is manufactured by pulverizing the original clinker so that each piece has an angular shape that negatively affects the freshness properties.

![Figure 3: Test results for mortar flow (15 times tapping)](image)

Figure 3: Test results for mortar flow (15 times tapping)
Masatoshi Nemoto, Hiromi Fujiwara, Masanori Maruoka, Makoto Kobayakawa, Takemi Sugawara and Takeya Kawato

Figure 4: Air content ratio of mortar

Figure 5: Additional rate for SP

Photo 1: State after mixing (GCL/S=100%)
3.2 Compressive strength

Figure 6 shows results of the compressive strength test. As the replacement ratio of GCL increases, the compressive strength increases. This is due to the densification in transition zone at interface between cement paste and fine aggregate and firmness of GCL, but additional work is needed to confirm this.

![Figure 6: Compressive strength of mortar](image)

3.3 Drying shrinkage

Figure 7 shows results of drying shrinkage test. As the replacement ratio of GCL increases, it is confirmed that drying shrinkage strain showed decrease tendency. In this test, when the GCL/S was 25%, the drying shrinkage showed minimum value (line: GCL/S25%-1). To confirm this, drying shrinkage of it was tested again. The result is also shown in Figure 7 (line: GCL/S25%-2). The result was almost same.

It is also considered that this is due to the densification in transition zone at interface between cement paste and fine aggregate and firmness of GCL, and mixture promotion has some effect on drying shrinkage, but additional work is needed to confirm this, too.

![Figure 7: Drying shrinkage test](image)
3.4 Accelerated carbonation test

Figure 8 shows results of the accelerated carbonation test. As the replacement ratio of GCL increases, the carbonation rate decreases.

The calcium ion and hydroxide ion derived from GCL fine aggregate may delay the progress of carbonation. The densification in the transition zone also may have some effect.

![Figure 8: Accelerated carbonation test](image)

3.5 Self-healing ability

Figure 9 shows the relationship between the amount of water leakage and the age after the start of the test.

The figure shows a large difference in the initial amount of water leakage caused by a difference in the roughness of the specimen’s split surface. Figure 10 shows the relationship between the water leakage reduction rate and the age after the start of the test. The figure shows a similar trend for every replacement rate.

It is presumed that this self-healing ability is due to the unhydrated cement component in all of the mixes after curing for 28 days. Since all of the mixes have the same unit cement content, they show almost the same self-healing ability. In this test, the final reduction rate for all of the replacement conditions was approximately 85%. Measurements will continue with additional experiments using more aged specimens.

Figure 11 shows the calculated crack width. All replacement rates showed a similar trend. The calculated crack width after 91 days was about 0.2 mm or less, which smaller than the crack width that would require repair as recommended by the Japan Concrete Institute.
Figure 9: Relationship between amount of water leakage and age after start of test

Figure 10: Relationship between water leakage reduction rate and age after start of test
4 CONCLUSIONS

This study examined the properties of mortar containing Belite - Gehlenite clinker (GCL) as a fine aggregate. From the results obtained in these experiments, the following conclusions were reached:

- As the replacement ratio of GCL increases, the volume of SP must be increased to obtain suitable freshness properties.
- As the replacement ratio of GCL increases, the compressive strength of mortar increases.
- As the replacement ratio of GCL increases, drying shrinkage strain decreases.
- As the replacement ratio of GCL increases, carbonation rate of mortar decreases.
- The self-healing ability of mortar tested in this study showed a similar trend for every replacement ratio. After 91 days, the water leakage reduction rate was about 80% and the calculated crack width was reduced to about 0.2 mm or less.

ACKNOWLEDGMENT

This work was supported by JSPS KAKENHI Grant Number 26420435.

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HIGH PERFORMANCE SUSTAINABLE MORTARS

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Key words: cementitious material, grout, repair mortar, binder blend, fly ash quality, life cycle assessment

Abstract. Cement production is an energy-intensive process, responsible for a significant amount of the world’s carbon dioxide emissions. Mixtures with so-called supplementary cementitious materials (SCM) like pozzolanic materials (pulverized fly ash or natural pozzolanas), latent hydraulic products like ground granulated blast furnace slag and even inert materials like limestone are therefore used. Such a clinker replacement with SCM may be able to overcome partially the problem of energy consumption, however, the resulting binder often lacks in performance compared to ordinary Portland cement (OPC).

This paper presents two high performance mortars in which part of the cement was replaced by a suitable combination of SCM to create a sustainable mortar without compromising the performance. Characteristics, mechanical and physical properties are discussed. Additionally, the environmental performance is analyzed through a simplified life cycle assessment (LCA).

1 INTRODUCTION

The annual global cement production reached 4.18 million metric tonnes, as of 2014, and is still increasing, with major growth coming from countries and regions like China, India, Middle East and Northern Africa [1].

The main use of cement is in concrete and cementitious materials. These are, with respect to the volumes used, the most successful materials in the world. This success is based on the simplicity of handling of both raw materials and fresh mix, as well as due to the ability of transformation at ambient temperature to a solid in short time, with complex shape and good or excellent properties. Further advantages are the low material costs and the energy balance of the materials. Nevertheless, the environmental impact of cement and concrete, if compared to alternative construction materials, cannot be neglected due to the huge volumes produced. Currently the cement production accounts for some 5-8 % of CO₂ emissions. There are increasing pressure and activities worldwide to innovate and increase sustainability of cementitious materials, concrete and constructions [2].

A possibility to improve the sustainability performance is the use of pozzolanic by-products either as additive in concrete and mortars for engineering, ecological or economic benefits, or as supplementary cementitious material (SCM) in production of
blended Portland cement. It is also a remedy to reduce the carbon footprint and accomplish substantial energy and cost savings due to reduction in clinker amount as it is partially replaced by supplementary cementitious materials (SCM). Fly ash, slag, silica fume and natural pozzolanas are well-known SCM. However, one disadvantage which is not resolved yet is related to the low early strength of blended cements, which also limits the level of substitution by SCM [3]. Many approaches, such as chemical activation [4] and ultrafine grinding [5], curing at elevated temperature [6] and addition of very fine lime [7], have been investigated to improve the early properties of blended cementitious materials. Most of these methods focus on increasing and improving the pozzolanic activity of the SCM, which is accompanied by difficulties in handling and/or cost increase.

Today, the incorporation of fly ashes of both types (i.e., with low and high calcium content) in cementitious systems for constructions is common practice. But the fly ash quantities produced worldwide are increasing and exceed the amount used. The non-used amount of fly ash causes environmental problems such as land, air and groundwater pollution [8]. In order to increase the usage of this by-product, it is necessary to investigate and understand its dynamic and pozzolanic potential, but also to overcome its shortcomings like slow reaction and fluctuation of quality.

The present work discusses the development and evaluation of high performance sustainable mortars using SCM. It shows that with appropriate raw materials it is possible to formulate sustainable dry mortar products without compromising the performance and thereby fulfill completely the requirements for high performance cementitious mortars.

2 EXPERIMENTAL PROCEDURE

Materials

The present study uses ordinary Portland cement (CEMI 52.5 R according to EN 197-1). Different fly ashes (all Class F according to ASTM classification) were used in order to blend with the cement to obtain a sustainable dry mortar. Different fractions of quartz sand with optimized grading curve as well as different additives, like superplasticizers and stabilizers, complete the formulations.

Test methods

The blended cement formulations were tested regarding fresh mortar properties (workability in time, flow, fresh density and air) as well as hardened mortar properties (mechanical: compressive and flexural strength and physical: shrinkage and expansion). The fresh properties were determined according to EN 1015-3 (without/with strokes) at 4, 10, 20, 30, and 45 (60) minutes. For selected mixes also Flow Cone (according to ASTM C939) and Flow Channel (according to EN 13395-2) were used.

The dry materials were mixed during 3 minutes in a Hobart-mixer before they were cast in 4 cm x 4 cm x 16 cm prism moulds. The testing of mechanical properties was done according to EN 12190-1 and of the physical properties according to EN 12617-4.

For the finally selected formulation, additional tests like adhesion (EN1542), E-modulus (EN 13412), anchoring by pull-out (EN 1881), freeze thaw de-icing salt test (EN 13687-1), carbonation (EN 13295), hydration kinetics, Baenziger Block (internal method
for evaluation of cracking tendency) and effective bearing area (grouts top surface / plate contact area) were performed.

3 RESULTS AND DISCUSSION

In this part, results of evaluation of suitable SCM and of development of CO₂ reduced mortars like grout and repair mortar will be discussed.

3.1 Evaluation of suitable SCM

A first approach towards developing high performance sustainable mortars was done regarding evaluation of different types of SCM. The cementitious binder was blended with slag or fly ash in amounts of 10, 15, 20, 28 wt% with respect to cement. The fresh properties (flow) over time and the mechanical properties (compressive strength) at 1, 7 and 28 days as well as physical properties (shrinkage) until 91 days were measured and are discussed in [9].

In this paper only the evaluation of fly ash will be discussed. The used fly ash amount was 10, 15, 20 and 28 wt% to cement. Fig. 1 presents the measured flow after 4, 10, 20, 30 and 45 minutes after mixing for fly ash blends. The reference sample is without SCM. These results show the positive contribution of fly ash on rheology and flow over time. This can be explained with the spherical particle shape of fly ash and a beneficial particle packing of cement and fly ash [10].

![Figure 1: Flow of fly ash blended cement grout.](image)

The mechanical performance was studied. The influence of cement substitution by fly ash on compressive strength was tested after 1, 7 and 28 days. Fig. 2 shows the negative effect of fly ash on the mechanical performance due to a slower reactivity of this pozzolan as previously reported [2-4]. Increasing the amount of fly ash decreases the compressive strength after 1, 7 and partly also 28 days. With a suitable activation of fly ash it may be possible to counteract this effect.
The influence of fly ash addition on the shrinkage properties was tested until 91 days as shown in Fig. 3. Obviously, the substitution of cement by fly ash has a positive influence on shrinkage behaviour, as a higher amount of fly ash causes a reduction of shrinkage [11].

These results show that the selected SCM (fly ash) is able to influence positively the workability and flow properties. The long pot life of the grout remains intact. The previously known shortcomings of SCM regarding early strength and strength development were also found in our investigations. A positive influence of SCM addition was detected for shrinkage: an improvement of shrinkage by about 15% was obtained (at 28 days). Therefore, it can be concluded that the selected fly ash is suitable for developing sustainable, CO2 reduced grout as well as other mortars.
3.2 Development of a CO2 reduced mortar

Based on the reported SCM evaluation results and in line with our requirements of the new product, it was decided to develop a blended cement grout and repair mortar with fly ash at the evaluated highest possible amount. In comparison to the formulation used in the previous evaluation, small adaptations of the formulation were made for this development.

High performance sustainable grout

The resulting flow is shown in Fig. 4. The new grout has a very stable flow without sedimentation, with a pot life of at least 60 minutes. For this grout, the flow properties acc. to the Flow Cone (according to ASTM C939) and the Flow Channel were determined. The ASTM cone value of this grout is below 50 sec. and the flow channel has value of 620 mm.

![Figure 4: Flow of the blended cement grout.](image)

The mechanical performance of this grout is shown in Fig. 5. The compressive strength after 1 day is 36 MPa, after 28 days it is larger than 80 MPa. It must the noted that this adapted formulation does not need any activator for the development of the mechanical properties, mainly in view of the good early compressive strength at 1 day. The flexural strength after 1 day is 7 MPa and after 28 days it is 12.5 MPa. Additional mechanical parameters were determined: adhesive strength of 3.3 MPa, E-modulus of 37.4 GPa. Further, durability tests such as freeze taw de-icing salt resistance and carbonation resistance were tested. The values of freeze taw and de-icing salt are around 3.4 MPa and the carbonation test was passed.

Measured physical properties, shrinkage (at 65 % r.H) and swelling (under water), were tested for up to 91 days and are shown in Fig. 6. The shrinkage values are below 0.6‰ at 28 days and 0.69‰ at 91 days. Our blended grout also shows low expansion under water until 91 days.

Tests regarding cracking tendency were done using an internally established method (Baenziger block) using large concrete blocks of 6 cm and 12 cm thickness, respectively. This test gives very good indication on large-scale cracking behaviour of cementitious
materials. Samples made from the new blended cement grout do not show cracks, even after long storage period outside. Thus, the new grout is suitable for applications involving large thickness without danger of cracking.

![Figure 5: Mechanical performance of the blended cement grout.](image)

**Figure 5:** Mechanical performance of the blended cement grout.

![Figure 6: Shrinkage and expansion of the blended cement grout.](image)

**Figure 6:** Shrinkage and expansion of the blended cement grout.

**High performance sustainable repair mortar**

Similar principle of the cement replacement was used for development of high-performance sustainable mortar. The resulting flow is shown in Fig. 7. The new repair mortar has a stable, soft and creamy consistency with a pot life of at least 60 minutes.

The mechanical performance of this mortar is shown in Fig. 8. The compressive strength after 1 day is 23 MPa, after 28 days it is larger than 60 MPa.

It must the noted that this adapted formulation also needs an activator for the development of the mechanical properties, mainly in view of the good early compressive
strength at 1 day. The flexural strength after 1 day is 5.3 MPa and after 28 days it is 9.1 MPa. Additional mechanical parameters were determined: adhesive strength of 4.1 MPa, E-modulus of 30.6 GPa. Further, durability tests such as freeze thaw de-icing salt resistance and carbonation resistance were also tested.

Figure 7: Flow of the blended cement repair mortar.

Figure 8: Mechanical performance of the blended cement repair mortar.

Measured physical properties, shrinkage (at 65 % r.H) and swelling (under water), were tested for up to 91 days and are shown in Fig. 9. The shrinkage values are below 0.5‰ at 28 days and 0.6‰ at 91 days. Our blended repair mortar also shows low expansion under water until 91 days.

Tests regarding cracking tendency were done using an internally established method (Baenziger block) using large concrete blocks of 6 cm and 12 cm thickness, respectively. Samples made from the new blended cement repair mortar do not show cracks, even after
long storage period. This new repair mortar is also suitable for applications involving large thickness without danger of cracking.

\[\text{Figure 9: Shrinkage and expansion of the blended cement repair mortar.}\]

4 LIFE CYCLE ASSESSMENT (LCA)

Life Cycle Assessment (LCA) is a standardized method [12] to assess and compare the inputs, outputs and potential environmental impacts of products and services over their life cycle. The comparison between the cement based mortars versus the mortars with 28% fly ash was made through a cradle to gate LCA for 1 kg mortar, as they have a similar performance. This means, it investigates the potential environmental impact of a product from raw material acquisition and processing to manufacturing and packaging. However, due to the fact that both mortars have a similar performance, and the production process and packaging are the same, the comparison focuses solely on the raw materials of the products.

The LCA was performed internally. First, an inventory analysis step was necessary, i.e., data collection and calculation procedures to quantify relevant inputs and outputs of the product system. The LCA was modelled using the GaBi 6 Software [13]. Product data collection was carried out by the manufacturer, including formulations of the products and transportation. The products were then modelled based on collected data and on datasets from commercial databases such as think step [13] and ecoinvent [14].

The impact assessment phase associates inventory data with environmental impact categories (calculated with the CML 2001 method). There are several impact categories which can be determined, however, for the scope of this paper the analysis focuses on the following three, which depict some of the main sustainability drivers in construction: carbon footprint, resource (and energy) efficiency, and pollutant emissions (also related to health and safety of the workers).

- Global Warming Potential (GWP 100 years): potential contribution to climate change due to greenhouse gases emissions. It is measured in kg (or ton) of carbon dioxide (CO2) equivalents for a 100-year time horizon.
• Cumulative Energy Demand (CED): total input of primary energy from renewable and non-renewable resources. It is measured in MJ.
• Photochemical Ozone Creation Potential (POCP): potential contribution to summer smog, related to ozone induced by sunlight on volatile organic compounds (VOC) and nitrous oxides (NOx). It is measured in kg of ethene equivalents (also known as ethylene).

**Allocation of impacts to Supplementary Cementitious Materials (SCM)**

When dealing with systems involving multiple products and recycling processes, allocation should be avoided as far as possible. Following the EN 15804 [15], when allocation cannot be avoided, inputs should be partitioned based on economic values when the difference in revenue from the co-products is high (more than 25%), which is mostly the case for SCM.

Policies regarding waste allocation are not globally consistent and there are agreements between industries on the allocation of impacts to co-products or wastes. Especially for SCM used in the construction industry, there is ongoing debate.

According to [16], in Europe Fly Ash and Granulated Blast Furnace Slag (GBFS) can no longer be considered as waste due to the European Union Directive 2008/98/EC, which states that a substance may be regarded as not being waste but as a by-product product if the following applies:

a) Further use is certain;
b) Can be used directly without any further processing other than normal industrial practice;
c) It is produced as an integral part of a production process; and
d) Further use is lawful.

Therefore, two scenarios were evaluated for the mortars regarding the fly ash impact allocation, to evaluate the influence in the results:
• Fly Ash as waste, burden-free, with only average transportation included.
• Fly Ash as co-product, using a dataset from think step, with economic allocation from the coal burning impacts.

4.1 LCA Results

The Fig. 10 and Fig. 11 below show the results for the three impact categories for both grout and repair mortar and both scenarios (fly ash as waste and as co-product).

![Figure 10:](image-url) Relative environmental impacts (%) for 1 kg formulation, considering fly ash as a co-product.
4.2 Discussion of LCA results

The results show that the binders are the main contributors to the impacts and that replacing 28% of the cement content with an SCM, while maintaining the performance of the mortars reduces the potential environmental impacts of the product. It also contributes to resource efficiency, as it replaces a virgin material (cement) with a waste from another process. In direct comparison to an equivalent performing product, reducing the amount of Portland cement with fly ash by 28% has demonstrated the following benefits:

- ~12% for grout and 0% for repair mortar reduction in primary energy demand (CED)
- ~17% for grout and 19% for repair mortar reduction in carbon dioxide emissions (GWP)
- ~16% for grout and 17% for repair mortar reduction in summer smog (POCP)

If the SCM is regarded as a waste, with no impacts allocated to it, the gains are of around 20% for the three impact categories for both types of materials, with exception of CED impact for repair mortar.

5 CONCLUSIONS

This contribution shows that the incorporation of fly ash into the cementitious binder of a grout and repair mortar leads to a mortar combining high performance characteristics and a reduced carbon footprint. The new mortars exhibit very good workability and long pot life, as well as excellent mechanical and physical properties. Early strength for grout of 37 MPa and more than 80 MPa, after 1 day and 28 days, respectively, is obtained without any additional activation. The early strength of the repair mortar of 23 MPa can be reached by using of suitable activator. The choice of the appropriate raw material together with an optimized formulation leads to blended cementitious products of highest quality.

As shown in the LCA analysis, in comparison to an equivalently performing mortar, the substitution of Portland cement by fly ash decreases the mortar’s potential environmental impacts (cradle to gate), namely carbon footprint, energy footprint and summer smog potential. It also contributes to resource efficiency, as it replaces a virgin material (cement) with a waste from other processes. The study presented in this paper shows that mortars with high performance characteristics and significantly reduced environmental impact is possible, and in the future, such products will increasingly come onto the market.
REFERENCES


IMPACT OF ALUMINATES ON SILICATES HYDRATION

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Abstract. Replacement of clinker by supplementary cementitious materials (SCMs) is one of the most promising routes to decrease the carbon footprint and embodied energy of Portland cement. However, increasing amounts of alumina-rich SCMs leads to a delay of cement hydration and a decrease of early mechanical strength. In this study, the passivation induced by aluminates on silicates hydration was investigated at the molecular level using Nuclear Magnetic Resonance (NMR). “In situ” $^{27}$Al MAS NMR experiments allowed quantifying tricalcium silicate (C$_3$S) hydration in presence of different concentration of aluminates. Since the natural abundance of $^{27}$Al is practically 100%, the NMR sensitivity is sufficient to realize this characterization in-situ with the necessary time-resolution. The advantage of this approach is that the sample is not dried thus insuring that the C-S-H structure is not perturbed. This study was complemented with isothermal calorimetry tests. The effect of specific surface area (SSA) of C$_3$S on its hydration kinetics was also evaluated. Results have shown that the delay of silicates hydration increases with the increase of aluminates concentration and the most pronounced effect is observed when coarser C$_3$S is used. Furthermore, the presence of aluminates in solution promotes the formation of AFm phases that are converted into another aluminate phase at later time of hydration.

1 INTRODUCTION

Portland cement is the most widely used manufactured material and its production contributes to 5 – 8 % of the worldwide CO$_2$ emissions thus playing a significant role in the climate change [1]. There are several strategies to minimize this effect, and clinker substitution is currently the most effective one. The use of aluminium-rich supplementary
cementitious materials (SCMs) such as slag, fly ashes as clinker replacement reduces the embodied emissions in cement; however, blended cements react slower than Portland cement compromising the early mechanical strength of cement and concrete [2]. To create the binder of tomorrow, fundamental knowledge has to be obtained with respect to the possible interactions between the main mineral components of blended cements. In particular, the possible delay of silicates hydration induced by the aluminates [3].

The passivating effect of aluminum ions on the dissolution rate of silicates has already been investigated to some extent in geochemistry [4-8] and the following possible mechanisms were proposed:

(a) formation of an insoluble silicates layer on the surface of silica due to the presence of aluminum ions [5]
(b) incorporation of aluminum into the silica framework [6,8] and
(c) adsorption of aluminate ions on silanol sites [7].

In the case of cementitious systems, the effect of aluminate ions has been also investigated [9-11]. Nicoleau et al. [9] concluded that under particular pH and Ca\(^{2+}\) concentration tricalcium silicate (C\(_3\)S) hydration is inhibited due to covalent bond formation between the aluminate ions and the silicate surface. In addition, Suraneni and Flatt [10] suggested that aluminate ions may act inhibiting step retreat rather than the opening of etch pits. Quennoz and Scrivener [11] discussed poisoning effect produced by aluminates acting mainly during nucleation and growth of C-S-H. Moreover, incorporation of Al into the C-S-H has been researched by Nuclear Magnetic Resonance (NMR) [12-19], that proved the formation of species containing pentahedral and octahedral aluminium. However, the passivation mechanism induced by aluminium still remains unclear.

The main objective of this work is to understand the effect of aluminates on the hydration of the main cement phase – C\(_3\)S. For this reason, C\(_3\)S hydration in presence of aluminates has been followed by isothermal calorimetry as well as by \(^{29}\)Si and \(^{27}\)Al solid-state Nuclear Magnetic Resonance (NMR) techniques.

2 EXPERIMENTAL

2.1 Materials

Pure triclinic C\(_3\)S was synthesized from stoichiometric quantities of CaCO\(_3\) and SiO\(_2\) (>99% pure, Sigma Aldrich). 10 g pellets pressed under a 350 bar load were fired in a platinum crucible at 1600 °C for 8 h and immediately quenched under a flow of compressed air to stabilize C\(_3\)S [20]. The final material was ground in a micronizing mill (McCrone) with agate grinding elements in order to achieve 3.6, 1.4 or 0.8 m\(^2\)/g BET specific surface areas (SSA), measured after degassing the powder at 200 °C during 1h. The synthesized C\(_3\)S was characterized by XRD and NMR. While the XRD and \(^{29}\)Si MAS NMR results were in line with what is expected for pure triclinic C\(_3\)S, \(^{27}\)Al magic angle spinning (MAS) NMR revealed the presence of approximately 0.1 weight % of Al coming from the SiO\(_2\) used as a precursor for the synthesis.

NaAlO\(_2\) solutions of different concentrations, 3, 30 and 60 mmol/l (pH 10.8, 11.8 and 12.1, respectively) were prepared dissolving defined quantities of solid NaAlO\(_2\) (>95% pure, VWR)
in ultrapure water (Milli-Q, Millipore). To adjust pH solid NaOH (>99% pure, Merck KGaA) was used.

2.2 Experimental methods

Pastes for isothermal calorimetry measurements were prepared by mixing 1 g of C₃S and 0.8 g of prepared solution in a plastic container during 3 min using a vortex mixer (Analog, VWR) at 2500 rpm. Experiments were performed at 23 °C using TAM Air microcalorimeter. The degree of hydration was estimated by dividing the cumulated heat released at a certain time by the enthalpy of total C₃S reaction determined as -520 J/g C₃S [21,22]. The time corresponding to the end of induction period was defined estimating the point after which deviation from the minimum heat flow takes place with subsequent increase in the heat flow.

Reproducibility of the data obtained during calorimetric experiments was proved by repeating randomly selected experiments for different C₃S and NaAlO₂ concentrations. Obtained data were found to be in a good agreement.

Hydration of C₃S samples with a SSA BET of 1.4 m²/g and 3.6 m²/g was first studied by NMR after stopping reaction at different times. C₃S with SSA BET of 1.4 m²/g was mixed 3 mM NaAlO₂ solution and the hydration was stopped at 1 h, 8 h, 1 day, 3 days and 7 days of curing by mixing during 5 minutes the pastes with iso-propanol (IPA) in 1 : 25 (C₃S : IPA) ratio. Afterwards, the powdered samples were filtered using a nylon 0.45 µm membrane filter and dried in a desiccator over silica gel until constant weight.

For in-situ NMR measurements (without stopping hydration), C₃S with SSA BET of 3.6 m²/g was mixed with 3 or 30 mM NaAlO₂ solution. The paste was then introduced into the NMR zirconia rotor using a syringe and a needle.

²⁷Al and ²⁹Si MAS NMR experiments were performed with 4-mm ZrO₂ rotors at spinning frequencies of 15 kHz and 7 kHz, respectively using a Bruker Avance-500 spectrometer (magnetic field of 11.7 T). Chemical shifts were referenced to 1 M aqueous solution of Al(NO₃)₃ for ²⁷Al or to tetrakis(trimethylsilyl)silane (TMS) for ²⁹Si with the accuracy of ± 0.1 ppm. The single-pulse ²⁷Al MAS NMR spectra were acquired with a π/6 pulse length of 2 µs, a recycle delay of 1 s and typically 3200 scans for in situ and 8000 scans for the measurements when hydration was stopped. The single-pulse ²⁹Si MAS NMR spectra were acquired with π/2 pulses of 5.9 µs, a recycle delay of 100 s and 560 number of scans. ¹H – ²⁹Si cross-polarization (CPMAS) spectra were acquired at 7 kHz spinning speed with a contact times of 5 ms and recycle delays of 10 s. A typical numbers of scans were 5016. Hartmann-Hahn matching was ensured by a ramp on the ²⁹Si rf field.

Based on the obtained one pulse ²⁷Al NMR spectra, the amount of the products was estimated comparing the integrated signal intensities. Quantities of different species containing Al were calculated using the program DMFit 2011 [23]. To fit the signals, Gaussian and Czjzek functions were used with fixed parameters varying only amplitudes through the whole set of experiments. In addition, a constant artifact signal of the probe was taken into account by introducing an extra broad peak with fixed parameters [16].
3 RESULTS

3.1 Effect of Al studied by calorimetry

Figures 1 and 2 show the heat flow and cumulative heat curves of C₃S hydration in presence of different concentrations of NaAlO₂ solutions.

Figure 1 shows that the length of the induction period increases, and that the appearance of the main hydration peak is retarded, with the increase in the concentration of NaAlO₂. Furthermore, while the increase of the induction period is about the same regardless the SSA of the used C₃S (Figure 3a), the delay induced by the aluminium on the time of appearance of the main hydration peak is more pronounced as the surface area of the C₃S decreases (Figure 3c), mainly when 60 mM NaAlO₂ is used. In addition, there is a linear trend between the time corresponding to the delay of the main hydration peak and the concentration of NaAlO₂ solution when added to C₃S with SSA of 1.4 m²/g but no clear one when mixed with finer or coarser powders (Figure 3c).

The addition of NaAlO₂ decreases the cumulative heat during the first 10 - 11 hours of hydration (see Figure 2). This depends on the concentration of NaAlO₂, but not on the surface area of starting C₃S. After approximately 11 hours there is an increase in the amount of heat released in the presence of NaAlO₂. Figures 2 and 3d-e show that this depends on the concentration of NaAlO₂ added and on SSA of the used C₃S. For instance, for C₃S with SSA equal 3.6 m²/g cumulated heat is equally enhanced by 30 and 60 mM NaAlO₂ solution, whereas for C₃S with SSA 0.8 m²/g this increase is only produced by 60 mM. Despite this dramatic increase in the cumulated heat after 15 hours of reaction in the presence of NaAlO₂, by 7 days of hydration this difference is minimized, and even almost disappears, in the case of hydration of finer C₃S (Figure 2, Figure 3e).

It is important to mention that the NaAlO₂ solutions used in the described experiments have different pH values. To distinguish if the observed effects on the calorimetry curves were induced by the presence of aluminate ions and not by the initial pH difference of the starting solutions, a set of additional experiments was performed. During those experiments the pH of the starting solutions containing 0 and 3 mM NaAlO₂ was adjusted with NaOH to 11.8, value corresponding to the pH of the 30 mM NaAlO₂ solution. Figure 4 confirms that this increase of pH has no detectable influence on C₃S hydration in absence and presence of NaAlO₂.
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**Figure 1**: Heat rate during hydration of C$_3$S with different NaAlO$_2$ concentrations. SSA of C$_3$S: 0.8 m$^2$/g (left), 1.4 m$^2$/g (middle) and 3.6 m$^2$/g (right).

**Figure 2**: Cumulated heat released during hydration of C$_3$S with different NaAlO$_2$ concentrations. SSA of C$_3$S: 0.8 m$^2$/g (left), 1.4 m$^2$/g (middle) and 3.6 m$^2$/g (right).
Figure 3: Summary of the calorimetric data pointing out a). Increase in induction period, b). Heat rate of the main hydration peak c). Increase in time corresponding to the end of acceleration period d). Increase in cumulated heat at 1 day e). Increase in cumulated heat at 7 days due to NaAlO$_2$ addition for C$_3$S with different surface areas as a function of NaAlO$_2$ concentration.

Figure 4: Effect of pH of the starting solution on the hydration rate of C$_3$S.

3.2 Effect of Al studied by NMR

The nature of the products that are formed during hydration of C$_3$S with NaAlO$_2$ solution was followed by $^{27}$Al MAS NMR. Since the natural abundance of this element is almost 100% no additional enrichment was necessary. Figure 5 shows NMR measurements obtained on the samples hydrated with 3 mM solution up to 7 days, and where hydration was
previously stopped. The spectra obtained on the non-hydrated C\textsubscript{3}S shows a resonance in the area of 4-fold Al with the center of gravity at 82.2 ppm corresponding to the presence of Al in the initial C\textsubscript{3}S and a broad resonance in the area of 6-fold Al coming from the used probe. During the first 8 h of hydration a small resonance with the center of gravity at 9.8 ppm is visible. As the hydration proceeds, the intensity of the peak at 82.2 ppm decreases and 5 additional signals appear, one in the area of 4-fold Al (69.8 ppm), two for 5-fold Al (35.8 and 20 ppm) and two for 6-fold Al (9.8 and 5 ppm). After 1 day of hydration no signal at 9.8 ppm is observed.

**Figure 5**: \textsuperscript{27}Al one pulse NMR data of the C\textsubscript{3}S hydrated with NaAlO\textsubscript{2} (c = 3 mM) between 1 h and 7 d. Hydration was stopped

\textsuperscript{29}Si one pulse and \textsuperscript{1}H – \textsuperscript{29}Si CP NMR show progress of C\textsubscript{3}S hydration with 3 mM NaAlO\textsubscript{2} up to 7 days (Figure 6). The spectra obtained confirm the formation of silicate dimers Q\textsuperscript{1}, longer chains containing Q\textsuperscript{2} species and the consumption of non-hydrated C\textsubscript{3}S (Q\textsuperscript{0}) over time. In addition, by \textsuperscript{1}H – \textsuperscript{29}Si CP NMR the presence of hydroxylated (Q\textsuperscript{0h}) species was established. Surprisingly no clear resonances attributed to Al in C-S-H (Q\textsuperscript{2}(1Al)) can be distinguished in either experiment. This resonance has been observed in the literature before between – 80 and – 82 ppm [13,15,17] but in our case the Al concentrations are most likely too low to see pronounced effect in the silicon spectra.
In-situ NMR measurements, where hydration was not stopped with isopropanol, were also performed (Figure 7). C₃S of 3.6 m²/g SSA was hydrated with 3 and 30 mM NaAlO₂ solution during 6 hours. The reaction was followed by $^{27}$Al MAS NMR. Comparing the data obtained in in-situ experiments with the data from the samples when hydration was stopped, both showed the same resonances for 4, 5 and 6-fold Al. Quantitative analyses of the obtained spectra confirms that as hydration takes place the amount of Al in C₃S decreased. In addition, the resonance at 9.8 ppm was clearly observed during hydration in the 30 mM NaAlO₂ solution already after 1 h 30 min of reaction but it disappeared as the 5 ppm and 69.8 ppm resonances started to grow. The quantitative analysis summarized in Figure 7 shows that the rate of formation of the phase at 5 ppm was higher when a 30 mM NaAlO₂ solution is added than in the case of a 3 mM one. Concerning the 5-fold Al species as they are formed in negligibly small amounts regardless of the NaAlO₂ concentration that was used, they were not included in the quantitative analyses.

Figure 6: $^{29}$Si one pulse (left) and $^1$H – $^{29}$Si CP (right) NMR data of the C₃S hydrated with NaAlO₂ (c = 3 mM) between 8 h and 7 d. Hydration was stopped. $Q_n$ refers to silicon atoms that are covalently bonded via bridging oxygen atoms to $0 \leq n \leq 2$ other silicon atoms [24]
Figure 7: Real time measurements by $^{27}$Al one pulse NMR using C$_3$S hydrated in a) 3 mM, b) 30 mM NaAlO$_2$ solution and quantitative analyses of the hydration products

4 DISCUSSION

By comparison to previous studies, in this paper we systematically studied the impact of NaAlO$_2$ on the C$_3$S hydration by varying the concentration of added aluminate ions and SSA of the used C$_3$S. In the presence of aluminates, C$_3$S hydration is retarded, being higher the delay with the increase of aluminium concentration in solution. Figure 4 proved that the increase of the initial pH when adding NaAlO$_2$ (up to pH = 12) does not have any impact on the hydration kinetics of C$_3$S. Furthermore, the SSA of the silicates does not seem to play a role on the extension of the induction period induced by the aluminates but clearly influences the position of the main hydration peak. The role of the SSA on the delaying effect of NaAlO$_2$ is in particular clear at concentrations of 60 mM NaAlO$_2$, where the retardation increases with the decrease of the SSA. This could suggest and support the hypothesis that aluminate ions probably act through a surface mechanism [9,10], by adsorption or forming covalent bonds on active kink sites, decreasing their reactivity.

Isothermal calorimetry curves (Figure 2) confirms that after the initial retardation of C$_3$S hydration, the presence of the aluminates increases the heat released and consequently the degree of reaction after 11 h of hydration. This increase was additionally proved by $^{29}$Si MAS
NMR (Figure 8) on C₃S hydrated for 1 day with NaAlO₂ 60 mM. In particular, ²⁹Si MAS NMR confirms higher reactivity of C₃S and consequently higher amount of C-S-H formed in the presence of NaAlO₂ compared to the sample that was just prepared with water.

![Figure 8](image)

**Figure 8**: Right: Degree of C₃S hydration with and without NaAlO₂ (c = 60 mM) addition studied by isothermal calorimetry (lines) and ²⁹Si one pulse NMR (squares) Left: ²⁹Si one pulse NMR spectra used to calculate degree of hydration

Based on the first results discussed in this study, NMR measurements performed in real time seem to be especially interesting as they prevent the use of any drying technique normally used to stop hydration. Based on previous research it is expected that during these experiments an acceleration of the C₃S hydration takes place, possible due to the increase of the temperature induced by frictional heating during spinning in the in-situ measurements [25]. Nevertheless, if results are compared with those obtained on the samples when hydration was stopped it can be seen that this type of experiments provide similar information on the nature of the products that are formed but continuously as hydration proceeds thus additionally minimizing experimental time.

Based on what has been previously reported in the literature most likely the resonance that are observed by ²⁷Al MAS NMR during C₃S hydration with NaAlO₂ are the following: at 69.8 ppm - Al in C-S-H, at around 35 ppm - Al³⁺ substituting Ca²⁺ in the interlayer of C-S-H, 9.8 ppm – AFm and 5 ppm – third aluminate hydrate (TAH) [12-19]. Nevertheless, additional techniques have to be used to confirm the formation of the mentioned phases. This is not an easy task as its concentration might be very low thus below the detection limit of many techniques. For example, this could explain the absence of resonance around -81.5 ppm in the ²⁹Si one pulse NMR spectra (Figure 6 and 8) attributed to Al in C-S-H (Q₂(1Al)).

## 5 CONCLUSIONS

- The presence of aluminates increases the induction period regardless of the SSA of the used C₃S. Furthermore, it also retards the main hydration peak; this delay is more pronounced with the decrease of the SSA.
- After 11 hours of C₃S hydration, the addition of NaAlO₂ promotes the increase of the degree of hydration of the silicates.
Al one pulse NMR experiments performed in real time and on the samples after hydration was stopped revealing similar products formed due to addition of NaAlO$_2$. Increased amount of NaAlO$_2$ promotes formation of phase appearing at 9.8 ppm that is most likely represent AFm but has to be confirmed by other techniques. At later time of reaction another aluminate phase at 5 ppm starts to grow and dominate in the system.

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MATERIAL PROPERTIES AND APPLICATION TO STRUCTURE OF LOW CARBON HIGH PERFORMANCE CONCRETE USING FLY ASH AND BLAST FURNACE SLAG

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Key words: Low Carbon, Fly Ash, Blast Furnace Slag, Material Property, On-site Test

Abstract. This paper describes the results of laboratory tests and on-site tests for concrete using 20% each of fly ash and blast furnace slag (Low Carbon High-Performance Concrete, LHC). In the laboratory tests, the material properties of strength development, resistance to cracking, and durability were investigated. The environmental impacts were also calculated. In the on-site tests, the production, transportation, placing, compaction and finishing of concrete were examined. The effects of concrete production seasons (spring, summer and winter) were also evaluated. It was found from the laboratory tests that the strength development, resistance to cracking, durability except for carbonation and reduction of environmental impact of LHC were equal to or greater than those of concrete using ordinary Portland cement. From the on-site tests, it was concluded that no separate storage facility for SCMs is required to produce LHC if blast furnace slag cement type B (JIS classification) is used instead of blast furnace slag as an SCM, and that the construction performance and quality of LHC is the same as those of ordinary concrete, but that it should be noticed that in the winter period, the finishing time and strength development are delayed.

1 INTRODUCTION

Recent years have seen various efforts by the construction industry around the world to reduce emissions of greenhouse gases, CO₂ in particular. Within the construction industry, the concrete manufacturing industry in particular emits large amounts of CO₂ during material
manufacturing, especially during cement manufacturing. One means of reducing CO\textsubscript{2} emissions during the production of concrete is to replace a portion of the cement with supplementary cementitious materials (SCMs) such as fly ash and ground granulated blast-furnace slag. Concrete that uses SCMs holds promise as an effective material for contributing to the creation of a low-carbon society in that it has a smaller environmental impact through the effective use of industrial by-products, as well as excellent performance in terms of long-term strength development and durability.

Studies on concrete using SCMs are being carried around the world, yielding many findings. For example, in Japan, the basic properties of concretes that use SCMs have been investigated [1] and such concrete has been applied to actual structures [2]. Outside Japan, research is being carried out on the strength and pore structure of ternary cement [3] and the strength of cement that uses SCMs such as fine limestone powder and ground granulated blast furnace slag [4].

Here, focusing on the cement and concrete fields in Japan, the production volume percentages by type of cement in 2014 were 69.2\% for ordinary Portland cement, followed by 21.6\% for blast furnace cement type B (defined by the JIS standard as having a mass ratio of blast furnace slag of 30\% to 60\%, and with an actual replacement ratio range of 40\% to 45\%). Thus, ordinary Portland cement and blast furnace cement type B account for more than 90\% of total cement production [5]. Thus in Japan, with the exception of special construction projects, use of concrete with a high SCM replacement ratio is relatively limited. Further, regarding the properties of concrete, it has been pointed out that a high SCMs replacement decreases early strength development and resistance to cracking. Therefore, it is important to consider the rational use of SCMs for reducing CO\textsubscript{2} emissions while retaining the basic properties of concrete. Against this background, the authors conducted a study on concrete that uses relatively little fly ash and ground granulated blast furnace slag. This study found that concrete with a replacement ratio of fly ash and ground granulated blast furnace slag of 20\%, respectively, provided good performance in terms of material properties and low carbon performance [6][7].

This paper reports the results of a study on low carbon high-performance concrete (hereafter, LHC), which is concrete with respective fly ash and ground granulated blast furnace slag replacement ratios of 20\%, carried out using laboratory tests and on-site tests. The laboratory tests, which were done to determine the material properties of LHC, investigated strength development, resistance to cracking, and durability. Further, CO\textsubscript{2} emissions were calculated to determine the environment impact of concretes with different mix proportions. On the other hand, the on-site tests examined the series of steps forming the construction process, from the production of LHC to its transportation, placing, compaction, and finishing, for the purpose of understanding the construction performance of LHC. The effect of the date of manufacture of LHC on the construction performance of concrete was also examined.

### 2 MATERIAL PROPERTIES OF LHC IN LABORATORY TESTS

#### 2.1 Test procedure

Table 1 lists the materials used for the concrete. Table 2 lists the combinations of binders

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of the concretes for laboratory experiments. The concretes used in the laboratory experiments were of three types, namely LHC, ordinary Portland cement alone (abbreviated as N) and blast furnace cement type B (with ground granulated blast furnace slag replacement ratio of 40%, abbreviated as BB).

Table 3 lists the mix proportions of the various concretes. The same water-binder ratio and sand-total aggregate ratio, 0.55 and 0.43 respectively, were used for all the types of concrete. The target values for fresh concrete, i.e. slump of 12 ± 2.5 cm and air content of 4.5 ± 1.5%, were achieved by adjusting the unit water content and admixture dosage. All the concretes were produced in a thermostatic laboratory at the temperature of 20 ± 2 °C.

Strength development, resistance to cracking, durability, and environmental impacts were set as the measurement items. Compressive strength (JIS A 1108) was measured for strength development. Standard curing was used as the curing method. The test specimens dimensions were φ100 × 200 mm, and tests were performed at the ages of 3, 7, 28, 56, and 91 days. The number of specimens for compressive strength test was three. For resistance to cracking, adiabatic temperature rise was measured to determine thermal resistance to cracking. The adiabatic temperature rise was measured until the age of 14 days using a water circulation apparatus. The number of specimen for adiabatic temperature rise test was one. Durability was assessed by measuring the relative dynamic modulus of elasticity through freeze-thaw testing, and the carbonation rate coefficient through accelerated carbonation testing. The specimens used for these two tests measured 100 × 100 × 400 mm. For the freeze-thaw test, the relative dynamic modulus of elasticity was obtained for up to 300 cycles in accordance with JIS A 1148. The accelerated carbonation test was performed in accordance with JIS A 1152 and JIS A 1153, measuring the carbonation depth under the conditions of temperature of 20 ± 2 °C, relative humidity of 60 ± 5%, and CO2 concentration of 5 ± 0.2%. The number of specimen for each durability test was two. The environmental impact reduction effect was determined through calculation of CO2 emissions at the time of concrete manufacturing. The level of CO2 emissions was determined by multiplying the basic unit [8] of CO2 emissions for the constituent materials of each concrete by the basic unit per 1 m3 of concrete.

### Table 1: Materials used for concrete

<table>
<thead>
<tr>
<th>Materials</th>
<th>Symbol</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary portland cement</td>
<td>OPC</td>
<td>Density: 3.15g/cm³, Blain: 3350cm²/g</td>
</tr>
<tr>
<td>Fly ash (JIS class II)</td>
<td>FA</td>
<td>Density: 2.39g/cm³, Blain: 4110cm²/g</td>
</tr>
<tr>
<td>Blast furnace slag</td>
<td>BFS</td>
<td>Density: 2.89g/cm³, Blain: 4200cm²/g</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>S</td>
<td>River sand, Density: 2.62g/cm³</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>G</td>
<td>Crushed stone, Density: 2.68g/cm³</td>
</tr>
<tr>
<td>AE water reducing agent</td>
<td>AD</td>
<td>Polycarboxylic acid based</td>
</tr>
</tbody>
</table>

### Table 2: Combination of binders of concrete

<table>
<thead>
<tr>
<th>Name</th>
<th>OPC</th>
<th>FA</th>
<th>BFS</th>
</tr>
</thead>
<tbody>
<tr>
<td>LHC</td>
<td>60</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>N</td>
<td>100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>BB</td>
<td>60</td>
<td>0</td>
<td>40</td>
</tr>
</tbody>
</table>
Table 3: Mix proportions of concrete in laboratory tests

<table>
<thead>
<tr>
<th>Name</th>
<th>BFS</th>
<th>s/a</th>
<th>Unit Contents (kg/m³)</th>
<th>Fresh concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>W</td>
<td>B</td>
</tr>
<tr>
<td>LHC</td>
<td>0.55</td>
<td>0.43</td>
<td>148</td>
<td>161</td>
</tr>
<tr>
<td>N</td>
<td></td>
<td></td>
<td>161</td>
<td>293</td>
</tr>
<tr>
<td>BB</td>
<td></td>
<td></td>
<td>157</td>
<td>171</td>
</tr>
</tbody>
</table>

2.2 Test results

Figure 1 shows the time-dependent compressive strength. The compressive strength of LHC was smaller than that of N up to the age of 28 days, but from the age of 56 days onward, it was larger than that of N. At the age of 91 days, the compressive strength ratio in relation to N was 1.21 for BB, and 1.07 for LHC. As a comparison of the compressive strength of LHC and BB shows that the hydration reaction occurs more gradually for the pozzolanic reaction (fly ash) than for the latent hydraulic property (ground granulated blast furnace slag), within the scope of the present study, the compressive strength of LHC was assumed to be inferior.

Figure 2 shows the ultimate value of adiabatic temperature rise and the parameter representing the rate of adiabatic temperature rise of each of the mixes. These values were obtained by approximation of the time variation of adiabatic temperature rise with equation (1). The ultimate value of adiabatic temperature rise ($Q_\infty$) of LHC was lower than that of N by 4.2 °C, and lower than that of BB by 5.9 °C. The parameter representing the rate of adiabatic temperature rise ($r$) of LHC was lower than that of N. Thus, LHC was found to have excellent thermal resistance to cracking owing to the ability to mitigate temperature rise.

$$Q(t)=Q_\infty[1-\exp(-r(t-t_{0,Q}))] \quad (1)$$

Where, $t$: age (day), $Q(t)$: adiabatic temperature rise at the age of $t$ (°C), $Q_\infty$: ultimate value of adiabatic temperature rise (°C), $r$: parameter representing rate of adiabatic temperature rise, $t_{0,Q}$: age at starting of temperature rise (day)

Figure 3(a) shows the relative dynamic modulus of elasticity at 300 cycles as the result of freeze-thaw testing. The relative dynamic modulus of elasticity at 300 cycles was a value of 92% or higher for all the mixes, and degradation of freeze-thaw resistance was not observed. Therefore, the freeze-thaw resistance of LHC was confirmed to show the same properties as normal concrete as long as a predetermined of air content in concrete is ensured. Figure 3(b) shows the accelerated carbonation rate coefficients. The accelerated carbonation rate coefficient of LHC was 1.3 times greater than that of N, and 1.2 times greater than that of BB.

Figure 3(c) shows the CO₂ emissions. The CO₂ emissions of LHC were found to be lower than those of N and BB by 43% and 6%, respectively, showing that LHC excels in terms of its environmental impact reduction effect, like BB.

Based on the above, the material properties of LHC excel compared with those of concrete using ordinary Portland cement in terms of strength development, resistance to cracking, durability except for carbonation, and environmental impact reduction effect.
3 PRODUCTION METHOD OF LHC AT READY-MIXED CONCRETE PLANT

3.1 Production method of LHC

The binders used for the LHC were fly ash and ground granulated blast furnace slag, each substituted for cement with the replacement ratio of 20%. Therefore, either the use of premixed binder, or the addition of fly ash and ground granulated blast furnace slag in the admixture silo is assumed as the method employed in the production of LHC at ready-mixed concrete plants. However, premixed binder involves blending and transportation costs and thus it is not generally suitable for application for LHC. Moreover, few ready-mixed concrete plants in Japan have two admixture silos, making the production of LHC difficult depending on the type of storage facilities available.

Consequently, the authors devised a method for the production of LHC that uses blast furnace cement type B, which is always available at ordinary ready-mixed concrete plants, instead of ground granulated blast furnace slag, as shown in Figure 4, for the purpose of mitigating the storage facility requirement.
3.2 Validity of production method of LHC

For the purpose of examining the validity of the production method described in Section 3.1, the fresh concrete properties and strength development of LHC were determined using laboratory tests. Table 4 lists the mix proportions of the various concretes. The mix proportions were of two types, using either ground granulated blast furnace slag or blast furnace cement type B as binder. Moreover, to improve resistance to cracking, expansive additive was substituted for fine aggregate at the ratio of 15 kg/m³.

Concrete production was performed at the temperature of 30 °C in the laboratory, assuming the ambient temperature during construction. Slump, air content, concrete temperature and compressive strength were selected as the measurement items.

No difference in fresh concrete properties, whether slump or air content, owing to differences in binder, could be seen, as shown in Table 5. Neither were differences in compressive strength observed, as shown in Figure 5, even as material age progressed.

From the above, even if blast furnace cement type B is used instead of ground granulated blast furnace slag in the production of LHC, there are no differences in fresh concrete properties and compressive strength, and LHC can be produced using the method.

Table 4: Mix proportions of concrete (case 1: laboratory test)

<table>
<thead>
<tr>
<th>No.</th>
<th>W/B</th>
<th>s/a</th>
<th>Unit Contents (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>OPC</td>
</tr>
<tr>
<td>1</td>
<td>0.545</td>
<td>0.438</td>
<td>158</td>
</tr>
<tr>
<td>2</td>
<td>0.545</td>
<td>0.438</td>
<td>158</td>
</tr>
</tbody>
</table>

Table 5: Comparison to properties of fresh concrete

<table>
<thead>
<tr>
<th>No.</th>
<th>Slump (cm)</th>
<th>Air (%)</th>
<th>C.T. (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.0</td>
<td>4.5</td>
<td>31</td>
</tr>
<tr>
<td>2</td>
<td>13.5</td>
<td>4.2</td>
<td>32</td>
</tr>
</tbody>
</table>

Fig. 5: Comparison to compressive strength

4 ON-SITE TESTS

4.1 Case 1: Application to slab and wall structure

4.1.1 Outline of on-site test in slab and wall structure

The construction of a slab and wall structure was performed for the purpose of grasping construction performance through the various steps of LHC production, transportation, placing, compacting, and finishing. Hazama Ando Corporation Technical Research Institute was selected as the construction location. Figure 6 shows the structure that was constructed. The slab dimensions were 4.4 m width, 10 m length, and 0.2 m thickness, and the wall dimensions were 2 m height and 0.2 m thickness, with projections ranging between 0.5 m to 0.6 m at five locations. The mix proportions of concrete were the two types shown in Table 6. The materials used for LHC and N were the same as those listed in Table 4. LHC was placed for the slab and the right portion of the wall, and N was placed for the left portion of the wall. AE agent for fly ash was used in the production of LHC in order to reduce air loss during transportation.

The slab and wall structure was constructed over two days. Specifically, the slab was constructed in August 2013, and the wall in September 2013. The concrete was prepared at a ready-mixed concrete plant located in Tsukuba City in Ibaraki Prefecture, and 15 m³ of LHC and 8 m³ of N were shipped. The transportation distance from the ready-mixed concrete plant to the construction site was about 30 minutes. A pump truck was used to place the concrete for the slab and wall, with pipe length of 20 m.

4.1.2 Results of on-site test in slab and wall structure

Table 7 lists the slump and air content when the concrete is about to be shipped (Plant), at the site during unloading (Site), and before and after pumping (Pumping Before/After). The slump and air content values of LHC at shipping and unloading approximated those of N. The use of AE agent for fly ash prevented air loss during transportation of LHC. The slump and air content before and after pumping showed the same tendencies for LHC and N.

The placing, compacting, and finishing of LHC and N during construction of the wall were assessed. Placing and compacting were done in a total of 4 layers of 0.5 m each. The placing
Table 6: Mix proportions of concrete (case 1: on-site test)

<table>
<thead>
<tr>
<th>Name</th>
<th>W/B</th>
<th>s/a</th>
<th>Unit Contents (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>W</td>
</tr>
<tr>
<td>LHC</td>
<td>0.545</td>
<td>0.439</td>
<td>158</td>
</tr>
<tr>
<td>N</td>
<td>0.545</td>
<td>0.439</td>
<td>169</td>
</tr>
</tbody>
</table>

[Note] Materials used were the same “Table 4”.

duration per layer was 17 to 26 minutes for LHC, and 18 to 26 minutes for N, thus no difference was observed between the two types of concrete. Now, looking at the finishing duration, the initial setting time of LHC was 8 hours 35 minutes, a delay of 2 hours 50 minutes compared with N (initial setting time of 5 hours 45 minutes). The delay in initial setting time means that the occurrence of cold joints can be reduced owing to the longer available compacting time, but this also has the effect of delaying the finishing start time. However, this problem is considered to be manageable by formulating a construction plan that delays the finishing start time in advance.

Figure 7 shows the temperature history of the wall structure. The temperatures of both LHC and N were measured by using thermocouples installed in the center of the wall. The temperature at the center of the wall was 41.7 °C for LHC and 45.7 °C for N, thus 4.0 °C lower for LHC compared with N. Based on these results, LHC is assumed to be less susceptible to thermal cracking than N. Moreover, observation of the wall structure after wall construction showed the absence of cracks due to thermal stress for both LHC and N.

Figure 8 shows the compressive strength of concrete collected during the wall construction. The compressive strength of LHC was equivalent to that of N at 28 days, but greater than that of N at 91 days, showing that LHC excels in strength development, similarly to the results described in Section 2.

Based on the results of the above-described examination of construction performance through the various steps of the construction of the slab and wall structure, the construction performance of LHC was found to be comparable to that of ordinary concrete.
Table 7: Properties of fresh concrete (case 1: on-site test)

<table>
<thead>
<tr>
<th>Name</th>
<th>Slump (cm)</th>
<th>Air (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plant</td>
<td>Site</td>
</tr>
<tr>
<td>LHC</td>
<td>18.0</td>
<td>14.0</td>
</tr>
<tr>
<td>N</td>
<td>18.0</td>
<td>11.0</td>
</tr>
</tbody>
</table>

[Note] Target value in site condition: slump=12±2.5cm, air=4.5±1.5%

4.2 Case 2: Application to leveling concrete

4.2.1 Outline of on-site test in leveling concrete

Taking into consideration the results of the construction performance over the various construction steps detailed in Section 4.1, application to leveling concrete was done for the purpose of determining the properties of LHC during its production and application during three different seasons, in addition to determining the quality of LHC concrete when it is shipped continuously from a ready-mixed concrete plant. The construction location was a water purification plant in the city of Kawasaki in Kanagawa Prefecture. The construction period was set in three different seasons, spring (June), summer (July), and winter (January). The leveling concrete in all three seasons was produced at a ready-mixed concrete plant in the city of Fuchu in Tokyo. The total volume of concrete that was applied was 735 m³, which breaks down as 204 m³ for spring, 155 m³ for summer, and 376 m³ for winter.

4.2.2 Selection of mix proportion

The proportioning conditions were nominal strength = 18, strength for proportioning = 23.0 N/mm², slump = 12 ±2.5 cm, and air content = 4.5 ±1.5%. To determine the water-binder ratio satisfying the proportioning conditions, three water-binder ratios, 0.45, 0.55, and 0.65, were used, and the compressive strength at 28 days was obtained for each. From the obtained compressive strengths values, the compressive strength was calculated in the 95% confidence interval, and the relationship between the binder-water ratio and compressive strength was obtained as shown in equation (2). From this equation, the water-binder ratio that satisfies the
strength for proportioning of 23.0 N/mm² was found to be 0.698. However, the upper limit of the water-binder ratio in a reference [9] having been set to 0.65, the water-binder ratio of the leveling concrete was set to 0.65.

\[ F_c = 20.6 \cdot B/W - 6.5 \]  

Where, \( F_c \): strength for proportioning (N/mm²), \( B/W \): binder-water ratio

Table 8 lists the mix proportions for each season. The mix proportion selection for spring was determined by first determining through test mixing that the strength for proportioning could be obtained 26.6 N/mm² at 28 days. Next, the concrete prepared in a full-scale mixer was loaded in a truck mixer agitator, and the changes in slump and air content over time were measured. As a result, it was confirmed that the prescribed slump and air content values for the 40 to 60 minute transportation time to the construction site were satisfied even as long as 75 minutes after the completion of mixing. The mix proportions for summer and winter were determined based on the performance of the ready-mixed concrete plant and existing knowledge [6].

4.2.3 Results of on-site test in leveling concrete

Table 9 lists the slump and air content measurement results. The slump and air content varied little across the seasons and good quality concrete was obtained in all seasons. Figure 9 shows the measurement results for compressive strength. For compressive strength, sealed curing was performed either in laboratory or on site up to the age of three days, followed by demolding and standard curing. The data in Figure 9 shows that when the outside air temperature is high (Spring: Site), a reduction in strength may occur if initial wet curing is not carried out sufficiently. Moreover, it should be noted that if the outside air temperature is low (Winter: Plant and Site), early strength development up to 3day may be delayed.

Next, with respect to construction performance, pumpability was found to be satisfactory, with the concrete being pumpable without blockage of the pump. Compacting and finishing could be executed in the same way as for ordinary concrete applied in other sections. Moreover, no trouble as crack was confirmed by hardened concrete after construction. However, it should be noted that application of trowel finishing in winter will delay initial setting time, and care should be taken to time the completion of finishing work accordingly.

<p>| Table 8: Mix proportions of LHC (case 2: on-site test) |</p>
<table>
<thead>
<tr>
<th>Seasons</th>
<th>W/B</th>
<th>s/a</th>
<th>Unit Content (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W</td>
<td>B</td>
<td>S₁</td>
</tr>
<tr>
<td>Spring</td>
<td>165</td>
<td>90</td>
<td>113</td>
</tr>
<tr>
<td>Summer</td>
<td>168</td>
<td>92</td>
<td>115</td>
</tr>
<tr>
<td>Winter</td>
<td>165</td>
<td>90</td>
<td>113</td>
</tr>
</tbody>
</table>

[Note] S₁ and S₂: crushed sand, S₃: hills sand, G₁ and G₂: crushed stone
Table 9: Properties of fresh concrete (case 2: on-site test)

<table>
<thead>
<tr>
<th>Seasons</th>
<th>Slump (cm)</th>
<th>Air (%)</th>
<th>C.T. (°C)</th>
<th>A.T. (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plant Site</td>
<td>Plant Site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spring</td>
<td>18.1</td>
<td>12.9</td>
<td>5.2</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>(17.0-19.5)</td>
<td>(11.5-14.5)</td>
<td>(4.5-6.1)</td>
<td>(4.1-5.4)</td>
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<tr>
<td>Summer</td>
<td>18.8</td>
<td>12.6</td>
<td>5.2</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>(18.0-21.0)</td>
<td>(11.0-14.5)</td>
<td>(4.5-5.5)</td>
<td>(3.3-4.5)</td>
</tr>
<tr>
<td>Winter</td>
<td>18.4</td>
<td>14.0</td>
<td>5.4</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td>(17.0-20.0)</td>
<td>(13.0-14.5)</td>
<td>(4.5-6.2)</td>
<td>(3.1-4.9)</td>
</tr>
</tbody>
</table>

[Note] Target value of slump and air in site condition: slump=12±2.5cm, air=4.5±1.5%
Upper value: average value, Lower value: (minimum value – maximum value)

Fig. 9: Compressive strength (case 2: on-site test)

5 CONCLUSIONS

This study carried out laboratory tests and on-site tests for the purpose of determining the material properties and construction performance of concrete with respective fly ash and ground granulated blast furnace slag replacement ratios of 20%. From the laboratory tests, the material properties of LHC excel compared with those of concrete using OPC and BB in terms of strength development, resistance to cracking, durability except for carbonation, and environmental impact reduction effect. From the on-site test, the construction performance of LHC was found to be comparable to that of ordinary concrete (OPC or BB). However, it should be noted that strength development at an early age in the case of winter construction is delayed, and that the finishing time is also delayed.

ACKNOWLEDGEMENT

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REFERENCES


MATERIAL PROPERTIES OF MINERALIZED FOAM AND ITS DENSITY DEPENDENCY – A META-STUDY

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Key words: foam concrete, lightweight mineralized foam, meta-study, systematic review, transient foam structure

Abstract. The dependency of mineralized foam towards their material properties, such as compressive strength or thermal conductivity, is widely studied and generally known. However, most of the research conducted on this topic so far has been focusing on a very narrow range of densities. This has led to the fact that the current level of knowledge on the thermo-mechanical performance of this versatile material is very concentrated and, therefore, not providing an overall figure that is covering a wide range of densities. To extend this level of knowledge, a meta-study was conducted at the Institute of Construction and Building Materials of the TU Darmstadt consisting of literature data combined with results taken from own lab tests. Aim of this study was to get a full picture of the interdependencies between various thermo-mechanical parameters of the hardened mineralized foam, while covering a wide range of densities. Final results showed a consistent trend, where literature data and lab test results turned out to be very complimentary.

1 INTRODUCTION

Due to their great band width of possible densities (100 kg/m³ to 2000 kg/m³), their heat insulation capabilities, and their structural strength, foam concretes and mineralized foams form an attractive material group for structural and insulating use. It enables a wide range of possibilities for using these materials in factory buildings or in in-situ applications [1]. Having said that, this variety of applications and material types creates several practical and scientific questions. Most important ones are on the possibility to produce a robust and very lightweight mineralized foam in a constant quality, how to optimize the heat insulation capacity of mineralized foam without a deterioration of its mechanical characteristics, what is the influence of the solid mater\(^1\) ratio in fresh mineral foam on its transient structure, and how does the consistency and hydration of the binder paste affect the very transient structure of mineral foams? Finding answers to these questions poses quite a challenge which can succeed only by adopting scientific methods. Therefore below, results of a meta-study about material properties of mineralized foam and its density dependency are recapitulated and discussed.

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\(^1\) The binder paste influences the foam structure already only due its relatively high density.
2 MATERIAL

2.1 Foam

Foam is gas dispersion in a fluid or solid continuous phase [2, 3]. The ISO 862 defines foam as a sequence of gas cells which are divided by thin fluid films. The confined gas volume is dispersed in a relatively small amount of liquid [5]. Like every system of non-miscible phases, foams aren't thermodynamically stable. The basic components tends toward separation and destabilization. Foam is, therefore, a multiphase system with a limited period of persistence [6, 7]. It is the structure of fluid foams that changes continuously because the surface potential tends to a minimum, by minimizing the interfaces between the phases while releasing the stored energy. Surfactants stabilize the foam structure but the destruction of the foam begins immediately after its formation [8, 9]. There are several processes inside a foam which cause its destruction (fig. 1) [7].

![Figure 1: Process of foam destruction [6, 10].](image)

As shown above, the structure of foam is significantly determined by the actual fraction of air. Given the fact, the specific weight can be a good criterion for characterizing a foam [11]. For aqueous foams, finding the right density is very often a point of scientific discussions [12–14]. Heavy foams tend to drain but are more flowable [15]. But when using them, more water is brought into the mineral foam with corresponding consequences for the cementitious matrix (fig. 4). Lighter aqueous foams are relatively stiff which becomes more explicit in case of applying an overdosage of foaming agent. This may lead to problems for getting foams with a constant void content [15]. Furthermore, the thin lamellas of a light foam with a low water content provoke a lower robustness of the aqueous foam against suction effects i.e. caused by cement particles [16]. Normal densities of the aqueous foams used for mineral foam production range between 60 kg/m³ and 80 kg/m³ [13, 14].

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2 Open porous materials don't have a dispersed phase therefore they aren't foams within this definition [4].

3 Foams with a high gas volume are called colloid agglomerated gas dispersion, otherwise it is by definition no foam but an emulsions of gas [1].
2.2 Foam concrete and mineralized foam

Foam concrete means a highly porous\textsuperscript{4} cementitious material without coarse aggregates\textsuperscript{5} [19–24]. In contrast to autoclaved aerated concrete, foam concrete hardens due to cement hydration. Besides, the two material types show different characteristics in the hardened state [25]. It's possible to differentiate between the various types of foam concrete using dry density or composition. In addition, the used production technique is formative for their characteristics [26, 27]. The publications analyzed for the proposed Meta-study showed many parallels when referring to the different ranges of densities, and when referring to the fact that these ranges were in most cases between 100 kg/m\textsuperscript{3} and 500 kg/m\textsuperscript{3} (cf. fig. 5). On the one hand this can be related to the fact that researches were used to specific compositions or, on the other hand, that when producing foam concrete – even in the lab – it is very hard to meet exactly the specified design density [28]. This shows the need to classify different types of foam concrete [26, 29, 30].

Typically the highest density discussed is about 1800 kg/m\textsuperscript{3}. This density is slightly lower than the 2000 kg/m\textsuperscript{3}, which is the normative limit of lightweight concrete [18]. The foam concretes ranging between 1400 kg/m\textsuperscript{3} and 1800 kg/m\textsuperscript{3} are often reinforced and find their use in housing or soil applications, however almost always outside Europe [26, 29–35]. For these particular application, in Europe the more expensive lightweight aggregates concrete is normally used [22, 36]. The densities of the second category of foamed cementitious material range between 800 kg/m\textsuperscript{3} and the previously mentioned 1400 kg/m\textsuperscript{3} approximately. It is used for structural applications but mostly as fillers or plugs in the mining or tunneling industry [26, 29, 37–40]. This material can, therefore, also be called ‘foam mortar’. A third category used as a robust filler or insulation material has a density of around 400 kg/m\textsuperscript{3} to 800 kg/m\textsuperscript{3}. This class can be considered to be part of the mineral (fresh state) or mineralized (hardened state) foams [26, 29, 41–43]. All other types of foams, with densities below a dry density of 400 kg/m\textsuperscript{3} are called lightweight mineral foams and/or mineralized foams, respectively. These types of foams is a relatively new subject that still requires a deep research effort, and is mostly used as an insulation material (tab. 1) [13, 44, 45].

\begin{table}[h]
\centering
\begin{tabular}{|c|c|}
\hline
\textbf{dry density range [kg/m\textsuperscript{3}]} & \textbf{category} \\
\hline
1400 – 1800 & (reinforced) foam concrete \\
\hline
800 – 1400 & foam mortar \\
\hline
400 – 800 & mineralized foam \\
\hline
< 400 & lightweight mineralized foam \\
\hline
\end{tabular}
\caption{Dry density ranges and proposed categorization for porous cementitious foams with aggregates D < 2 mm.}
\end{table}

\textsuperscript{4} The porosity normally is way higher than in an air-entrained concrete [17].

\textsuperscript{5} For this reason actually it is not really correct to define it a concrete. It would be better to speak from foam mortar [18].
3 METHODS

3.1 Meta-study

A meta-study or extensive data analysis serves primarily as a way to provide a tendency overview of data gathered from a specific subject. To this end, more than 40 specific studies have been combined and evaluated in order to draw conclusions from a wide range of literature data and own lab results. Present meta-study addresses test results of "foam concrete". Each study-topic was done on a series of tests with various properties that was depending on the density of a particular type of foam concrete. Based on the experimental results the tendency calculations of the meta-study were performed.

3.2 Foam

The here discussed foamed cementitious materials are based on an aqueous foam principle, which is produced with a foam generator. This device merges the three essential substances, water, air and surfactant, into a fresh foam. The concentration of the foam agent can be changed by adjusting the flow ratio between the in the flowing water and air phases. In an air intrusion device, using compressed air, the gas gets merged in the continuous phase and enclosed by the aqueous foam (cf. fig. 2) [46].

A porous membrane follows the air intrusion flow leading to a homogenized pore size distribution of the aqueous foam [6, 15]. Here the pores in the accelerated foamed fluid get divided by the shear forces induced by the obstacles [16].

3.3 Mineralized foam

All foamed cementitious materials discussed in this context are essentially a mix of aqueous foam and a binder matrix. This technique has the methodical advantages that all components can be individually admeasured and analyzed before mixing. This makes the calculation of the mass flow more transparent, and interdependencies between the used materials can be easily identified and considered. Thus for production a three step procedure is often used. First the binder paste will produced in a specific mixer such as a mortar or
slurry mixer, which ensures a homogenous paste due to the high shear forces [47, 48]. Next, the foam will be generated as mentioned before. Finally, both components are mixed together in a classical concrete mixer. In this regard it important to consider that big density differences between foam a binder paste often cause obstructions, which may hinder a fast and efficient mixing (cf. fig. 3). Own research showed that shaft mixers are quite suitable for a successful mixing of these two phases [49, 50].

**Figure 3**: Schematic representation of the mixing systems used for producing mineral foam. From left to right: single-shaft compulsory mixer, two-wave mixer, bucket mixer.

### 3.4 Mix design

For analyzing the different test results of foam concrete it is essential to refer to a unique mix design. Such a method exists for ordinary concrete but it does not exist for foam concrete. The following design method serves as a way to classify the different data received from the evaluated articles of the meta-study.

In order to categorize the importance of the various material characteristics, dry density $\rho_d$ is defined as the principal design parameter. Independent of the water cement ratio $w/c$ used for the binder paste, it is assumed that it has no significant influence on the dry density. Higher water to cement ratios ($> 0.4$) will produce pore volumes, which in a first step are relatively small, once comparing with the pore volume produced by foam. For lower water cement ratios, the water needed for a chemical optimum will be counterbalanced by water taken from the aqueous foam. For practical reasons, like measuring consistency and viscosity, water is determined from the total mass of binder and solid additives. On this basis their sum $m_f$ can be calculated using formula 1.

$$m_f = \rho_d / (1 + w/c) \quad (1)$$

It is possible to fill the rest of a virtual cubic meter with aqueous foam $V_F$ left by summarizing the individual volumes of the binder paste components $V_w$ and $V_f$, using the respective material densities (formula 2).

$$V_F = 1 m^3 - V_w - V_f \quad (2)$$

As mentioned before, the fresh density $\rho_{fr}$ of mineral foam is an important parameter to control the quality during the production process (formula 3).

$$\rho_{fr} = \Sigma (m_F, m_w, m_f) \quad (3)$$

Not considering the water in the aqueous foam $m_{Fw}$, as this is done in many publications, is only acceptable for higher dry densities [51]. The mentioned water amount is directly proportional to the created porosity $\phi$ of the foam and so indirectly proportional to the density of the mineralized foam (formula 4 and 5).
\[ m_{\Sigma w} = m_{Fw} + m_F \]  
\[ m_{\Sigma w} = \rho_F + \phi \cdot 1 \text{ m}^3 \]  

From this, the paste, and thus also the amount of mixing water with a given constant water cement ratio, is indirectly proportional to the porosity. For these reasons, together with the real water cement ratio \( w/z_{\text{real}} \), the amount of water in the aqueous foam \( \phi_{mFw} \), relative to the total water content, rises disproportionally with declining densities (formula 6 and 7).

\[ \phi_{mFw} = m_{Fw} / m_{\Sigma w} \]  
\[ w/z_{\text{real}} = m_{\Sigma w} / m_z \]  

Figure 4 shows this deviated relationship. There the real water cement ratio and the share of the water of the aqueous foam is plotted against the design density of the mineralized foam. The given graphs reflect the limiting cases of the water cement ratios 0.4 and 0.6 together with the foam densities of 60 kg/m³ and 80 kg/m³.

**4 RESULTS**

**4.1 Density – compressive strength**

Results of the meta-study regarding the relationship between compressive strength and density of foamed concrete and mortar are plotted in figure 5. Based on own experimental results, in figure 6, the same relation for lightweight mineralized foams has been provided. In this figure, the characteristic compressive strength is shown for a foam with a dry density of 180 kg/m³ and has an average value of 0.12 N/mm² (5 %-quantile, 90 % confidence band)\(^6\). The distribution curves on the single values as well as on the broad confidence band could be

\(^6\) In order to calculate this single value for one density all results were moved parallel along the linear regression function.
an indication that different failure types occurred, which could be attributed to the different pore size distributions or pore forms (fig. 6).

\[ \text{Figure 5: Relationship between 28-day compressive strength and density of foamed cementitious material as a result of the meta-study [13, 32, 33, 37, 39, 40, 42, 52–61].} \]

\[ \text{Figure 6: Relationship between compressive strength and dry density of mineralized foam (about 100 single values).} \]

4.2 Density – thermal conductivity

The results of the thermal conductivity showed a nearly linear distribution (fig. 7). Especially for lightweight mineralized foams, results plotted in figure 8 show this observation. The regression function represents the average value of thermal conductivity depending on the dry density. From the Gauss distribution, a characteristic value of 0.059 W/(m · K) for a mineralized foam with a design density of 180 kg/m³ is calculated.
Figure 7: Relationship between thermal conductivity and density of foamed cementitious material as a result of a meta-study [56, 58, 60, 62, 63].

Figure 8: Left: Individual values of thermal conductivity at an average sample temperature of 10 °C as a function of dry density of dried cementitious foams. Right: Normal distribution of the parallel shifted values along the regression function [64].

5 CONCLUDING SUMMARY

Figure 6 shows a linear relationship between the density and compressive strength for a mineralized foam where densities range between 120 kg/m³ and 260 kg/m³. The global dependencies shown in figure 5 are characterized by a decreasing gradient for the lower densities. In figure 7 the distribution of the thermal conductivity of mineralized foam for densities ranging between 200 kg/m³ and 1600 kg/m³ is nearly linear. Single evaluated investigations show, however, always a significant scatter of the distribution towards the lower densities. This can lead to a conclusion that there is a potential for a minimized thermal conductivity once lowering the density of mineralized foam. At the same time also the strength will decrease so there is for every application an optimum of thermal conductivity and mechanical strength (fig. 9). This correlation has been quantified and provided in fig. 9.
The relatively large spread in results show that there have to be other factors than the density that influence the mechanical or structural-physical characteristics of foamed cementitious materials. Besides the composition of the continuous phase, it is most probably the pore structure of the hardened foams that plays a dominant role. This pore structure is the result of the transient characteristic of the produced mineral foam. Especially for cementitious foams it is possible to analyze the thermo-dynamic processes that occurs inside the foam, and which causes the quasi-meta-stable (instable) nature of the fluid foams. The cementitious hardening process allows to freeze the actual three dimensional structure of a fluid foam in various states. On this bases, values like compressive strength and thermal conductivity are strongly depending on this and are, therefore, still subject of scientific research.

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MECHANICAL PROPERTIES OF FIBER REINFORCED CEMENTITIOUS COMPOSITES WITH HIGH AMOUNTS OF FLY ASH AS CEMENT REPLACEMENT

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Key words: Fly Ash, Strain Hardening, Fibers

Abstract. Fiber reinforced cementitious composites (FRCC) made with hybrid Polyvinyl Alcohol Fibers and high amounts of fly ash (FA), show favorable mechanical properties in terms of large tensile strain capacity and ductility, and offer advantageous prospects for innovative products in construction [1]. The amount of cement replacement by FA in the composite in the order of 60%, reaching 34% by weight of total mixed materials is contributing towards the reduction of CO₂ emissions from cement production, recycling a material (fly ash) that would otherwise be disposed in the field. By eliminating the use of coarse aggregate exploitation of natural resources is decreased even further. Addition of fibers increases the toughness and ductility in structural components offering a novel opportunity for reduced amounts of transverse reinforcement, enhancing the durability of the structural member and limiting damage in strong earthquakes. Due to the high strength and very high tensile strain capacity it offers new opportunities particularly in the precast concrete industry. Experimental test results on the composite with and without the use of short discontinuous fibers (made of polyvinyl alcohol PVA) under uniaxial compression, extension and four point bending loads illustrated the beneficial structural behavior of this class of sustainable materials, owing to the addition of fibers which controls cracking and deterioration, and to the fine packing of particles achieved through the very large fly-ash content and the absence of coarse aggregate, which render the porosity very low, favorably contributing towards the reduction of permeability.

1 INTRODUCTION

This year’s COP21 United Nations Conference on climate change has issued a resolution to limit the rate of global warming through a global reduction of emissions responsible. The production of Portland cement is accountable for 5% of global CO₂ emissions, owing to the extreme heat required for cement production: a ton of cement requires 4.7 million BTU of energy, equivalent to about 180kg of coal, and generates nearly a ton of CO₂. Cement’s production is growing by 2.5% annually, and is expected to rise from 2.55 billion tons in 2006 to 3.7-4.4 billion tons by 2050 with the equivalent amount of CO₂ emitted in the environment.
Additionally to the emissions, cement and concrete production is responsible for the excessive consumption of lime, clay and burning energy required in the process, also for the use of water and gradated crushed aggregate materials. According with the 1987 UN definition, “sustainable development” is “meeting the present needs without compromising the ability of future generations to meet their needs”. With the increasing development of the build environment throughout the globe and the trends of people in the developing world to adopt to a consumer-driven lifestyle, the question of concrete’s sustainability gains new perspective. Recycling of aggregates is pursued as one alternative at the cost of yet more energy and resources in order to process materials to a re-usable state. One of the most promising attempts for sustainable development for concrete is the use of fly ash (FA), a byproduct of the energy industry that otherwise ends up in wastelands creating lots of environmental problems.

1.1 Cement Emissions

CO₂ emissions are generated both from the heating of limestone in cement-production but also from the fossil fuels burned to create that heat. This process known as calcination, accounts for 50% of CO₂ emissions for cement production. The remaining 40% comprises indirect emissions from burning of fossil fuels. Additionally the electricity used to power additional plant machinery, and the final transportation of cement, represents another source of indirect emissions and account for 5-10% of the industry’s emissions. One of the most promising techniques to reduce emissions from the calcination process is the use of blended cement made by replacement of some of the limestone-based clinker with other materials, primarily FA [3].

Fly ash has been incorporated into the modern codes as a cement replacement that is limited to a maximum of 20%. This was based on concerns raised in early experimental studies where coarser FA with high amounts of carbon content had been used. Originally, FA had been used in massive concrete structures such as dams as a means to reduce thermal cracking and therefore increase water-tightness. The beneficial results of FA are owing to the particle size (40% under 10 microns - greater early age strengths), and to the lower reactivity that can reduce the water content in the mix (20% reduction of water with 50% FA) resulting in a denser microstructure (with or without the use of aggregates); these benefits come at the expense of plastic shrinkage cracking, an issue that ought to be addressed by developing new formulas of composites.

1.2 Relevance of the present research to sustainable design

Structural engineers may meet the requirements of sustainable structural design either by using technologies that address the ecological and natural resources aspect, and/or by the design of durable and resilient structures with longer service life. Neglecting the connection between the advent in material technologies, structural design, and the objectives of sustainable design leads to structures with a shorter service-life, resource-intensive requiring more extensive maintenance [4].

This research concerns a material engineered to balance all sustainability objectives that consider the energy embodied within a structure. Increased sustainability was achieved through partial replacement of Portland Cement (PC) with FA, elimination of coarse
aggregate (max. aggregate size < 300μm) and through addition of fibers to increase energy absorption under mechanical load. This is characterized as high-performance fiber-reinforced cementitious composite (HPFRCC) designed to resist large tensile and shear forces while remaining compatible with ordinary concrete in almost all other respects such as compressive strength and durability [1].

Previous research regarding the use of FA has documented the many beneficial aspects of its use in concrete such as low permeability and high durability ([5], [6]). Also it was shown for high-strength concrete that replacement levels of PC at a content by weight of 35% are effective in controlling generation of heat of hydration [7]. Indeed it was shown [8] that HVFAC (High Volume FA Composites with more than 50% of cement replacement) exhibits lower shrinkage, creep and water permeability, and higher modulus of elasticity as compared to conventional concrete (CC). But based on a review of the existing literature, there is a lack of evidence regarding the structural performance of this class of materials containing unconventionally high fractions of cement replacement, i.e. their behavior in reinforced concrete structural members. Without this background, there is no quantitative basis for safely implementing ECC in structural design. Ongoing studies at the University of Cyprus are intended to study this particular issue. In this context, a series of tests are undertaken to characterize the structural performance of members under tension, compression, flexure and shear of a mix of 60% replacement of cement with FA, without the use of coarse aggregate, with and without the use of fibers. The experimental program, test results, and analyses for this study are presented in the following sections.

1.3 Background of the State of the Art and Standards

Fly Ash is defined by ASTM [9] as “the finely divided residue that results from the combustion of ground or powdered coal and that is transported by flue gases”. The Standard of the ACI classifies FA into three categories: class N, F, and C based on their chemical compositions [10]. FA as cement replacement in normal concrete has been a long standing standardized practice used extensively in construction as a replacement or supplementary material in amounts of 15–25% of cement [11]. Recently, higher amounts have been considered - for example, FA was used at about 30–75% mass replacement of hydraulic cement to reduce heat generation in construction of the Thames Barrage in the United Kingdom and the Upper Stillwater Dam in the United States [10]. However, specification for the use of FA in the production of SHCC (Strain Hardening Cementitious Composites) has not yet been incorporated in Modern Codes, consistently with the fact that this application is rather recent and therefore not fully supported with the critical mass of data and evidence regarding the structural performance of the resulting members. In response to this emerging needs, studies such as the one presented herein are intended to provide the essential lacking evidence that can support the development of design rules for general practical use.

2 EXPERIMENTAL PROGRAM

2.1 Materials

The target of the research was to examine the mechanical properties of a self-consolidating cementitious mix, that has a high percentage of FA replacement and can maintain its
rheological properties with the addition of fibers. The mix design selected was ECC-M45 [12] suitable also for large scale casting. The materials and quantities were adjusted to the local available options. Thus, the cement used was Portland Composite Cement En 197-1 Cem II / A-M (L-S) 42.5 R. This type of Blended Cement is produced using pure calcite limestone and is more impermeable and dense as compared to OPC, with a higher degree of workability and reduced plastic shrinkage. The compressive strength of this particular cement is at 28 days equivalent to that of 42.5 R OPC and superior at 2 and 7 days. Silica sand (>95% Si) used had a maximum grain size of 300μm.

The FA was of type F and was imported. This class of FA has pozzolanic properties (a pozzolan is a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value but which will, in finely divided form and in the presence of water, react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties [9]. Class F FA is typically a byproduct of combustion of anthracite or bituminous coal for the production of electricity, but may also be obtained from burning sub-bituminous coal and lignite. Class F FA has been used in other studies as cement replacement at fractions ranging from 20-30% of the mass of cementitious material but in this research the amount of FA used was 120% by weight of the amount of cement (that is, one part of hydraulic material in the final mixture comprised 45% cement and 55% fly-ash). In the present investigation 12mm long, 39μm diameter PVA fibers were used. The nominal tensile strength was 1600MPa, Young’s Modulus was 40GPa and the density was 1300kg/m³. The fibers used were specially coated. It has been shown in previous studies by the authors that the coating on PVA fibers increases the dispersion of the hydrophilic fibers and the final ductility of composites by changing the interface bond properties between the fibers and the matrix [13]. Mix details for the two types of composites with and without the use of fibers are listed in Table 1. The water and superplasticizer were adjusted according to the decrease of workability in the case of SHCC induced by the fibers in the mix. Mixing procedure for the mix with fibers (SHCC) is as follows: a) All dry materials are first added in the drum (cement, fly-ash, sand) and stirred until adequately mixed, b) then 90% of the mixed water and plasticizer are slowly added to the mix, c) after the mix reaches a self-consolidating fluidity the fibers are added to the mix together with the remaining water and plasticizer.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement</th>
<th>Fly Ash</th>
<th>Sand</th>
<th>Water</th>
<th>Super Plasticizer</th>
<th>Fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHCC</td>
<td>1.00</td>
<td>1.20</td>
<td>0.80</td>
<td>0.60</td>
<td>0.017</td>
<td>2% (per volume)</td>
</tr>
<tr>
<td>HVFA</td>
<td>1.00</td>
<td>1.20</td>
<td>0.80</td>
<td>0.55</td>
<td>0.012</td>
<td>-</td>
</tr>
</tbody>
</table>

2.1 Specimen Design

Tests were carried out on a series of specimens to determine the mechanical properties of the mix with (SHCC) and without the use of fibers (HVFA). The tests comprised of direct tension, uniaxial compression and prism specimens under four point bending. Three identical specimens were tested for each type of test and mix design.
Displacement controlled direct tensile tests were carried out on dog bone specimens. The testing setup and mounting of specimens used specially designed hardware for this purpose. Displacement rate was 0.0025mm/s and the gauge length of the measuring device was 100mm, with the specimen cross section at the critical location being 25mmx50mm. Also uniaxial compression tests were performed on cylinder specimens to define the stress-strain law ($\sigma_c - \varepsilon_c$) under compression. Cylinder specimens 100mm in diameter and 200mm in height, having an aspect ratio of 2 in order to eliminate the local end effects, were tested under displacement control conditions. A closed-loop, servo controlled compressive testing machine was used. A loading rate ranging from 10 µm/s to 30 µm/s was applied following the recommendations of Concrete library of JSCE (1984). Axial deformations were measured using two linear variable differential transducers (LVDTs) mounted on opposite sides of the specimen with a gage length equal to 140mm. An additional LVDT measured the deformation over the entire length of the specimen, through the displacement of the platens. Circumferential elongation was measured using a circumferential extensometer (chain), placed at the middle of the height of the coupon. An electronic data acquisition system was used to record deformations and the corresponding loads.

Additionally to the tested dog-bone direct tensile tests and cylinders under compression, tests on beams under four point bending were carried out. The distance between the load points for the four point bending beams was 100mm with a cross section of 100x100mm. The beams were supported on end rollers at a distance of 100mm from the intermediate loads. Beams extended 25 mm over the supports. Beams were tested under displacement control at a displacement rate of 1.5µm/s. Five LVDTs were used to record the vertical deflection at the supports, at the load points and at the middle of the beams.

3 RESULTS AND DISCUSSION

3.1 Direct tensile properties

The initiation of cracks in concrete is governed by its tensile properties, therefore it is one of the most important parameters needed to be determined prior to any analysis of a concrete member. Even though it’s rather difficult to perform uniaxial tension tests on normal cement composites, in the case of fiber cementitious composites this becomes more possible. In the case of the HVFA mix the three samples that were casted for uniaxial tension, cracked during their de-molding. The SHCC mix on the other hand demonstrated strain hardening response in uniaxial tension as illustrated in Fig. 1(a). Strain was calculated as the elongation measured by the equipment mounted on the specimens’ two opposite sides (Fig. 1 (b)) and divided by the gage length of 100mm. During the formation of multiple cracks, stress capacity of the cross section increased. First cracking stress $f_{ty}$ and corresponding first cracking strain $\varepsilon_{ty}$ depend primarily on the matrix composition and fault distribution in the cross section. Past the yield point multiple cracks form and fibers bridging the cracks elongate from the stress they transfer but simultaneously they also pull-out from the matrix at the crack locations. Formation of cracks saturates the full length within the measuring range and even spreads outside that, on the wider cross sections of the specimen. Beyond a limiting strain value which depends on the competition of interfacial bond and the tensile strength and stiffness of the fibers, new cracks cease to form and crack localization is evident (this point defines the maximum tensile stress $f_{tmax}$ and respective strain $\varepsilon_{tmax}$); as bond strength increases with age.
due to ongoing hydration, there is a growing tendency with age, of the occurrence of increasing crack-widths and local fracturing of fibers witnessed by a marked reduction in the available strain ductility $\varepsilon_{\text{max}}$.

![Figure 1](image1.png)

**Figure 1:** (a) Stress-strain diagrams, (b) testing setup and (c) multiple cracking of direct tension test on SHCC

### 3.2 Uniaxial Compression

![Figure 2](image2.png)

**Figure 2:** (a) Stress - axial/lateral strain diagrams under compression of HVFA and SHCC mixes, (b) failure cracks for HVFA (c) failure cracks for SHCC

Figure 2 (a) compares the stress - axial strain - lateral strain diagrams obtained from compression tests conducted on specimens of the mix with the fibers (SHCC) and of the same matrix without the fibers (HVFA). The compressive strength of the latter reaches 58MPa as maximum from the three tested samples and 50.37MPa as the mean average, but with the addition of fibers the strength decreases to a maximum obtained value of 52MPa and 45.08MPa as average. An increase by more than 30% in the axial deformation that corresponds to peak load as compared to the corresponding values for plain mixes occurred in the case of the matrix with the fibers (average strain at peak stress from -0.0032 for the HVFA to -0.0041 for the SHCC), as well as a stable descending branch. Lateral deformation is responsible for the rate of descend of the post peak branch. Fibers mobilized in the lateral direction can bridge cracks, transferring load and limiting lateral expansion of the cylinder.
under compression. This is evidenced by the restricted growth of post-peak lateral strain of the mix with fibers as compared to the plain concrete specimens. The stress-strain curves suggest that past the peak load, concrete with fibers behaves as if internally confined. The intensity of confinement is directly related to the materials' stress-strain behavior under tension.

3.3 Flexural Test – Four point bending

The four point bending test is preferably used for determining the material properties in tension since the central portion of the tested beam is under a uniform stress state with regards to the three point bending test where maximum stresses are concentrated only under the central load. This is more representative of the actual stress capacity as for brittle materials the flaws that could be located at any point and not directly under the central load, are related to crack initiation and tensile strength of the composite. The Load-Deflection diagrams obtained from the four point bending tests (Fig. 3) shows the increase in fracture energy due to the addition of fibers. Yield and ultimate moments at midspan are calculated from the applied total loads \( P \) according with \( M_{pb} = P \cdot a / 2 \), where \( a=100\text{mm} \). The mix without the fibers (HVFA) collapses suddenly past the yield load as the crack rapidly penetrates through the height of the cross section. In this type of test, collapse could occur at any point within the constant moment region where moments attain their maximum value, at some weak cross section, due to internal flaws and microcracking. The properties obtained through the 4-point test for this type of mix differ from the ones obtained from a normal concrete mix of the same compressive strength, owing to the absence of coarse aggregates that would give higher stiffness, and due to the denser microstructure obtained via the use of FA. In the case of the fiber reinforced composite (SHCC) multiple cracking is observed after the first crack, as if the section had been reinforced. The cracking extends outside the constant moment region indicating the strain hardening behavior of the material. The “yield” moment which corresponds to first cracking is used to determine the flexural cracking strength of the composites as it marks the end of the elastic branch. At the extreme tension fiber flexural stress is estimated from: \( f_{t,\text{cr}} = M / (b h^2 / 6) \). The load reached by the HVFA mix is 11.1 kN \( (M_y = M_u = 0.65\text{kNm}) \) while the yield load for SHCC is 24.37 kN \( (M_y = 1.22\text{kNm}) \) and ultimate total load is almost double, at 40.25 kN \( (M_u = 2.01\text{kNm}) \). Estimation of the flexural stress from the flexural moment is valid only for the linear part of behavior where the neutral axis is at specimen’s mid-height and the stress distribution may be assumed linear to the maximum (yield) strength at the top and bottom surfaces; thus calculation is exact up to yielding (the term corresponds to the end of the ascending branch of the load-displacement response). After yielding, for the SHCC mix fibers bridge the full depth of the tension region of the cross section up till the maximum stress, so the full cross section is participating in the transfer of load. In this context, the equation is no longer exact, but it is used to convert the load to a nominal stress value of the various specimens for comparison purposes. Therefore \( f_{t,\text{cr}} = 3.35\text{MPa} \) for the HVFA mix, whereas \( f_{t,y} = 7.32\text{MPa} \) and \( f_{t,u} = 12.06\text{MPa} \) for the SHCC flexural strength at the yield point and at the ultimate. The direct tensile strength may be estimated from this measured flexural tensile strength value using Eq. 1 as suggested by the Model Code [15]. Equation (1) is used to calculate the first cracking stress \( f_{t,\text{cr}} \) of the material in direct tension; \( h_b \) is the cross sectional depth of the flexural specimen. Cracking
strain ($\varepsilon_{t,cr}$) is obtained from the ratio of cracking stress $f_{t,cr}$ divided by the elastic modulus ($E$). The coefficient $A_{fl}$ relating flexural to tensile strength is 0.6 and the direct tensile strength is 2MPa for the HVFA and 4.39MPa at yield and 7.24MPa at maximum for the SHCC mix.

$$f_{t,cr} = A_{fl} \cdot f_{t,fl}, \quad \text{where}, \quad A_{fl} = \frac{a_{fl} \cdot h_{b}^{0.7}}{1 + a_{fl} \cdot h_{b}^{0.7}} = 0.6, \quad a_{fl} = 0.06$$

(1)

Figure 3: (a) Total Applied Load-deflection under point loads under 4-point bending of HVFA and SHCC mixes, (b) failure cracks for HVFA (c) failure cracks for SHCC

### 3.4 Material Properties

Average compressive strength from the experimental results is 50.37MPa for HVFA and 45.08MPa for the SHCC mix. Mean deviation is determined as 8MPa [15] and the characteristic values for the mixes is 42.37MPa and 37.08MPa respectively. Tensile strength can be determined from the compressive characteristic values [15] as per Equation 2. Based on Eq. 2 the tensile strength of the HVFA mix is 3.65MPa and for the SHCC is 3.36MPa.

for $C<50$ then $f_{cm}=0.3 \cdot (f_{ck})^{2/3}$

(2)

Compared to the values attained from the four point bending test and Eq. 1, the tensile strengths of the HVFA and SHCC concretes as estimated from compressive strength deviate by +83% ($f_{t,cr}$=2MPa from 4-point bending) and -30% ($f_{t,cr}$=4.39MPa from 4-point bending), respectively. This difference could affect the estimation of the capacity of a structural member if the wrong tensile strength is adopted. The assumed relationship between tensile to flexural and tensile to compressive strength of the material is obtained from equations calibrated with normal-weight concrete [15]. Based on the results listed above it seems that this calibration is no longer valid when mixes with high amounts of FA, lack of coarse aggregate, low water content with or without the use of fibers, such as those studied herein, are considered. The tensile strength of the structural members’ material controls both the SLS and the ULS design criteria of a structure, being used to determine numerous detailing parameters, e.g. thickness of the reinforcement cover, bond strength and development length requirements, and the shear strength. Thus, extending Modern Codes to include SHCC materials rides on the ability to adequately estimate their mechanical properties in direct and indirect tension. Tensile
properties estimated indirectly from the compressive or the flexural strength (Section 3.2 and 3.3) are much higher than the average yield tensile strength (≈2.5MPa) and ultimate tensile strength (≈2.8MPa) obtained from direct tension experiments (Section 3.1).

4 FINITE ELEMENT MODELING

The four point bending tests of the two mixes are modeled in Abaqus with the Concrete Damage Plasticity model. Modulus of Elasticity is taken equal to 20GPa for both mixes based on the uniaxial compression experimental results (based on the load and deformation values at 40% of the ultimate load). Poisson’s ratio is 0.3 and the density of the material is 2000kg/m$^3$. The supports of the specimen are a sliding and a rotational pin. Load is applied as vertical displacement. The dilation angle is chosen to be 50° based on the lateral deformation measurements of the uniaxial compression test. The volume/or size of the mesh is verified based on the fracture energy consumed in the cracks. A relation between the mesh size ($l_c$) and the materials’ tensile strain properties ($\varepsilon_1$, $\varepsilon_u$) (Eq. 3) should be adopted during simulation to eliminate mesh size effects on the results. Figure 4 (a) shows the correlation between different mesh sizes and the correctly defined $\varepsilon_1$ and $\varepsilon_u$ values as determined by the Model Code. With this procedure there is only a difference by 5.31% between the peak load results of the same model with different mesh sizes.

$$\varepsilon_1 = \varepsilon_{cr} + \frac{w_1}{l_c} \quad \text{and} \quad \varepsilon_u = \varepsilon_{cr} + \frac{w_u}{l_c} \quad (3)$$

Figure 4: (a) Load-Deflection Curves of 3D FE analysis of HVFA mix with different mesh size and tensile strength, (b) comparison of experimental results with FE analysis

The results from the FE simulation presented in Figure 4 (a) (greyscale) are calculated with a tensile strength of 3.65MPa as determined by the Model Code (Eqn. 2), based on the characteristic compressive strength. In the case of the HVFA mix this tensile strength value gave 50% higher total load value as compared with the experimental result (in the FE simulation ultimate load is 15kN, whereas in the experiment it is 11kN). This suggests that Eqn. 2 which was calibrated for normal concrete does not apply in the case of the mixes studied herein and tensile strength is in the order of $0.16\cdot(f_{ck})^{2/3}$ - almost half the value prescribed by the Model Code [15] for normal concrete. When the tensile strength of the HVFA composite incorporated in the FE analysis is taken equal to 2MPa, as calculated from the four point bending test, the ultimate load from the analysis reaches up to 12kN, a value very close to the experimental results’ average. In Figure 4(b) the experimental results from
the four point bending tests are compared to the Load-Deflection curves obtained from FE modeling with the two different tensile strengths, i.e., 2 and 3.65MPa estimated from the four-point bending and uniaxial compression tests, respectively. From this Figure it may also be concluded that the flexural stiffness of the experimental results is almost half the value obtained from the FE analysis. This is due to the Damage Plasticity Model used in the F.E. simulation that considers that elastic stiffness is the same both for the compressive and the tensile properties of the material, something unrealistic since the tensile and compressive properties in a mix consisting of multiple materials are not actually the same.

![Figure 5: Stress and strain results of FE modeling of the HVFA (mix without fibers) at the beginning of crack localization and at the last step of the analysis](image)

### Table 3: Tensile properties of HVFA mix for FE simulation

<table>
<thead>
<tr>
<th>Mesh size</th>
<th>( f_{ct} )</th>
<th>( w_{1} = G_F / f_{ctm} )</th>
<th>( w_{u} = 5 G_F/f_{ctm} )</th>
<th>( w_{1}/l_c )</th>
<th>( w_{u}/l_c )</th>
<th>( \varepsilon_{1} = \varepsilon_{cr} + \frac{w_{1}}{l_c} )</th>
<th>( \varepsilon_{u} = \varepsilon_{cr} + \frac{w_{u}}{l_c} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.0</td>
<td>0.0739</td>
<td>0.3695</td>
<td>0.00739</td>
<td>0.03695</td>
<td>0.00757</td>
<td>0.0371</td>
</tr>
<tr>
<td>B</td>
<td>3.65</td>
<td>0.0405</td>
<td>0.2025</td>
<td>0.00405</td>
<td>0.02025</td>
<td>0.00423</td>
<td>0.0204</td>
</tr>
</tbody>
</table>

In the case of FE modeling of the SHCC specimens, the tensile stress-strain curve used was that obtained from the uniaxial tension test. When these properties \( f_{ty} = 2.8\text{MPa}, \varepsilon_{ty} = 0.00014, f_{tu} = 3.1\text{MPa}, \varepsilon_{tu} = 0.008015 \) were used in the simulation, the resulting Load-Deflection curve gave a good approximation of the yield-point, but it could not simulate the full range of the strain hardening behavior up to the experimental ultimate load. The dimensions of the specimen’s cross section for the uniaxial test, the percentage and flaws’ distribution in the small cross section (25mmx50mm) as compared to the dimensions of the beams under four point bending (100mmx100mm) and the limitations of the gripping mechanism transferring stresses through the uniaxial tension test give false ultimate stress and strain values for the composite and this is obvious when using those parameters in numerical simulation. In order to find the proper material properties in tension, an inverse analysis of the four point bending tests was used based on the theoretical proposal made in [16] for three point bending tests. The inverse analysis gave the following tensile properties for the SHCC: \( f_{ty} = 3.564\text{MPa}, \)}
εty=0.00054, fyu=5.26MPa, εtu=0.01. In Figure 6 the Load-Deflection curves of the experiment is compared to the FE analysis with tensile properties based on the direct tensile test results (DT) and the inverse analysis (INV). The FE simulation with tensile properties from the inverse analysis shows good agreement with the experimental curves.

**Figure 6**: Load-Deflection curves of FE modeling of the SHCC (mix with fibers) and experimental results and strain and stress distribution at maximum load of INV FE analysis

### 12 CONCLUSIONS

- One of the most important products that may be used for a more ecological design of structures is fly ash; however, in order to better fulfill the overall sustainability design cement replacement by fly ash is accompanied with the addition of synthetic dispersed fibers so as to enhance the longevity of the structure as well as its resilience, deformation capacity, durability and overall resistance of the resulting structure to natural disasters such as earthquakes by optimizing the use of materials.

- The resulting materials may be considered as functionally graded in the sense that the CO2 footprint is substantially reduced while desirable structural properties such as ductility and resilience are achieved without an inordinate amount of confining steel–reinforcement; this is another synergistic effect towards improved sustainability objectives.

- In this study, the experimentally determined properties in uniaxial tension and compression of two mix designs with high volume cement replacement with fly ash, with and without the use of fibers, were used to simulate analytically, the behavior of those composites in flexure, by comparing FE modeling analysis results to the experimental load-deflection curves of small beams in flexure.

- Cement replacement by up to 60% with FA may yield composites that attain compressive strengths up 58MPa, while the addition of fibers in the same mix lowered the strength by as much as 6MPa. However, the addition of fibers increased by 30% the axial deformation associated with peak load, restrained lateral expansion at peak load and controlled the compression failure giving a stable postpeak descending branch.

- Tensile, flexural and compressive strength of a material as proposed by Model Code [15] are connected with equations calibrated with normal concrete. In the case of mixes with high amounts of FA, lack of coarse aggregate, low water content with or without the use of fibers it is shown that these calibrated empirical expressions do not apply. As tensile strength is a very important analysis parameter, procedures for its proper determination

-
must be incorporated into Modern Codes for these types of materials.

- Results obtained from direct tensile tests may underestimate the ultimate strength and strain capacity of the composite when this is placed in a 3-D state of stress. Flaws and size effects (in terms of possible voids during compaction for the estimation of stress from the specimens’ area, and the gauge length used in estimation of average strain from crack widths) may result in lower values than what may be supported in the presence of a more complex stress state. An inverse analysis of flexural tests is needed in order for the experimental results to coincide with the FE analysis.

REFERENCES

PAPER AS ADDITIVE IN CONCRETE MIXTURES FOR LOW RESISTANCE BLOCKS

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Key words: Waste, Concrete, Paper.

Abstract. The production of waste is an inevitable phenomenon which occurs daily and its proportions depend on the size of population and local economic development. The usage of recycled materials in civil constructions may be an important way to eliminate urban waste, being a way of reducing the costs of deposition and treatment, and also a way of minimizing the aggressive effects of this type of material on the environment. Thus, we can highlight the usage of paper in the production of concrete, since this material represents one of the greatest types of waste in the world. It can thus be reusable and, moreover, produce a concrete with greater benefits for the environment, mainly because of paper’s limited recyclability. This study aims to analyze the behavior of concrete with the addition of paper. Thus, behavior and properties of concrete made with different proportions of paper added to the mixture were analyzed, while also setting a standard mixture and analyzing differences between both concrete. A total of 56 specimens were produced with the same cement, aggregate and water amounts, whilst several specimens contained different quantities of cut and shredded paper. Slump tests were performed prior to the casting to determine the effects of the paper addition over workability. After the proper molding and curing, specimens were submitted to axial compressive strength tests at the ages of seven, fourteen, twenty-one, and twenty-eight days. Every concrete containing cut or shredded paper presented significantly loss of workability regardless of the amount added. Concretes with shredded paper presented loss of strength as the paper amount in the mixture was increased, while the addition of cut paper increased compressive strength in every quantity tested. It was also estimated the amount of paper allowed in 1 m³ of concrete based on the results, reaching up to almost 11 thousand A4 sheets. Based on the disclosed data, it is possible to emphasize the importance of producing low resistance concrete blocks with the use of recycled paper, as a way to reduce impacts caused by this type of waste on the environment, contributing to environmental construction.

1 INTRODUCTION

Concrete is the most used concrete material in the world¹. In Brazil, its usage can be up to 90% of all materials used in civil construction. Therefore, it is perceived an increase of studies and analysis regarding concrete made with the addition of other substances, not only to study its behaviour but also to provide some sort of help to the environment, dealing directly with
According to CEMPRE\textsuperscript{2}, Brazil’s urban solid residue production average is of 0.7 kg/hab.day, where in big cities such as São Paulo and Rio de Janeiro the rates can be up to 1.0 kg/hab.day. Based on that, it is estimated the daily disposal of a total of 140,000 tons of waste. From this total, 76\% end up in irregular dumps, without any kind of treatment, contributing to soil and water bodies contamination, and increase of diseases.

Pietrobelli\textsuperscript{3} mentions that the 3R theory, which stands for to reduce, to reutilize and to recycle, has been a permanent procedure, aiming to minimize this situation. Recycling, as every other process, may generate residue and most of times require great investments. Even with these restrictions, it is presented as the best solution.

On the other hand, Nascimento\textsuperscript{4} states that the 5R theory, which stands for to reduce, to reutilize, to recycle, to refuse and to rethink, is required for the processes of environmental education since it is a much practical on a daily basis. Considering the 5R, to rethink is to think about socio-environmental production processes; to refuse consists in avoiding excessive and unnecessary consumption, refusing products that may damage the environment; to reduce means the diminution of waste production; to reutilize is to provide a new utility for materials once considered useless; to recycle is the process of transforming used materials into something new and reusable.

Based on that, it is possible to think about utilizing paper in concrete mixtures, being the paper a kind of residue that is commonly found in great quantities in Brazil’s environment, which allows it to be somehow reused. Paper, unfortunately, has a limited recyclability, which means it can only be recycled a number of times. Adding paper in concrete mixtures presents a suitable destination for either recyclable or non recyclable paper, removing this type of material definitely from the environment.

2 PAPER RECYCLING AND THE ENVIRONMENT

Brazil is one of the greatest producers of paper and other cellulose derivatives, being globally competitive. Besides providing supply for the internal market, the country even exports products, mostly to Latin America, Europe and North America, which sums up to 4\% of all national exportations.

BRACELPA\textsuperscript{5} states that Brazil is the 11\textsuperscript{th} greatest worldwide paper producer, producing 9.4 millions of tons of paper only in 2009. With large productions, recycling sectors tend to increase as well, which Brazil is also a reference. The rates of recovered paper over produced paper have been growing on the past years: in 1990, 36.5\% of all produced paper was recovered and recycled, in 2008 the rate raised to 43.7\%. BRACELPA\textsuperscript{5} also mentions that companies have recycled 70\% of all cardboard boxes used in other activities.

Recycling process can be defined as the transformation of waste into new products, allowing disposed materials to be inserted into the industry, reducing consumption of raw materials, energy and environmental pollution.

According to Embalixo\textsuperscript{6}, paper takes three months to decompose itself in nature and each ton of recycled paper frees three cubic meters of space in sanitary landfills. Also, Oliveira and Carvalho\textsuperscript{7} assert that a ton of recycled paper prevents twenty trees from being cut down. However, unfortunately, it is believed that only 26\% of all brazilians recycle, and a great part
of all produced paper ends up in improper areas and on the streets, contributing for environment degradation.

BRACELPA\textsuperscript{5} also exposes that paper recycling presents some advantages in the productive chain which evolves the practice. It creates job opportunities, warming the local economy. Under the economic point of view, recycling reduces production costs and raw material consumption. It reduces the volume of disposed materials, which is of great importance since brazilian landfills are commonly saturated. Besides that, due to its calorific power, paper can be used in energy recycling, and such practice may gain more importance in a near future.

Even though recycling accounts for uncountable benefits to the environment and the economy, there some obstacles that arise along the process: community’s negligibility over the matter, lack of proper incentive from organizations, insufficient funds, and unskilled labor.

In addition, it is important to highlight the fact that the paper loses its potential each time it is recycled, since cellulose fibers lengths are significantly reduced within the process, leading to a low lifespan, which means it will never be the same as the primary batch. Such problem, however, is not observed in the recycling process of other materials such as aluminum, which presents a high recyclability index, allowing it to be reused as many times as necessary without losing its original properties.

2.1 PAPER USAGE IN CIVIL CONSTRUCTION

Even though paper can not be recycled indefinitely, its presence in the making of new products is growing everyday. In civil construction, paper residue has been used in some construction materials, such as ecological tiles and compressed soil blocks.

Yoshimura, Yoshimura and Wiebeck\textsuperscript{8} assert that tiles made from residue, paper in particular, are called ecological tiles (eco-tiles), and possess the same basic characteristics of conventional tiles: protect buildings from weather conditions. Eco-tiles conventionally commercialized in Brazil run against the market of fibrocement tiles, having lightweight-ness and flexibility as main advantages. Eco-tiles also help to reduce noise, working as an acoustic isolator, and contribute to lower temperature indoors. Besides supporting paper recycling, tile production also stimulates local businesses.

The authors\textsuperscript{8} also mentions that regarding environmental impact, it not expected that eco-tiles emit any aggressive substance, since it is a material made from paper, asphalt and paint, and its usage is restricted to civil construction. It is important to highlight that the production of 1,000 m\textsuperscript{2} of eco-tiles consume an average of 3,800 kg of paper residue, which is an outstanding performance environmentally, contributing to the creation of new paper collectors’ cooperatives.

According to Batista\textsuperscript{9}, compressed soil bricks with paper in their composition is a great option as well as eco-tiles. The idea came from a research made by professor Márcio Busson, from Universidade de Brasília (FAU/UNB), where compressed blocks were made from a mixture of soil and fibers from kraft paper, originated from cement sacks. The brick was developed by mixing soil, paper fiber and some cement to enhance its properties, and then compressed. The brick was named “Krafterra” (Kraftsoil). Cement sacks are a common type of residue found in civil construction and are not often incorporated in any production process, emphasizing the viability of this recycling practice as an alternative construction material.
Batista\textsuperscript{9} asserts that the technique utilized in Kraftsoil bricks is quite similar to the making of compressed soil bricks (CSB). CSBs are mostly used in the construction of popular homes, since it is more affordable and is created from soil and other additives, where its most common composition is 88\% of raw dirt and 12\% of cement. The main difference between CSB and Kraftsoil is that 6\% of dirt is replaced with paper fiber, lowering costs.

According to Batista\textsuperscript{9}, impact resistance and fire resistance testes were performed and results surprised the authors. Comparing CSB with Kraftsoil bricks, the bricks containing paper in its composition performed better in fire resistance, qualifying as a fire-safe material. Also, the fibers allowed the blocks to be more flexible. Kraftsoil bricks, however, were not effective when tested for water absorption, presenting results 7.6\% higher than regular CSBs, therefore being susceptible to infiltrations. To solve this problem, the authors added a kind of sap to enhance water resistance. Despite the good results in some tests, the authors believe that it is too early to fully utilize this sort of technology in civil construction. The next research step is to evaluate the Kraftsoil behavior in an experimental construction site.

3 RESEARCH METODOLOGY

Besides theoretical studies, the research procedures consisted in two areas: confection of concrete specimens and concrete testing.

3.1 Paper preparation

Seeking understanding over Portland cement concrete’s behavior when paper is added to the mixture and its usage in low resistance blocks, several tests were performed following brazilian standards (NBRs). The results were obtained by comparing a standard concrete mixture, and the same mixture with the addition of paper.

Mixing proportions of 1:2:3 (cement, sand and rock, respectively), based on volume, and water-cement ratio of 0.6 were adopted to make standard specimens. This proportion has a theoretical resistance of 20 MPa.

The addition of paper was performed in two different ways. Initially, the paper was cut in little pieces and added dry into the concrete, with amounts of 20\%, 40\% and 60\% of the cement volume. To cut the paper, an office paper disposal machine (figure 1) was used. After passing through the machine, the paper was cut in smaller pieces and was dosed according to the desired volume (figure 2).

![Figure 1: Office paper cutting machine.](image1)

![Figure 2: Cut paper.](image2)
Shredded paper was also considered as an alternative. To shred the paper, the same initial process was performed, but a shredder (regular blender) and water were also necessary (figure 3). The result was a sort of paste due to the great amount of water incorporated (figure 4). Thereafter, water excess was removed and the desired volume of paper was added to the concrete (figure 5). Proportions used were the same as used before: 20%, 40% and 60% of cement volume. Even though the paper was almost dry, it still had a small amount of moisture in it. The moisture content of the paper was measured and the total water volume in the shredded paper was calculated in order to maintain the exact desired water-cement ratio of 0.6.

3.2 Specimens preparation

A total of 8 specimens for each type of concrete were prepared, totaling 56. A rupture calendar was prepared, in order to determine dates which specimens were going to be submitted to compressive strength testing. Rupture dates of 7, 14, 21 and 28 days were chosen, while testing two specimens at a time. The specimens were cylindrical, with a diameter of 10 cm and 20 cm of height.

The brazilian standard NBR 5738/2015 regulates the correct procedures to mold and prepare specimens. Figures 6 and 7 illustrate the concrete confection using both cut and shredded paper,
as well as the other materials.

Figure 6: Concrete with cut paper.  

Figure 7: Concrete with shredded paper.

Having prepared the concrete mixtures, the specimens were correctly molded and compacted as recommends the standard. Figure 8 illustrates the process. The molding is performed by filling in two layers of equal volume, while compacting each layer individually by stroking the layers twelve times with a metal rod. The excess is leveled with the top of the mold.

Figure 8: Specimens being prepared.

After 24 hours, the specimens were unmolded and submitted to submerged curing until the rupture date.

3.3 Slump test

To determine workability and consistency, produced concretes were also submitted to slump tests. The standard that regulates the procedure is the NBR 67/1998, and the consistency is determined by a slump cone. The cone is filled with fresh concrete in three layers of equal volume, and each layer must be stroked 25 times with a metal rod. After the cone has been filled with concrete, excess is cut off level with the top of the cone and the cone is then removed. The distance that the concrete falls or slumps is measured. Figure 9 illustrates the process.
3.4 Compressive strength testing

The test that measures compressive strength of concrete specimens is regulated by the brazilian standard NBR 5739/2007. In order to correctly submit the specimens to axial compressive strength, it is necessary treat the surface of the cylinders in order to prevent the existence of forces other than axial. The process is called capping, and is made with a fine layer of mortar. With the top leveled, the tensions formed by the compressive strength are purely axial. Having successfully applied the leveling layer and let it cure, the specimens are placed on the appropriate equipment to initiate the test. Figures 10 and 11 present the process.

4 RESULTS AND ANALYSIS

4.1 Slump testing

Tables 1 and 2 provide results obtained from consistency tests for all types of concrete produced with the study.

Table 1: Slump results - Cut paper.

<table>
<thead>
<tr>
<th>Cut paper %</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>120</td>
</tr>
<tr>
<td>20%</td>
<td>30</td>
</tr>
<tr>
<td>40%</td>
<td>15</td>
</tr>
<tr>
<td>60%</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 2: Slump results – Shredded paper.

<table>
<thead>
<tr>
<th>Shredded Paper %</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>120</td>
</tr>
<tr>
<td>20%</td>
<td>5</td>
</tr>
<tr>
<td>40%</td>
<td>0</td>
</tr>
<tr>
<td>60%</td>
<td>0</td>
</tr>
</tbody>
</table>
Analyzing both tables, it is possible to affirm that paper addition causes a loss of workability when compared to the standard concrete, without any paper addition. Also, as the paper amount was increased, consistency also increased, reaching such a point concrete would slump slightly or not at all. This means that the cement paste volume reduced when related to the aggregate amount.

4.2 Compressive strength results

To obtain compressive strength results, specimens were submitted to rupture testing at the ages of 7, 14, 21 and 28 days. Two specimens were tested at a time, and the greatest value between both results was collected.

4.2.1 Standard concrete

Compressive strength results to specimens made with standard concrete mixture, without any paper addition, are presented on table 3.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard Concrete - 1:2:3</td>
</tr>
<tr>
<td>7</td>
<td>12.89</td>
</tr>
<tr>
<td>14</td>
<td>18.91</td>
</tr>
<tr>
<td>21</td>
<td>19.84</td>
</tr>
<tr>
<td>28</td>
<td>20.22</td>
</tr>
</tbody>
</table>

4.2.2 Concrete with cut paper

Compressive strength results to specimens made with concrete with the addition of cut paper are presented on table 4.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Paper amount: 20%  Paper amount: 40%  Paper amount: 60%</td>
</tr>
<tr>
<td>7</td>
<td>14.78  18.65  17.99</td>
</tr>
<tr>
<td>14</td>
<td>19.29  21.81  20.60</td>
</tr>
<tr>
<td>21</td>
<td>22.54  26.96  25.93</td>
</tr>
<tr>
<td>28</td>
<td>24.75  27.01  27.26</td>
</tr>
</tbody>
</table>
The following graphic presented on figure 12 shows a comparison between compressive strength results obtained from testing standard specimens and ones containing cut paper in the mixture.

![Figure 12: Testing results – Cut paper – Days x MPa.](image)

Based on figure 12, it is possible to perceive that cut paper addition in the mixture improved significantly the specimens’ resistance. This was observed regardless of the concrete’s age. Adding 20% of cement’s volume as cut paper increased compressive strength at 28 days in 22.40%, while the addition of 40% and 60% improved resistance in 33.60% and 34.80%, respectively. Such improvement may be due to the fact that paper is an extremely absorbent material and if the water-cement ratio is reduced, a significantly increase in compressive strength occurs. This fact can be proved when slump tests results are taken into consideration. After adding paper, a considerable loss of workability was observed, which means the water-cement ratio was reduced. In order to bypass this issue, other additives may be used, such as plasticizers admixtures, or the water-ratio may be recalculated.

### 4.2.3 Concrete with shredded paper

Compressive strength results of concretes containing 20%, 40% and 60% of cement’s volume as shredded paper are presented on table 5.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Paper amount: 20%</th>
<th>Paper amount: 40%</th>
<th>Paper amount: 60%</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>14.11</td>
<td>11.73</td>
<td>10.09</td>
</tr>
<tr>
<td>14</td>
<td>18.05</td>
<td>16.28</td>
<td>11.98</td>
</tr>
<tr>
<td>21</td>
<td>20.60</td>
<td>18.91</td>
<td>13.21</td>
</tr>
<tr>
<td>28</td>
<td>23.67</td>
<td>19.17</td>
<td>16.20</td>
</tr>
</tbody>
</table>

Figure 13 displays a comparison between results obtained from testing standard specimens and ones containing shredded paper in the mixture.
Shredded paper affected concretes’ strength significantly, but their behavior differs from what was observed previously. While cut paper improved compressive strength values regardless of the added amount, shredded paper improved strength until a certain limit and, after that, it weakened the specimens. At 28 days, 20% of paper in the mixture caused a 17% increase in compressive strength, while 40% and 60% weakened strength in 5% and almost 20%, respectively.

Comparing slump test results, it is possible to infer that shredded paper is greatly absorbent, since it reduced slump results to 0. With 20% of paper, compressive strength was still acceptable, since it was increased, whereas other amounts lowered compressive strength values. This leads to assume that after a certain amount of shredded paper in the mixture, the water-cement ratio dropped to a point that hydration of all cement particles was not possible, weakening the mixture. Considering that quantities greater than 20% caused such strength reduction, it is possible to affirm that adding 20% of the cement’s volume as shredded paper is close to the ideal amount.

4.2.4 Cut paper vs. Shredded paper

The following table 6 displays strength results for both cut paper and shredded paper.

<table>
<thead>
<tr>
<th>Amount</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cut paper</td>
</tr>
<tr>
<td>20%</td>
<td>24.75</td>
</tr>
<tr>
<td>40%</td>
<td>27.01</td>
</tr>
<tr>
<td>60%</td>
<td>27.26</td>
</tr>
</tbody>
</table>

Shredded paper is greatly absorbent, since it reduced slump results to 0.

The strength’s improvement caused by 20% of shredded paper was not as significant as the one observed with cut paper additions possibly due to the fact that the shredded presents some sort of agglomeration. This agglomeration improves the size of paper particles, causing paper to become like a granular material, which may reduce strength since it is weaker compared to other granular materials in the mixture.

5 CONCRETE, PAPER AND ENVIRONMENTAL IMPACTS

As mentioned earlier, one of the main objectives of this study is to provide a suitable way to reduce environmental impacts caused by paper waste. While preparing paper to be used in the
experiment, as 3.1 states, it was possible to measure overall paper quantity used in each mixture. The paper used in the experiment was gathered from many offices nearby (A4 sheets, 75 g/m²). Table 7 displays paper quantity used to produce 12 specimens of concrete, which is equivalent to 0.019 m³.

<table>
<thead>
<tr>
<th>Number of sheets</th>
<th>Quantity</th>
<th>Cut paper</th>
<th>Shredded Paper</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20%</td>
<td>31.4</td>
<td>68.4</td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td>62.8</td>
<td>136.8</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>94.2</td>
<td>200</td>
</tr>
</tbody>
</table>

Table 7: Number of A4 (75 g/m²) paper sheets to produce 0.019 m³ of concrete.

Besides that, it is important to predict how many paper sheets would be required to produce 1 m³ of concrete using each presented method. The required quantities are shown on table 8.

<table>
<thead>
<tr>
<th>Number of sheets</th>
<th>Quantity</th>
<th>Cut paper</th>
<th>Shredded Paper</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20%</td>
<td>1,652.6</td>
<td>3,600.0</td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td>3,305.2</td>
<td>7,200.0</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>4,957.8</td>
<td>10,800.0</td>
</tr>
</tbody>
</table>

Table 8: Number of A4 (75 g/m²) paper sheets to produce 1 m³ of concrete.

Shredded paper has a bulk density of 203.3 kg/m³, while cut paper has less than half of that value: 93.3 kg/m³. Based on the tables above, it is possible to infer that shredded paper is the best option when comparing both alternatives, since it allows the addition of more than twice the amount of paper, in weight, while retaining the same volume. Even though shredded paper addition over 40% lowered compressive strength, the results were still acceptable to produce low resistance concrete blocks.

6 CONCLUSIONS

This study aimed to analyze compressive strength behavior in order to determine whether or not it is viable to achieve the proposed objective, which is the production of low resistance concrete blocks with paper as additive. Studies presented above proved that paper addition is completely viable, although it requires further studies to better adequate paper proportions in the mixtures, or even a transformed form other than cut or shredded.

While comparing both alternatives presented, cut and shredded, shredded paper in great quantities caused a considerable loss of compressive strength, but values obtained were all greater than 15 MPa, leading to infer that it indeed can be used in the production of non-structural low resistance concrete blocks. Simultaneously, the usage of shredded paper doubles the amount of paper inserted in the mixture, therefore, removing twice the paper once thrown
on the environment.

Also, besides regular concrete blocks, the produced low resistance concrete can be used in a few different ways, such as paving blocks, sealing blocks, flooring base or curb elements.

As suggestions for future studies include better adequacy of paper amounts in different mixtures and the element behavior when analyzing other properties, such as lifespan, thermal and acoustic isolation, fire resistance and permeability.

REFERENCES

POSSIBLE REUSING OF HOUSEHOLD CERAMIC WASTES AS MINERAL ADMIXTURES IN ECOLOGICAL CEMENT/CONCRETE

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Key Words: X-ray method; Chemical properties; Strength; Pozzolanic activity and Frattini test

Abstract: Both the Ceramic wastes and the pollution of the cement industry can cause strong damage to the environment and the sustainable development. In the present study, the pozzolanic activity of household ceramic waste powder was investigated by SAI test and Frattini Test; the possibility of the partial substitution of Portland cement blended with ceramic waste powder was analyzed. The results indicate that the compressive strengths of mortar containing ceramic waste at both early age and after 28 days were generally increased as the ceramic waste ratio increased up to at least 15% replacement, and the ground ceramic waste show clear pozzolanic activity.

1 INTRODUCTION

Since 2002, the output of the cement industry in China has been larger than any other country in the world. It is known that around 1 ton of carbon dioxide is emitted for each ton of cement production (Fig.1 (a)). The total cement output was about 2.48 billion tons in 2014, and this trend will continue due to economic growth. Consequently, it is important to take effective measures to reduce the carbon dioxide pollution caused by the cement industry.

Fig.1. Pollution of cement industry and ceramic waste on the environment
(a. Cement industry in China; b. Household ceramic waste)
At the meantime, China is the world’s leading manufacturer of ceramics; as a result, more than 100 million tons of ceramic wastes are produced each year in China. The ceramic waste is a serious problem, as there is between 10% and 20% waste from the processing procedures. Only in Guangdong province in south China near Hong Kong, the ceramic waste and waste sludge are about 10 million tons each year. The disposal of this waste not only occupies large areas of land, but can also severely pollute the environment (Fig.1 (b)). Hence, correct treatment of ceramic waste is necessary to preserve the environmental degradation. The reuse of such waste cannot only protect the environment, but also offers a great advantage in partial replacement of cement in concrete for example. As mentioned, more than 2 billion tons of cement is produced annually in China, so 5% replacement of cement would require some 10 million tons of pozzolanic materials and reduce the similar amount of CO₂ emission. That may show significant advantage to both the environment and the sustainable development of the construction industry.

There have been several previous studies that have investigated the use of powdered igneous or metamorphic rocks, as pozzolanic material [1-3] and the use of ceramic waste as aggregate in concrete [4-5]. The investigation on the ceramic roof tile wastes and clay bricks as pozzolanic admixture has been also been conducted [6-7].

In this paper, our attention is focused on the household ceramic (TC) waste from China. In reviewing the literature, few data was found on this topic. The aim of this study is to evaluate the possibility of partial substitution of Portland cement with household waste to produce environmental friendly cement or concrete.

The powdered ceramic waste was used to replace cement in the ratio of 5%, 15% and 25%. The chemical composition, the flowability and crystal structures, especially the pozzolanic activity of the ceramic waste powder is investigated. The results of the study are highly significant for the sustainable development of construction materials as they indicate that the ceramic waste can be reused in partial substitution of Portland cement, and the market for the recycled new pozzolanic materials could be enormous.

2 EXPERIMENT

The experimental program had the following three main aims:
(a) Investigation of the workability of mortars containing household ceramic wastes in comparison with normal cement mortar and cement mortar with fly ash addition.
(b) XRD analysis of ceramic waste powder and its comparison with normal cement and fly ash.
(c) Evaluation of the pozzolanic activity in terms of the Frattini test and SAI test, compressive strength of samples containing household ceramic wastes is compared with that of normal cement and cement mortar with fly ash addition.

2.1 Materials

The household ceramic wastes were obtained from south China. The mix design is listed in Table 1. Cement CEM I 42.5 (CEM-I) with a specific surface area of 330 m²/kg, fly ash (FA) with the specific surface area of 516 m²/kg and household ceramic waste powder with a specific surface area 530 m²/kg were used. The ceramic wastes replace 5%, 15% and 25% of cement. The sand used conformed to ISO 679: 2009 [8]. The water/binder ratio was 0.5.
Table 1 Mix-design of mortars

<table>
<thead>
<tr>
<th></th>
<th>Cement (g)</th>
<th>TC/FA (g)</th>
<th>Water (g)</th>
<th>ISO sand (g)</th>
<th>W/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM-I</td>
<td>450</td>
<td>0</td>
<td>225</td>
<td>1350</td>
<td>0.5</td>
</tr>
<tr>
<td>TC/FA5</td>
<td>427.5</td>
<td>22.5</td>
<td>225</td>
<td>1350</td>
<td>0.5</td>
</tr>
<tr>
<td>TC/FA15</td>
<td>382.5</td>
<td>67.5</td>
<td>225</td>
<td>1350</td>
<td>0.5</td>
</tr>
<tr>
<td>TC/FA25</td>
<td>337.5</td>
<td>112.5</td>
<td>225</td>
<td>1350</td>
<td>0.5</td>
</tr>
</tbody>
</table>

2.2 Grain size analysis

Ceramic wastes were ground by a ball mill for 24 hours. The particle size distribution of cement, fly ash and ceramic waste powder (measured by laser granulometry) is illustrated in Fig. 2. It can be seen that for any particular size the percentage of ceramic waste powder and fly ash was greater than that of cement. The most part (almost 100%) of the ceramic waste powder and fly ash was smaller than 40 μm.

![Fig.2. Particle size distribution of materials used](image)

2.3 Chemical analysis

The alkali environment of mortar or concrete matrix is an important factor that affects the durability. The aim of the present investigation was to use the recycled ceramic powder as a part of the mortar or concrete. Hence, the matrix should have a similar alkali ambient to normal cement. The lower boundary of pH value of ceramic waste powder was limited, in order to avoid the reduction of the general alkali value of the concrete matrix. The evaluation of the pH-value of cement, ceramic waste powder and fly ash solution was carried out (see Table 2). It can be seen that pH-value of ceramic waste powder solution is about 10.4 and could be suitable for using in cement or concrete matrix.

Table 2 Comparison of the pH-values

<table>
<thead>
<tr>
<th></th>
<th>CEM</th>
<th>TC</th>
<th>FA</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH value</td>
<td>12.46</td>
<td>10.41</td>
<td>11.94</td>
</tr>
</tbody>
</table>

The chemical compositions of CEM, FA and TC were analyzed using X-ray fluorescence and results are presented in Table 3.
Table 3 Comparison of the chemical composition of CEM, FA and TC

<table>
<thead>
<tr>
<th></th>
<th>CEM-I(%)</th>
<th>TC(%)</th>
<th>FA(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>66.9</td>
<td>0.57</td>
<td>8.25</td>
</tr>
<tr>
<td>SiO₂</td>
<td>16.8</td>
<td>71.03</td>
<td>49.1</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.36</td>
<td>18.22</td>
<td>26.2</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>4.19</td>
<td>2.59</td>
<td>10</td>
</tr>
<tr>
<td>SO₃</td>
<td>3.89</td>
<td>-</td>
<td>0.533</td>
</tr>
<tr>
<td>MgO</td>
<td>1.77</td>
<td>0.33</td>
<td>1.23</td>
</tr>
<tr>
<td>K₂O</td>
<td>1.12</td>
<td>4.66</td>
<td>1.58</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.311</td>
<td>-</td>
<td>1.88</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.263</td>
<td>1.03</td>
<td>0.641</td>
</tr>
<tr>
<td>SrO</td>
<td>0.253</td>
<td>-</td>
<td>0.198</td>
</tr>
<tr>
<td>ZrO₂</td>
<td>0.0646</td>
<td>0.28</td>
<td>0.1</td>
</tr>
<tr>
<td>ZnO</td>
<td>0.0429</td>
<td>0.2</td>
<td>0.0361</td>
</tr>
<tr>
<td>Cl</td>
<td>0.0339</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PbO</td>
<td>-</td>
<td>0.58</td>
<td>-</td>
</tr>
<tr>
<td>CdO</td>
<td>-</td>
<td>0.18</td>
<td>-</td>
</tr>
<tr>
<td>Rb₂O</td>
<td>-</td>
<td>0.11</td>
<td>-</td>
</tr>
<tr>
<td>BaO</td>
<td>-</td>
<td>-</td>
<td>0.157</td>
</tr>
<tr>
<td>Sum</td>
<td>99.9984</td>
<td>99.78</td>
<td>99.9051</td>
</tr>
</tbody>
</table>

The main chemical components of ceramic waste powder are similar to those of FA and consist mainly of SiO₂ and Al₂O₃, and they are expected to contribute to the pozzolanic activity of ceramic waste powder. The high content of potassium oxide (K₂O) may have a negative effect and the compressive strength of the samples may be reduced.

2.4 Workability of different mixtures

Table 4 illustrates the flowability of mortars with different ratios of ceramic waste powder and fly ash substitutions.

Table 4 Flowability of different fresh mortars

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CEM-I</th>
<th>TC5</th>
<th>TC15</th>
<th>TC25</th>
<th>FA5</th>
<th>FA15</th>
<th>FA25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flowability(mm)</td>
<td>176.5</td>
<td>175</td>
<td>189</td>
<td>188</td>
<td>177</td>
<td>184</td>
<td>189</td>
</tr>
</tbody>
</table>

From Table 4, it can be seen that the ceramic waste powder does not exert negative influence on the workability of the mortar if the substitution ratio is lower than 25%.

2.5 XRD analysis

The X-ray diffraction (XRD) testing of cement, fly ash and ceramic waste is carried out to evaluate and compare the crystal structures and amorphous behavior of the materials. The results are presented in Figs. 3.
CEM in Fig.3a shows very slight crystalline behavior. The peak intensity is about 250 counts. The calcium magnesium aluminum is the main crystalline forms detected while calcium silicate is the minor compound. Fig.3b shows that quartz of low degree crystallinity is the main compound with synthesized mullite and sillimanite as minor compounds in the fly ash. Fig.3c reflects that quartz of low degree crystal is the main compound detected in the test accompanied with calcium aluminum oxide and synthesized berlinite as the minor compounds in the ceramic waste powder. The low
average intensity of TC shows low crystalline degree. Compared to cement and fly ash, it can be seen that ceramic waste powder shows few higher peak intensities due to the slightly crystalline structure (Fig. 3c). It is noted that the average crystalline degree of TC was similar to that of fly ash. The ceramic waste also illustrates clear amorphous structure, which indicates that the crushed household ceramics waste could be used as both pozzolanic material and as fine filler.

2.6 Pozzolanic activity

In order to evaluate the pozzolanic activity, the Frattini test and SAI test were conducted [9-13]. Studies indicate that the Frattini test (direct) and the strength activity index (indirect) are found to be the most accurate and reliable methods to assess pozzolanic reaction of calcined clays to be used in blended cements[11-12], when two direct tests (Frattini and saturated lime) and two indirect tests (strength activity index and electrical conductivity) were evaluated.

In addition, Malquori[13] has suggested that an evaluation of pozzolanic materials for purpose of their addition to Portland cement must be based on two factors: (1) the mechanical strength of mortars and concretes made with a Portland pozzolan mixture, and (2) the reduction of free calcium hydroxide in the hardened pozzolanic cement.

Therefore, in this research work the Frattini test and the SAI test were used to evaluate the pozzolanic activity of ceramics blended cement.

2.6.1 Strength activity index (SAI) of different mortars

The most important quality for pozzolanic activity is seen in the compressive strength. In order to evaluate the compressive strength of blended cement, the concept of Strength Activity Index (SAI) is used. SAI is defined as the ratio of the compressive strength of cement with additional waste powder to the compressive strength of cement without any addition [14–17]. The ceramic waste powder may show pozzolanic activity, if SAI is greater than 0.65[14], 0.8[16] or 0.75[17] with cement replacement of 30%, 20% and 30 after 28 days.

Prismatic specimens of 40 mm x 40 mm x 160 mm were cast and wet cured at 20°C [14-15]. Results of compressive strengths and SAI values of various mortars at different ages are listed in Tables 5 and 6.

### Table 5 Comparison of compressive strengths of various mortars at different ages

<table>
<thead>
<tr>
<th></th>
<th>CEM-I</th>
<th>TC5</th>
<th>TC15</th>
<th>TC25</th>
<th>FA5</th>
<th>FA15</th>
<th>FA25</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1d</strong></td>
<td>8.95</td>
<td>6.31</td>
<td>9.92</td>
<td>10.64</td>
<td>12.10</td>
<td>9.73</td>
<td>8.50</td>
</tr>
<tr>
<td><strong>7d</strong></td>
<td>37.82</td>
<td>36.07</td>
<td>32.79</td>
<td>30.66</td>
<td>36.99</td>
<td>32.64</td>
<td>28.13</td>
</tr>
<tr>
<td><strong>28d</strong></td>
<td>45.57</td>
<td>46.52</td>
<td>49.57</td>
<td>44.06</td>
<td>49.55</td>
<td>45.57</td>
<td>38.18</td>
</tr>
</tbody>
</table>

### Table 6 Comparison of SAI values of various mortars at different ages

<table>
<thead>
<tr>
<th></th>
<th>CEM-I</th>
<th>TC5</th>
<th>TC15</th>
<th>TC25</th>
<th>FA5</th>
<th>FA15</th>
<th>FA25</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1d</strong></td>
<td>1</td>
<td>0.71</td>
<td>1.11</td>
<td>1.19</td>
<td>1.35</td>
<td>1.09</td>
<td>0.95</td>
</tr>
<tr>
<td><strong>7d</strong></td>
<td>1</td>
<td>0.95</td>
<td>0.87</td>
<td>0.81</td>
<td>0.98</td>
<td>0.86</td>
<td>0.74</td>
</tr>
<tr>
<td><strong>28d</strong></td>
<td>1</td>
<td>1.02</td>
<td>1.09</td>
<td>0.97</td>
<td>1.09</td>
<td>1</td>
<td>0.84</td>
</tr>
</tbody>
</table>
In comparison with the CEM samples without waste powder, it can be seen that: The compressive strength of the samples with 15% and 25% of ceramic waste powder increased 11% and 19% after 1 day, respectively.

After 28 days, the compressive strength of the samples with the mix ratio of 5% and 15% of ceramic waste powder increased slightly, whereas TC25 decreased slightly (3.3%). This implies that 15% of TC could be a possible upper bound for an effective substitution.

- The compressive strength of the samples decreased with the increasing of waste powder and fly ash after 7 days.
- All the SAI values of TC5, TC15 and TC25 samples without heat treatment are higher than 0.8 and the values of FA, indicating that ceramic waste powders show good pozzolanic activity, which could be very cost-efficient for industrial use.

2.6.2 Frattini Test

The Frattini test was conducted after 8 days curing at 40°C according to the EN 196-5 2011[18]. Results are presented as a hyperbolic curve related to the calcium ion [CaO] concentration versus hydroxyl ion [OH⁻] concentration (Fig.4).

![Fig.4. Frattini test results for blended cement containing TC and FA powder](image)

The interaction relationship of [CaO] and [OH⁻] concentrations expresses the lime solubility curve and divides the [CaO] - [OH⁻] domain into an upper pozzolanic inactive region and a lower pozzolanic active region. The theoretical maximum [CaO] [19] concentration can be calculated according to Eqn. (1)

$$Max\ [CaO] = \frac{350}{([OH^{-}]-15)}$$  \hspace{1cm} (1)

The [OH⁻] of various mortars can be compared with the Max [CaO] and the results quantified as the difference between the two values, and they are expressed as the reduction of the Max [CaO]% illustrated in Table 7.
Table 7 Difference of standard calcium hydroxide concentration to the measured concentration

<table>
<thead>
<tr>
<th>Number</th>
<th>[OH] mmol/L</th>
<th>[CaO] mmol/L</th>
<th>Theoretical max[CaO] mmol/L</th>
<th>[CaO] reduction %</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC5</td>
<td>53.93</td>
<td>5.37</td>
<td>8.99</td>
<td>40.3</td>
</tr>
<tr>
<td>TC15</td>
<td>53.07</td>
<td>5.33</td>
<td>9.19</td>
<td>42</td>
</tr>
<tr>
<td>TC25</td>
<td>47.29</td>
<td>4.74</td>
<td>10.84</td>
<td>56.27</td>
</tr>
<tr>
<td>FA5</td>
<td>49.86</td>
<td>5.81</td>
<td>10.04</td>
<td>42.13</td>
</tr>
<tr>
<td>FA15</td>
<td>43.66</td>
<td>5.06</td>
<td>12.21</td>
<td>58.64</td>
</tr>
<tr>
<td>FA25</td>
<td>41.52</td>
<td>3.59</td>
<td>13.2</td>
<td>72.8</td>
</tr>
</tbody>
</table>

2.6.3 Comparison of SAI test and Frattini Test

For evaluation of the pozzolanic activity of the waste material, it is important to consider the experimental method used. The Frattini test and the SAI were found to be the most accurate and reliable methods to assess pozzolanic reaction of various materials in blended cements.

The correlation between measured pozzolanic activity of six test materials using the Frattini test and the strength activity index test is illustrated in Fig. 5.

It can be seen that there is a significant correlation ($R^2 = 0.78$) between the Frattini test results and the SAI results.

3 CONCLUSIONS

Mortars with Portland cement, fly ash blended cement, the household ceramic waste powder with cement substitution ratios of 5%, 15% and 25% were experimentally investigated. The pozzolanic activity is an important precondition for recycling of the ceramic waste. The present investigation of ceramic waste powder led to the following conclusions:

- The ceramic waste powder did not have a negative influence on the workability of the mortar for substitution ratio of cement up to 25%.
- Low degree crystalline quartz is the main compound of household ceramic waste.
Yining Ding, Huiwen Dong, Yongchao Zhang and Cecilia Azevedo

powder, and the ceramic waste shows amorphous structure.

- The compressive strength of TC 5 and TC 15 were higher than those of TC 25 after 7 and 28 days. This implies that 15% of TC could be a possible upper bound for an effective substitution ratio of cement.

- All the SAI values of TC 5, TC 15 and TC 25 samples without heat treatment were higher than 0.8 after 7 days.

- The results of Frattini test can be quantified where [OH⁻] is in the range 40-55 mmol/l.

- The ceramic waste powder shows high pozzolanic reactivity from 1 day to 28 days by Frattini test and SAI test. There is significant correlation between the Frattini and SAI test results (R² = 0.84).

The crushed household ceramic waste powder could be used in cement as both pozzolanic active material and as fine filler. The substitution ratio of 15% of TC appears to be both a reactive efficient point and a possible economically favourable point.

ACKNOWLEDGEMENT

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REFERENCES


PROPERTIES OF ALKALI-ACTIVATED FLY ASH MORTARS MADE WITH MULTIPLE ACTIVATORS

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Key words: Alkali Activation, Fly ash, Transport Properties, Durability Properties

ABSTRACT In this study, class F fly ash was activated with an alkaline solution composed of different dosages of sodium hydroxide and sodium silicate. Alkali-activated fly ash mortars were prepared using a constant solution-to-binder ratio of 0.46 and fine aggregate-to-binder ratio of 2. Two sodium hydroxide concentrations of 5 and 10 M, and sodium silicate-to-total solution ratios of 0.20 and 0.60 were used. Upon batching, the studied mortars were sealed-cured for 3 hours at 60°C prior to de-molding and curing at 85°C until testing. The evaluated properties included workability, compressive strength, modulus of rupture, modulus of elasticity, absorption, void content, rapid chloride migration, acid attack resistance, abrasion depth, and resistance to freezing and thawing cycles. Compressive strength was evaluated after 1, 3 and 7 days of curing. Other properties were evaluated after 7 days of curing.

The studied alkali-activated fly ash mortars displayed satisfactory performance. In particular, alkali-activated fly ash mortars developed high strength, appropriate abrasion resistance, and excellent resistance against acid attack. Test results showed that an increase in sodium hydroxide concentration allowed for improvements in strength, stiffness, absorption, void content, depth of wear, and resistance to freezing and thawing and chloride penetration. However, these properties were affected by the sodium silicate-to-total solution ratios. When 5M sodium hydroxide was used, the properties of the studied alkali-activated mortars improved with increases in sodium silicate content. The contrary was found for the mortars having 10M sodium hydroxide.

1. INTRODUCTION

With approximately 1.35 billion tons produced per year, concrete is considered to be the most widely used construction material in the world [1]. Ordinary Portland Cement (OPC) is the main binder used for concrete production; however, a ton of carbon dioxide (CO₂) arises per production of every ton of OPC [2,3]. Overall, OPC production accounts for approximately 7% of total CO₂ emissions released into the atmosphere per year worldwide [4,1,3]. In addition to the environmental concerns, OPC concrete faces durability issues as it is not impermeable, allowing for water and other harmful liquids to enter the system, causing reinforcement corrosion, and concrete spalling [4,5]. Nearly 40% of 600,000 bridges...
constructed in the United States face corrosion problems, totaling to an estimated 50 billion dollars in repair costs [2].

Recently, alkali-activated binders have shown promise as sustainable and durable materials. For the purpose of this study, class F fly ash was the selected binder activated using different dosages of alkaline solutions. Fly ash is a by-product from coal combustion in powered steam generating plants and is considered to be a recyclable material when used for concrete production. When combined with an alkaline activator, such as sodium hydroxide (NaOH) and sodium silicate as used for this study, a strong and durable binder can be produced.

The main reaction product formed in alkali activation of class F fly ash is an alkaline aluminosilicate hydrate gel, known as N-A-S-H gel or geopolymer gel. The alkaline activation process can be broken down into three steps: “destruction-coagulation”, “coagulation-condensation” and “condensation-crystallization” [6]. In the first step, the Me-O, Si-O-Si, Al-O-Al and Al-O-Si bonds in the fly ash are broken down to form Si-O-Na+ bonds and also aluminate complexes such as Al(OH)4- or Al(OH)63-. In the second step, the bonds formed in step one begin to form a coagulated structure along with polycondensation. In the third stage, the main reaction product, known as N-A-S-H gel or geopolymer gel, is formed. The chemical composition of the geopolymer gel is dependent on many variables, such as alkaline activator solution type and concentration, curing conditions, and binder type.

While studies conducted on alkali-activated fly ash binders have focused primarily on the microstructural properties of paste samples, this study aimed at finding various macro-level properties of alkali-activated fly ash mortars using sodium hydroxide and sodium silicate as the alkaline activators. Properties evaluated were workability, compressive strength, modulus of rupture, modulus of elasticity, absorption, void content, rapid chloride migration, acid attack resistance, abrasion depth, and resistance to freezing and thawing.

2. EXPERIMENTAL PROGRAM

2.1. Materials and Mixture Properties

Class F fly ash was used as the sole binder for this study. Its chemical properties are listed in Table 1. Fine aggregates were obtained from a Southern Nevada quarry. Sodium hydroxide and sodium silicate were used as the alkaline activator solutions. The sodium silicate used contained 55.9% by weight of water and SiO2/Na2O of 2/1.

<table>
<thead>
<tr>
<th>Silicon Dioxide (SiO2)</th>
<th>Aluminum Oxide (Al2O3)</th>
<th>Iron Oxide (Fe2O3)</th>
<th>Sulfur Trioxide (SO3)</th>
<th>Calcium Oxide (CaO)</th>
<th>Moisture</th>
<th>Loss on Ignition</th>
</tr>
</thead>
<tbody>
<tr>
<td>59.93%</td>
<td>22.22%</td>
<td>5.16%</td>
<td>0.38%</td>
<td>4.67%</td>
<td>0.04%</td>
<td>0.32%</td>
</tr>
</tbody>
</table>
A constant solution-to-binder ratio and fine aggregate-to-binder ratio of 0.46 and 2, respectively, were used to produce test specimens. Two sodium hydroxide concentrations were considered, namely 5 and 10 M. Two sodium silicate-to-total solution ratios by weight were investigated, 0.20 and 0.60. Table 2 presents the mixture proportions of the mortars investigated in this study.

<table>
<thead>
<tr>
<th>Sodium Silicate-to-Total Solution Ratio</th>
<th>NaOH Concentration (M)</th>
<th>Fine Aggregate (g)</th>
<th>Fly Ash (g)</th>
<th>Activator Solution (g)</th>
<th>Sodium hydroxide</th>
<th>Sodium Silicate</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>5</td>
<td>19641.7</td>
<td>9820.9</td>
<td>3614.1</td>
<td>903.5</td>
<td>5</td>
</tr>
<tr>
<td>0.60</td>
<td>10</td>
<td>19641.7</td>
<td>9820.9</td>
<td>1807.0</td>
<td>2710.6</td>
<td></td>
</tr>
</tbody>
</table>

*Each liter of 5M sodium hydroxide contained 200 g NaOH plus 973.9 g water. A liter of 10M sodium hydroxide included 400 g NaOH plus 903.4 g water.

### 2.2. Casting and Test Methods

Fly ash and fine aggregate was mixed together for approximately two minutes prior to the addition of the alkaline solution and further mixing for another two minutes. Afterward, consistency was checked, and once achieved, the slump flow was evaluated according to ASTM C 1437-01. Dependent on the workability of the mixture, total vibration time varied from 5 to 10 seconds, with a longer vibration time given for the mixtures having a lower slump flow.

Once mortar mixtures were cast, molds were sealed with a four-layers plastic wrap and placed inside an oven at 60°C for 3 hours prior to de-molding. Once de-molded, samples were wrapped again in a four-layers plastic and placed into an oven at 80°C until testing. For compressive strength measurements, samples were cured in for 1, 3 and 7 days. Modulus of elasticity, modulus of Rapture, absorption/void content, rapid chloride migration, abrasion, and resistance to freezing and thawing cycles and acid attack, samples were evaluated using 7-day cured specimens.

Compression tests used 50×100 mm cylinders under a constantly increasing compressive load until failure occurred. Flexural strength test was conducted by placing beam-shaped (50×50×200 mm) mortar specimens under a four-point loading system according to ASTM C78. Elastic modulus was also determined according to ASTM C469-05 using 100-mm diameter by 200-mm height cylindrical samples. Rapid chloride migration (RMT) was evaluated using 50×100 mm disks according to NT BUILD 492. The testing involved placing
the bottom of sample in a sodium chloride (NaCl) tank and the top of sample in a 0.3 M NaOH solution. A current was then allowed to pass through the two ends of sample for 24 hours at a voltage of 10V. Upon completion, test sample was split in half and sprayed with silver nitrate. After about 15 minutes, a white line appeared on the sample indicating the extent to which chloride had penetrated into the test sample. The depth of white line was measured in different locations and then averaged.

ASTM C 642-97 was used to determine the absorption and void content of the studied alkali-activated mortars. The test, referred to as water absorption, involved measuring the oven dry mass of 50×100 mm disk-shaped samples followed by placing them in water for 24 hour periods and measuring their masses until the difference in mass between two 24 hour periods was less than 0.5%, with the last mass labeled as the saturated surface dry mass. Test samples were then boiled for five hours and left to cool down for at least 14 hours before their masses were measured and labeled as the saturated surface dry after boiling mass. Test samples were then submerged in water and their masses were measured and labeled as the saturated surface dry after boiling and immersion mass. With these measured masses, the void content and absorption were calculated.

For resistance to freezing and thawing measurements, mass of 50 mm diameter by 100 mm height cylindrical samples was measured. Samples were then labeled and immersed in water for 7 days prior to measuring the mass of each sample. Test samples were placed in an individual container and filled with distilled water, leaving approximately 3.175 mm of excess water above the sample. Containers were then placed in a freezer at -20°C for 24 hours and then left at room temperature to thaw for 24 hours. This was accounted for as one complete freeze-thaw cycle. Samples were subjected to 50 freeze-thaw cycles, with the mass measured after 10, 20, 30, 40, and 50 freeze-thaw cycles.

Acid resistance was assessed in terms of mass and strength losses. The mass of 50 mm diameter by 100 mm height cylindrical sample was recorded prior to immersion in water and in sulfuric acid solution as well as after being immersed for up to 8 weeks. The difference between the sample prior to immersion and the mass recorded after 8 weeks was found and reported as total mass loss. Additionally, compressive strength of samples was measured after 1, 2, 4 and 8 weeks of immersion in sulfuric acid.

Two 150-mm cube samples were utilized per each matrix tested for abrasion resistance testing. ASTM C 779, Procedure C, standard for assessing the abrasion resistance of horizontal concrete surface, was used for abrasion resistance testing. The cube sample was fixed in place with the use of clamps and attached to steel plate under the drive shaft of the test apparatus. The steel plate was leveled once the sample was placed to aid in maintaining the sample positioned horizontally throughout the test duration. Once the sample was placed and fixed, twelve (12) 13 mm diameter steel ball bearings were positioned on the bottom of the bearing plate on the test apparatus. The drive shaft was then lowered until the ball bearings came into contact with the top surface of the sample. An air supply was used in order to aid in the removal of loose particles during testing. With that said, the air valve with approximately 40 psi air pressure was opened. The dial gauge affixed to a cantilever flange and a digital stopwatch was then set to zero. Once the set up was completed, the power was turned on.
Measurements were taken every 30 seconds for either 20 minutes or until a wear depth of 3 mm was reached.

3. RESULTS AND DISCUSSION

3.1. Fresh Properties

Table 3 reports the results of slump flow test. As can be seen, workability of alkali-activated mortars decreased with increased sodium hydroxide concentration. The slump flow reduced by averagely 41.4%, when molarity of sodium hydroxide increased from 5 to 10M. With increased sodium hydroxide concentration, a higher degree of reactivity occurred, resulting in reduction in workability. Also, increased sodium hydroxide concentration decreased the total initial water content found in the matrix, contributing to decreased workability.

Workability also decreased with increases in sodium silicate-to-total solution ratio. On average, the slump flow of the studied mixtures decreased by 42.7%, when sodium silicate content increased from 20 to 60%. The observed reduction can be related to the lower water content of sodium silicate in comparison with that of sodium hydroxide solution. While almost 83 and 70% of sodium hydroxide solutions were water for molarities of 5 and 10M, respectively, the sodium silicate solution contained nearly 56% water. The lower water content resulted in reduction of flow of mixtures having higher sodium silicate-to-total solution ratio. In addition, with an increase of sodium silicate, the overall SiO$_2$/Na$_2$O increased, which in turn reduced the flow of the matrix [7].

<table>
<thead>
<tr>
<th>Sodium Silicate-to-Total Solution</th>
<th>5M NaOH</th>
<th>10M NaOH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>15.2</td>
<td>8.9</td>
</tr>
<tr>
<td>0.60</td>
<td>8.7</td>
<td>5.1</td>
</tr>
</tbody>
</table>

3.2. Mechanical Properties

3.2.1. Compressive Strength

The compressive strength of the studied alkali-activated fly ash mortars are presented in Table 4. As can be seen, the compressive strength improved with increases in sodium hydroxide concentration, in particular in case of mixtures having 20% sodium silicate. The 1-, 3-, and 7-day compressive strengths of 10M mixtures containing 20% sodium silicate was 10.7, 6.37, and 6.05 times of 5M mixtures having similar sodium silicate content, respectively. The improvements were negligible for mixtures having 60% sodium silicate content. With increased sodium hydroxide concentration, the amount of N-A-S-H gel increased, leading to higher strength development for the evaluated alkali-activated fly ash mortars [8,9,10,11].
Compressive strength also increased with increased sodium silicate-to-total solution ratio for the mixtures containing 5M NaOH; however, it decreased when sodium silicate content of 10M mixtures increased. The 7-day compressive strength of 5M mortars improved 361.8% when sodium silicate content was increased from 20 to 60%. The similar increase in sodium silicate content resulted in 20.4% reduction in the 7-day compressive strength of 10M mixtures. In general, increased sodium silicate content in the mixture allowed for higher Si ions to form, leading to the formation of a more compact structure, which in turn improved the compressive strength. Although more compact structures were formed with more Si ions in corresponding gels due to the presence of silicate ions, there was a limit associated with this phenomenon. At higher concentrations of sodium hydroxide, such as 10 NaOH as used in this study, there was an insufficient amount of sodium hydroxide available to hydrate fly ash adequately, resulting in higher amounts of non-reacted fly ash particles and ultimately lower compressive strength [12,7]. Similarly, Joseph and Mathew found that the insufficient amounts of sodium hydroxide hindered on the dissolution process within the entire geopolymerization process. Also, the pH of sodium silicate was lower than sodium hydroxide and there was a higher \([\text{SiO(OH)}_2]^2-\) content for higher sodium silicate-total solution ratio. A higher \([\text{SiO(OH)}_2]^2-\) content hindered on the condensation process of the reaction product formation [13]. In conjunction with the results found in this study, it can be said that for higher sodium hydroxide concentration, increased sodium silicate-to-total solution ratio allowed for higher \([\text{SiO(OH)}_2]^2-\) content that prevented steady formation of reaction product and lowered compressive strength of the studied alkali-activated fly ash mortars.

<table>
<thead>
<tr>
<th>Sodium Silicate-To-Total Solution</th>
<th>NaOH Concentration</th>
<th>1 Day</th>
<th>3 Days</th>
<th>7 Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20 5M</td>
<td></td>
<td>5.88</td>
<td>11.87</td>
<td>12.60</td>
</tr>
<tr>
<td>0.60 5M</td>
<td></td>
<td>44.02</td>
<td>57.79</td>
<td>58.19</td>
</tr>
<tr>
<td>0.20 10M</td>
<td></td>
<td>62.94</td>
<td>75.67</td>
<td>76.20</td>
</tr>
<tr>
<td>0.60 10M</td>
<td></td>
<td>46.97</td>
<td>59.41</td>
<td>60.64</td>
</tr>
</tbody>
</table>

3.2.2. Flexural Strength (Modulus of Rupture)

Table 5 shows flexural strength of the studied alkali-activated fly ash mortars. As can be seen, modulus of rupture increased with increases in molarity of solution. The flexural strength of 10M mixtures was 3.15 and 1.18 times of that of 5M mixtures for sodium silicate contents of 20 and 60%, respectively. Similar to the compressive strength results, this trend can be related to the formation of higher N-A-S-H gel which is responsible for strength development. Flexural strength of 5M mixture increased by increases in sodium silicate content, whereas the opposite behavior was observed for 10M mixtures. By increasing sodium silicate content, flexural strength increased by 91.95% for 5M mixtures and decreased by 28.19 % for 10M mixtures. These observations are similar to those found for compressive strength.
Table 5. 7-day flexural strength of alkali-activated fly ash mortars

<table>
<thead>
<tr>
<th>Sodium Silicate-to-Total Solution</th>
<th></th>
<th>Flexural Strength (MPa)</th>
<th></th>
<th>Modulus of Elasticity (GPa)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td></td>
<td>2.98</td>
<td></td>
<td>9.40</td>
<td></td>
</tr>
<tr>
<td>0.60</td>
<td></td>
<td>5.72</td>
<td></td>
<td>6.75</td>
<td></td>
</tr>
</tbody>
</table>

3.2.3. Modulus of Elasticity

Table 5 also presents modulus of elasticity of the studied alkali-activated fly ash mortars. As can be seen, modulus of elasticity increased with increases in sodium hydroxide concentration. Similar to the compressive strength, the increases were significant for 20% sodium silicate content and marginal for 60% sodium silicate content. Modulus of elasticity of mixtures having 20 and 60% sodium silicate increased by 366.67 and 9.6% when molarity increased from 5 to 10M, respectively. As described for compressive strength results, increased sodium hydroxide concentration allowed for higher reaction product formation and thus, higher strength development [11].

Modulus of elasticity also increased with increased sodium silicate-to-total solution ratio for the mortars containing 5M NaOH, while the opposite occurred for mortars containing 10M NaOH. This behavior is similar to the trend observed for of compressive strength.

3.3. Transport Properties

3.3.1. Absorption

The results of absorption test are presented in Table 6. Absorption and void content decreased with increases in sodium hydroxide concentration. On average, absorption and void content of 10M mixtures were 45.4 and 35.5% lower than those of 5M mixtures, respectively. Increased sodium hydroxide concentration allowed for higher amounts of produced N-A-S-H gel to ultimately forming a stronger, more compact structure. A more compact structure resulted in a lower absorption.

Absorption also decreased with increased sodium silicate-to-total solution ratio for the mortars containing 5M NaOH, while the opposite occurred for the mortars containing 10M NaOH. As discussed previously, increased sodium silicate allowed for a more compact structure due to the higher silicate ions available, however, the improvement was limited. At higher concentration of sodium hydroxide, the increased amounts of sodium hydroxide hindered the geopolymerization process, resulting in a less compact matrix and ultimately in higher absorptions.
Table 6. Absorption of alkali-activated fly ash mortars

<table>
<thead>
<tr>
<th>Sodium Silicate-to-Total Solution</th>
<th>5M NaOH</th>
<th>10M NaOH</th>
<th>5M NaOH</th>
<th>10M NaOH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>7.86</td>
<td>3.60</td>
<td>12.4</td>
<td>8.4</td>
</tr>
<tr>
<td>0.60</td>
<td>6.82</td>
<td>4.32</td>
<td>16.5</td>
<td>10.1</td>
</tr>
</tbody>
</table>

3.3.2. Rapid Chloride Migration

The penetrated chloride depth of the selected alkali-activated fly ash mortars is documented in Table 7. The depth of penetrated chloride decreased with increases in sodium hydroxide concentration. The reduction due to increase of molarity from 5 to 10 were 73 and 43% for sodium silicate contents of 20 and 60%, respectively. A higher reaction product was formed with increased sodium hydroxide concentration, allowing for lower permeable void contents. With that said, liquids were unable to penetrate the system, thus decreasing the amount of chloride penetration for mortars containing a higher concentration of sodium hydroxide.

Chloride depth results were also found to decrease with increased sodium silicate-to-total solution ratio for mortars made with 5M NaOH (28.8 % decrease), while the contrary occurred for mortars made with 10M NaOH (48.3% increase). Higher amounts of sodium silicate for mortars containing 5M NaOH had a higher geopolymerization rate, which led to a more compact structure and lower chloride ion penetration. For mixtures containing 10M NaOH, the increased amount of sodium silicate adversely affected geopolymerization process, allowing for higher amounts of chloride to penetrate the system.

Table 7. Rapid chloride migration depth for alkali-activated fly ash mortars

<table>
<thead>
<tr>
<th>Sodium Silicate-to-Total Solution</th>
<th>5M NaOH</th>
<th>10M NaOH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>33.68</td>
<td>9.20</td>
</tr>
<tr>
<td>0.60</td>
<td>23.97</td>
<td>13.64</td>
</tr>
</tbody>
</table>

3.4. Durability Properties

3.4.1. Freezing and Thawing Resistance

Mass loss due to freezing and thawing was measured after every 10 cycles of freezing-thawing and the results are displayed in Table 8. The mass loss was found to decrease with increased sodium hydroxide concentration. The improvements were averagely 89.6 and 67.9%
for sodium silicate content of 20 and 60%, respectively. With increased sodium hydroxide concentration, a more compact structure was formed and limiting the amount of liquid penetration allowed. With that said, lower amounts of water were absorbed, limiting the deterioration caused by freezing and thawing.

The freezing and thawing mass loss decreased with increased sodium silicate-to-total solution ratio for the mortars made with 5M NaOH (averagely 54.9% reduction), while the opposite trend emerged for mortars containing 10M NaOH (averagely 34.2% increase). These results can be related to the strength and transport properties. The stronger mixtures were able to withstand deterioration due to freezing and thawing. The stronger mixtures also had lower permeable void content contributing to their resistance.

<table>
<thead>
<tr>
<th>Sodium Silicato-to-total Solution</th>
<th>NaOH Concentration</th>
<th>Mass Loss (%)</th>
<th>Number of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>5M</td>
<td>8.46 58.68</td>
<td>70.1 100</td>
</tr>
<tr>
<td>0.60</td>
<td>5M</td>
<td>3.15 20.79</td>
<td>39.7 50.95 58.99</td>
</tr>
<tr>
<td>0.20</td>
<td>10M</td>
<td>0.68 5.29</td>
<td>8.98 11.64 15.15</td>
</tr>
<tr>
<td>0.60</td>
<td>10M</td>
<td>0.33 4.58</td>
<td>12.34 22.56 31</td>
</tr>
</tbody>
</table>

3.4.2. Resistance to Acid Attack

Table 9 reports the strength loss and total mass loss of alkali-activated fly ash contained mortars immersed 8 weeks in sulfuric acid.

<table>
<thead>
<tr>
<th>NaOH Concentration</th>
<th>Sodium Silicate-to-Total Solution</th>
<th>Total Acid Mass Loss (%)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5M</td>
<td>0.2</td>
<td>2.61</td>
<td>Before, 1 week, 2 weeks, 4 weeks, 8 weeks</td>
</tr>
<tr>
<td>5M</td>
<td>0.6</td>
<td>0.13</td>
<td>43.64, 43.58, 43.04, 43.10, 42.62</td>
</tr>
<tr>
<td>10M</td>
<td>0.2</td>
<td>0.40</td>
<td>72.39, 69.56, 69.78, 71.57, 73.61</td>
</tr>
<tr>
<td>10M</td>
<td>0.6</td>
<td>1.00</td>
<td>60.64, 61.55, 59.78, 59.00, 58.47</td>
</tr>
</tbody>
</table>
It can be seen that the mass and strength losses were minimal for the alkali-activated mortars. In fact, the developed mortars had excellent resistance against acid attack. This observation can be attributed to low calcium oxide content of fly ash as the sole binder of the studied mixtures.

### 3.4.3. Resistance to Wear

Table 10 displays the final abrasion depth of the studied alkali-activated fly ash mortars after 20 minutes. Generally speaking, increases in molarity of solution improved abrasion resistance. Increasing sodium silicate content was also useful in reducing the abrasion depth. It can be seen that while abrasion depth of 5M mixture having 20% sodium silicate was 2.5 mm, it was reduced to almost 1 mm through an increase in molarity to 10M or an increase in sodium silicate content to 60%.

<table>
<thead>
<tr>
<th>Sodium silicate to total solution ration</th>
<th>5M NaOH</th>
<th>10M NaOH</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>2.51</td>
<td>0.93</td>
</tr>
<tr>
<td>0.60</td>
<td>1.01</td>
<td>1.17</td>
</tr>
</tbody>
</table>

### 4. CONCLUSIONS

This study assessed fresh, mechanical, transport, and durability properties of fly ash mortars activated with sodium hydroxide and sodium silicate. The following conclusions can be drawn based on the results presented:

1. Workability was found to decrease with increases in sodium hydroxide concentration and sodium silicate-to-total solution ratio.
2. Compressive strength, flexural strength and modulus of elasticity increased with increases in sodium hydroxide concentration, as well as increased sodium silicate-to-total solution ratio for the mortars containing 5M NaOH. The strength properties decreased with increased sodium silicate-to-total solution ratio for the mortars containing 10M NaOH.
3. Absorption, void content, chloride penetration depth and mass loss due to freezing and thawing cycles decreased with increased sodium hydroxide concentration. These properties decreased with increased sodium silicate-to-total solution ratio for the mixtures containing 5M NaOH, while the opposite occurred for the mortars containing 10M NaOH. The excessive freeze-thaw induced mass loss precludes the use of alkali-activated mortars in severe freezing and thawing environments.
4. Alkali-activated fly ash mortars provided an excellent resistance against acid attack. The strength and mass losses of all the mixtures were negligible after 8 weeks of immersion in sulfuric acid.

5. The abrasion depth of the studied mortars reduced with increases in molarity of NaOH solution and sodium silicate content of the alkaline activator.

5. REFERENCES


PROPERTIES OF HIGH FLUIDITY CONCRETE USING FINE POWDER OF MELT-SOLIDIFIED SLAG FROM MUNICIPAL WASTE AS AN ADMIXTURE

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Key words: Melt-Solidified Slag Made From Municipal Waste, High Fluidity Concrete

Abstract. In Japan, most of the melt-solidified slag made from municipal waste is made by quenching process. The melt-solidified slag made by quenching process has the latent hydraulic property. Several properties of high fluidity concrete which contains the fine powder of the melt-solidified slag as an admixture, such as density, compressive strength, modulus of elasticity and drying shrinkage are examined varying kind of melt-solidified slag and specific surface area of the slag powder. The experiments revealed that fine powder of melt-solidified slag from municipal waste which contains little metallic aluminum in the components is effectively used as an admixture of high fluidity concrete.

1 INTRODUCTION

In Japan, part of municipal waste is fused at a high temperature more than 1,200°C to adapt the upper limit of dioxins in the residual material, in accordance with the "Law Concerning Special Measures against Dioxins" enforced on January 2000. Production quantity of melt-solidified slag made from municipal waste is about 850,000,000kg every year. The melt-solidified slag made from municipal waste is recommended to use as aggregate for concrete and road. The research on slag utilization as aggregate for concrete has been conducted by many researchers [1] [2]. Two Japanese industrial standards have been regulated. One is JIS A 5031 "Melt-solidified slag aggregate for concrete derived from municipal solid waste and sewage sludge" and the other is JIS A 5032 "Melt-solidified slag material for road construction derived from municipal solid waste and sewage sludge" which were regulated in 2006. Nevertheless small portion of the slag is used effectively.

Considering that a large amount of energy is used to produce the slag, a higher price
material should be replaced by the slag. Most of the melt-solidified slag made from municipal waste is made through quenching process. Because of this, it exerts the latent hydraulic property [3]. This paper describes several properties of high fluidity concrete which contain the fine powder of the melt-solidified slag as an admixture, such as, density in oven-dry condition, compressive strength, static modulus of elasticity and drying shrinkage.

**Table 1:** Factors and levels of experiments adopted in the experiments

<table>
<thead>
<tr>
<th>Factors</th>
<th>Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Melting system</td>
<td>Direct melting (A),</td>
</tr>
<tr>
<td></td>
<td>Ash melting (B, C)</td>
</tr>
<tr>
<td>Type of melting furnace</td>
<td>Shaft furnace type gasification melting furnace (A),</td>
</tr>
<tr>
<td></td>
<td>Electric resistance type ash melting furnace (B),</td>
</tr>
<tr>
<td></td>
<td>Plasma type ash melting furnace (C)</td>
</tr>
<tr>
<td>Area of production</td>
<td>a, b, c, d, e</td>
</tr>
<tr>
<td>Specific surface area</td>
<td>4500 (cm²/g), 6000 (cm²/g)</td>
</tr>
</tbody>
</table>

**Table 2:** Materials

<table>
<thead>
<tr>
<th>Materials</th>
<th>Density (g/cm³)</th>
<th>Fineness modulus</th>
<th>Specific surface area (cm²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary portland cement</td>
<td>3.15</td>
<td>-----</td>
<td>3280</td>
</tr>
<tr>
<td>Type B portland blast-furnace slag cement</td>
<td>3.04</td>
<td>-----</td>
<td>3820</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>2.62</td>
<td>2.92</td>
<td>-----</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>2.96</td>
<td>6.5</td>
<td>-----</td>
</tr>
<tr>
<td>High-range water-reducing admixture</td>
<td>1.04~1.08</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Air-entraining admixture</td>
<td>1.04~1.08</td>
<td>-----</td>
<td>-----</td>
</tr>
</tbody>
</table>

**Table 3:** Manufacturing method and production area of melt-solidified slag

<table>
<thead>
<tr>
<th>Melt-solidified slag</th>
<th>Manufacturing method</th>
<th>Area of production</th>
<th>Density (g/cm³)</th>
<th>Specific surface area (cm²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Direct melting</td>
<td>a, b</td>
<td>2.81</td>
<td>4560</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Ash melting</td>
<td>c, d, e</td>
<td>2.76</td>
<td>5780</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.78</td>
<td>4590</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.79</td>
<td>4460</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.79</td>
<td>6070</td>
</tr>
</tbody>
</table>
2 EXPERIMENTAL PROCEDURES

2.1 Factors and levels of experiments

Table 1 shows factors and levels adopted in the experiments. Table 2 shows the materials used in the experiments. Table 3 shows the melt-solidified slag made from municipal waste used in the experiments. Table 4 shows the mixture proportion of high fluidity concrete. Binder content of all mixes was 600kg/m³. Type B portland blast-furnace slag cement which contain blast-furnace slag powder about half in weight was used as the binder at a content of 600kg/m³ for Base mix. Ordinary portland cement at a content of 300kg/m³ and fine powder of the melt-solidified slag at a content of 300kg/m³ were used as the binder for all mixes except Base mix. The sign of mix expresses combination of manufacturing method and production area of the melt-solidified slag. All mixes were steam cured.

2.2 Test methods

Slump flow of concrete was measured according to JIS A 1150 which corresponds to ISO 1920-2. Air content of concrete was measured according to JIS A 1128 which corresponds to ISO 4848. Compressive strength determination were made on 100×200 cylinders, according to JIS A 1108 which corresponds to ISO 1920-4. Compressive strength values were obtained at several ages from 1 day to a year. And static modulus of elasticity of concrete was measured according to JIS A 1149 "Method of test for static modulus of elasticity of concrete". Drying shrinkage was measured using embedded strain gauge for a year.

3 RESULTS

3.1 Slump flow, air content and density

Table 5 shows the results of slump flow, air content and density of high fluidity concrete used in this study. All high fluidity concrete showed good flowability. Mix C-e expanded after molding as shown in Figure 1. The expansion seems to be caused by the chemical reaction between the metallic aluminum in the slag and Ca(OH)₂ produced through hydration [4].

3.2 Compressive strength and static modulus of elasticity

Figure 2 shows the relation between compressive strength and age. Compressive strength of mix A-a which contain slag manufactured by shaft furnace type gasification melting furnace was nearly equal to that of Base mix which contain blast-furnace slag cement. Compressive strength of concrete made from other slag is lower than that of mix A-a and Base mix. It is shown from Figure 2 that compressive strength of concrete used slag C-e is markedly low.

Figure 3 shows the relation between compressive strength and density at the age of 28 days. Figure 4 shows the relation between static modulus of elasticity and density at the age of 28 days. These figures show that compressive strength and static modulus of elasticity decrease with the decrease of density. This indicates that compressive strength and static modulus of elasticity decrease with the increase of void which is generated after molding.
Table 4: Concrete mix data

<table>
<thead>
<tr>
<th>Mix</th>
<th>Slump flow (mm)</th>
<th>Air (%)</th>
<th>W/C (%)</th>
<th>s/a (%)</th>
<th>Water (kg/m³)</th>
<th>Cement (kg/m³)</th>
<th>Melt-solidified slag (kg/m³)</th>
<th>Fine aggregate (kg/m³)</th>
<th>Coarse aggregate (kg/m³)</th>
<th>High-range water-reducing admixture (kg/m³)</th>
<th>Air-entraining admixture (kg/m³)</th>
</tr>
</thead>
</table>
| Base*   | 28.3 46.0 170 600* | --- 755 | 888 3.4 | 0.009 
| A-a     | 27.8 45.9 167 300** | 300 752 | 885 3.9 | 0.012       |
| A-b     | 27.8 45.9 167 300** | 300 752 | 885 3.6 | 0.009       |
| B-c     | 680 ±50 4.5 ±1.0 | 30 46.0 180 300** | 300 734 | 861 4.8 | 0.009       |
| C-d     | 27.8 46.0 167 300** | 300 752 | 882 3.6 | 0.009       |
| C-e(4500) | 28.3 45.9 170 300** | 300 747 | 879 4.2 | 0.009       |
| C-e(6000) | 28.3 45.9 170 300** | 300 747 | 879 4.2 | 0.009       |

* Type B portland blast-furnace slag cement which contain pulverized blast-furnace slag about half in weight.
** Ordinary portland cement

Table 5: Slump flow, air content and density

<table>
<thead>
<tr>
<th>Mix</th>
<th>Slump flow (mm)</th>
<th>Air content (%)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>725</td>
<td>3.5</td>
<td>2.42</td>
</tr>
<tr>
<td>A-a</td>
<td>725</td>
<td>4.2</td>
<td>2.46</td>
</tr>
<tr>
<td>A-b</td>
<td>635</td>
<td>3.9</td>
<td>2.32</td>
</tr>
<tr>
<td>B-c</td>
<td>735</td>
<td>5</td>
<td>2.38</td>
</tr>
<tr>
<td>C-d</td>
<td>630</td>
<td>4.2</td>
<td>2.41</td>
</tr>
<tr>
<td>C-e(4500)</td>
<td>650</td>
<td>4.5</td>
<td>2.27</td>
</tr>
<tr>
<td>C-e(6000)</td>
<td>660</td>
<td>4.0</td>
<td>2.30</td>
</tr>
</tbody>
</table>

Figure 1: Concrete after molding (mix C-e)

Figure 2: Relation between compressive strength and age
Figure 3: Relation between compressive strength and density (28 days).

Figure 4: Relation between static modulus of elasticity and density (28 days).

Figure 5: Relation between drying shrinkage and age.

Figure 6: Relation between drying shrinkage and density (28 days).

Figure 7: Relation between compressive strength and drying shrinkage (28 days).

Figure 8: Relation between static modulus of elasticity and drying shrinkage (28 days).
3.3 Drying shrinkage

Figure 5 shows the relation between drying shrinkage and age. Drying shrinkage of mix A-a is nearly equal to that of Base mix. Values of drying shrinkage of mixes except A-a are larger than that of mix A-a. The figure shows that drying shrinkage of mix C-e is significantly large in comparison with that of other mixes.

Figure 6 shows the relation between drying shrinkage and density at the age of 28 days. Figure 7 shows the relation between compressive strength and drying shrinkage at the age of 28 days. Figure 8 shows the relation between static modulus of elasticity and drying shrinkage at the age of 28 days.

These figures show that drying shrinkage increases with the decrease in density and that compressive strength and static modulus of elasticity decrease with the increase of drying shrinkage. It is conceivable that reaction between metallic aluminum and Ca(OH)$_2$ produced through hydration generates void in concrete, and the void causes decrease in density, compressive strength and static modulus of elasticity, and increase in drying shrinkage.

4 CONCLUSIONS

- Void generated through the reaction between the metallic aluminum in slag and Ca(OH)$_2$ produced through hydration causes decrease in density of concrete.
- Decrease in density causes decrease in compressive strength and elastic modulus, and increase in drying shrinkage.
- Fine powder of melt-solidified slag made from municipal waste which contains little metallic aluminum in the components is effectively used as an admixture for high fluidity concrete.
- In the experiments, high fluidity concrete used a kind of melt-solidified slag manufactured by a shaft furnace type gasification melting furnace obtained the same compressive strength and drying shrinkage as high fluidity concrete used portland blast-furnace slag cement as binder.

REFERENCES

PROPERTIES OF SELF CONSOLIDATING CONCRETE CONTAINING NATURAL POZZOLAN

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Email: nader.ghafoori@unlv.edu

Key words: Self Consolidating Concrete, Natural Pozzolan, Modulus of Elasticity, RCPT

ABSTRACT The influence of Natural Pozzolan (NP) on properties of Self Consolidating Concrete (SCC) was investigated. A constant cementitious material of 475 kg/m$^3$ was used. The NP replaced a portion of Portland cement at the levels of 15 and 30% by mass. A uniform water-to-cementitious materials ratio of 0.4 was used to produce 3 distinct mixtures including reference concrete. The evaluated properties included slump flow, J-ring, $T_{50}$, compressive strength, modulus of elasticity, water absorption, void content, rapid chloride permeability, rapid chloride migration, and mass loss due to freezing and thawing.

The results of this investigation showed that fresh and transport properties of SCC mixtures were improved by increases in replacement level of natural pozzolan. On the other hand, slight reductions were observed in early and mid-aged strength properties of the studied NP contained self consolidating concretes. Durability properties of the studied SCCs improved by partial replacement of NP with Portland cement.

1. INTRODUCTION

Global Portland cement production currently accounts for 7% ($2.1\times10^9$ tonnes) of anthropogenic carbon dioxide (CO$_2$) emissions annually [1]. Because kiln-fired Portland cement is an energy-intensive material, requiring 4–5 GJ per ton of cement [2], about half of these emissions occur through combustion of fossil fuels. The remaining emissions result from calcination of limestone. Overall, one kg of Portland cement clinker releases 0.87 kg of CO$_2$ to the atmosphere [3].

In recent years, Self Consolidating Concrete (also known as Self Compacting Concrete and abbreviated as SCC) has gained wide use for placement in congested reinforced concrete structures with difficult casting conditions [4]. SCC also offers other advantages when compared with ordinary vibrated concrete including higher flowability, shorter construction period, lower labor costs, lesser screeding and better self-leveling, higher construction quality and productivity, and a better work environment through construction site noise reduction [5, 6]. In order to reach high flowability and to maintain sufficient viscosity to prevent segregation, SCC generally contains a higher paste volume than ordinary vibrated concrete. The need for a higher cementitious material content may result in a higher production cost for self-consolidating concrete [7].
One way to reduce the production cost of SCC, and to address the sustainability concerns as related to cement production, is to utilize pozzolanic materials as replacement for a portion of the paste volume [7]. Silica fume, fly ash, slag, and natural pozzolan are the most widely used supplementary cementitious materials in ordinary vibrated concrete [8,9]. While an extensive literature is available on the use of first three supplementary materials in SCC [10, 11, 12], there are significantly fewer studies reported on the use of natural pozzolan in self-consolidating concretes.

Kemal Celik et al. [13] incorporated Saudi Arabian aluminum–silica rich basaltic glass as a natural pozzolan in self-consolidating concrete. An increase in the amount of NP, as a partial replacement of OPC, resulted in a decrease in the required superplasticizer content or T50 which implied an increase in the flowability of the studied concretes. While in 30% replacement, the NP contained SCCs had the same compressive strength as the reference concrete, it had a lower compressive strength at the ages 7, 28, 91 and 365 days when NP replaced 50% by mass of Portland cement. The results of this study also revealed that incorporation of natural pozzolan in SCCs reduced gas permeability and chloride penetration migration coefficient of concrete.

Belaidi et al. [14] examined the effects of substitution of cement with natural pozzolan on the rheological and mechanical properties of SCC. They observed an improvement of workability in the SCCs containing up to 15% of NP as replacement of cement mass. However, the workability of 20 and 25% NP-contained concretes decreased significantly as compared to that of the reference mix. A reduction of compressive strength was observed with inclusion of NP as compared to reference concrete. The results indicated that NP increases strength at late age curing (90 days and beyond).

H. Said et al. [15] compared the hydrochloric and sulfuric acid behaviors of a SCC containing Algerian natural pozzolan with SCC containing limestone powder. They revealed that, the NP-contained SCCs are stronger than the limestone power contained concretes.

In an attempt to have a full spectrum of properties examined, the study presented herein reports on fresh, mechanical, transport, and durability properties for natural pozzolan-contained self-consolidating concrete. Moreover, the findings are compared to the properties of the companion reference concrete.

2. MATERIALS AND MIXTURE PROPORTIONS

The class N natural pozzolan was obtained from a source in Nevada. The chemical compositions of Portland cement and natural pozzolan are presented in Table 1. A comparison between physico-chemical properties of the natural pozzolan and the requirements of ASTM C618 is shown in Table 2. The physical properties of the limestone coarse and fine aggregates are reported in Table 3. A Polycarboxylate-based High Range Water-Reducing Admixture (HRWRA), complying with the requirements of ASTM C 494 Type F, was also utilized.

Table 4 summarizes the mixture proportions and constituents of the studied SCCs. In addition to the reference concrete (without natural pozzolan (NP0)); two natural pozzolan-contained
SCCs were batched by replacing 15 and 30% of cement mass with natural pozzolan (NP15 and NP30, respectively). In an attempt to achieve a uniform target flowability (spread flow of 673±25 mm (26.5 in), and visual stability index of zero) the amount of required superplasticizer amongst the studied mixtures varied. The results are shown in Table 4.

### Table 1: Chemical composition of utilized cement and natural pozzolan

<table>
<thead>
<tr>
<th>Compound/property</th>
<th>Cement</th>
<th>Natural pozzolan</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcium oxide (CaO)</td>
<td>61.32</td>
<td>3.2</td>
</tr>
<tr>
<td>Silicon Dioxide (SiO2)</td>
<td>21.68</td>
<td>68.8</td>
</tr>
<tr>
<td>Aluminum Oxide (Al2O3)</td>
<td>4.85</td>
<td>8.5</td>
</tr>
<tr>
<td>Iron oxide (Fe2O3)</td>
<td>4.40</td>
<td>1.1</td>
</tr>
<tr>
<td>Magnesium oxide (MgO)</td>
<td>2.60</td>
<td>---</td>
</tr>
<tr>
<td>Sodium oxide (Na2O)</td>
<td>0.25</td>
<td>2.6</td>
</tr>
<tr>
<td>Potassium oxide (K2O)</td>
<td>0.71</td>
<td>3.9</td>
</tr>
<tr>
<td>Sulfur trioxide (SO3)</td>
<td>1.90</td>
<td>0.1</td>
</tr>
</tbody>
</table>

### Table 2: Chemical and physical properties of natural pozzolan according to ASTM C618

<table>
<thead>
<tr>
<th>Chemical requirements</th>
<th>Class N, ASTM C618</th>
<th>Natural Pozzolan results</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2 + Al2O3 + Fe2O3 (%)</td>
<td>Min, 70.0</td>
<td>78.4</td>
</tr>
<tr>
<td>Sulfur trioxide (SO3) (%)</td>
<td>Max, 4.0</td>
<td>0.1</td>
</tr>
<tr>
<td>Moisture content (%)</td>
<td>Max, 3.0</td>
<td>0.7</td>
</tr>
<tr>
<td>Loss on ignition (%)</td>
<td>Max, 10.0</td>
<td>3.4</td>
</tr>
</tbody>
</table>

#### Physical requirements

<table>
<thead>
<tr>
<th>Property</th>
<th>Class N, ASTM C618</th>
<th>Natural Pozzolan results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>---</td>
<td>2.29</td>
</tr>
<tr>
<td>Blaine</td>
<td>---</td>
<td>6088</td>
</tr>
<tr>
<td>Amount retained when wet-sieved on 45 µm (-325) sieve (%)</td>
<td>Max, 34</td>
<td>9.4</td>
</tr>
<tr>
<td>Strength activity index, at 7 days, percent of control*</td>
<td>Min, 75</td>
<td>85</td>
</tr>
<tr>
<td>Strength activity index, at 28 days, percent of control*</td>
<td>Min, 75</td>
<td>92</td>
</tr>
<tr>
<td>Water requirement, percent of control</td>
<td>Max, 115</td>
<td>98</td>
</tr>
<tr>
<td>Autoclave expansion or contraction (%)</td>
<td>Max, 0.8</td>
<td>0.00</td>
</tr>
</tbody>
</table>

*Note that, the activity index of natural pozzolan is determined based on ASTM C311 by replacing 20% of cement with natural pozzolan.
Table 3 Aggregate properties

<table>
<thead>
<tr>
<th>Aggregate type</th>
<th>Specific gravity (Oven-Dry)</th>
<th>Specific Gravity (Saturated-Surface Dry)</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine (0–4.75 mm)</td>
<td>2.755</td>
<td>2.777</td>
<td>0.81</td>
</tr>
<tr>
<td>Coarse (4.75–19 mm)</td>
<td>2.747</td>
<td>2.768</td>
<td>0.79</td>
</tr>
</tbody>
</table>

Table 4 Mixture proportions and constituents

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>NP Dosage (% by mass of binder)</th>
<th>Binder content (kg/m³)</th>
<th>Water to binder ratio</th>
<th>Cement (kg/m³)</th>
<th>Natural pozzolan (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Fine Agg. (kg/m³)</th>
<th>Coarse Agg. (4.75–9 mm) (kg/m³)</th>
<th>Coarse Agg. (9–12.5 mm) (kg/m³)</th>
<th>Admixture (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP0</td>
<td>0</td>
<td>475</td>
<td>0.40</td>
<td>475</td>
<td>0.00</td>
<td>190</td>
<td>1004.7</td>
<td>416.9</td>
<td>341.1</td>
<td>2.763</td>
</tr>
<tr>
<td>NP15</td>
<td>15</td>
<td>475</td>
<td>0.40</td>
<td>403.75</td>
<td>71.25</td>
<td>190</td>
<td>992.7</td>
<td>411.9</td>
<td>337</td>
<td>2.916</td>
</tr>
<tr>
<td>NP30</td>
<td>30</td>
<td>475</td>
<td>0.40</td>
<td>332.5</td>
<td>142.5</td>
<td>190</td>
<td>980.7</td>
<td>406.9</td>
<td>332.9</td>
<td>2.611</td>
</tr>
</tbody>
</table>

3. MIXING AND TESTING METHODS

A counter-current horizontal pan mixer with 0.0283 m³ (1 ft³) capacity was used to batch the studied concrete. For the mixing sequence, first the coarse aggregate and 1/3 of the mixing water was added. Following the first 2 minutes of mixing, the fine aggregate and 1/3 of the water was incorporated and mixed for another 2 minutes. Afterward, the cement, natural pozzolan and remaining water was added. After mixing for 2 minutes, the HRWRA was slowly introduced and allowed to mix for an additional 2 minutes. At this point, fresh concrete was allowed to rest for 2 minutes before remixing resumed for one minute. The elapsed time of the total mixing sequence was 11 minutes or 7 minutes once cement became in contact with water. Immediately after mixing, freshly-mixed concretes were poured in molds without any external vibration. After casting, specimens were kept covered in the laboratory for 24 hours (room temperature of 23 ± 1 °C) before being demolded and placed in a moisture room to cure for different ages prior to testing.

The slump flow and dynamic stability expressed in Visual Stability Index (VSI) for the studied SCCs were measured based on ASTM C1611 standard. The T₅₀ time (flow rate) test, as an indirect measure of viscosity, is the time in second it takes for the fresh SCC’s horizontal spread to reach a diameter of 50 cm (20 inches) from the moment the cone is lifted. The T₅₀ test was evaluated based on ASTM C1611. The J-ring test ASTM C1621, was used to determine the passingability of self-consolidating concretes.

Cylindrical specimens having dimensions of 76×152 mm (3×6 in) were used to conduct compression tests, in accordance to ASTM C39 and C469 to obtain compressive strength and modulus of elasticity, respectively.
ASTM C642 was used to measure water absorption and void content of the studied SCC at the age of 28 and 90 days. Rapid chloride permeability test (ASTM C1202) and rapid migration test (NT Build 443 or AASHTO TP 64-03) were also carried at the ages of 28 and 90 days.

ASTM C672 specifies the mass loss due to the freeze–thaw cycling of SCC specimens (76×152 mm (3×6 in)) being constantly submerged in NaCl solution. This type of exposure simulates applications subjected to deicing chemicals or seawater. Samples were subjected to 56 freeze-thaw cycles with the mass measured after 7, 14, 21, 28, 35, 42, and 56 freezing and thawing cycles. For all the mentioned tests, at each age three samples were tested and their average value was reported as the value of the related mixture design.

4. RESULTS AND DISCUSSION

4.1. FRESH PROPERTIES

The measured slump flow, VSI, T₅₀ (rate of flowability), and J-ring (passingability) for each studied mixtures is shown in Table 5. The measurements reported are the average of three tests.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Slump Flow (mm)</th>
<th>VSI</th>
<th>HRWRA Dosage (% by mass of binder)</th>
<th>J-ring (mm) (h₂-h₁)</th>
<th>T₅₀ (Seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP0</td>
<td>673</td>
<td>0</td>
<td>0.5764</td>
<td>25.4</td>
<td>2.61</td>
</tr>
<tr>
<td>NP15</td>
<td>673</td>
<td>0</td>
<td>0.5534</td>
<td>12.7</td>
<td>2.34</td>
</tr>
<tr>
<td>NP30</td>
<td>673</td>
<td>0</td>
<td>0.4724</td>
<td>12.7</td>
<td>2.03</td>
</tr>
</tbody>
</table>

As can be seen in Table 5, all three SCC types met the target slump flow of 673 ± 25 mm (26.5 ± 1 in). Moreover, VSI of 0 was recorded for all studied SCCs, indicating an excellent display of dynamic stability, and resistance to bleeding and segregation.

The HRWRA dosage for the studied SCC mixtures decreased with an increase in the amount of NP. The admixture dosage decreased 4 and 22% for the SCCs containing 15 and 30% Natural Pozzolan, respectively. This is attributed to the 98% required water for the utilized natural pozzolan as shown in Table 2, which necessitated lesser amount of water to meet the target flowability.

The J-ring test (difference in concrete diameter inside and outside J-ring) results of the SCCs containing natural pozzolan showed improvement in passingability when compared to that of
reference concrete. On average, the SCCs containing natural pozzolan showed 12 mm lesser J-ring as compared to that of the reference mixture.

The $T_{50}$ flow times, which indicates the rate of flowability or viscosity (by inference) of SCC, mixtures improved with inclusion of natural pozzolan. $T_{50}$ decreased by 10 and 23%, respectively, when natural pozzolan replaced 15 and 30% by mass of Portland Cement when compared with the result obtained for the reference concrete.

Improvement in $T_{50}$ and passingability of NP-contained SCCs can be due to the reduction in the amount of HRWRA than that reported for the control concrete. The reduction in HRWRA demand resulted in the NP-contained SCCs to have better viscosity (rate of flow) and improved passingability [16].

4.2. MECHANICAL PROPERTIES

Figure 1 reports the results of compression test. The results revealed that the compressive strength decreased with increases in NP content at all ages except for 140 days. The gap in compressive strength between natural pozzolan-contained SCCs and the reference concrete widened with an increase in NP content and decreased with increases in curing age. The 7-, 28-, and 90-day compressive strength of the NP15 marginally decreased by 7, 0.1 and 0.4%, respectively, as compared to that of the reference concrete. These decreases are minimal and remain within typical concrete variations. Once the natural pozzolan replaced 30% by mass of Portland cement, the compressive strength of NP30 experienced a larger drop in compressive strength throughout the first 90 days of curing. The compressive strength of the NP30 SCCs decreased by nearly 24, 11, and 9% at the age of 7, 28, and 90 day of curing, respectively, when compared to that of the reference concrete. When curing age was extended to 140 days, the compressive strength of both NP15 and NP30 SCCs surpassed that of the control concrete by 3 and 6%, respectively. These findings are similar to those reported by Belaidi et al. [14].

Modulus Of Elasticity (MOE) of the studied SCCs are shown in Table 6. As can be seen, elastic modulus of the studied SCC mixtures decreased with increases in the amount of natural pozzolan. 28-day MOE for NP15 and NP30 decreased by roughly 3, and 8%, respectively, as compared to that reported for the control SCC. When natural pozzolan replaced 15 and 30% by mass of Portland cement, the 90-day MOE of the studied SCCs averagely decreased by 4% when compared to that obtained for the reference concrete.

The observed decrease in the strength of SCCs due to the inclusion of NP can be explained by the fact that the large amount of pozzolan content (more than 15%) in concrete leads to the surplus of small-sized particles, resulting in a slow separation of grain particles to cause reduction in packing density and strength of cementitious materials paste [17, 18]. The increase in the strength of the studied NP contained SCCs with curing age is due to the continued pozzolanic reactivities of the cement and Natural Pozzolan [19]. It should also be noted that the use of polycarboxylate-based HRWRA at high dosages may also lengthen the setting time and negatively affect the early compressive strength of SCCs incorporating Natural Pozzolan, particularly when large dosage of NP are used [20].
Table 6 Modulus of Elasticity

<table>
<thead>
<tr>
<th>Modulus of Elasticity (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28days</td>
</tr>
<tr>
<td>NP0</td>
</tr>
<tr>
<td>NP15</td>
</tr>
<tr>
<td>NP30</td>
</tr>
</tbody>
</table>

4.3. TRANSPORT PROPERTIES

Table 7 illustrates the water absorption and void content of the studied SCC samples at the ages of 28 and 90 days of curing. Water absorption and void content of SCC samples improved by inclusion of Natural Pozzolan. The 28-day water absorption of SCC samples decreased by 16, and 20% when NP replaced 15 and 30% by mass of Portland cement, respectively. The 28-day void content also improved by 17, and 23% for the SCCs containing 15 and 30% NP by mass of total cementitious materials, respectively, as compared to that of the reference concrete.

At the age of 90 days, the water absorption of NP15 and NP30 SCCs were 21 and 27% lower than that of reference concrete. These reductions were 30 and 28%, respectively, for the void content of the NP contained SCCs.
Table 7 Water absorption and void content

<table>
<thead>
<tr>
<th></th>
<th>28 days</th>
<th></th>
<th>90 days</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water absorption (%)</td>
<td>Void Content (%)</td>
<td>Water absorption (%)</td>
<td>Void Content (%)</td>
</tr>
<tr>
<td>NP0</td>
<td>3.27</td>
<td>7.85</td>
<td>3.34</td>
<td>7.98</td>
</tr>
<tr>
<td>NP15</td>
<td>2.76</td>
<td>6.51</td>
<td>2.64</td>
<td>6.31</td>
</tr>
<tr>
<td>NP30</td>
<td>2.60</td>
<td>6.05</td>
<td>2.44</td>
<td>5.78</td>
</tr>
</tbody>
</table>

The results of Rapid Chloride Permeability Test (RCPT) and Rapid Migration Test (RMT) at 28 and 90 days of curing are shown in Table 8. The test results showed improvements in transport properties for the studied NP-contained SCCs when compared to that of the reference concrete. The 28-day RCPT decreased by nearly 3 and 35%, when NP replaced 15 and 30% by mass of Portland cement, respectively. The 90-day RCPT also improved by approximately 53 and 76%. For the rapid migration test, an improvement of 5 and 6% was observed at 28 days of curing; and 44 and 55% at 90 days of curing when natural pozzolan replaced 15 and 30% by mass of Portland cement, respectively.

The improvement in the transport properties of NP-contained SCCs could be due to the filler effects of Natural Pozzolan, improving transition zone and providing a more dense microstructure [21, 22, and 23].

Table 8 - Transport Properties

<table>
<thead>
<tr>
<th></th>
<th>28 days</th>
<th>90 days</th>
<th>28 days</th>
<th>90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP0</td>
<td>3943.33</td>
<td>2305.00</td>
<td>23.70</td>
<td>14.89</td>
</tr>
<tr>
<td>NP15</td>
<td>3836.67</td>
<td>1092.33</td>
<td>22.39</td>
<td>8.39</td>
</tr>
<tr>
<td>NP30</td>
<td>2566.67</td>
<td>542.33</td>
<td>22.33</td>
<td>6.76</td>
</tr>
</tbody>
</table>

4.4. DURABILITY PROPERTIES

The results of freeze-thaw cycling tests are documented in Figure 2. As can be seen, until 42 cycles, not only there was not any mass loss, but also the SCC samples gained weight which is attributed to the penetration of salt water into the samples. The net weight of absorbing salt water and loosing particles (Mass loss) was negligible until 42 freezing and thawing cycles. Beyond this point, the SCC samples started to loose mass. As shown in Figure 2, NP-contained concretes had lower mass loss than the reference concrete for up to 56 cycles. The mass loss of the studied SCCs after 56 cycles are presented in Table 9. As can be seen, the 56-cycles mass loss decreased by 6 and 16% when NP replaced 15 and 30% by mass of Portland cement.
cement, respectively. This improvement is attributed to the positive influence of NP on pore structure, as corroborated by the reported results of transport properties [21, 22].

![Figure 2 Freeze thaw cycling test](image)

<table>
<thead>
<tr>
<th>Weight change after 56 cycles</th>
<th>NP0</th>
<th>P15</th>
<th>P30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass loss after 56 cycles</td>
<td>-0.39%</td>
<td>-0.36%</td>
<td>-0.31%</td>
</tr>
</tbody>
</table>

### 5. CONCLUSIONS

This study evaluated fresh, mechanical, transport, and durability properties of natural pozzolan contained self-consolidating concrete. The following conclusions can be drawn based on the results presented:

1. Flow properties of the studied SCCs improved with increases in the amount of Natural Pozzolan.
2. Compressive strength and modulus of elasticity slightly decreased with increases in NP dosage at the curing ages of 7, 28 and 90 days. However, at 140-day curing, both studied NP contained SCCs displayed compressive strengths exceeding that of the control concrete.
3. An increase in the amount of NP, as a partial replacement of OPC, resulted in improvement of water absorption, void content, RCPT, and RMT of the studied SCCs.
4. Mass loss due to freezing and thawing cycles decreased with an increase in replacement level of OPC by natural pozzolan.

6. REFERENCES


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Recycled aggregate: compliance with legal requirements

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Key words: construction and demolition waste, aggregates, reuse

Abstract

Construction and demolition waste (C&DW) has acquired particular prominence in Europe in the present context of sustainability. The C&DW generated in Spain accounts for one of the country’s most significant waste flows. Despite the present crisis, the national construction industry generated 79.4 million tonnes of such waste in 2009-2013. Against this backdrop, the use of recycled aggregate is becoming standard practice in construction, particularly in civil works. According to the European Aggregates Association (UEPG), 183 million tonnes were used in Europe in 2012. This paper reports on the analysis of the physical, chemical, mineralogical and mechanical properties and chemical composition of aggregates sourced from construction and demolition waste management plants. The findings are compared to the legal requirements presently in place and a practical guide is put forward for the possible use of this material in construction.
1 INTRODUCTION

The pressure brought to bear on finite and often scant natural resources by the demand and competition for raw materials that characterise modern societies is debilitating the environment and hastening its degeneration. Ever since the industrial revolution, economic growth has been patterned on a linear ‘extract-produce-consume-eliminate’ model that assumes resource abundance, availability, ease of acquisition and inexpensive elimination. That attitude, however, constitutes an increasingly obvious threat to the competitiveness and sustainability of modern societies. Migration toward a circular economy in which waste-containing materials are reused to produce new products or raw materials is consequently imperative to guaranteeing the intelligent, sustainable and integrated growth described in Europe’s 2020 Strategy. In that context, recycling or valorising industrial waste will have a leading role to play in the decades to come [1].

Construction and demolition waste (C&DW) is a particularly prominent item on Europe’s present sustainability agenda, for it constitutes one-third of the total waste generated in the EU [2]. The 27 member countries are estimated to produce around 460 million tonnes of construction and demolition debris yearly [3]. Spain alone generated 27 million tonnes in 2012, according to data published in the Nationwide Waste Management Plan 2016-2022 (Spanish initials, PEMAR) [4]. That number, a drastic drop from the 42 million tonnes recorded in 2007, is in keeping with the overall downward trend observed in the construction industry in recent years.

Present C&DW reuse rates range widely in the EU-27, from 1 % in Cyprus to over 85 % in the Netherlands, Denmark, Estonia and Germany. The mean rate for the EU-27 is under 50 %, far below the 70 % target defined for 2020 [2]. In Spain in particular, it varies from 14 to 17 % (depending on the source), i.e., a value lower than the mean in Europe and other industrialised regions [5].

C&DW is treated in specific plants [6] where, after preliminary inspection, it is subjected to a series of operations (initial screening, crushing, magnetic separation, hand-picking and sieving to name a few) that may vary depending on the initial composition of the waste, end product requirements and plant technology.

A number of researchers [7-15] have focused on the possibility of using recycled construction and demolition waste aggregate in construction to reduce the volume of waste stockpiled in landfills. Such applications would also create jobs, further sustainable development by lowering CO2 emissions and reduce the exploitation of natural resources [16]. These authors reported that recycled aggregates can be used in civil works provided they meet the requirements laid down in the existing legislation.

In this study, recycled C&DW aggregate from several waste management plants in the Spanish region of Extremadura was characterised to assess its possible aptness for use in construction. To that end, the chemical, mineralogical, physical and mechanical properties of the samples were analysed and the findings compared to the requisites established in the existing legislation on aggregate for use in concrete manufacture and hot mix asphalts (HMA).
2 EXPERIMENTAL

2.1 Materials

Four recycled aggregates were collected from two permanent recycling plants in the Spanish region of Extremadura and stored in the laboratory for subsequent study (Figure 1). They were pre-conditioned prior to characterisation, essentially by heating at 50 °C to a constant mass (approximately 72 hours).

![Figure 1: Example of the recycled aggregates analyzed: a) Coarse aggregate and b) Fine aggregate](image)

The dried aggregate was then characterised for mineralogy, chemistry, geometry (particle size distribution and, fines content), and physical (absorption, flakiness index, fracture faces) and mechanical (Los Angeles coefficient) properties.

2.2 Methodology

The chemical composition of the recycled aggregates was determined with pressed powder XRF analysis performed on a Bruker S8 TIGER XRD spectrometer. The readings were processed with SPECTRA plus QUANT EXPRESS software.

X-ray diffraction (XRD) mineralogical studies were conducted on a Bruker D8 Advance diffractometer fitted with 1.54 Å CuKα radiation, a Lynxeye detector with a 3 mm antiscatter slit and a (0.5%) Ni K-beta filter (i.e., Kα2 was not eliminated). Readings were taken at 20 diffraction angles ranging from 5° to 60°.

<table>
<thead>
<tr>
<th>Property</th>
<th>Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle size distribution</td>
<td>EN 933-1 [17]</td>
</tr>
<tr>
<td>Composition</td>
<td>EN 933-11 [18]</td>
</tr>
<tr>
<td>Water uptake</td>
<td>EN 1097-6 [19]</td>
</tr>
<tr>
<td>Flakiness index</td>
<td>EN 933-3 [20]</td>
</tr>
<tr>
<td>Fracture face</td>
<td>EN 933-5 [21]</td>
</tr>
<tr>
<td>Los Angeles coefficient</td>
<td>EN 1097-2 [22]</td>
</tr>
</tbody>
</table>

Table 1: Physical and mechanical properties of the recycled aggregates
Table 1 lists the properties studied and methodology used for the geometric, physical and mechanical characterisation of the recycled aggregate. Representative samples of the aggregate were selected for these tests further to the procedure laid down in European standard EN 932-2 [23].

3 RESULTS AND DISCUSSION

3.1 Chemical and mineralogical composition

Figure 2 depicts the chemical composition found for the recycled aggregate. The main constituents detected were silicon oxide (>48.0 wt%), aluminium oxide (>10.0 wt%), iron oxide (>3.0 wt%) and calcium oxide (>6.5 wt%), in addition to other minority oxides.

The values for SiO₂, Al₂O₃, CaO and loss ignician (LOI) lay within the ranges observed in previous studies by Limbachiya et al. [24] and Medina et al. [5, 11] for this type of recycled aggregate.

According to the XRD analysis, irrespective of recycled aggregate typology, the mineralogical composition consistently included quartz, feldspars, phyllosilicates, calcite and hematites. These findings are consistent with the mineralogy determined by Vegas et al. [25] for other C&DW generated on the Iberian Peninsula.

3.2 Geometrical requirements

Further to the particle size distribution of the recycled aggregate shown in Figure 3, the top sizes found for the coarse recycled aggregate (AG-1 and AG-2) and recycled fines (AF-1 and AF-2) were 20 and 4 mm, respectively.

These aggregates also proved to be graded, ensuring more intense interaction between particles and hence greater concrete or HMA compactness and mechanical strength [26].
The figure also shows the particle size range recommended by the structural concrete code presently in effect in Spain (EHE-08) for fines (<4 mm). The recycled fines lay within the range specified and had a percentage of fine particles (<0.063 mm) under the EHE-08 [27] ceiling. Table 542.8 on particle size ranges for aggregate in Spain’s general specifications for road asphalt PG-3 [28] sets out the recommended particle size ranges by type of hot mix asphalt. The mix of coarse and fine aggregates conformed to all those recommendations.

### 3.2 Compositional classification

Both aggregates comprised primarily stony materials (Rc > Ru > Rb > Ra), as the data in Table 2 show; the minority components included glass, metal and wood.

<table>
<thead>
<tr>
<th>Constituent (wt.%)</th>
<th>AG-1</th>
<th>AG-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete and mortar (Rc)</td>
<td>46.98</td>
<td>42.58</td>
</tr>
<tr>
<td>Unbound aggregate (Ru)</td>
<td>44.92</td>
<td>56.37</td>
</tr>
<tr>
<td>Clay-based materials (Rb)</td>
<td>7.15</td>
<td>0.66</td>
</tr>
<tr>
<td>Bituminous materials (Ra)</td>
<td>0.56</td>
<td>0.08</td>
</tr>
<tr>
<td>Floating particles (FL)</td>
<td>0.17</td>
<td>0.02</td>
</tr>
<tr>
<td>Gypsum (Y)</td>
<td>0.04</td>
<td>0.00</td>
</tr>
<tr>
<td>Others</td>
<td>0.19</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Based on the data in the table, AG-1 can be classified as a recycled mixed aggregate and AG-2 as a recycled concrete aggregate as defined in EHE-08 [27], for their Rc+Ru contents were respectively observed to be <95 % and >95 %.

### 3.2 Physical and mechanical requirements

Table 3 compares the physical and mechanical characteristics of the recycled aggregates to the requisites laid down in Spanish structural concrete code (EHE-08) and the
recommends in the country’s general specifications for roads and bridges (PG-3) for coarse and fine aggregates used the manufacture of concrete and hot mix asphalts, respectively. The values given are consistent with the findings reported by other authors for recycled aggregates [29-33].

Table 3: Physical and mechanical properties of the recycled aggregates

<table>
<thead>
<tr>
<th>Property</th>
<th>AF-1(1)</th>
<th>AF-2(1)</th>
<th>AG-1(2)</th>
<th>AG-2(2)</th>
<th>EHE-08(3)</th>
<th>PG-3(4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water absorption (wt%)</td>
<td>5.39</td>
<td>4.42</td>
<td>5.27</td>
<td>3.63</td>
<td>≤ 5</td>
<td>-</td>
</tr>
<tr>
<td>Flakiness index (wt%)</td>
<td>-</td>
<td>-</td>
<td>9.55</td>
<td>20.85</td>
<td>&lt; 35</td>
<td>≤ 25</td>
</tr>
<tr>
<td>Crushed and broken surfaces (wt%)</td>
<td>Cc*</td>
<td>-</td>
<td>99</td>
<td>97</td>
<td>-</td>
<td>≥ 90</td>
</tr>
<tr>
<td>Los Angeles coefficient (wt %)</td>
<td>-</td>
<td>-</td>
<td>36</td>
<td>27</td>
<td>≤ 40</td>
<td>≤ 30</td>
</tr>
</tbody>
</table>

Notes
(1) Recycled fine aggregate; (2) Recycled coarse aggregate; (3) Requisites for aggregate used in concrete manufacture; (4) Recommendations laid down for aggregate used in hot mix asphalt for road bases and category T2 heavy vehicle traffic; *Crushed particles; **Completely rounded particles

As the table shows, all the recycled aggregates were found to be EHE-08 compliant, with the exception of the water absorption coefficients exhibited by AF-1 and AG-1. This would not constitute a constraint where the aggregate replacement ratio is under 100 %, however, for the water absorption value in the recycled - natural coarse aggregate mix would be lower than 4.5 % and hence compliant with the recommendations in Annex 15 to EHE-08 on the use of recycled aggregate.

The materials also conformed to all the recommendations for aggregate used in hot mix asphalts for road bases and T2 heavy vehicle traffic, except for the Los Angeles coefficient exhibited by aggregate AG-1, which was slightly higher than the value for AG-2. As above, this would not constitute a limitation where the aggregate replacement ratio is under 100 %, for the resistance to fragmentation in natural aggregate is <25.

4 CONCLUSIONS

The conclusions drawn about the recycled aggregate analysed here are listed below.
- It comprises primarily silicon and aluminium oxide.
- It is characterised by a graded particle size distribution and the fines are EHE-08-compliant.
- It meets most of the physical and mechanical requirements and recommendations laid down in EHE-08 and PG-3.
- Where the replacement ratio is under 100 %, it can be used to manufacture structural and non-structural concrete as well as hot mix asphalt for road bases and category T2 heavy vehicle traffic.
5 ACKNOWLEDGEMENTS

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REFERENCES


Research on spray type high ductility PVA fiber concrete used for the deep roadway supporting key technology

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Key words: roadway support; fiber reinforced concrete; high toughness; shotcrete layer

Abstract. The supporting structural of deep coal mine roadway, whose surrounding rock has the characteristics of soft rock engineering, is required to be of high strength and certain toughness. The biggest disadvantage of common shotcrete as supporting material is the great brittleness, poor toughness and easy cracking destruction under the rock pressure. To solve this problem, the technique of adopting high ductility PVA fiber shotcrete as the supporting material is proposed in this article. Moreover, the PVA fiber dispersion problem is solved by using horizontal mixer to mix and bag the fiber, fly ash, sand and other materials together on the ground in advance, the under-well concrete mixing problem is handled by the study of coal mines with small spiral type mixer, and the mixer equipment selection and spraying techniques are introduced. Real practice has been carried out in No.34 mining area of cheji colliery. Result shows that the strength of the shotcrete layer reached 32 Mpa, the toughness $\tau_{30}$ reached 45.65 which is 38 times higher than common concrete, the injection efficiency reached 3.8 m$^3$/h. It provides a new method for deep mine roadway and soft rock tunnel supporting, which has a broad prospect.

1 INTRODUCTION

In deep mine, with the increase of mining depth, the ground stress increases gradually, if the appropriate support is not be taken, the deformation of roadway surrounding rock is more severe and the support is more difficult. And the accumulation of deformation will lead to the unstable failure of mine roadway. At present, the following methods are generally used in the support of deep rock roadway in coal mine, such as bolt-shotcrete support, bolt-mesh-cable
support, bolt-beam-mesh support. These ways of supports all use sprayed concrete shotcrete layer which is poor toughness, low strength and great brittleness. Under the condition of large deformation and high stress of surrounding rock, the brittle failure occurs easily, and the shotcrete layer is easy to crack and fall off, so that the supporting failed and personal safety is threatened. According to existing research, Dry sprayed concrete is commonly used in coal mine, of which the compressive strength is only 14 ~ 18MPa and the flexural toughness $I_{20}$ is only 1.19. Therefore, the research of support with the PVA composites with high strength and high ductility (the compressive strength is about 30MPa, the flexural toughness is above 30) is proposed. If the traditional supporting methods continue to be used, the roadway is often destroyed in the service period and needs to be renovated frequently. The cost of repairing a meter roadway is about ¥3800 every year, which seriously influenced the normal production of the mine and the economic benefit of coal mine enterprises.

2 CHARACTERISTICS OF ECC MATERIALS

ECC (Engineered Cementitious Composite) is an improved product for FRC (Fiber Reinforced Concrete). The research and development of ECC adopted the integrated design method which is based on the combination of unique structure and material and it is random distribution and short fiber reinforced cementitious composite designed by micro mechanical. ECC usually uses cement, mineral admixture, quartz sand as matrix, and fiber as reinforcement material. If the fiber volume fraction is 2%, the ultimate tensile strain is usually within the range of 3% ~ 7% and in saturation state the width of the multi-slit cracking is less than 0.1mm. ECC has a tensile strength performance similar to metal material, and its ultimate tensile strain is equivalent to the plastic deformation capacity of steel. ECC is a kind of deformable concrete material similar to metal material, so it is commonly known as bendable concrete. As shown in Figure 1.

3 PREPARATION METHOD OF PVA-ECC

Engineered cementitious composite(ECC) also can be called as the super high toughness PVA fiber concrete which is made of polyvinyl alcohol fiber (PVA) [1]. Combined with Super-Plasticizer and air entraining agent, the super high toughness fiber concrete can be obtained with good macro properties and micro interfacial adhesion. Super-Plasticizer is used to control the coagulation phenomenon of cement particles, and Viscosity Modifier can make cement particles reach to the stable state by reducing the shear viscosity of cement paste [2]. Viscosity Modifier is useful to improve the fluidity of ECC and avoid the segregation phenomenon, but some bubbles (Volume content is about 20%) are produced which are relatively large and can not be spilled in the mixing process. The workability of the super high toughness fiber concrete is mainly influenced by the small size particles. With appropriate water cement ratio, the mixture in the mixing process can be liquefied, which can
decrease the consistency of super high toughness fiber concrete in the mixing process and make the cement, sand, other solid particles and fibers be evenly distributed, so as to improve the fluidity and maintain a good and stable microstructure.

3.1 Test condition

(1) Modified polypropylene fiber is domestic PVA fine fiber, and the material parameters of this kind of fiber are as follows: Tensile strength is greater than 1560MPa, length is 12mm, equivalent diameter is 39μm, Strong acid and alkali resistance; flavourless; no water absorption; good dispersion; elongation at break of fiber is 7%; flame retardant and antistatic performance, cost-effective, good for human safety, suitable for large areas of Engineering.

(2) The test adopted Huilong PO.42.5 brand Portland cement produced by Yongmei Group Huilong Cements limited company.

(3) The size of common river sand used in the test is less than 0.5 mm.

(4) Fly ash adopted in the test is from Gaozhuang power plant’s first-order fly ash. The test block size of compression strength test is 100mm×100mm×100mm, and the test is conducted according to GB/T50081-2002 common concrete mechanical properties the test method standard. The test block size of tensile stress and strain test is 250mm×40mm×20mm. Considering the discreteness of the test results, each ratio has 3 groups, and each group has 3 pieces to test. Select out the group of the minimum date discreteness, and using its arithmetic mean as the experimental results.

3.2 Result of tests

In the tests, the content of polyvinyl alcohol fiber (PVA) is different in proportions, including the reference concrete C25, a total of 5 Series, in order to compare the effect of reinforcing and toughening. The content of polyvinyl alcohol fiber (PVA) is expressed by volume ratio. In the tests, the volume content of polyvinyl alcohol fiber were 2.0%, 1.8%, 1.5%, 1.2% and 0%, respectively, corresponding to PVA2.0, PVA1.5, PVA1.8, PVA1.2 and C, concrete mix proportion is shown in Table 1. These 5 series of Concrete mix proportion are carefully chosen after comparing the results of a large number of tests, and the content of the comparison includes the effect of the reinforcing and toughening and the requirements of spray process for shotcrete and so on. Test results are shown in Table 2 and Figure 2.

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Fiber content (P/%)</th>
<th>Water content (kg/m³)</th>
<th>Sand rate (%)</th>
<th>Water-binder ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0</td>
<td>360</td>
<td>80</td>
<td>0.40</td>
</tr>
<tr>
<td>PVA2.0</td>
<td>2.0</td>
<td>360</td>
<td>80</td>
<td>0.40</td>
</tr>
<tr>
<td>PVA1.8</td>
<td>1.8</td>
<td>360</td>
<td>80</td>
<td>0.40</td>
</tr>
<tr>
<td>PVA1.5</td>
<td>1.5</td>
<td>360</td>
<td>80</td>
<td>0.40</td>
</tr>
<tr>
<td>PVA1.2</td>
<td>1.2</td>
<td>360</td>
<td>80</td>
<td>0.40</td>
</tr>
</tbody>
</table>

The results of tests show that the strength of the high ductility concrete has reached more than 2.2 times the strength of the sprayed concrete. Bending toughness reached more than...
Yuanzhi Bi, Shumei Gao

Table 2 The average strength and bending toughness of specimen

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Compressive strength /MPa</th>
<th>Bending toughness I20</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>35.55</td>
<td>1.35</td>
</tr>
<tr>
<td>PVA2. 0</td>
<td>29.32</td>
<td>58.64</td>
</tr>
<tr>
<td>PVA1. 8</td>
<td>31.54</td>
<td>47.75</td>
</tr>
<tr>
<td>PVA1. 5</td>
<td>32.35</td>
<td>43.96</td>
</tr>
<tr>
<td>PVA1. 2</td>
<td>33.45</td>
<td>38.56</td>
</tr>
</tbody>
</table>

Fig.2 Force-displacement curve of PVA2.0

43.44 times. So it meets the engineering requirements.

4 CONDITION OF ROADWAY IN THE TESTS

The roadway of the tests is Down-Dip Seam Gateway in Cheji Mine of Yongmei Group, which is buried deep in the ground of -635m to -700m. The roadway is used for mining ventilation of 34 mining area. Designed length is 1100m and designed slope is 10°~20°. Service life is 20 years. The roadway roof of which the thickness is 5.15m is made of fine sandstone and the roadway floor of which the thickness is 6.3m is made of aluminum mudstone. The roadway section is a semi-circular arch section, and the specifications are as follows: Width: 4500mm, arch radius: 2250mm, the height of wall: 1400mm, the thickness of floor: 100mm, the height of base: 100mm. 17 bolts and 3 cables were constructed with the method of Full-section construction of the roadway. Bolt size is φ20mm×2500mm and its spacing is 600mm×600mm. Cable size is φ18.9mm×7200mm and its spacing is 1500mm×1000mm.

5 CONSTRUCTION TECHNOLOGY

5.1 Mechanical equipment selection and mixing technology

(1) Sand, fiber and fly ash were evenly stirred on the ground by Horizontal SJ-750 mixer in
advance. After mixed 2 min, the mixture was bagged and transported to the underground by mine car. Two different types of JPZ-7I-L spiral mixers are used in the underground. One is used for mixing various materials under dry condition, and the other is used to mix water and water-reducing agent to turn them into wet sprayed material, as shown in Figure 3. Finally, pour the mixture into the feed inlet of wet spraying machinery to spray them onto the surface of rock. Efficiency of the mixer is 3.8 m³ per hour which is correspond to wet spraying machinery. The type of Wet Spraying Machinery is the explosion-proof PS5I-H, which is specially designed for wet sprayed fiber concrete in the coal mine.[6]

(2) Wet spurting craft applied in concrete spraying mechanical can improve working environment.

![Fig.3 Spray PVA-ECC matching equipment](image)

1—spiral mixer  2—spiral mixer  3—wet spraying machinery

![Fig.4 Spraying process](image)

5.2 Spraying technology

(1) To improve the toughness of the shotcrete layer, high ductility fiber reinforced concrete support should be applied in the underground roadway. And in order to ensure the quality of construction, wet spurting craft should be taken. The shotcrete layer is divided into two layers. The first shotcrete layer of which the thickness is 30 ~ 40mm is used to make the surrounding rock surface level with plain concrete. After the strength of the first shotcrete layer reached 8 ~ 10MPa, drill and then install anchor, steel meshes and anchor cable in designed position.
The second shotcrete layer made of PVA-ECC materials was constructed at the place that is 35~60m away from the joint and its thickness is 60 ~ 80mm.

(2) It is an important part of wet spurting craft to control the amount of water, because too much water can influence the result of wet spraying. People should take into account the composition of sand and gravel transport system when adding water. Productivity of the wet spurting craft is 3.8m³/h and working wind pressure is 0.25~0.35MPa. Spraying technology is shown in Figure 4. Effect of shotcrete layer in underground roadway is shown in Figure 5. After sprayed, the underground roadway should be leveled with in 15-25mins to keep the surface of the roadway smooth.

6 TEST EFFECTS AFTER SPRAYING
A big penal was sprayed same as shotcrete layer in the construction site to be a sample. After 28 days’ maintenance, a piece of the penal is tested. Its strength reach 32MPa and bending toughness $I_{20}$ is 45.65, which is 38 times higher than common shotcrete. Roadway in test section has been used more than 3a. There is still no cracking or peeling off phenomenon on shotcrete layer. It can still be normally used. But roadway out of test section which is supported by common shotcrete has cracked the spray coat has fallen off. It has been repaired. As a consequence, in this comparison between the two kinds of roadway, PVA high ductility materials applied to support deep mine roadway has more active effect.

7 CONCLUSIONS
(1)Roadway in test section has been constructed for 3a and is still in normal use without peeling off phenomenon. However, the roadway outside the test section has been renovated because of cracking and peeling phenomenon. These show that roadway in test section has obvious effect.

(2) Supporting structure in deep mine requires a certain amount of deformation to adapt to the deformation and energy release of surrounding rock in the early stage and a certain amount of support strength in the later stage. The strain-hardening characteristics of PVA meet the requirements of surrounding rock support in deep mine, and provide new materials and new structures for the surrounding rock support of deep mine roadway.

(3) The strain-hardening characteristics of PVA-ECC are the result of the steady state cracking, which is the source of PVA's unique toughness. That the strain hardening of PVA is a process of injury accumulation is different from metal materials. So it is also called quasi strain-hardening.
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(4) Because of the hydrophilic properties of PVA, PVA-ECC can be combined with cement soundly. In order to ensure that the fibers are not pulled off when they are pulled, the surface of the fibers should be covered with oil to reduce bond strength between fiber and cement. Strain-hardening characteristics of PVA-ECC concrete material provide a high ductility material for deep roadway support. With the Popularization of domestic PVA, the price is only one-fourth of the imported PVA, so the price advantage has been more and more obvious. And the application prospects of PVA will be broader.

REFERENCE

REUSE OF WASTE DISCARDED BY THE CERAMIC INDUSTRY AS HIGH QUALITY COMPONENTS OF CONCRETE

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Key words: Recycled Concrete, waste ceramic industry, high strength

Abstract. Numerous studies have already been conducted on the use of construction and demolition waste (CDW) for the production of concrete, grounded for reasons of environmental nature, due to the generation of large volumes of waste and the difficulty of its management.

The objective of the research is to expand the study to non-hazardous industrial waste, in order to give value to these wastes as high quality components of concrete, since in technical terms it can be concluded that the quality of CDW is lower than the quality of natural aggregates.

This study evaluates the behavior of non-hazardous waste produced by the ceramic industry reused as components of concrete. The Density, water absorption and particle size of the ceramic waste is determined.

An extensive test program is performed in order to evaluate the influence of replacing natural expensive aggregates for this type of waste in order to obtain HPC, then performing a valorizing this product in a performing way.

Different types of HPC were analyzed modifying cement content, but also aggregates grading.

Results show a that this type of byproducts can be used to reach concretes with strength equivalent to those made with the most selective aggregates.

More efficient and sustainable concretes can be produced, even in the high level of quality requirements

1 INTRODUCTION

Numerous studies have already been conducted on the use of construction and demolition waste (CDW) for the production of concrete, grounded for reasons of environmental nature, due to the generation of large volumes of waste and the difficulty of its management.

Furthermore, there is a legal obligation of having waste plans. In the Waste Framework Directive (2006/12/EC) [1], it is specified that Member States shall be required to draw up as
soon as possible one or more waste management plans. These plans include wastes other than CDW. Other than the legal obligations, numerous organizations, such as, the European Environment Agency, the US Environmental Protection Agency or the World Bank are working to reduce the amount of waste produced.

Actually, the World Bank Group-Global Environment Facility (GEF) Program has integrated more than 790 investment projects and programs in 120 countries, granting actions to combat major environmental issues [2].

By reusing the waste produced we reduce the amount of raw materials and natural resources used. Therefore,

The objective of the research is to expand the study to non-hazardous industrial waste, in order to give value to these wastes as high quality components of concrete, since in technical terms it can be concluded that the quality of CDW is lower than the quality of natural aggregates, such as the GEAR Project [3] and [10] concluded.

The GEAR[3] project states that in the case of structural and nonstructural concrete defined by their resistance feature, it should be noted that:

- The dosage may require more cement, based on resistance to be achieved.
- Compressive strength, for equivalent amounts of cement in a conventional mass concrete decreases about 2.5% for every 10% of CDW substitution in structural concrete. The compressive strength decreases about 2% for every 10% of CDW substitution in nonstructural and cleaning concrete.

High Performance Concrete (HPC) is a type of concrete with three characteristics: high strength, high durability and high workability. In order to achieve these properties, HPC needs less water and more cement than conventional concrete. In order to achieve high strength and high durability, there must be very little voids. Therefore very fine aggregates and silica fume, which has a lower diameter than cement, are used in HPC to fill the voids between the aggregates [8].

In the area of Castellón the ceramic industry is very important. Due to the legal obligations of the waste plans, most of the waste created is reused in the production process, but there are still two kinds of ceramic waste that cannot be reused and are still discarded into dumping sites. This type of waste are clearly different to those from CDW as show a very low absorption, and are clean and strong. Since its quality seems promising and the particle size distribution of these two ceramic wastes are very similar to the particle size distribution of the aggregates used in HPC, it was decided to study the properties of a HPC substituting the natural aggregates by those ceramic waste and compared the compressive strength of the mortars produced with the standard raw materials with the compressive strength when the commercial products were substituted by the ceramic waste products.

2 CHARACTERISTICS OF CERAMIC WASTES STUDIED

TAU produces different types of ceramic products, such as technical porcelain, stoneware, etc... In this study it was used two different kinds of waste that TAU produces and actually are disposed of in landfills.

The called "coarse" waste product in this study is obtained from the rejections produced at the sieving process of the materials grinded at the continuous ball mill. The rejection product is mainly crushed silica stone.
The "fine" waste product is produced at the grinding wheel, one of the last steps of the productive process, which consists in polishing and shaping the final product. The powder produced is mixed with water, collected at a gutter and directed to the company’s waste water treatment plant. The decanted sludge is dried in a filter press. This sludge is the waste used in the study.

The particle size of the coarse waste product is in the range from 1 to 4mm, and the particle size of the fine waste product is less than 0,125 mm. These waste products will be compared, respectively, with commercial silica flour and commercial aggregates. TAU has performed waste characterization tests to both ceramic waste products and they are classified as non-hazardous waste products.

3 EXPERIMENTAL PROGRAM

In order to determine if the ceramic waste products were suitable to be used as HPC, test were conducted to determine their density, water absorption (UNE-EN_1097-6:2014) [5] for the coarse aggregate (table 1) and particle size distribution ( figure 1 and table2) (UNE-EN_933-1-2012) [6] for the fine aggregate.

Table 2 also shows the size distributions of the others components used in this work to produce HPC, including 3 commercial aggregates with nominal sizes: 0,5/2mm (NA1); 0,25/1,25mm (NA2) and 0,063/0,63mm (NA3), cement I 42,5R, silica fume 940-U undensified produced by Elkem Microsilica (SF) and silica flour U-S500 produced by SIBELCO.

The FCW shows a grading close to this one of the cement and similar to this of the silica flour, then it seems logical to be propose to replace this expensive material. The coarse waste is higher than the more coarse aggregate used for HPC in this study.

<table>
<thead>
<tr>
<th>DENSITY</th>
<th>SAMPLE 1</th>
<th>SAMPLE 2</th>
<th>AVERAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_a$</td>
<td>2,65</td>
<td>2,65</td>
<td>2,65</td>
</tr>
<tr>
<td>$\rho_{rd}$</td>
<td>2,54</td>
<td>2,53</td>
<td>2,54</td>
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<tr>
<td>$\rho_{ssd}$</td>
<td>2,59</td>
<td>2,57</td>
<td>2,58</td>
</tr>
<tr>
<td>WATER ABSORPTION (%)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WATER ABSORPTION (%) at 24 h</td>
<td>1,61</td>
<td>1,76</td>
<td>1,69</td>
</tr>
</tbody>
</table>
Water absorption fine aggregates: 0.1%. The accepted usually accepted water absorption in codes [7] for both fine and coarse aggregates is 5%.

Table 2. Particle Size Distribution Waste Ceramic and others concrete components

<table>
<thead>
<tr>
<th>D(mm)</th>
<th>NA1</th>
<th>Coarse ceramic waste (CCW)</th>
<th>NA2</th>
<th>NA3</th>
<th>Silica Flour</th>
<th>Fine ceramic waste (FCW)</th>
<th>CEMI42,5R</th>
<th>Silica Fume(SF)</th>
</tr>
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<tbody>
<tr>
<td>4</td>
<td>100%</td>
<td>100% 100% 100% 100% 100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
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<tr>
<td>2</td>
<td>97%</td>
<td>6% 100% 100% 100% 100%</td>
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<td>100%</td>
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<td>20%</td>
<td>26%</td>
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<td>100%</td>
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<td>0.001</td>
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</tr>
<tr>
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</table>
To analyze the viability of replacing natural aggregates by the proposed waste four base HPC were used as reference. The base concretes were defined by the cement content, and the solid components grading.

Two different percentages of cement, 15% or 20% of the volume, which is equivalent respectively to 360 kg/m³ or 481kg/m³. The amount of water was fixed to a constant value of 217 kg/m³ which resulted in W/C ratios of 0.6 for the specimens containing 360kg/m³ of cement and 0.45 for the specimens containing 481kg/m³ of cement. A silica fume dosage was fixed at 20% of the cement for all concretes.

Two different solid components grading was propose for each one of the cement content. This grading was defined by packing theory model we followed is the A&A modified model, which was studied in previous experimental projects performed by the Camacho [8]. According with this theory the grading of all solids including binders must to fit a model defined by the equation

$$\frac{CPFT}{100\%} = \frac{D^q - D_{Min}^q}{D_{Max}^q - D_{Min}^q}$$

(1)

Two values of the q exponent (q=0.25 or q=0.35) were analyzed for each cement dosage and then 4 reference concrete were defined.

Three recycled concretes were derived from each one of the base concretes: a) replacing aggregate NA1 aggregate by CCW, b) replacing silica flour by FCW. c) replacing both materials.

The criteria of replacing aggregates was to define the final mix proportioning of different aggregates in every concrete by fitting the grading to the theoretical one.

It is possible, by varying the proportions of the various products to approach the model distribution curve by minimizing the average standard deviation calculated at all the sieves opening used. Figure 2 shows an example result for one of the concretes.

Figure 2. Results for q=0.35 and 20% of cement
The 16 different mixes composition were studied (Table 3)
The mixes composition can see in table 4.

Consistency test were performed for every mortar mixture by using a mini cone in order to
determine the amount of plasticizer admixture necessary to obtain a diameter between 20-22
cm. This way we could compare the results of different mortar mixtures for a similar
workability. A superplasticizer Sika 20 HE was used

One the final mix design was defined a 100 mm cubic specimen was cast. The specimens
were conserved for 7 or 28 days in a 20 °C an 95% RH chamber.

<table>
<thead>
<tr>
<th>Table 3. Mixes studied</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Fine ceramic waste (FCW)</td>
</tr>
<tr>
<td>Fine natural aggregate (Silica flour)</td>
</tr>
</tbody>
</table>

The amount of plasticizer needed in order to obtain the same workability is slightly less for
the mixes containing coarse ceramic waste, being always less than 6 kg/m³. Considering as
binders all of the components with a particle size distribution equal or smaller to cement, the
ratio Plasticizer/Binders range is in between 0,6% and 1,12%.

Several studies have proven that the order in which the components are introduced in the
mixture has an influence in the compression strength of the specimens [9], [10]

In order to perform the mix, first the aggregates and ceramic waste were introduced in the
mixture, followed by the cement and silica fume. Aggregates and binders were mixed during 1
minute and subsequently water and the plasticizer admixture were added. The total mixing
time varies between 4’30” and 5’30”.

five batches were performed for each concrete. One to define the admixture dosage and
test the consistency and four more batches with the same dosage, obtaining four cubic
specimens of 100 mm side. Specimens were tested in compression, two at 7 days and the
other two at 28 days, following the UNE-EN 12390-3 Standard [11].

4 RESULTS

In tables 5 and 6 and figures 3 and 4 it may see the compression strength of the 16
mixtures after 7 and 28 days. All of the mixtures give compression strengths greater than 50
MPa after 7 days and 71 MPa after 28 days.
<table>
<thead>
<tr>
<th>(kg/m³)</th>
<th>q=0.25</th>
<th>q=0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C15%</td>
<td>C20%</td>
</tr>
<tr>
<td>CCW</td>
<td>448</td>
<td>449</td>
</tr>
<tr>
<td>NA2</td>
<td>337</td>
<td>333</td>
</tr>
<tr>
<td>NA3</td>
<td>522</td>
<td>531</td>
</tr>
<tr>
<td>FCW</td>
<td>408</td>
<td>277</td>
</tr>
<tr>
<td>CEMI42.5R</td>
<td>360</td>
<td>481</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>72</td>
<td>96</td>
</tr>
<tr>
<td>Plasticizer</td>
<td>5</td>
<td>5,3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(kg/m³)</th>
<th>q=0.35</th>
<th>q=0.35</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C15%</td>
<td>C20%</td>
</tr>
<tr>
<td>CCW</td>
<td>583</td>
<td>584</td>
</tr>
<tr>
<td>NA2</td>
<td>403</td>
<td>398</td>
</tr>
<tr>
<td>NA3</td>
<td>577</td>
<td>586</td>
</tr>
<tr>
<td>FCW</td>
<td>153</td>
<td>220</td>
</tr>
<tr>
<td>CEMI42.5R</td>
<td>360</td>
<td>480</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>72</td>
<td>96</td>
</tr>
<tr>
<td>Plasticizer</td>
<td>3,5</td>
<td>5,2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(kg/m³)</th>
<th>q=0.25</th>
<th>q=0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C15%</td>
<td>C20%</td>
</tr>
<tr>
<td>CCW</td>
<td>448</td>
<td>449</td>
</tr>
<tr>
<td>NA2</td>
<td>337</td>
<td>333</td>
</tr>
<tr>
<td>NA3</td>
<td>522</td>
<td>531</td>
</tr>
<tr>
<td>Silica Flour</td>
<td>408</td>
<td>277</td>
</tr>
<tr>
<td>CEMI42.5R</td>
<td>360</td>
<td>481</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>72</td>
<td>96</td>
</tr>
<tr>
<td>Plasticizer</td>
<td>5,9</td>
<td>5,8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(kg/m³)</th>
<th>q=0.35</th>
<th>q=0.35</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C15%</td>
<td>C20%</td>
</tr>
<tr>
<td>CCW</td>
<td>583</td>
<td>584</td>
</tr>
<tr>
<td>NA2</td>
<td>403</td>
<td>398</td>
</tr>
<tr>
<td>NA3</td>
<td>577</td>
<td>586</td>
</tr>
<tr>
<td>Silica Flour</td>
<td>153</td>
<td>220</td>
</tr>
<tr>
<td>CEMI42.5R</td>
<td>360</td>
<td>480</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>72</td>
<td>96</td>
</tr>
<tr>
<td>Plasticizer</td>
<td>4,5</td>
<td>5,9</td>
</tr>
<tr>
<td>Formula</td>
<td>Average compressive strength 7 days (Mpa)</td>
<td>Max (MPa)</td>
</tr>
<tr>
<td>-------------</td>
<td>------------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>0,25/15%/FCW/CN</td>
<td>73,9</td>
<td>74,7</td>
</tr>
<tr>
<td>0,25/15%/FCW/CCW</td>
<td>68,3</td>
<td>69,7</td>
</tr>
<tr>
<td>0,25/15%/FN/CN</td>
<td>58,0</td>
<td>58,9</td>
</tr>
<tr>
<td>0,25/15%/FN/CCW</td>
<td>59,1</td>
<td>60,53</td>
</tr>
<tr>
<td>0,25/20%/FCW/CN</td>
<td>75,4</td>
<td>79,8</td>
</tr>
<tr>
<td>0,25/20%/FCW/CCW</td>
<td>80,1</td>
<td>81,1</td>
</tr>
<tr>
<td>0,25/20%/FN/CN</td>
<td>70,0</td>
<td>71</td>
</tr>
<tr>
<td>0,25/20%/FN/CCW</td>
<td>64,4</td>
<td>66,82</td>
</tr>
<tr>
<td>0,35/15%/FCW/CN</td>
<td>51,2</td>
<td>51,79</td>
</tr>
<tr>
<td>0,35/15%/FCW/CCW</td>
<td>47,1</td>
<td>47,6</td>
</tr>
<tr>
<td>0,35/15%/FN/CN</td>
<td>55,1</td>
<td>56,8</td>
</tr>
<tr>
<td>0,35/15%/FN/CCW</td>
<td>52,5</td>
<td>52,73</td>
</tr>
<tr>
<td>0,35/20%/FCW/CN</td>
<td>67,1</td>
<td>68,46</td>
</tr>
<tr>
<td>0,35/20%/FCW/CCW</td>
<td>57,5</td>
<td>64,1</td>
</tr>
<tr>
<td>0,35/20%/FN/CN</td>
<td>63,4</td>
<td>63,88</td>
</tr>
<tr>
<td>0,35/20%/FN/CCW</td>
<td>71,7</td>
<td>72,23</td>
</tr>
</tbody>
</table>

**Figure 3.** Compressive Strength at 7 days

In tables 7 and 8 can see the average compression strength at 7 and 28 days for coarse ceramic waste and coarse natural aggregate.
Table 6. Compressive Strength at 28 days

<table>
<thead>
<tr>
<th>Composition</th>
<th>Average compression strength 28 days (MPa)</th>
<th>Max (MPa)</th>
<th>Min (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,25/15%/FCW/CN</td>
<td>96,3</td>
<td>98,6</td>
<td>94</td>
</tr>
<tr>
<td>0,25/15%/FCW/CCW</td>
<td>96,6</td>
<td>98</td>
<td>95,1</td>
</tr>
<tr>
<td>0,25/15%/FN/CN</td>
<td>81,9</td>
<td>83,38</td>
<td>80,44</td>
</tr>
<tr>
<td>0,25/15%/FN/CCW</td>
<td>84,5</td>
<td>85,93</td>
<td>83,08</td>
</tr>
<tr>
<td>0,25/20%/FCW/CN</td>
<td>99,6</td>
<td>107</td>
<td>92,2</td>
</tr>
<tr>
<td>0,25/20%/FCW/CCW</td>
<td>95,6</td>
<td>99</td>
<td>92,26</td>
</tr>
<tr>
<td>0,25/20%/FNONAT/CN</td>
<td>105,5</td>
<td>109,7</td>
<td>101,2</td>
</tr>
<tr>
<td>0,25/20%/FN/CCW</td>
<td>86,7</td>
<td>92,95</td>
<td>80,5</td>
</tr>
<tr>
<td>0,35/15%/FCW/CN</td>
<td>79,1</td>
<td>80,28</td>
<td>77,91</td>
</tr>
<tr>
<td>0,35/15%/FCW/CCW</td>
<td>77,5</td>
<td>77,93</td>
<td>76,99</td>
</tr>
<tr>
<td>0,35/15%/FN/CN</td>
<td>78,6</td>
<td>79,54</td>
<td>77,61</td>
</tr>
<tr>
<td>0,35/15%/FN/CCW</td>
<td>74,5</td>
<td>76,01</td>
<td>72,99</td>
</tr>
<tr>
<td>0,35/20%/FCW/CN</td>
<td>93,1</td>
<td>102,06</td>
<td>84,04</td>
</tr>
<tr>
<td>0,35/20%/FCW/CCW</td>
<td>90,9</td>
<td>93,7</td>
<td>88,12</td>
</tr>
<tr>
<td>0,35/20%/FN/CN</td>
<td>71,6</td>
<td>82,33</td>
<td>60,93</td>
</tr>
<tr>
<td>0,35/20%/FN/CCW</td>
<td>99,7</td>
<td>103,84</td>
<td>95,62</td>
</tr>
</tbody>
</table>

Figure 4. Compressive Strength at 28 days
The results show that all the specimens give compression strength values greater than or equal to 50MPa. The compression strength of the specimens at 7 days is on average 72% of that at 28 days.

In addition, the coarse ceramic waste product on average has a slightly lower compression strength than that obtained by the specimens containing natural coarse aggregates. The compression strength of the specimens made for \( q = 0.35 \) at 7 days is lower for those specimens containing fine ceramic waste products than for the specimens containing silica flour. However, on average the compression strength at 28 days of the specimens made with ceramic waste products give the best strength. The fine ceramic waste products thus achieve a higher resistance, but it takes longer to achieve it, while the silica flour gives a higher resistance at 7 days but increases more slowly thereafter.

In addition to the results relating to the compression strength of the test specimens corresponding to the workability of the selected criterion, additional tests can also be analyzed, in particular at the level of the plasticizing additive. By increasing the amount of additive used increase the workability, until the saturation limit is reached. The workability remains constant beyond the saturation limit. The minimum diameter that gave an acceptable workability is 20cm. When the diameter exceeded 22cm, the compression strength diminished. Recycled coarse ceramic waste requires less additive but more mixing time than natural coarse aggregates.

### Table 7. Average Compression Strength

<table>
<thead>
<tr>
<th>Average compression strength Mpa</th>
<th>Coarse ceramic waste</th>
<th>Coarse natural aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td>15% Cement</td>
<td>20% Cement</td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>28 days</td>
<td>7 days</td>
</tr>
<tr>
<td>Fine ceramic waste q=0,25</td>
<td>68,3</td>
<td>96,6</td>
</tr>
<tr>
<td>q=0,35</td>
<td>47,1</td>
<td>77,5</td>
</tr>
<tr>
<td>Fine natural (Silica Flour) q=0,25</td>
<td>59,1</td>
<td>84,5</td>
</tr>
<tr>
<td>q=0,35</td>
<td>52,5</td>
<td>74,5</td>
</tr>
</tbody>
</table>

### Table 8. Average Compression Strength

<table>
<thead>
<tr>
<th>Average compression strength Mpa</th>
<th>Coarse natural aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td>15% Cement</td>
<td>20% Cement</td>
</tr>
<tr>
<td>7 days</td>
<td>28 days</td>
</tr>
<tr>
<td>Fine ceramic waste q=0,25</td>
<td>73,9</td>
</tr>
<tr>
<td>q=0,35</td>
<td>51,2</td>
</tr>
<tr>
<td>Fine natural (Silica Flour) q=0,25</td>
<td>58,0</td>
</tr>
<tr>
<td>q=0,35</td>
<td>55,1</td>
</tr>
</tbody>
</table>

5 ANALYSIS

The results show that all the specimens give compression strength values greater than or equal to 50MPa. The compression strength of the specimens at 7 days is on average 72% of that at 28 days.

In addition, the coarse ceramic waste product on average has a slightly lower compression strength than that obtained by the specimens containing natural coarse aggregates. The compression strength of the specimens made for \( q = 0.35 \) at 7 days is lower for those specimens containing fine ceramic waste products than for the specimens containing silica flour. However, on average the compression strength at 28 days of the specimens made with ceramic waste products give the best strength. The fine ceramic waste products thus achieve a higher resistance, but it takes longer to achieve it, while the silica flour gives a higher resistance at 7 days but increases more slowly thereafter.

In addition to the results relating to the compression strength of the test specimens corresponding to the workability of the selected criterion, additional tests can also be analyzed, in particular at the level of the plasticizing additive. By increasing the amount of additive used increase the workability, until the saturation limit is reached. The workability remains constant beyond the saturation limit. The minimum diameter that gave an acceptable workability is 20cm. When the diameter exceeded 22cm, the compression strength diminished. Recycled coarse ceramic waste requires less additive but more mixing time than natural coarse aggregates.
6 CONCLUSIONS
- From the experimental program carried out, it may conclude that all tests gave results in compression corresponding to the high-performance concrete, which was the purpose of the program, since the compressive strengths are greater than 71 MPa after 28 days. The ceramic waste products are therefore a priori an acceptable product in the manufacture of HPC.
- However, before validating the usability, it must continue conducting other tests such as flexural and durability tests of concrete.
- Finally, the economic value of the use of such wastes, indeed logistics and waste treatment costs, can justify an promising use replacing materials like silica flour that is an expensive material. In addition to improving the sustainability of this HPC.

REFERENCES
[3] Program GEAR Project
SIMPLIFICATIONS FOR CONSIDERING THE CONTRIBUTION OF THE REINFORCEMENT IN THE COMPRESSION ZONE FOR DESIGNING MORE EFFICIENT RC FRAME ELEMENTS

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Key words: Reinforced Concrete, Frame Design, Resisting Moment, Axial-Moment Diagram

Abstract. The scope of this paper is to present an analytical method that permits the resisting moment evaluation of reinforced concrete frame elements. The most important feature of the method consists in the fact that, the contribution of the reinforcement located in the compressive zone is taken into account. While in the majority of other design methods, where the compressed reinforcement is usually neglected or is considered only through the use of more complex mathematical models, the alternative presented here enables such complex calculations to be performed without resorting to iterations of any kind. As long as the compression state at sectional level can be identified and localized on the axial-moment diagram, it can be shown that, for common reinforcing percentages and arrangements, the calculus of the resisting moment is straightforward or it can be reduced to simple operations of linear interpolation.

1 INTRODUCTION

The optimal design of reinforced concrete frame elements is of primary concern in obtaining significant material cost reductions and efficient structural frame systems. Despite the availability of a multitude of design methods, starting from the simple ones to the more complex, there are still doubts that the existing analytical tools are exploited to their full potential. If some of the mechanisms which develop in reinforced concrete frame elements are heavily simplified or even neglected in design, then the possibility of building inefficient and higher cost structural systems may occur.

One simplified concept that is commonly used in frame design is to neglect the contribution of the compressed reinforcement for elements that are subjected to bending and axial forces. While this can be applied for beams or light compression members, it can generate severe errors when used for columns and heavily compressed beams. In order to minimize such potential design errors, an analytical method that considers the contribution of the compressed reinforcement is a prior necessity for an efficient frame design.
2 THE CONSTRUCTION OF THE AXIAL-MOMENT INTERACTION DIAGRAMS

The majority of the methods used for the resisting moment evaluation are based on the equilibrium equations performed at the cross-section level. If both axial and bending forces are present, then two equilibrium equations are necessary in order to determine the interior resultant forces. The two equilibrium equations are expressed in relation (1). The significance of the used terms and the reduction of the internal forces to the centroid of the cross-section are presented in Figure 1.

\[
\begin{align*}
A_{s1} \cdot \sigma_s(\varepsilon_s) + A_{s2} \cdot \sigma_s(\varepsilon_s) + \left( b \cdot \lambda \chi \right) \cdot \eta \sigma_c(\varepsilon_{c,\text{max}}) &= N_{\text{Rd}} \geq N_{\text{Ed}} \\
F_{s1} \cdot z_{s1} + F_{s2} \cdot z_{s2} + F_c \cdot z_c &= M_{\text{Rd}} \geq M_{\text{Ed}}
\end{align*}
\]

The following terms are used in relation (1): \( \sigma_s(\varepsilon_s) \), \( \sigma_s(\varepsilon_s) \) and \( \sigma_c(\varepsilon_{c,\text{max}}) \) - stress of the tension reinforcement, of the compression reinforcement and of the extreme concrete fiber in compression, respectively, calculated with the material stress-strain relations; \( \varepsilon_{s1} \), \( \varepsilon_{s2} \) - strain in the tensile and compression reinforcement, respectively; \( \varepsilon_{c,\text{max}} \) - strain in the extreme concrete fiber in compression; \( A_{s1}, A_{s2} \) - tension and compression reinforcement areas, respectively; \( b \) - overall width of the cross-section; \( x_c \) - neutral axis depth; \( \phi \) - curvature; \( \lambda, \eta \) - factors defining the effective height of the compression zone and the effective concrete strength, respectively; (\( \lambda=0.8 \) and \( \eta=1 \) for concrete strength classes \( \leq \text{C50/60} \), according to Eurocode 2 [1]); \( z_{s1}, z_{s2} \) and \( z_c \) - the lever arm measured from the cross section centroid to the center of the tension reinforcement, the center of the compression reinforcement and the center of the effective compression zone, respectively; \( M_{\text{Rd}}, N_{\text{Rd}} \) - the internal resisting moment and resisting axial force, respectively; \( M_{\text{Ed}}, N_{\text{Ed}} \) - the external design moment and design axial force, respectively.

The material stress-strain relations are usually taken from design codes. Eurocode 2 [1] recommends two relations for modeling the reinforcing steel and three relations for modeling the compressed concrete behavior. These are presented in Figure 2 and equations (2) to (6).

The design stress-strain diagram with hardening – for reinforcing steel:

\[
\sigma_i = \begin{cases} 
\varepsilon_s \cdot E_s \rightarrow & \text{if } 0 \leq \varepsilon_s < \varepsilon_{y} \\
\varepsilon_{sh} \cdot E_{sh} \rightarrow & \text{if } \varepsilon_{y} \leq \varepsilon_s \leq \varepsilon_{iu}
\end{cases}
\]

Figure 1: The internal stress distribution
The design stress-strain diagram without hardening – for reinforcing steel:

\[
\sigma_s = \begin{cases} 
\varepsilon_s \cdot E_s & \rightarrow \text{if} : 0 \leq \varepsilon_s < \varepsilon_{sy} \\
\frac{f_{yd}}{\varepsilon_{yd}} & \rightarrow \text{if} : \varepsilon_{sy} \leq \varepsilon_s \leq \varepsilon_{su}
\end{cases}
\]  
(3)

The nonlinear diagram – for concrete:

\[
\sigma_c = \left( \frac{k\eta - \eta^2}{1 + (k - 2)\eta} \right) f_{cm}
\]  
(4)

The parabola-rectangle diagram – for concrete:

\[
\sigma_c = \left[ 1 - (1 - \frac{\varepsilon_c}{\varepsilon_{c2}})^n \right] f_{cd} \rightarrow \text{if} : 0 \leq \varepsilon_c \leq \varepsilon_{c2} \\
\frac{f_{cd}}{\varepsilon_{cd}} \rightarrow \text{if} : \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{ca2}
\]  
(5)

The bilinear stress-strain relation – for concrete:

\[
\sigma_c = \begin{cases} 
\varepsilon_c \left( \frac{f_{cd}}{\varepsilon_{c3}} \right) & \rightarrow \text{if} : 0 \leq \varepsilon_c \leq \varepsilon_{c3} \\
\frac{f_{cd}}{\varepsilon_{cd}} & \rightarrow \text{if} : \varepsilon_{c3} \leq \varepsilon_c \leq \varepsilon_{ca3}
\end{cases}
\]  
(6)

The terms from equations (2) to (6) have the following meaning: \( \varepsilon_s \) and \( \sigma_s \) - strain and stress of the reinforcing steel, respectively; \( \varepsilon_{sy} \) - yield tensile strain in the reinforcement; \( \varepsilon_{su} \) - ultimate tensile strain in the reinforcement; \( f_{yd} \) - design yield strength of reinforcement; \( \varepsilon_c \) and \( \sigma_c \) - strain and stress of the compressed concrete, respectively; \( \varepsilon_{c2} \) and \( \varepsilon_{c3} \) - strain at reaching the maximum strength for the parabola-rectangle diagram and the bilinear stress-strain relation, respectively; \( \varepsilon_{ca2} \) and \( \varepsilon_{ca3} \) - ultimate strain for the parabola-rectangle diagram and the bilinear stress-strain relation, respectively; \( f_{cd} \) - design value of concrete compressive strength; \( E_s \) - design value of modulus of elasticity of reinforcing steel; \( E_{sh} \) - the hardening coefficient; \( E_m = (f_t - f_{yd}) / (\varepsilon_{su} - \varepsilon_{sy}) \), where \( f_t \) - tensile strength of reinforcement; \( n \) - exponent (equals 2 for concrete strength classes \( \leq \text{C50/60} \)); \( \eta = \varepsilon_s / \varepsilon_{c1} \) and \( k = 1.05 \cdot E_m \cdot |\varepsilon_{c1}| / f_{cm} \), where: \( \varepsilon_{c1} \) - is the

![Design stress-strain diagram (with hardening)](image)

![Design stress-strain diagram](image)

**Figure 2:** Material stress-strain diagrams for steel reinforcement (left) and concrete (right)
compressive strain in the concrete at the peak stress; $f_{cm}$ - is the mean value of concrete cylinder compressive strength; $E_{cm}$ - is the secant modulus of elasticity of concrete.

While the behavior of the reinforcement is considered to be linear for both domains (i.e. the elastic and the post-elastic domains), it can be seen from Figure 2 that the concrete behavior can be represented as: completely nonlinear, linear for both domains, or nonlinear for the first domain and linear for the second domain.

Finally, the terms representing the lever arms $z_{s1}$, $z_{s2}$ and $z_c$ measured from the centroid of the cross-section have the following mathematical expressions:

$$z_{s1} = h/2 - d_1$$
$$z_{s2} = h/2 - d_2$$
$$z_c = h/2 - \lambda x_c / 2$$

where: $h$ - overall depth of a cross-section; $d_1$ - depth to tensile reinforcement; $d_2$ - depth to compression reinforcement.

Starting from the maximum tension state and ending with the maximum compression state or vice versa, all the possible values for $M_{Rd}$ and $N_{Rd}$ can be obtained by resolving equation (1). The $M_{Rd}$-$N_{Rd}$ interaction diagrams are constructed by representing all the resulted values in form of a graph (see Figure 3).

The construction of the interaction diagrams can be performed for six deformation domains: $1a$, $1b$, $2a$, $2b$, $2c$ and $3$. All the domains are bounded by seven deformation states that are known, and are named as: $1a$, $1b$, $2a$, $2b$, $2c$, $3a$ and $3b$. The graphical representation of these domains and the separation states between them are presented in Figure 3. The strains for the tension reinforcement and the extreme concrete fiber in compression are expressed below for all six deformation domains:

- the $1a$ domain: $\varepsilon_{s1} = \varepsilon_{su}$ $0 \leq \varepsilon_{c, \max} \leq \varepsilon_{su}$
- the $2b$ domain: $0 \leq \varepsilon_{s1} \leq \varepsilon_{sy}$ $\varepsilon_{c, \max} = -\varepsilon_{cu}$

- the $1b$ domain: $\varepsilon_{s1} = \varepsilon_{su}$ $-\varepsilon_{cu} \leq \varepsilon_{c, \max} \leq 0$
- the $2c$ domain: $\left(-\varepsilon_{c, \max} \cdot d_1\right)/h \leq \varepsilon_{s1} \leq 0$ $\varepsilon_{c, \max} = -\varepsilon_{cu}$

- the $2a$ domain: $\varepsilon_{s1} \leq \varepsilon_{s1} \leq \varepsilon_{su}$ $\varepsilon_{c, \max} = -\varepsilon_{cu}$
- the $3$ domain: $-0.002 \leq \varepsilon_{s1} \leq \left(-\varepsilon_{c, \max} \cdot d_1\right)/h$ $-\varepsilon_{cu} \leq \varepsilon_{c, \max} \leq -0.002$

where: $\varepsilon_{cu}$ - ultimate compressive strain in the concrete.

It can be seen from relations (8) that all six domains represent the ultimate states or the failure domains for reinforced concrete sections (i.e. failure occurs for strains reaching the maximum tensile strain in the tensile reinforcement or the maximum compression strain in the extreme compressed concrete fiber, whichever appears first [2,3]).

Finally, the $M_{Rd}$-$N_{Rd}$ interaction diagrams are constructed for each domain by varying the curvature $\phi$ and resolving equation (1) for the resulted deformed state. The internal deformed state is totally known as long as the plane section hypothesis (i.e. the Bernoulli hypothesis) is respected. For the $2a$, $2b$ and $2c$ domains where $0 \leq x_c \leq h$, the neutral axis depth $x_c$ may be used
instead of the curvature as the varying parameter. The relation between the curvature and the neutral axis depth is as stated in equation (9) (see Figure 1 and Figure 3).

\[
x_c = \frac{|\varepsilon_{cu}|}{\phi}
\]

(9)

3 METHODS FOR CALCULATING THE RESISTING MOMENT

The most accurate method to determine the resisting moments of reinforced concrete frames is to generate the axial-moment interaction diagrams. If this is done, then the resisting moment can be obtained for any design or resisting axial force. Otherwise, simplified procedures that apply separately for columns and beams are often used in practice.

3.1 Calculation methods for rectangular columns

As mentioned before, accurate values are obtained only if the axial-moment interaction diagrams are generated. The downside of this practice relies in the necessity of performing iterative calculations for every deformation state. While this can be done easily by using computer programs, it is an inefficient and a time consuming procedure if done by hand.

Much faster methods can be formulated by using design charts as those stated by Beeby and Narayanan [4] for example. Despite the efficiency of such graphical methods, there are still inconveniences regarding the errors which may occur when reading the values on charts. Not to mention that the charts have to be permanently at the user’s disposal.

Anyway, if design charts are available, the resisting moment can be obtained graphically for any axial load if the following factor is evaluated:

\[
\omega = \frac{A_s \cdot f_{yd}}{bh \cdot f_{cd}}
\]

(10)

where: \( \omega \) - factor that represents the ratio between the reinforcement resistance and the concrete resistance to axial forces (usually takes values between 0.10 and 1.00); \( A_s \) - total

Figure 3: Internal strain distribution for the six failure domains
reinforcement area that is considered for the resisting moment evaluation.

If \( \omega \) and the design axial force \( N_{Ed} \) are known, then the resisting moment \( M_{Rd} \) can be determined accordingly to Figure 4.

In the absence of any axial-moment interaction diagrams, whether they are generated through iterations or not, some simplistic methods which neglect the contribution of the compressed reinforcement or assume the same stress in both tension and compression reinforcements (\( \sigma_{s1} = -\sigma_{s2} = f_{yd} \)) are usually used in practice. Whatever the case, it is clear that these assumptions are not correlated with the real deformation states presented in Figure 3 and therefore, such methods should be avoided as much as possible.

### 3.2 Calculation methods for rectangular beams

The main difference when evaluating the resisting moment for beams is that the influence of the axial force is neglected (i.e. \( N_{ed} \approx 0 \)). This statement permits the use of simpler relations and still having satisfying results. Depending on the neutral axis depth approximation, there are two relations that are commonly used in practice:

\[
M_{Rd} = A_{s1} f_{yd} \cdot 0.9d \quad \text{or} \quad M_{Rd} = A_{s1} f_{yd} \cdot \left( d - \frac{\lambda_{x}}{2} \right)
\]

where \( \lambda_{x} = \frac{A_{s1} f_{yd}}{\eta f_{yd}} \) and \( d \) - effective depth of a cross-section. The contribution of the compressed reinforcement is neglected in both cases.

The same as for columns, if more precise results are needed, then other calculation methods which rely on axial-moment interaction diagrams must be used. Whether these interaction diagrams are generated through iterations or are obtained from the available design charts, the resisting moment will be determined for an axial force equal to zero if this assumption can be applied. Obviously, the method is also valid for beams subjected to axial forces that cannot be neglected.

### 4 SIMPLIFICATIONS FOR THE RESISTING MOMENT EVALUATION

In order to formulate a method for calculating the resisting moment by considering the compressed reinforcement and to be in perfect consistency with the real deformation states, the following simplifications are necessary for a proper practical use.

#### 4.1 Choosing the proper material stress-strain relations

Choosing from the available material stress-strain relations can make a big difference between the obtained results. The relations accepted by Eurocode 2 [1] are presented in Figure 2 and equations (2) to (6).

The resisting moments and axial forces are often identified as the internal forces associated with the failure conditions [2]. Therefore, as long as the tensioned concrete is neglected in
design, the influence of the chosen concrete stress-strain relation is reflected by the design value of the compressive strength only. In other words, the concrete influence stands only in the stress value that is associated with the ultimate compression strain. Under these conditions, either the parabola-rectangle diagram or the bilinear stress-strain relation can be used to model the concrete behavior (see Figure 2).

By contrast, the chosen steel stress-strain relation can increase the complexity of the evaluation process. If the hardening effect is neglected, then the stress in the reinforcing steel has the same value all over the plastic domain (i.e. between $\varepsilon_{sy}$ and $\varepsilon_{su}$). Therefore, the design stress-strain diagram (without hardening - see Figure 2) will be further used.

4.2 Considering only the failure domains that are acceptable for design

It has been mentioned earlier that the evaluation of the resisting moment for beams can be made in the same manner as for columns, but taking the design axial force equal to zero. This restricts the calculation only for the $2a$ domain (see Figure 3). Since tensile reinforced concrete columns are rarely encountered in design, the $1a$ and $1b$ domains are usually unused and therefore, their construction is not necessary. If the ductility aspect is also taken into account, then columns subjected to compression forces associated with $2c$ and $3$ domains (see Figure 3) will be generally avoided. This ensures that compression will not be encountered in all reinforcement. Under these considerations, the only deformation domains that are usually undertaken by columns are the $2a$ and $2b$ domains.

4.3 Splitting the $2a$ domain into three failure domains

The calculus performed for the $2a$ domain is difficult because the reinforcement in compression undergoes three deformation states: a plastic domain in tension, an elastic domain in tension and compression, and a plastic domain in compression. The existence of last mentioned domain (the plastic domain in compression) is not possible if the inequality from relation (13) is valid (i.e. the compressed reinforcement strain is lower or equal to the yield strain while the tensile reinforcement strain is at yield strain: $\varepsilon_{s1} = \varepsilon_{sy}$ and $\varepsilon_{s2} \leq -\varepsilon_{sy}$).

$$d_2 \geq \frac{\varepsilon_{cu} - \varepsilon_{sy}}{\varepsilon_{cu} + \varepsilon_{sy}} d$$

(13)

In other words, the strain in the compressed reinforcement cannot exceed the yield strain for the balance point (i.e. the point situated on the axial-moment interaction diagram which is associated with the $IIb$ deformation state: $\varepsilon_{c,max} = \varepsilon_{cu}$ and $\varepsilon_{s1} = \varepsilon_{sy}$; see Figure 3). Since $\varepsilon_{cu} = \varepsilon_{cu3} = 0.0035$ (see Eurocode 2 [1]) and $0.0015 \leq \varepsilon_{sy} \leq 0.0022$ (i.e. the values are calculated for reinforcement having $f_{yk} = 345$ MPa and $f_{yk} = 500$ MPa, respectively), it can be verified by using relation (13) that $d_2 \geq (0.4\pm0.23)d$. The condition is unrealistic for common design solutions, which proves the existence of the plastic domain in compression.

The division of the $2a$ domain is necessary because the stress of the compressed reinforcement cannot be deducted using a single mathematical relation, even if the function defining the stress of the reinforcement $\sigma_s(\varepsilon_{s2})$ is continuous in $\varepsilon_{s2} = \pm \varepsilon_{sy}$. By dividing the $2a$ domain into three deformation states, at least two advantages are achieved. The first one eliminates the necessity of performing iterations for the plastic domains in tension and compression because the stress in the reinforcement is constant (i.e. the stress-strain diagram
without hardening is adopted). The second advantage is gained by neglecting the plastic domain in tension since such high tensile forces in columns are unlikely to occur. As a consequence, only two of the three domains (i.e. the elastic domain in tension and compression, and the plastic domain in compression) are suitable for design purposes.

The new-formed domains are presented in Figure 5, and are named accordingly to the 2a domain: 2a’ – the plastic domain in tension; 2a’’ – the elastic domain in tension and compression; 2a’’’ – the plastic domain in compression.

The deformation states IIa’ and IIa’” that separate the 2a’, 2a’’ and 2a’’’ domains are defined by ε\(_{c,max}\) = ε\(_{cu}\), and \(\varepsilon_{s2} = \varepsilon_{sy}\) for the first deformation state and \(\varepsilon_{s2} = -\varepsilon_{sy}\) for the second one (see Figure 5).

### 4.4 Analyzing the linearity for the considered failure domains

Following an extended study upon the shape of the axial-moment interaction diagrams on the failure domains of interest (i.e. the 2a’’, 2a’’’ and 2b domains), it has been observed that in certain conditions the resisting axial force varies linearly with the resisting moment. To prove this statement, the first and the second derivatives were calculated for all the three domains. By adapting equation (1) to these domains, the following equations were obtained:

\[
\begin{align*}
N_{rd} &= A_1 f_{ud} \pm A_2 \left(\varepsilon_{cu} - \varepsilon_{ca}\right) E_s - Bx \quad \rightarrow \text{for the 2a'' domain} \\
M_{rd} &= A_1 f_{ud} \left(\frac{h}{2} - d_1\right) + A_2 \left(\frac{\varepsilon_{cu} - \varepsilon_{ca}}{x_c} d_2 - \varepsilon_{cu}\right) E_s \left(\frac{h}{2} - d_2\right) + Bx \left(\frac{h}{2} - \frac{\lambda x}{2}\right) \\
N_{rd} &= A_1 f_{ud} - A_2 f_{ud} - Bx \\
M_{rd} &= A_1 f_{ud} \left(\frac{h}{2} - d_1\right) + A_2 f_{ud} \left(\frac{h}{2} - d_2\right) + Bx \left(\frac{h}{2} - \frac{\lambda x}{2}\right) \quad \rightarrow \text{for the 2a'''' domain}
\end{align*}
\]
\[
\begin{align*}
N_{rd} &= A_{s1} \left( \frac{\varepsilon_{cu} d}{x_c} - \varepsilon_{cu} \right) E_s - A_{s2} f_{yd} - B x_c \\
M_{rd} &= A_{s1} \left( \frac{\varepsilon_{cu} d}{x_c} - \varepsilon_{cu} \right) E_s \left( \frac{h - d_1}{2} \right) + A_{s2} f_{yd} \left( \frac{h - d_2}{2} \right) + B x_c \left( \frac{h - \lambda x_c}{2} \right)
\end{align*}
\rightarrow \text{for the 2b domain}
\]

where \( B = b \lambda \eta f_{yd} \). The superior sign in equation (14) is taken as the sign of the compressed reinforcement strain (see Figure 5).

It can be seen that equations (14) to (16) are formulated with respect to a single parameter \( x_c \) (i.e. the neutral axis depth). This permits a simpler evaluation of the first derivative:

\[
\frac{dN_{rd}(x_c)}{dM_{rd}(x_c)} = \frac{dN_{rd}(x_c)}{dx_c} \cdot \frac{dx_c}{dM_{rd}(x_c)} = N'_{rd} = \frac{N''_{rd}}{M''_{rd}}
\]

where \( N'_{rd} \) and \( M'_{rd} \) represent the first derivatives with respect to \( x_c \) in Lagrange's notation.

After performing the calculations for all three domains, the following results were obtained:

\[
\begin{align*}
N'_{rd} &= \frac{B x_c^2 + C}{B \lambda x_c^3 - B \frac{h}{2} x_c^2 - C \left( \frac{h}{2} - d_1 \right)} \rightarrow \text{for the 2a'' domain} ; \\
M'_{rd} &= \left( \frac{x_c}{2} \right) \\
N'_{rd} &= \frac{1}{\lambda x_c - \frac{h}{2}} \rightarrow \text{for the 2a''' domain} ; \\
M'_{rd} &= \left( \frac{x_c}{2} \right) \\
N'_{rd} &= \frac{B x_c^2 + D}{B \lambda x_c^3 - B \frac{h}{2} x_c^2 + D \left( \frac{h}{2} - d_1 \right)} \rightarrow \text{for the 2b domain} ; \\
M'_{rd} &= \left( \frac{x_c}{2} \right)
\end{align*}
\]

where \( C = A_{s2} \varepsilon_{cu} d_2 E_s \) and \( D = A_{s1} \varepsilon_{cu} d_1 E_s \).

The second derivative is calculated starting from equation (17):

\[
\frac{d}{dM_{rd}(x_c)} \left( \frac{dN_{rd}(x_c)}{dx_c} \right) = \frac{d}{dx_c} \left( \frac{dN_{rd}(x_c)}{dx_c} \right) \cdot \frac{dx_c}{dM_{rd}(x_c)} = \frac{N''_{rd}}{M''_{rd}}
\]

where \( N''_{rd} \) and \( M''_{rd} \) represent the second derivatives with respect to \( x_c \).

The relations for the second derivative with respect to \( x_c \) are presented below:

- for the 2a'' and 2b domains:
  \[
  \begin{cases}
    N''_{rd} = 2B x_c \\
    M''_{rd} = (3 \lambda x_c - h) B x_c
  \end{cases}
  \]

- for the 2a''' domain:
  \[
  \begin{cases}
    N''_{rd} = 0 \\
    M''_{rd} = \lambda
  \end{cases}
  \]

The linearity of a function on a certain domain can be verified if the first derivative is constant, or if the second derivative is equal to zero. Through the use of such mathematical methods, a parametric study is conceived in order to study the influence of the \( \omega \) factor and the \( d/h \) ratio upon the linearity of the \( M_{rd} - N_{rd} \) interaction diagrams. The study is referred only to columns, because the relations (11) and (12) offer satisfying results for beam elements that are not subjected to axial forces. The values for the \( \omega \) parameter are taken staring from 0.10 to 1.00. For the \( d/h \) ratio the values are the following: 0.05, 0.10, 0.15, 0.20.
The calculated results for the first and second derivatives were analyzed with the use of the standard deviation principle. If the standard deviation results are near zero, then the results obtained through equations (18) to (22) are closed to the mean value. Therefore, the first derivative results are constant for the whole domain, while for the second derivative are equal to zero. The values are calculated for the whole range of the $2a''$, $2a'''$ and $2b$ domains. The relation for the uncorrected sample standard deviation (i.e. the standard deviation) is expressed below:

$$s_n = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (x_i - \bar{x})^2}$$ \hspace{1cm} (23)

where: $s_n$ - standard deviation of the sample; $N$ - size of the sample (i.e. the number of calculation points); $x_i$ - value of the sample (i.e. the calculated derivatives); $\bar{x}$ - mean value.

The standard deviation results for the first derivative are presented for all the domains in Tables 1, 2 and 3.

**Table 1**: Standard deviation results for the first derivative for the $2a''$ domain

<table>
<thead>
<tr>
<th>$d_{1/h}$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>0.05</td>
<td>0.14</td>
<td>0.11</td>
<td>0.09</td>
<td>0.07</td>
<td>0.06</td>
<td>0.05</td>
<td>0.05</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>0.1</td>
<td>0.49</td>
<td>0.40</td>
<td>0.34</td>
<td>0.30</td>
<td>0.26</td>
<td>0.24</td>
<td>0.21</td>
<td>0.20</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>0.15</td>
<td>1.23</td>
<td>1.04</td>
<td>0.90</td>
<td>0.79</td>
<td>0.71</td>
<td>0.64</td>
<td>0.59</td>
<td>0.54</td>
<td>0.50</td>
<td>0.47</td>
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<tr>
<td>0.2</td>
<td>3.32</td>
<td>2.77</td>
<td>2.37</td>
<td>2.08</td>
<td>1.85</td>
<td>1.67</td>
<td>1.52</td>
<td>1.40</td>
<td>1.29</td>
<td>1.20</td>
</tr>
</tbody>
</table>

*The market values do not correspond to a linear domain*

**Table 2**: Standard deviation results for the first derivative for the $2a'''$ domain

<table>
<thead>
<tr>
<th>$d_{1/h}$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>10.32</td>
<td>3.25</td>
<td>1.75</td>
<td>1.12</td>
<td>0.79</td>
<td>0.59</td>
<td>0.47</td>
<td>0.38</td>
<td>0.31</td>
<td>0.26</td>
</tr>
<tr>
<td>0.15</td>
<td>24.38</td>
<td>4.73</td>
<td>2.33</td>
<td>1.44</td>
<td>0.99</td>
<td>0.73</td>
<td>0.57</td>
<td>0.45</td>
<td>0.37</td>
<td>0.31</td>
</tr>
<tr>
<td>0.2</td>
<td>175.17</td>
<td>8.01</td>
<td>3.40</td>
<td>1.97</td>
<td>1.31</td>
<td>0.95</td>
<td>0.72</td>
<td>0.57</td>
<td>0.47</td>
<td>0.39</td>
</tr>
</tbody>
</table>

*The market values do not correspond to a linear domain*

**Table 3**: Standard deviation results for the first derivative for the $2b$ domain

<table>
<thead>
<tr>
<th>$d_{1/h}$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
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<td>0.05</td>
<td>6.24</td>
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<td>1.39</td>
<td>0.92</td>
<td>0.66</td>
<td>0.50</td>
<td>0.40</td>
<td>0.32</td>
<td>0.27</td>
<td>0.23</td>
</tr>
<tr>
<td>0.1</td>
<td>10.32</td>
<td>3.25</td>
<td>1.75</td>
<td>1.12</td>
<td>0.79</td>
<td>0.59</td>
<td>0.47</td>
<td>0.38</td>
<td>0.31</td>
<td>0.26</td>
</tr>
<tr>
<td>0.15</td>
<td>24.38</td>
<td>4.73</td>
<td>2.33</td>
<td>1.44</td>
<td>0.99</td>
<td>0.73</td>
<td>0.57</td>
<td>0.45</td>
<td>0.37</td>
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</tr>
<tr>
<td>0.2</td>
<td>175.17</td>
<td>8.01</td>
<td>3.40</td>
<td>1.97</td>
<td>1.31</td>
<td>0.95</td>
<td>0.72</td>
<td>0.57</td>
<td>0.47</td>
<td>0.39</td>
</tr>
</tbody>
</table>

*The market values do not correspond to a linear domain*

By analyzing the results of the first derivative, it is clear that the relation between $M_{Rd}$ and $N_{Rd}$ on the $2a'''$ domain is highly nonlinear, the standard deviation results being much higher.
than zero. It can be observed that the results over this domain are not influence by the $\omega$ parameter (see Table 2).

Over the $2a''$ domain instead, the results are between zero and one, except for those marked in Table 1. Extra calculations are necessary in order to see how standard deviation values closer to one influence the final results. Therefore, the resisting moments and axial forces ($M_{Rd}$ and $N_{Rd}$) were calculated with the general procedure stated by equation (1), but also through linear interpolation between the points that define the $IIa'$ and $IIa''$ deformation states (see Figure 5). It was found that the results do not differ more than 5% for standard deviation values lower than one. Following this argument, it can be concluded that the $2a''$ domain is linear for $d_1/h \leq 0.15$, except the case when $\omega \leq 0.2$ and $d_1/h = 0.15$ (see Table 1).

Over the $2b$ domain no linearity rule can be identified from studying the first derivative, even if the majority of the standard deviation results are lower than one. Therefore, the same standard deviation principle was applied to the second derivative results. These are presented in Table 4. It was found that a linear dependency can be accepted if both standard deviation results for the first and the second derivative are higher enough. The results obtained through the use of equation (1) were compared with the results obtained through linear interpolation between the values associated with the $IIb$ and $IIc$ deformed states (see Figure 5). It was observed that the differences are less than 5%, except for those cases where $\omega = 0.1$ and $d_1/h \leq 0.10$ (see Table 3 and Table 4).

Table 4: Standard deviation results for the second derivative for the $2b$ domain

<table>
<thead>
<tr>
<th>$d_1/h$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
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<td>0.132</td>
<td>0.095</td>
<td>0.071</td>
<td>0.056</td>
<td>0.044</td>
<td>0.036</td>
</tr>
</tbody>
</table>

*The market values do not correspond to a linear domain

5 CONSIDERATIONS FOR SIMPLIFYING CALCULATION PROCESS

The simplifications discussed above are meant to ease the design process. It can be seen that, under certain conditions, a linear dependency can be identified for the $M_{Rd}$ and $N_{Rd}$ values. The linearity property is valid for the $2a''$ and $2b$ failure domains, and the resisting moment can be determined through linear interpolation between the known deformed states: $IIa'$ and $IIa''$ for the $2a''$ domain, and $IIb$ and $IIc$ for the $2b$ domain (see Figure 5). This procedure eliminates the necessity of performing iterations for finding the correct deformation state. Under the $2a'''$ domain instead, since both reinforcement are at yield (i.e. one in tension and the other one in compression) then the resisting moment can be calculated directly (i.e. without resorting to iterations of any kind) because the stress in the reinforcement is known. This is a consequence of neglecting the hardening effect by adopting the proper design stress-strain diagram.

Therefore, in order to calculate the resisting moment for a certain design axial force ($N_{Ed}$), the internal resisting moments and the internal axial forces (i.e. $M_{Rd}$ and $N_{Rd}$) have to be
determined for each $IIa'$, $IIa''$, $IIb$ and $IIC$ deformed states (see Figure 5). This can be done by using relation (1) or the more detailed relations (14) to (16).

If $N_{rd}(I1a') \geq N_{ed} \geq N_{rd}(I1a'')$, and $d_1/h \leq 0.15$, but $\omega > 0.2$ if $d_1/h = 0.15$, then:

$$M_{rd} = \left( N_{ed} - N_{rd}(I1a') \right) \frac{M_{rd}(I1a') - M_{rd}(I1a'')}{N_{rd}(I1a') - N_{rd}(I1a'')} + M_{rd}(I1a'')$$  \hspace{1cm} (24)

If $N_{rd}(I1a'') \geq N_{ed} \geq N_{rd}(I1b)$, then:

$$M_{rd} = \left( N_{ed} - N_{rd}(I1a'') \right) \frac{M_{rd}(I1a'') - M_{rd}(I1b)}{N_{rd}(I1a'') - N_{rd}(I1b)} + M_{rd}(I1b)$$  \hspace{1cm} (25)

Finally, if $N_{rd}(I1b) \geq N_{ed} \geq N_{rd}(I1c)$, and $d_1/h > 0.10$ if $\omega = 0.1$, then:

$$M_{rd} = \left( N_{ed} - N_{rd}(I1b) \right) \frac{M_{rd}(I1b) - M_{rd}(I1c)}{N_{rd}(I1b) - N_{rd}(I1c)} + M_{rd}(I1c)$$  \hspace{1cm} (26)

$M_{rd}(I1a')$, $M_{rd}(I1a'')$, $M_{rd}(I1b)$ and $M_{rd}(I1c)$ represent the resisting moments for the $IIa'$, $IIa''$, $IIb$ and $IIC$ deformed states, respectively; while $N_{rd}(I1a')$, $N_{rd}(I1a'')$, $N_{rd}(I1b)$ and $N_{rd}(I1c)$ represent the resisting axial forces for the $IIa'$, $IIa''$, $IIb$ and $IIC$ deformed states, respectively.

12 CONCLUSIONS

The proposed simplifications presented under this study are meant to increase the efficiency of the resisting moment evaluation procedure. No iterations or design charts are required, because a proper use of the available material stress-strain relations can generate linear dependencies on the axial-moment diagrams (i.e. the axial forces vary linearly with the corresponding moments on the axial-moment diagram). This applies for certain failure domains, while for others the results can be obtained directly by applying a single mathematical relation. Moreover, the proposed calculation process is consistent with the real deformed state which develops at the cross-sectional level, and therefore, the obtained results respect the equilibrium conditions. The compressed reinforcement contribution is considered.

REFERENCES

STRUCTURAL BEHAVIOUR OF RECYCLED CONCRETE: MECHANICAL STRENGTHS AND SHRINKAGE

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Key words: recycled concrete; compressive strength; tensile splitting; shrinkage

Abstract. In this paper the mechanical properties of recycled aggregate concrete (RAC) were characterized by replacing different percentages of natural coarse aggregate with recycled coarse aggregate (0%, 20%, 50% and 100%) and two different water to cement ratios 0.50 and 0.65. The results made it possible to establish the differences between the mechanical properties of conventional concrete and the recycled concrete depending on the replacement percentage and the quality of recycled coarse aggregates. This experimental program was carried out at the laboratory of University of A Coruña and at the laboratory of the Università Politecnica delle Marche. Overall, 16 different concrete mixtures were manufactured. Firstly, consistency and density were identified. Then, compressive strengths at 3, 7 and 28 days were obtained in order to define the time-dependent curve. After that, tensile splitting strength was defined, and finally, shrinkage curves were obtained. It was thus found that the density and mechanical strengths of recycled aggregate concrete decreases as the recycled aggregates content increases. On the other hand, recycled concrete shrinkage increases with the replacement percentage and shows a delayed time-dependent development due to internal curing effect, which also affects compressive strength.

1 INTRODUCTION AND OBJECTIVES

Many countries promote sustainable construction based on reuse of coarse aggregates from crushed structural concrete, with the objective of preserving natural resources and reducing space for waste storage. However, the use of recycled concretes is not yet widespread. Aiming to popularize and encourage its use in different application fields, different studies have been conducted about this topic [1-11].

The use of recycled coarse aggregates have a remarkable influence on concrete properties. Some authors [12, 13] have noted for 100% replacement ratio reductions up to 10-35% in
compressive strength or 40% in modulus of elasticity [14-16] while tensile strength shows scattering results, ranging from a slight increase [14, 15] up to 20-30% reduction [17, 18]. Nevertheless, recycled aggregates from concrete demolition offer good enough features for use in structural reinforced concrete; indeed, with a recycled aggregate content up to 20-30% the reduction in mechanical properties is not conspicuous [2, 3, 14].

Regarding strain behaviour, recycled concretes show higher values shrinkage than reference one. This increase can rise 70% for concretes fully made of recycled aggregates [16] or similar results to conventional concretes when replacement ratios are 20-30% [16, 19, 20].

The great number of researches involving structural recycled concrete [3, 9, 10, 17, 20-22] attempt to determine its behaviour in structural members and to establish trustworthy design guidelines. This is especially important for those countries whose codes and standards do not yet include recommendations of use for high replacement ratios. However, current literature states different results about recycled concrete behaviour. These differences are mainly due to quality of recycled aggregates used, which is determined by aggregates composition [18], absorption capacity [23] and quantity of attached mortar [24]. In accordance with this, it is not only necessary to know the influence of recycled aggregates content, but also the aggregate quality.

In order to determine the general properties of concretes made with recycled coarse aggregates, since a practical point of view, this research was carried out with similar materials in two different laboratories, one in Spain and the another one in Italy. Both countries are regulated by the same European standards (EN) and international codes (EC-2, MC-10), however the properties of this material are slightly diverse due to different origin plant or company. It is widely accepted that recycled aggregate can be classified by its composition, taking into account the percentage of virgin aggregate or with old mortar, masonry,…

Regarding the amount and quality of recycled aggregates, this research is focused on the use of recycled coarse aggregate from demolition of concrete structures to make concrete with different replacement ratios: 20%, 50% and 100%.

The proposed experimental program attempts to obtain useful guidelines to design structural concrete made up with recycled coarse aggregates, determining the influence of recycled aggregate taking into account its quality and quantity.

2 EXPERIMENTAL PROGRAM

In this study, different types of concrete have been designed with different water-to-cement ratios: 0.50 and 0.65. Each one is constituted by four series of concretes with different replacement ratios of recycled coarse aggregate: 20%, 50%, 100% and a reference concrete (replacement ratio is considered as 0%).

In addition, two different kind of recycled aggregate were used, one with water absorption capacity of 5% and the other one of 7%, named as I and II respectively.

This experimental program was carried out in different laboratories, first one in Spain at the laboratory of the La Coruña University and the second one in Italy at the laboratory of the Università Politecnica delle Marche.

Overall, 16 different concrete mixtures were manufactured, which are referred to hereafter as CX-YZ, being X the number referred to the water-to-cement ratio (50 or 65), Y is the percentage of recycled coarse aggregate, 0, 20, 50 or 100%, and finally, Z the letter I or II in
order to distinguish the concretes made with recycled aggregates of water absorption capacity of 5% or 7%. Therefore, C50-20I means concrete with a water to cement ratio of 0.50, made with a 20% replacement percentages and recycled coarse aggregates with 5% of water absorption capacity.

Firstly, consistency of fresh concrete was identified. Then, compressive strengths at 3, 7 and 28, days were studied in order to define a correct time-dependent curve of compressive strength concretes with different replacement ratios. After that, tensile splitting strength was defined at 28 days, and finally, shrinkage tests were carried out.

2.1 Materials

The cement used was always CEM I 52.5 R, according to EN 197-1.

Two different superplasticizers were used as water reducing admixture in order to obtain suitable workability for the first and the second part of the experimental activity: both are constituted by an aqueous solution of modified vinyl copolymers and carboxylic acrylic ester polymer, but that used in Spain had a water reduction power of 30% if dosed at 1% by weight of cement while the one used in Italy had a water reduction power of 38% at the same dosage.

As fine aggregate, natural sand (0–4 mm) was used with a fineness modulus of 3.71. Concerning natural coarse aggregates, two different fractions were used: fine gravel with aggregate size 4–12 mm (6–12 mm), and gravel with 8–20 mm (11–20 mm), whose fineness moduli were 6.20 and 7.37, respectively.

Then, a coarse recycled fraction of 4-16 mm was used. These aggregates were obtained from real demolition debris of structural concrete with different origin, one with a water absorption capacity of 5.40% and the other one with greater absorption capacity, 6.43%. These aggregates were made up mainly of concrete, more than 80% is old mortar and aggregates, Fig. 1. The original concrete strength class was unknown and likely different for waste concrete coming from different sources. The content in recycled concrete fractions of chlorides, sulphates, organic materials were evaluated according to the methods recommended by EN 1744-1 (parts 7, 11, 12, 14 and 15) and the presence of alkali–silica reactive materials according to the method recommended by EN 8520-22. No organic or alkali–silica reactive materials were detected; concerning the amount of chlorides and sulphates they were below 0.04% (by weight) and 0.15% (by weight) respectively. Table 1 collects the main physical properties of the aggregates, both natural and recycled.

![Figure 1: Composition of recycled coarse aggregate](image-url)
Table 1: Basic properties of the aggregates used

<table>
<thead>
<tr>
<th>Phase</th>
<th>0-4N</th>
<th>8-20N</th>
<th>4-12N</th>
<th>4-16R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density in oven-dry conditions (EN 1097-6)</td>
<td>I</td>
<td>g/cm³</td>
<td>2.58</td>
<td>2.60</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td></td>
<td>2.54</td>
<td>2.57</td>
</tr>
<tr>
<td>Water Absorption (EN 1097-6)</td>
<td>I</td>
<td>%</td>
<td>2.22</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td></td>
<td>2.00</td>
<td>3.00</td>
</tr>
</tbody>
</table>

2.2 Concrete mixtures

Concrete mixtures were designed according to the Faury method, Table 2. Recycled aggregates have a high absorption capacity, due mainly to the adhered mortar [21-28]. This feature influences concrete properties, particularly its consistency and workability, which are reduced when recycled aggregates are introduced into the mix. This decrease is due to the absorption of mixing water by the recycled coarse aggregates.

Table 2: Mix proportion 1 m³

<table>
<thead>
<tr>
<th>Concrete C65</th>
<th>0%</th>
<th>20%</th>
<th>50%</th>
<th>100%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement kg</td>
<td>275.00</td>
<td>275.00</td>
<td>275.00</td>
<td>275.00</td>
</tr>
<tr>
<td>Water Kg</td>
<td>178.75</td>
<td>178.75</td>
<td>178.75</td>
<td>178.75</td>
</tr>
<tr>
<td>0-4N kg</td>
<td>918.49</td>
<td>938.05</td>
<td>962.73</td>
<td>1005.18</td>
</tr>
<tr>
<td>8-20N kg</td>
<td>486.19</td>
<td>372.47</td>
<td>218.29</td>
<td>0.00</td>
</tr>
<tr>
<td>4-12N kg</td>
<td>457.65</td>
<td>350.60</td>
<td>205.48</td>
<td>0.00</td>
</tr>
<tr>
<td>4-16R kg</td>
<td>0.00</td>
<td>180.77</td>
<td>423.77</td>
<td>756.46</td>
</tr>
<tr>
<td>w/c</td>
<td>0.65</td>
<td>0.65</td>
<td>0.65</td>
<td>0.65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete C50</th>
<th>0%</th>
<th>20%</th>
<th>50%</th>
<th>100%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement kg</td>
<td>380.00</td>
<td>380.00</td>
<td>380.00</td>
<td>380.00</td>
</tr>
<tr>
<td>Water kg</td>
<td>190.00</td>
<td>190.00</td>
<td>190.00</td>
<td>190.00</td>
</tr>
<tr>
<td>0-4N kg</td>
<td>781.43</td>
<td>794.31</td>
<td>811.37</td>
<td>838.29</td>
</tr>
<tr>
<td>8-20N kg</td>
<td>665.44</td>
<td>512.76</td>
<td>303.34</td>
<td>0.00</td>
</tr>
<tr>
<td>4-12N kg</td>
<td>307.93</td>
<td>237.28</td>
<td>140.37</td>
<td>0.00</td>
</tr>
<tr>
<td>4-16R kg</td>
<td>0.00</td>
<td>187.51</td>
<td>443.71</td>
<td>807.97</td>
</tr>
<tr>
<td>w/c</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Numerous attempts have been made to define the best way to mitigate the effects of the high absorption capacity of the recycled aggregates and to prevent the decrease in the water to cement ratio. As a result, several alternatives have been proposed: working with dry aggregates while increasing the amount of water incorporated in the mixer [25], pre-soaking the recycled aggregates for 24 hours [20-22], pre-wetting them for 10 minutes [26] or sprinkling them [14]. The chosen option for this experimental program was to pre-wet the recycled aggregates for 10 minutes before mixing.

2.3 Test procedures

Regarding fresh concrete, characterization included the measurement of consistency using the slump test according to EN 12350-2.

For hardened concrete, density at time of demolding, 1 day after casting and at 28 days was
obtained. It was also determined compressive strength (EN 12390-3) at 3, 7 and 28 days using cubic specimens of 100x100x100 mm.

In addition, tensile splitting at 28 days was evaluated according to EN 12390-6. In the first phase (I), cylindrical specimens of 150x300 mm were used. However, in the second one (II) cubic specimens were used to determine the tensile splitting strength.

Finally, prismatic specimens were prepared for each concrete mixture according to EN 11307 in order to obtain shrinkage curves.

3. RESULTS AND DISCUSSION

In order to determine the influence of aggregate quality, two different types of recycled coarse aggregate were used to carry out this experimental, which were identified by their water absorption capacity (WA). This is one of the most important feature to determine the aggregate quality. In this case, both types of recycled aggregate have a WA under 7%.

With these aggregates, compressive and tensile splitting strength and shrinkage of recycled concretes were analysed and compared with those of the conventional one made with the same dosage.

3.1 Consistency and density

The basic properties of the different concretes are collected in Table 3.

Fresh concrete was characterized using the slump-test to obtain the concrete consistency. Density of fresh and hardened concrete decreases as the replacement rate of recycled coarse aggregate increases mainly due to the adhered mortar of the recycled coarse aggregates [3-4; 9; 29-31; 34-36]. It can be observed that concretes made with 100% recycled coarse aggregates have experienced density drops of 4%.

3.2 Compressive strength

Fig. 2 shows the compressive strength at 28 days. It can be observed that these ones decrease as replacement percentage increases.

Analysing concretes made with aggregates I, it can be observed that C50-I recycled concretes with a replacement ratio of 20, 50 and 100% showed a decrease compared to conventional concrete of 11%, 18% and 31%, respectively. If the C65-I concretes are considered, these drops are of 2%, 7% and 22% for 20%, 50% and 100% replacement ratios, respectively.

On the other hand, C50-II concretes have experienced a similar trend to C50-I ones, showing reductions of the compressive strength of 1%, 17% and 30% for C50-20II, C50-50II and C50-100II, respectively. However, C65-II concretes have showed a greater influence of recycled coarse aggregates, with decreases up to 25% and 39% for C65-50II and C65-100II.

The quality of recycled aggregate is also an important feature to take into account. In this case, recycled aggregates I have lower water absorption capacity than recycled aggregates II, which is directly related with the aggregate quality [28]. However, the water absorption capacity ranges from 5.4 to 6.4, and therefore, this slight difference made both concrete series to have a similar behaviour.
Table 3: Basic properties of concretes

<table>
<thead>
<tr>
<th></th>
<th>0%</th>
<th>20%</th>
<th>50%</th>
<th>100%</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concretes C50-I</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Admixture (%)</td>
<td>0.43</td>
<td>0.20</td>
<td>0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>Slump values (cm)</td>
<td>5</td>
<td>6</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Density (t/m³)</td>
<td>2.39</td>
<td>2.36</td>
<td>2.31</td>
<td>2.28</td>
</tr>
<tr>
<td>f_c (28 days) (MPa)</td>
<td>64.13</td>
<td>57.19</td>
<td>52.74</td>
<td>44.20</td>
</tr>
<tr>
<td>f_t (28 days) (MPa)</td>
<td>3.86</td>
<td>3.28</td>
<td>2.86</td>
<td>2.43</td>
</tr>
<tr>
<td>E_c (28 days) (MPa)</td>
<td>39077</td>
<td>35958</td>
<td>35380</td>
<td>25941</td>
</tr>
</tbody>
</table>

| **Concretes C65-I** |       |       |       |       |
| Admixture (%)   | 0.98  | 0.83  | 0.42  | 0.08  |
| Slump values (cm) | 6     | 7     | 6     | 5     |
| Density (t/m³)  | 2.38  | 2.36  | 2.32  | 2.28  |
| f_c (28 days) (MPa) | 53.85 | 52.83 | 49.85 | 42.04 |
| f_t (28 days) (MPa) | 3.67  | 3.67  | 2.88  | 2.82  |
| E_c (28 days) (MPa)  | 36918 | 32294 | 28553 | 25474 |

| **Concrete C50-II** |       |       |       |       |
| Admixture (%)   | 0.70  | 0.70  | 0.70  | 0.70  |
| Slump values (cm) | 1     | 3     | 10    | 15    |
| Density (t/m³)  | 2.42  | 2.37  | 2.37  | 2.29  |
| f_c (28 days) (MPa) | 68.23 | 67.70 | 56.35 | 47.86 |
| f_t (28 days) (MPa) | 3.80  | 3.77  | 3.59  | 2.88  |
| E_c (28 days) (MPa) | 34533 | 30847 | 28026 | 27019 |

| **Concretes C65-II** |       |       |       |       |
| Admixture (%)   | 0.70  | 0.70  | 0.70  | 0.70  |
| Slump values (cm) | 15    | 17    | 18    | 15    |
| Density (t/m³)  | 2.40  | 2.39  | 2.35  | 2.29  |
| f_c (28 days) (MPa) | 63.16 | 58.67 | 47.15 | 38.60 |
| f_t (28 days) (MPa) | 3.91  | 3.61  | 3.15  | 2.70  |
| E_c (28 days) (MPa) | 27802 | 24000 | 23328 | 22282 |

Figure 2: Compressive strength at 28 days
By focusing the attention on the compressive strength evolution over time, it can be noted that the recycled concretes have developed particularly low strengths at early ages comparing with those obtained at 28 days, Fig 3.

This effect is due to the use of recycled coarse aggregates under pre-saturation conditions leads to bleeding, which causes a different time-dependent development of the concrete strengths. The extra water inside aggregate pores bleeds during the cement paste hydration leading to a weaker paste-aggregate matrix (interfacial transition zone (ITZ) that reduces recycled concrete compressive strength, especially at early ages [9, 17, 28]. This reduction in compressive strength influences also the recycled concrete deformability.

From 3 to 7 days, it has been observed a different behaviour pattern among concretes with high content of recycled aggregate and low replacement ratios. In fact, concretes with 20% of recycled coarse aggregate underwent a similar evolution to conventional one; increasing 17-25%; while the recycled concretes with 50% and 100% of replacement showed higher increases of compressive strength, 36-47%.

From 7 to 28 days, lower increments of compressive strengths were detected in all concretes. Again a different trend was observed, depending on the recycled aggregate content. On the one hand, concretes with 0% and 20% of replacement percentage developed increases of 10%, while concretes with high content of recycled aggregate raised 16-25%.

Therefore, it was noted that recycled concretes up to 20% of replacement provide features and time-dependent evolution similar to the reference concrete. However, recycled concretes with high replacement percentages (50-100%) show a different pattern.

### 3.3 Tensile splitting strength

Analysing tensile splitting strengths, it was found a similar trend to that observed for compressive strength, Fig. 4.

C65 and C50 recycled concretes experienced reductions of tensile splitting strength in the same extent than compressive one, between 23-37% for 100% replacement rate. Again, 20% replacement ratios develop similar values to that obtained for control concrete.
3.4 Shrinkage

Fig. 5 shows the experimental shrinkage curves. It can be seen that shrinkage growths gradually as the replacement percentages of recycled coarse aggregates increases [10; 16]. In this way, at 90 days it has been obtained that shrinkage is up to a 55-78% higher in concretes made with 100% recycled coarse aggregate than in conventional ones.

It can be seen that recycled concretes have developed particularly low shrinkage values at early ages, showing a curing effect due to the use of recycled coarse aggregates under pre-saturation conditions (internal curing effect). It is known that, these differences get reduced at later ages and recycled concretes shrinkage shows a time-dependent development similar to that obtained with the conventional ones.

It has been thus noted a delay on the shrinkage development of recycled concretes. This effect, which has been already detected in compressive strength, can be attributed to the use of pre-saturated recycled aggregates, with a high water content inside their particles. This leads to bleeding during the early ages reducing then the shrinkage at this time.

4. CONCLUSIONS

The mechanical behaviour of recycled concretes with different replacement ratios of
recycled coarse aggregate at different ages is determined in this research. Based on these experimental results the following conclusions can be drawn:

- As already noted by different authors, the density and mechanical strength, compressive and tensile splitting, of recycled aggregate concrete decreases as the recycled aggregates content increases. When the compressive strength of 100% replacement recycled aggregate concretes is analyzed, these reductions reach 22-31%. Tensile splitting strength decreases also when replacement percentage rises.
- Regarding time-dependent of compressive strength, it has been noted that recycled concretes have developed particularly low strengths at early ages comparing with those obtained at 28 days. This effect is due to the bleeding effect caused by the use of recycled coarse aggregates under pre-saturation conditions.
- Shrinkage of recycled concretes increases with the replacement percentage of recycled coarse aggregate. In addition, it has been noted a delay on the shrinkage development of recycled concretes at early ages. This effect, known as internal curing, is due to the use of pre-saturated recycled aggregates with a high water content inside their particles, which leads to bleeding during the early ages reducing then the shrinkage at this time.

REFERENCES


STUDY OF BUCKLING OF SMA REINFORCEMENTS IN CONCRETE ELEMENTS

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Key words: Structural engineering, concrete structures, Ni-Ti, instability.

Abstract. Shape Memory Alloys (SMA) are becoming more and more useful in construction world every day. This material can be attractive to use in seismic design due to its superelasticity and high deformation capacity. However, its low young modulus in relation to steel can cause buckling of compressed bars. This phenomenon can cause significant reductions of strength and deformation capacity. In order to prevent this from happening, the structural codes [1, 2, 3] restrict the relationships between diameters of transversal and longitudinal reinforcements, and determine minimum separations of transversal reinforcement, always talking about steel. There is no design regulation to prevent buckling of SMA bars.

Research of reinforcement buckling requires numerical models and experimental campaigns to understand and quantify this phenomenon. However, in scientific literature, there is a lack of experimental campaigns specifically focused on the study of reinforcement buckling, inserted into a concrete section. The rigorous detection of onset of reinforcement buckling is not an accurately measured variable in literature. In addition, there is a total absence of experimentation when considering very high performance concrete (VHPC) and SMA reinforcements. For all of these reasons, there is insufficient data at one’s disposal to be able to evaluate numerical models or to draw conclusions about reinforcement buckling.

In this work is presented the experimental results about buckling of SMA in form or nickel and titanium (Ni-Ti) in VHPC columns under compressive and eccentric loading.

1 INTRODUCTION

Consequences of buckling of the compressed passive reinforcement are a drastic decrease
of the expected ductility. Buckling of reinforcements can happen in reinforced concrete elements subjected to bending and axial stresses. This may occur because either insufficient or incorrect arrangement of transverse reinforcement, or concrete cover degradation, among other causes.

On the other hand, shape memory alloys (SMAs) are a new class of metallic alloys capable of undergoing very large deformations while reverting back to their original undeformed shape through either the application of heat (what is called “shape memory effect”), or the removal of the load (superelastic effect) [4]. Whether the former or the latter effect is triggered off depends on the room temperature and the characteristics of the material. The capability of SMAs to recover their shape (Figure 1) are due to the existence of two crystalline structures: the austenite phase is stable at low stress and high temperature, and as a result of temperature change or applied stress is transformed in the martensitic phase through a displacive (diffusionless) phase transformation [5].

On a macroscopic level, superelastic SMAs develop a flag-shaped hysteresis under cyclic axial loading, which provides both recentering and energy dissipation as seen in Figure 1. Total recovery of axial strain can be achieved for strains up to 5-6%. The unique properties of SMAs have led to the development of a variety of applications and devices over the past few decades in fields such as biomedical, aerospace and automobile, taking advantage of the excellent corrosion resistance, damping characteristics, recentering capability and low- and high-cycle fatigue properties [6]. While applications in these areas have emerged, many other fields have started to probe the properties of SMAs. Namely, the recentering and damping properties associated with SMAs have prompted interest in the civil engineering field, in particular for the use of SMAs in seismic applications intended to fit the needs of performance-based design, currently used in the design of structures in earthquake prone areas.

![Figure 1: Idealized superelastic shape memory alloy stress-strain behaviour [5]](image)

Theoretical models exist in literature to determine when the buckling of the compressed reinforcements occurs: computational models like full finite element and fiber discretization models, and analytical models like minimization energy approach or stiffness based methods. The aforementioned types of models do not take into account the effect of the concrete cover
continuously and the transverse reinforcement discretely as it happens in reality.

Among those who use the energy minimization are Dhakal and Maekawa (2002c) [7] and Tallat (2007) [10], who model stirrups in a distributed way and do not consider concrete cover effect against buckling; Dhakal (2006) [8] shows how to incorporate the effect of concrete cover in a distributed way as well; Campione (2011) [9], who assumes the stiffness of the stirrups and the cover continuously. Moreover, Papia et al. (1988) [11] use a model based on stiffness of the system and model stirrups discretely.

Regarding computational models, they only consider the bars as isolated elements, without interaction with the stirrups or buckling between two infinitely stiff ones. They are computationally demanding, so they are not valid for design. Full finite elements are used by [12, 13, 14] and fiber discretization models are use by [15, 16].

Despite the existence of models that predict the buckling of the steel reinforcements, there are not accurate experimental campaigns focused exhaustively on the study of buckling in reinforced concrete elements to get critical strains. They are only based on observation data. Regarding SMA reinforcements, there are no experimental campaigns evaluating the buckling of this kind of bars. This is due to shape memory alloys are new materials in the construction world and they are not widespread.

2 OBJECTIVES AND IMPORTANCE OF RESEARCH

The results of an experimental campaign carried out with the aim of finding accurately buckling strains of the nickel-titanium (Ni-Ti) SMA reinforcements inside a fibre reinforced concrete section are presented. This will let know the maximum rotation that can be reached in plastic hinges prior to buckling of reinforcements. High Strength Fiber Reinforced Concrete (HSFRC) and Very High Performance Concrete (VHPC) have been used.

3 THEORETICAL MODEL

A theoretical model has been developed for calculating the load and the critical buckling length of the compressed passive reinforcement. This takes into account the effect of transverse reinforcement discretely, and the positive effect of the concrete cover continuously. The model is based on that of Papia et al. (1988) [11], which considers hoops discreetly, and it is extended to take into account the concrete cover continuously.

3.1 General description and assumptions

This model is based on the equilibrium of forces in the passive compressed rebar, which are discrete forces caused by the stirrups ($F_j$), force distributed due to the concrete cover ($F_c$), axial force in the bar ($P$) and bending moment at built-in end ($M_0$), shown in figure 2. The stirrups are modelled as springs with a certain axial stiffness ($\alpha_s$) in contact with the longitudinal bar and the concrete cover as an element which has a distributed stiffness ($\alpha_c$) along the instability length. It has been adopted as an hypothesis that the stiffness of the stirrups is the same for all of them and distributed stiffness of concrete cover is the same at all times.
3.2 Analytical procedure

The analytical procedure starts with the differential equation of the deflected bar, assuming the symmetry simplification:

$$E_r \cdot I \cdot \frac{d^2 y_j(x)}{dx^2} = -(P \cdot y_j(x) - M_0 - M_{sj}(x) - M_{cj}(x))$$

(1)

where:

- $E_r$: Reduced modulus in circular section [11]. $E_r$ depends on the elastic modulus ($E_s$) of Ni-Ti and on the tangent modulus at the onset of instability ($E_h$).
- $y_j(x)$: Coordinate of “y” displacement of “j” segment depending on x.
- $I$: Inertia moment of longitudinal rebar.
- $M_0$: Flexural moment at built-in end in the bar.
- $M_{sj}(x)$: Flexural moment due to lateral loads generated by stirrups.
- $M_{cj}(x)$: Flexural moment due to lateral loads generated by concrete cover.

Following the procedure of Papia et al. (1988) [11], eq (1) can be solved using boundary and symmetry conditions, and with $y_j(x) \geq 0$ condition is possible to obtain the dimensionless critical load:

$$c_c = P / (\pi^2 E_r I / l^2)$$

(2)
4 EXPERIMENTAL RESULTS

4.1 Specimens

An experimental campaign of 4 rectangular supports under eccentric monotonic loading has been performed. Two of them were made of HSFRC and two of VHPC. Tested supports were 1.35 m long and had a section of 20x20 cm (Figure 3). Two types of fibers are used: Type A has a slenderness of 80 and 30 mm long. Type B has a slenderness of 26 and 13 mm long. The elastic modulus of all fibers is $E_f = 200\,000$ MPa. HSFRC has 40 kg/m$^3$ of type A fibers. VHPC has 60 kg/m$^3$ of type A fibers and 90 kg/m$^3$ of type B fibers. Concrete compressive strength is obtained by testing cylindrical samples with 15 cm diameter and 30 cm long. The average strength is 81.3 MPa for HSFRC and 111.3 MPa for VHPC and the coefficient of variation is 0.052 and 0.045 respectively.

The longitudinal compressed reinforcements are made Ni-Ti (approximately 50% of each component) and they are 12 mm diameter. The elastic modulus of Ni-Ti is 65 GPa and the yield stress is 420 MPa. Tensioned bars are made of B 500 SR steel and are 16 mm diameter. This fact, and a load eccentricity of 10 cm (half of the side), ensures that both reinforcements are yielded when buckling occurs, according to current design criterion. Stirrups vary its separation depending on the specimens (5 or 10 cm).

![Figure 3: Left, frontal side of a tested specimen; right, state of compressed side after testing](image)

4.2 Onset of buckling detection

Instrumentation used for capturing the buckling strain was steel gauges positioned along longitudinal reinforcements. 8 gauges were located in specimens with 10 cm of stirrups separation and 14 gauges in specimens with 5 cm of stirrups separation. When a sharp drop in strain is registered, while applied load is increasing, it would mean that the bar is beginning to
It has also been utilized 3 concrete gauges for measuring the deformation in the compressed side and steel gauges in tensioned longitudinal reinforcements. They were located in the central section. This instrumentation defines the strain plane up to either concrete cover failure or the buckling of the compressed reinforcements.

After the failure of the concrete cover, the damage is located in a particular area, forming a plastic hinge (Figure 3). Photogrammetry is used for getting the strain plane after a rupture of the cover. This allows to know the position of the neutral fibre. All this allows to know the buckling strain of the bar in the centre of it, since gauges were arranged to capture buckling and not to measure the average strain in the directrix.

4.3 Results

The main result after testing the 4 specimens is that Ni-Ti shape memory alloy rebars does not buckle. Strain gauges do not detect a drop in the registered strain while applied load is increasing. In a previous research, buckling strains of steel rebars were captured in analogous specimens with this procedure. For this reason the procedure is validated.

The explanation for which Ni-Ti rebars does not buckle is because this kind of alloys posses and asymmetric behaviour in tension with respect to compression. In tension, the slope of the non-linear plateau ($E_h$) is low. In compression this slope is higher, as well as the yield stress. According to the theoretical model presented, $E_r$ depends on $E_s$ and $E_h$. For this reason $E_r$ of Ni-Ti alloys is high, compared with steel reinforcements. Therefore looking at eq (2) the higher $E_r$, the higher $c_{cr}$ and $P$.

5 CONCLUSIONS

The results of the conducted experimental campaign provide several conclusions that are set out below:
- Reduced modulus $E_r$ of Ni-Ti rebars is higher than B 500 SR steel rebars.
- The higher $E_r$, the higher $c_{cr}$ and critical load. $E_r$ depends on $E_s$ and $E_h$. Not only is important the elastic modulus $E_s$ but also the slope of the non-linear plateau $E_h$.
- In case of tested Ni-Ti, in spite of having $E_s = 65$ GPa (30% of steel modulus), the high $E_h$ causes that Ni-Ti does not buckle and steel reinforcements does according to previous research carried out previously in ICITECH.

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STUDY OF ENVIRONMENTALLY FRIENDLY BEDDING MORTARS PREPARED WITH RECYCLED AGGREGATES AND BIOMASS ASH

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Key words: Biomass Ash, Concrete Recycling, C&D waste, Recycled Aggregate Mortar, Sustainability.

Summary. This work is aimed at evaluating the possibility of reusing waste from demolished concrete building as aggregate for bedding mortars replacing virgin aggregate. Besides to recycled aggregate the mortars studied in this paper were prepared with another kind of waste: a biomass ash coming from paper mill sludge, which is a by-product of paper production. In order to evaluate the quality of joining mortars made of recycled aggregate and biomass ash, both mechanical behaviour of cementitious mortars and the interaction between mortar and brick in terms of bond strength developed at the interface mortar–brick were studied. The experimental results show that mortars containing recycled aggregates develop lower mechanical strength with respect to the reference cementitious mortar, particularly when recycled aggregates and biomass ash are used together. Nevertheless, the bond strength at the interface between the mortar and the brick comes out to be higher if an inorganic primer is used. However, concerning bedding mortars, the mechanical performance of the overall mortar–brick system, strictly related to the mortar–brick adhesion, makes the mortar bond strength certainly more important than its mechanical strength.

1 INTRODUCTION

Concrete is the most used construction material, with estimated annual production of 10 billion cubic meters. Since 60–80% of the concrete volume is taken by aggregates, the overall consumption of natural aggregates is very high, generating huge pressure on surrounding ecosystems. The environmental impact of aggregate extraction is particularly severe regarding sands, with distinct problems associated to different extraction or production technologies. Sand extraction from seaside increases erosion and retreating coastline, harming inland protection and fauna and flora habitats, and changing wave and tide behaviour. Sand extraction from riverbed or lakebed alters flow regimes, affecting surrounding structures and local ecosystems. Finally, the production of fine aggregates presents high energetic cost and raises
difficulties concerning fresh concrete, given their high angularity [1].

Construction and demolition wastes (C&DW) has been identified as a priority waste stream by the European Union (EU). It accounts for approximately 25-30% of all waste generated in the EU. The level of recycling and material recovery of CDW varies greatly (between less than 10% and over 90%) across the Union. However, in most cases it is reused for backfilling applications more than recycling.

Figure 1: CDW statistics across EU (source: http://ec.europa.eu/environment/waste/construction_demolition.htm).

In 2002, the CEN/TC 154 drew the EN 12620 “Aggregates for concrete” in which artificial or recycled aggregates are considered beside natural aggregates for use in concrete.

Among other things studies carried out on this topic reported that, in recycled-aggregate concrete, the fine recycled-aggregate fraction is particularly detrimental to both mechanical performances and durability of concrete. In addition, these studies showed that, the presence in the recycled-aggregate of materials other than concrete (that are mainly crushed bricks and tiles) reduces concrete performance due to their higher porosity and consequently their lower density and higher water absorption [2-6]. For this reason the more recent approach is to recycle for concrete production only the coarse recycled fraction [7-10].

In particular, the Italian Standards UNI EN 12620 and UNI 8520 allow the use of the coarse recycled fraction coming from building demolition only for low-strength concretes (under 25 MPa strength class) and the use of the coarse recycled fraction coming from demolished concrete for concretes with strength class up to 35 MPa. In this way a huge
amount of fine recycled materials cannot be reused neither in concrete neither in roadbeds or floor foundation (where higher particle size is requested). In addition, due to the reasons above mentioned, before their input in recycling plant materials coming from building demolition need to be separated into concrete rubble and brick rubble. Consequently, these materials are available not only as miscellaneous rubble but also as either concrete scraps or as crushed bricks, and they proved to be suitable for replacing virgin sand in mortars [11-14].

Besides to recycled aggregate the mortars studied in this paper were prepared with another kind of waste: a biomass ash coming from paper mill sludge, which is a by-product of paper production. Paper mill sludge is composed of mineral fillers, inorganic salts, small cellulose fibres, water and organic compounds. Paper mill sludge is often burnt in order to reduce the waste disposal and sometimes to recover heat. This process is achieved by incineration at high temperature (> 800°C). During incineration, paper and organic compounds are burned out, whereas mineral fillers and inorganic salts are transformed into the corresponding oxides at higher temperatures. CaO, Al₂O₃, MgO and SiO₂ are the most abundant oxides in incinerated paper mill sludge [15-16]. The obtained paper mill sludge ash is classified as waste, and at present it is mainly conferred to landfill at high costs.

2 EXPERIMENTS

Four different combinations were tried by alternatively using virgin sand and recycled concrete fine fraction (recycled sand), by using either ordinary cement or cement replaced (20% by weight) with biomass ash.

Mortars were prepared with two different levels of fresh workability, in order to check the influence of this parameter on mortar mechanical performance and mortar adhesion to masonry support.

Mortar adhesion to brick was checked by means of triplet tests, also in the presence of an inorganic primer made of a cement paste in which microcement was used (particle size under 0.050 mm) in order to improve bond at the interface mortar-brick.

3.1 Materials

As aggregate for mortars, either quartz sand or recycled concrete scraps ground by means of a rotating ball mill were used. The particle size distributions of quartz sand and recycled concrete fractions, all characterized by the same maximum size of 4 mm, were determined according to EN 933-1. Specific gravities in saturated-surface-dried conditions, determined according to EN 1097-6, were 2.63 and 2.33 relative specific gravity, respectively, and water absorptions were 4.2% and 8.6%, respectively.

As binder, commercial portland-limestone blended cement type CEM II/B-L 32.5 R according to EN-197/1 was used. In alternative, biomass ashes from paper mill sludge combustion (BIO) were used. Biomass ashes were characterized by a percentage of material passing through the sieve of 0.150 mm of about 80%. This information was taken into account during SCC mixture proportions, in order to correctly control the amount of filler introduced. An image of BIO obtained by means of scanning electron microscopy is reported in Figure 2. Its chemical composition is roughly 40% CaO, 30% SiO₂, 15% Al₂O₃, 5% MgO and other minor constituents (10% as a whole).
3.2 Mortar mixture proportions

The dosage of quartz sand was 3:1 with respect to cement, while the dosage of recycled sand was reduced keeping into account the lower volumic mass of recycled particles (in this way the same volume of inert particles was obtained in all the tested mixtures). In two cases the cement was partially replaced by biomass ash (by 20% by weight of cement).

Fresh consistency of the mortar was equal to $130 \pm 10$ mm (evaluated according to EN 1015-3) for a first series of mixtures (Table 1), then equal to $165 \pm 10$ mm for the second series (Table 2). Water dosage was set to achieve the prefixed value of consistency.

**Table 1**: Mixture proportions of the mortars with fresh consistency of 130 mm (g/litre of mortar)

<table>
<thead>
<tr>
<th>MIXTURES</th>
<th>Quartz sand</th>
<th>Recycled sand</th>
<th>Cement</th>
<th>Biomass ash</th>
<th>Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAT-130</td>
<td>1350</td>
<td>-</td>
<td>450</td>
<td>-</td>
<td>230</td>
</tr>
<tr>
<td>BIO-NAT-130</td>
<td>1350</td>
<td>-</td>
<td>360</td>
<td>90</td>
<td>240</td>
</tr>
<tr>
<td>REC-130</td>
<td>-</td>
<td>1110</td>
<td>450</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>BIO-REC-130</td>
<td>1110</td>
<td>-</td>
<td>360</td>
<td>90</td>
<td>320</td>
</tr>
</tbody>
</table>

**Table 2**: Mixture proportions of the mortars with fresh consistency of 165 mm (g/litre of mortar)

<table>
<thead>
<tr>
<th>MIXTURES</th>
<th>Quartz sand</th>
<th>Recycled sand</th>
<th>Cement</th>
<th>Biomass ash</th>
<th>Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAT-165</td>
<td>1350</td>
<td>-</td>
<td>450</td>
<td>-</td>
<td>260</td>
</tr>
<tr>
<td>BIO-NAT-165</td>
<td>1350</td>
<td>-</td>
<td>360</td>
<td>90</td>
<td>265</td>
</tr>
<tr>
<td>REC-165</td>
<td>-</td>
<td>1110</td>
<td>450</td>
<td>-</td>
<td>320</td>
</tr>
<tr>
<td>BIO-REC-165</td>
<td>1110</td>
<td>-</td>
<td>360</td>
<td>90</td>
<td>340</td>
</tr>
</tbody>
</table>
### 4 RESULTS AND DISCUSSION

#### 4.1 Compression and bending tests

Prismatic specimens (40 x 40 x 160 mm) were manufactured, cast and wet cured at 20°C up to 42 days. The compressive and flexural strengths were evaluated according to EN 1015-11. Results obtained in terms of compressive and flexural strengths are reported in Figures 3-4 and 5-6 for the two levels of workability, respectively.

The use of recycled sand and biomass ash replacing virgin sand and 20% of cement produced a strength loss of about 20% and 25% in the case of higher and lower mortar fluidity, respectively. This result is coherent with previous studies showing that the higher is the water/cement the lower is the strength loss due to the use of recycled aggregate [5, 8, 10].

#### 4.2 Triplet tests

Three red prismatic (250 x 120 x 50 mm) bricks were used for triplet tests (test procedure derived from UNI EN 1052-3). The value of Initial Rate of Absorption (IRA) was about 2 kg/m²/min. IRA represents the mass of water absorbed per unit area in 1 minute by the brick face in contact with the mortar, when immersed to a depth of 3 mm in water.

The tested model is composed of three bricks, it has a symmetric structure thus avoiding eccentric loads. The applied load was measured and at the same time the vertical displacement of the central brick was also monitored. Usually, at the end of the test only one joint was cracked, so the bond strength was calculated dividing the maximum load by twice the fracture area where brick and mortar were in contact (approximately 120 x 200 mm).

![Figure 3: Compressive strengths vs. curing time for the mortars with higher fresh consistency](image-url)
**Figure 4**: Compressive strengths vs. curing time for the mortars with lower fresh consistency

**Figure 5**: Flexural strengths vs. curing time for the mortars with higher fresh consistency
Tests were carried out on five triplet specimens for each mortar mixture reported in Table 1 and Table 2, results obtained are included in Figure 7. In this figure also the results obtained by using the mortar mixtures reported in Table 2 and an inorganic primer applied on the surface of brick were reported. This inorganic primer was spread just few minutes before mortar layer.

**Figure 6:** Flexural strengths vs. curing time for the mortars with lower fresh consistency

**Figure 7:** Bond strengths measured by means of triplet tests after 28 days of curing
It can be observed that the workability level influences the bond strength more than the mechanical strength of the mortars. In fact, higher fluidity (165 mm fresh consistency) is obtained with higher water/cement and the related mortars are weaker, but they are able to show higher adhesion with brick with respect to those prepared with lower fluidity (130 mm fresh consistency), due to better permeation of mortar within the porous surface of the brick, as already found in other studies [11-13]. The use of recycled sand instead of natural sand in some cases improves bond strength and in other cases reduced it. The use of biomass ash generally reduced bond strength because the grain size is higher with respect to cement (see Fig. 1). In fact, the use of inorganic primer made of a cement paste, in which the binder was a microcement (with particle size under 0.050 mm) proved to be extremely effective in improving adhesion. The reason is just the size of the particles, the finer they are the higher is the ability to permeate brick surface and to guarantee good interlock, as already found in a previous study [17]. In fact, the use of inorganic primer allowed to modify the kind of rupture: not more detachment at the interface mortar-brick (see Fig. 8 left side), but shear failure inside the mortar layer (see Fig. 8 right side). The results obtained showed values of bond strength more than double in the absence of biomass ash, and even fivefold when biomass ash is used. Further studies concerning the shear strength of mortars prepared with recycled sand and biomass ash are necessary to better interpret such surprising results.

Figure 8: Mortar REC-130: loss of adhesion at the interface mortar-brick (left), shear failure within the mortar in the presence of inorganic primer (right)

5 CONCLUSIONS

The following general conclusions can be drawn from the study reported in this paper:

- The use of recycled sand and biomass ash replacing virgin sand and 20% of cement, respectively, produced a strength loss of about 20% and 25% in the case of higher and lower mortar fluidity, respectively;
In terms of bond strength, the workability level influences the bond strength more than the mechanical strength of the mortars; the use of inorganic primer proved to be extremely effective in improving adhesion, because it allowed to modify the kind of rupture: not more detachment at the interface mortar-brick, but shear failure inside the mortar layer; this effect proved to be particularly effective if the mortar prepared with recycled sand and biomass ash is coupled with brick.

In conclusion, the use of materials coming from C&D waste recycling instead of sand as well as of biomass ash for the production of bedding mortars, especially in the presence of an inorganic primer, proved to be profitable not only for the obvious environmental advantages but also in terms of mortar–brick interface, which is generally recognized as the weak chain link of the masonry assemblage.

REFERENCES


STUDY OF MECHANICAL PROPERTIES OF HIGH PERFORMANCE CONCRETE WITH ADDITION OF STABILIZED NANO-SILICA

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Key words: Nano-silica, High Performance Concrete, Mechanical Properties

Abstract. Nanomaterials can improve the mechanical properties and durability of concrete. In the case of high performance concrete (HPC), a good dispersion of materials is an essential factor for improving its properties. In this study, the influence of the addition of stabilized nano-silica in compressive and splitting tensile strength of HPC was evaluated. Three different concrete mixes were produced: one containing stabilized nano-silica, another containing stabilized nano-silica combined with silica fume, and a reference concrete. The incorporation of the nano-silica in the polycarboxylate superplasticizer contributed to a better application and efficiency of the nano-silica in the concrete, since the nano-silica was easily homogenized in the concrete mix. Results showed a substantial increase in the concrete compressive strength as well as splitting tensile strength, even with the reduction in cement consumption. The results also suggest that there is a synergy between the nano-silica and silica fume and can be a good alternative to produce concrete with smaller cement content and high mechanical strength. Therefore, concretes with nano-silica stabilized can be a good option for sustainable development.

1 INTRODUCTION

High performance concrete (HPC) is a concrete, which has, at the same time, high workability, mechanical strength and durability. The use of mineral admixtures in HPC can influence favorably its properties by physical effect, due to the presence of very fine size particles, and by the chemical reactions associated with pozzolanic or cementitious materials. The properties that are favorably affected are the rheological behavior of fresh concrete mix, as well as the mechanical strength and durability of hardened concrete. In general, due to better packing of particles and lower demand for water, the fresh concrete containing mineral
cementitious materials has reduced tendency to segregation and exudation, resulting in better cohesion and workability [1].

Nanomaterials can improve the mechanical properties and durability of concretes. For HPC, good dispersion of the materials is an essential factor to improve its properties. The greatest challenge to the success of the use of nanomaterials in the cement is to obtain an even distribution of the nanoparticles. A better dispersion of these nanoparticles requires simultaneously a good dispersal of the cement particles [2].

Previous investigation has shown the influence of nano-silica particles dispersed in water solution (15% in suspension) on the mechanical properties (tensile and compressive strength) and durability of concrete (water absorption and depth of chloride penetration). The results showed that these concrete properties with addition of nano-silica had better performance than the reference concrete [3].

Another study [4] analyzed the influence of nano-silica and silica fume on the performance of concrete. The water/cement ratio was equal to 0.50. The results showed that the simultaneous addition of 6% of silica fume and 1.5% of nano-silica as a partial replacement of Portland cement improved the compressive strength, electrical resistance as well as reduce significantly the capillary absorption. The use of colloidal nano-silica in suspension and silica fume has been shown to improve the mechanical properties of self-compacting concrete [5].

In this scenario, the objective of this work was to evaluate the mechanical properties (compressive and splitting tensile strength) of the HPC with addition of nano-silica stabilized. The influence of combining stabilized nano-silica with silica fume on the HPC was evaluated.

2 MATERIALS AND EXPERIMENTAL PROCEDURES

2.1 Materials

In this research, concrete composition materials were selected to provide better particle size distribution and packing of aggregates, in order to obtain concrete with good compactness. The fine aggregate composition consisted of 70% of natural sand (quartz) and 30% of artificial sand (originated from gneiss). The natural and artificial sands presented fineness modulus 2.30 and 2.20, respectively. The size distribution of the gneiss coarse aggregate was 60%, 30% and 10% of aggregates with maximum dimensions of 25 mm, 12.5 mm and 6.3 mm, respectively. The Table 1 presents the particle size distribution of the aggregates.

Brazilian Type CP V ARI [ASTM Type III] cement (ABNT NBR 5733)[6] was used in all concretes. This kind of cement was chosen since it does not to contain any pozzolanic materials or slag.

The silica fume used was spherical and had an amorphous SiO2 content greater than 85%, a specific surface of 20000 m²/kg and a specific gravity of 2.220 g/cm³. The typical particle size was between 200 nm and 1 µm. The silica fume was in the form of a suspension [7].

The nano-silica (Silicon NS AD 200) was stabilized and dispersed homogeneously into a polycarboxylate superplasticizer, which enabled a good homogeneity in the concrete mass at the time of mixing. The typical nano-silica particle size was between 3 and 200 nm [7].

A multifunctional plasticizer MURAPLAST FK 93, which is compatible with superplasticizer admixtures, was used in all concretes.
The employed water was locally supplied and satisfied the requirements of Brazilian standard NBR 15900-1 \[8\].

**Table 1**: Particle size distribution of the aggregates

<table>
<thead>
<tr>
<th>Mesh (mm)</th>
<th>Natural sand</th>
<th>Artificial sand</th>
<th>Coarse aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Accumulated retained (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>8.6</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>12.5</td>
<td>56.7</td>
<td>0.3</td>
<td></td>
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<tr>
<td>9.5</td>
<td>0.0</td>
<td>91.0</td>
<td>43.3</td>
</tr>
<tr>
<td>6.3</td>
<td>0.8</td>
<td>0.0</td>
<td>99.1</td>
</tr>
<tr>
<td>4.8</td>
<td>1.5</td>
<td>0.1</td>
<td>99.4</td>
</tr>
<tr>
<td>2.4</td>
<td>4.8</td>
<td>3.4</td>
<td>99.1</td>
</tr>
<tr>
<td>1.2</td>
<td>16.7</td>
<td>27.1</td>
<td>99.4</td>
</tr>
<tr>
<td>0.6</td>
<td>33.3</td>
<td>47.1</td>
<td>99.4</td>
</tr>
<tr>
<td>0.3</td>
<td>78.0</td>
<td>63.0</td>
<td>99.4</td>
</tr>
<tr>
<td>0.15</td>
<td>95.3</td>
<td>78.8</td>
<td>99.4</td>
</tr>
<tr>
<td>Total</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>

### 2.2 Concrete mix proportion

Three different concretes were produced designing to achieve a characteristic compressive strength $f_{ck} = 45$ MPa (at 28 days of age) and a slump equal to 100 ± 20 mm. The Table 2 synthetizes the dosage of the mixtures studied.

**Reference concrete**: 1 : 1.135 : 2.124 (cement: fine aggregates: coarse aggregates). The water/cement ratio was equal to 0.41 and the cement content was 503 kg/m$^3$ of concrete. The amount of the multifunctional plasticizer was 0.90% with respect to cement content.

**Concrete with stabilized nano-silica**: 1 : 1.662 : 2.558 (cement: fine aggregates: coarse aggregates). The water/cement ratio was equal to 0.41 with a cement content of 424 kg/m$^3$. The amount of superplasticizer admixture containing the nano-silica was 0.39% with respect to the cement content. The amount of the multifunctional plasticizer was 0.90% with respect to cement content.

**Concrete with stabilized nano-silica and silica fume**: 1 : 1.650 : 2.557 (cement and silica fume: fine aggregates: coarse aggregates). The water/cement and water/cementitious materials ratios were respectively 0.425 and 0.41. The cement content was 409 kg/m$^3$. The amount of silica fume (in the form of a suspension) corresponded to 7% with respect to the cement content while the superplasticizer admixture containing the nano-silica was 0.39% (with respect to the cement and silica fume content). The amount of the multifunctional plasticizer was 0.90% with respect to cement and silica fume content.

The amount of water present in the silica fume suspension was considering in the water/cementitious materials ratio in order to keep the same water/binders ratio.
Table 2: Proportions and consumption of the materials by cubic meter of concrete

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Reference concrete</th>
<th>Concrete with stabilized nano-silica</th>
<th>Concrete with stabilized nano-silica and silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>water/cement</td>
<td>0.41</td>
<td>0.41</td>
<td>0.425</td>
</tr>
<tr>
<td>water/binders</td>
<td>0.41</td>
<td>0.41</td>
<td>0.41</td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>503</td>
<td>424</td>
<td>409</td>
</tr>
<tr>
<td>Silica fume (suspension) (kg)</td>
<td>-</td>
<td>-</td>
<td>28.63</td>
</tr>
<tr>
<td>Natural sand (kg)</td>
<td>396</td>
<td>490</td>
<td>486</td>
</tr>
<tr>
<td>Artificial sand (kg)</td>
<td>174</td>
<td>215</td>
<td>214</td>
</tr>
<tr>
<td>Coarse aggregate 25 mm (kg)</td>
<td>640</td>
<td>651</td>
<td>650</td>
</tr>
<tr>
<td>Coarse aggregate 12.5 mm (kg)</td>
<td>320</td>
<td>325</td>
<td>325</td>
</tr>
<tr>
<td>Coarse aggregate 6.3 mm (kg)</td>
<td>107</td>
<td>108</td>
<td>108</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>206</td>
<td>174</td>
<td>174</td>
</tr>
<tr>
<td>Multifunctional plasticizer (kg)</td>
<td>4.52</td>
<td>3.82</td>
<td>3.82</td>
</tr>
<tr>
<td>Polycarboxylate superplasticizer admixture containing nano-silica (kg)</td>
<td>-</td>
<td>1.65</td>
<td>1.65</td>
</tr>
</tbody>
</table>

The concretes were cast into cylindrical metal molds (10 x 20 cm) according to the Brazilian Standard NBR 5738 [9] using a metal rod. Slump was determined according to NBR NM 67 [10]. After 24 hours, the specimens were stripped from the molds and kept in a curing room until the date of testing. The curing room conditions satisfied the temperature and humidity requirements of NBR 9479 [11].

2.3 Concrete mechanical properties

The evaluated concrete mechanical properties were the compressive strength and splitting tensile strength. Compressive strength was determined according to the requirements of the NBR 5739 [12] at the ages of 7, 14 and 28 days. Five specimens were used at each age.

The splitting tensile strength was evaluated according to the requirements of NBR 7222 [13]. Two specimens were used for each concrete to the ages of 7 and 28 days. For this test, a metal apparatus adapted to the specimens was employed which assured the alignment of the applied load.

A 2000 kN servo-controlled digital electric hydraulic ram, manufactured by SOLOCAP, was used in all tests.
3 TEST RESULTS AND ANALYSIS

3.1 Fresh concrete

The measured slumps were equal to 100 mm, 110 mm and 120 mm, respectively for the reference concrete, concrete with stabilized nano-silica, and the concrete with the mix of stabilized nano-silica and silica fume. The fact of nano-silica been dispersed into a polycarboxylate superplasticizer facilitated its use and effectiveness in the concrete. This is due to easy of nano-silica homogenization when it is immersed into the polycarboxylate admixture in comparison with the nano-silica in powder.

The spherical shape of the silica fume also contributed to improving the workability of concrete as has been shown elsewhere [1].

3.2 Concrete compressive strength

The results of the average concrete compressive strength and corresponded standard deviations are shown in Table 3.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Reference concrete</th>
<th>Concrete with stabilized nano-silica</th>
<th>Concrete with stabilized nano-silica and silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>54.8 ± 3.0</td>
<td>61.8 ± 0.6</td>
<td>66.7 ± 1.8</td>
</tr>
<tr>
<td>14</td>
<td>58.3 ± 1.2</td>
<td>66.0 ± 2.2</td>
<td>69.8 ± 1.4</td>
</tr>
<tr>
<td>28</td>
<td>62.0 ± 1.3</td>
<td>69.3 ± 0.5</td>
<td>73.9 ± 1.8</td>
</tr>
</tbody>
</table>

The results for the concrete compressive strength satisfy the requirements for the specified $f_{ck}$ of 45 MPa at the age 28 days. The standard deviations values were, for all concretes and all ages, less than 5% of the corresponding measured strength. The results also show that there is a beneficial effect with the addition of the nano-silica due to its physical and chemical effects (pozzolanic reaction), increasing significantly the concrete compressive strength in all ages investigated. The analysis of these results also reveals that there is a synergy when stabilized nano-silica and silica fume are combined: the corresponding compressive strength of this concrete was always the highest measured one in all ages. In addition to increased compressive strength, the concrete with nano-silica showed a reduction in cement content of 15.7% (concrete with stabilized nano-silica) and 18.7% (concrete with the mixture of two pozzolans). The improvement in compressive strength when adding the nano-silica confirm the results obtained by other investigations [3,4,5].

3.3 Concrete splitting tensile strength

The results of the average concrete splitting tensile strength and their corresponded standard deviations are shown in Table 4. The results show an increase in tensile strength with increasing age for the three concretes analyzed. This increase was most evident in the
concrete produced with the combination of stabilized nano-silica and silica fume. This fact reflects the synergy between these two mineral cementitious materials. Improved splitting tensile strength with the addition of nano-silica was also obtained in previous study \[^5\].

<p>| Table 4: Average concrete splitting tensile strength and corresponded standard deviations |
|---------------------------------|---------------------------------|---------------------------------|</p>
<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Splitting tensile strength (MPa)</th>
<th>Reference concrete</th>
<th>Concrete with stabilized nano-silica</th>
<th>Concrete with stabilized nano-silica and silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>3.3 ± 0.6</td>
<td>3.3 ± 0.4</td>
<td>3.5 ± 0.2</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>3.6 ± 0.2</td>
<td>3.8 ± 0.2</td>
<td>4.8 ± 0.7</td>
<td></td>
</tr>
</tbody>
</table>

4 CONCLUSIONS
- Nano-silica dispersed in the polycarboxylate superplasticizer facilitated its use and improved its efficiency in the concrete, since the nano-silica was easily homogenized in the mix. This fact led to a higher slump without cohesion loss and material segregation.
- In relation to the mechanical properties, the results showed a substantial increase in the concrete compressive as well as splitting tensile strength demonstrating that the stabilized nano-silica improved HPC mechanical properties.
- There was a synergy when stabilized nano-silica and silica fume are combined: the corresponding compressive and splitting tensile strength of this concrete was always the highest measured one in all ages.
- In addition to the improvement of the mechanical properties, the concrete with stabilized nano-silica showed a reduction in cement consumption indicating that its use is very advantageous, especially when mixed with silica fume. Therefore, concretes with nano-silica stabilized can be a good option for sustainable development.

REFERENCES


SULPHATE RESISTANCE OF CONCRETE CONTAINING RECYCLED GRANULATED STEEL AS A PARTIAL REPLACEMENT OF FINE AGGREGATE

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Key words: Recycled Granulated Steel, Compressive Strength, Sulphate test, Scanning electron microscopy.

Abstract. The aim of this paper is to investigate the performance of concrete containing Recycled Granulated Steel (RGS) as a partial replacement of Natural Fine Aggregates (NFA) when exposed to a 5% sodium sulphate solution. RGS was introduced as a replacement to NFA at up to 30% by weight. Concrete specimens were immersed in a sodium sulphate solution to be tested in compression according to ASTM C1012 and ASTM C452. A comparison was also performed against the control specimens in terms of mechanical strength and micro-structural transformation of concrete specimens through Scanning Electron Micrograph (SEM) images. The results of this study revealed that the compressive strength of concrete containing RGS reduced by 15% compared to the control concrete when exposed to 5% sodium sulphate solution for 28 days. Moreover, RGS concrete specimens experienced maximum 4.4% volume change after sulphate exposure due to the formation of ettringite while the control specimens experienced 2% volume change.

1 INTRODUCTION

Concrete structures are susceptible to deterioration under chemical exposure. Use of recycled aggregate in concrete might aggravate the situation while exposed to an adverse environment. Hence it is of paramount importance in evaluating the long term durability properties of recycled aggregate concrete besides the basic mechanical properties of concrete. Construction of concrete structures with recycled aggregate might be a challenge in the coastal areas of Bangladesh. To investigate the proper utilization of recycled material, durability of concrete is a vital criterion as the performance of concrete significantly depends on the durability properties. Study on the usage of recycled material in concrete has acquired a high level of attention in recent years as evident from recent research [1-7]. From a thorough review on
numerous studies, it was revealed that researchers are working on improvising the production of economical and environment friendly concrete structure using various recycled aggregates such as demolished structural units, industrial waste, crushed scallop shells, ceramic waste, FRP scrap, grounded glass waste, etc [1-7]. In this succession, recycled granulated steel (RGS) in concrete was introduced by authors of the current research work. Through preliminary experimental studies, it was found that usage of recycled granulated steel (RGS) as a partial replacement of the natural fine aggregate (NFA) serves an excellent profit to refute the current demand of good quality natural fine aggregate. Recycled granulated steel (RGS) is a by-product produced in the steel re-rolling mills. Every year steel re-rolling mills in Bangladesh produce a significant amount of granulated steels which are kept unused. However, the properties of RGS are completely different from the natural aggregate and it needs to fulfill the mechanical and durability properties of concrete for its further approval on the large scale commercial use to produce green concrete. As, steel is receptive to salty environment due to the formation of corrosion, the use of recycled granulated steel in concrete construction is a challenge, especially in the coastal areas of Bangladesh. The present study addresses a gap in the literature in its investigation of the application of granulated steel as an alternative partial replacement of fine aggregates in concrete. The primary objective of this research work is to perform a detailed experimental investigation on the physical and mechanical response of control and RGS based concrete specimens after exposure to a sodium sulphate solution. Experiments were carried out to investigate the physical sulphate attack by highlighting the change in mechanical strength. Later on, microscopic analyses were conducted to investigate the formation ettringite due to intrusion of sulphate salts through the micro cracks of porous concrete specimens through SEM images.

2 MATERIALS AND METHODS

2.1 Materials

Cement: Portland Composite Cement (PCC) was used as the binding material. The cement used was fresh and without any lumps. The specific gravity was found to be 3.15. Other properties are described in Table 1.

<table>
<thead>
<tr>
<th>S.I. No.</th>
<th>Particulars</th>
<th>Experimental result</th>
<th>Standard Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Normal consistency</td>
<td>29%</td>
<td>25~30%</td>
</tr>
<tr>
<td></td>
<td>Setting Time (Minutes)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Initial</td>
<td>128</td>
<td>Not less than 45</td>
</tr>
<tr>
<td></td>
<td>Final</td>
<td>215</td>
<td>Not more than 300</td>
</tr>
</tbody>
</table>

Fine Aggregate: Locally available Sylhet sand passing through 4.75 mm sieve was used in the current study. Fineness Modulus and specific gravity were found to be 2.47 and 2.64, respectively. The gradation result for the NFA is presented in Figure 1.
**Coarse Aggregate**: Crushed stone chips with a maximum size of 19 mm were used. Fineness Modulus and specific gravity of the coarse aggregate were found to be 6.96 and 2.70, respectively. The gradation curve for the NCA is illustrated in Figure 2.
Recycled Granulated Steel (RGS): RGS was used as a partial replacement (up to 30%) of NFA on a weight basis. Fineness Modulus was determined as 3.22 and specific gravity was found to be 7.90. The gradation curve for the RGS is shown in Figure 3.

![Figure 3: Gradation curve for RGS](image)

2.2 Mix proportions

Four distinct concrete batches comprised of 24 cylinders having dimensions of 100 mm diameter and 200 mm height were prepared for this research work. Based on previous research works on concrete with recycled aggregate [1] and from practical casting work of concrete structure, the mix ratio of current program was fixed as 1:2:3 in regular weight basis of each ingredient, and an effective water-binder ratio (w/c ratio) of 0.41 was chosen as per high strength concrete’s demand. All types contained same cement, fine aggregate and coarse aggregate mix ratio where only the amount of granulated steel varied. The mix proportions for four different batches are shown in Table 2. Among the batches one is the control specimen and the other three batches held their identity as 10, 20, and 30 percent partial replacement of NFA with RGS in weight basis. After casting they were demolded within 24 hours to maintain a curing period of 28 days. After this period a set of 24 cylinders comprised of six cylinders from each batch were taken out from curing container, wiped with dry towels and then tested under compression according to ASTM C39.
Table 2: Quantity of ingredients for producing 1m³ concrete

<table>
<thead>
<tr>
<th>Batch identity</th>
<th>Cement (kg)</th>
<th>NFA (kg)</th>
<th>NCA (kg)</th>
<th>RGS (kg)</th>
<th>Water (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix-1 (Control)</td>
<td>164.3</td>
<td>328.59</td>
<td>492.9</td>
<td>0</td>
<td>67.36</td>
</tr>
<tr>
<td>Mix-2 (10% RGS)</td>
<td>164.3</td>
<td>311.79</td>
<td>492.9</td>
<td>16.8</td>
<td>67.36</td>
</tr>
<tr>
<td>Mix-3 (20% RGS)</td>
<td>164.3</td>
<td>277.15</td>
<td>492.9</td>
<td>51.44</td>
<td>67.36</td>
</tr>
<tr>
<td>Mix-4 (30% RGS)</td>
<td>164.3</td>
<td>242.50</td>
<td>492.9</td>
<td>86.09</td>
<td>67.36</td>
</tr>
</tbody>
</table>

2.3 Preparation for sulphate test

The sulphate test was performed in accordance with ASTM C1012. A sulphate bath was prepared one day before the use with 5% sodium sulphate and stored at 23±2°C. In the storage container the ratio of volume of sulphate solution to the volume of concrete cylinder was 4±0.5. After this preparation, the concrete specimens were immersed in the sulphate solution having an extension period of another 28 days to investigate the impact of sulphate environment. During this period Sulfuric Acid was added every day to maintain the pH of sulphate bath around 6.5 to 7. Finally after 56 days of total curing period the rest of the specimens were taken out from sulphate bath to examine compression test, physical degradation and microstructure properties.

3 RESULTS AND DISCUSSION

The current study primarily focused on the durability properties of concrete using RGS as a partial replacement of NFA. The effect of sulphate rich environment on the mechanical properties of different concrete mixes containing RGS and control mix were evaluated. Specimens from each mixes were also tested in Scanning Electron Microscope to observe the micro-structural changes.

3.1 Change in compressive strength

The change in compressive strength of concrete with different RGS replacement is demonstrated in Table 3. All the mixes experienced a reduction in strength except Mix-1 experienced a slight enhancement in compressive strength of 4.49%. Mix 2, 3 and 4 experienced a reduction in compressive strength which was 0.29%, 1.46% and 15.23%, respectively after the sulphate exposure. It should be noted that highest reduction change is observed for Mix-4(30% RGS). It can also be mentioned here that difference is raised from
Mix-2 to Mix-4 compared to control mix. The most significant point is that any of the RGS modified specimens have not failed to qualify to achieve the specified value of mechanical strength as per ASTM code after the sulphate exposure. This indicates the compatibility of RGS to be used as partial replacement of fine aggregate in concrete. The compressive strength of different concrete mixes before and after sulphate exposure are illustrated in Figure 4 and 5, respectively.

![Compressive Strength (Before Sulphate Test)](image1)

**Figure 4:** Compressive strength of cylindrical specimens (before sulphate exposure)

![Compressive Strength (After Sulphate Test)](image2)

**Figure 5:** Compressive strength of cylindrical specimens (after sulphate exposure)
### Table 3: Change in Mechanical Strength due to Sulphate Exposure

<table>
<thead>
<tr>
<th>Test Condition</th>
<th>Before Sulphate Exposure</th>
<th>After Sulphate Exposure</th>
<th>Remark (Increase/Decrease)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Avg. (MPa)</td>
<td>Avg. (MPa)</td>
<td></td>
</tr>
<tr>
<td>Mix-1 (Control)</td>
<td>33.721 (2.86)</td>
<td>35.235 (2.29)</td>
<td>0.0449 ↑</td>
</tr>
<tr>
<td>Mix-2 (10% RGS)</td>
<td>28.345 (2.36)</td>
<td>28.263 (2.41)</td>
<td>0.0029 ↓</td>
</tr>
<tr>
<td>Mix-3 (20% RGS)</td>
<td>28.789 (1.56)</td>
<td>28.370 (1.01)</td>
<td>0.0146 ↓</td>
</tr>
<tr>
<td>Mix-4 (30% RGS)</td>
<td>33.757 (2.29)</td>
<td>28.617 (1.77)</td>
<td>0.1523 ↓</td>
</tr>
</tbody>
</table>

*Note: Six specimens per mix, values in parentheses are Standard Deviation*

4 MICRO-STRUCTURAL ANALYSIS

In order to explain the changes in microstructure of the experimental specimens due to sulphate exposure, the concrete specimens were cut with cutting equipment from every mix into specified size required for scanning. The cut surface of hardened concrete specimens were then examined with scanning electron microscope (JEOL, Model JSM- 6490LA, and Japan). Typical micrographs with different zooming capacity are demonstrated from Figure 6 to 9. The microstructure of 28 days cured concrete specimens are illustrated in Figure 6 and 7. These images explain the morphological change in microstructure after 28 days curing period. Due to lower specific gravity of natural fine aggregate than RGS, control specimens usually possess higher void ratio than the other six modified mixes with RGS. Thus these specimens offer enough available space for the formation of ettringite in their voids due to sulphate exposure. On the contrary due to the higher density, RGS modified specimens will offer paucity in available spaces for the formation of ettringite in their voids and as a matter of consequences these specimens will experience the formation of cracking due to expansion of volume. These transformation of microstructure are showed in Figure 8 and 9. Here the images of modified specimens indicate the formation consecutive lower amount of ettringite in micro voids from 10% RGS to 30% RGS.
Figure 6: Scanning Electron Micrograph (500X) showing the formation of microstructure in concrete after 28 days curing period: a) Control, b) 10% RGS, c) 20% RGS, and d) 30% RGS

Figure 7: Scanning Electron Micrograph (1000X) showing the formation of microstructure in concrete after 28 days curing period: a) Control, b) 10% RGS, c) 20% RGS, and d) 30% RGS
Figure 8: Scanning Electron Micrograph (1000X) showing the formation of ettringite in concrete due to sulphate exposure of 28 days: a) Control, b) 10% RGS, c) 20% RGS, and d) 30% RGS

Figure 9: Scanning Electron Micrograph (5000X) showing the formation of ettringite in concrete due to sulphate exposure of 28 days: a) Control, b) 10% RGS, c) 20% RGS, and d) 30% RGS
5 CONCLUSIONS

Based on the results the following conclusions can be derived:

a) The use of Recycled Granulated Steel (RGS) as a partial replacement serves an exclusive alternative of naturally available fine aggregate. For 30% RGS replacement, the compressive strength decreased by around 15% compared to the control concrete. However, it is comparable to the target strength (i.e., 28 days concrete strength has to be 24 MPa) set by the industries in the context of Bangladesh.

b) SEM images revealed that RGS replaced specimens filled their voids due to the formation of ettringite in their microstructure after sulphate exposure.

REFERENCES

SUSTAINABILITY ASSESSMENT OF PRECAST CONCRETE SEGMENTS FOR TBM TUNNELS

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Key words: Sustainability, Precast Concrete Segments, TBM tunnels, Fibres

Abstract.
Nowadays, several tunnels bored with Tunnel Boring Machines (TBM) are already in service and various dozens are being constructed around the world. The internal support of these usually consist of precast concrete segmental rings with a low – moderate amount of concrete since these elements are mainly compressed in service conditions and low tensile stresses only appear during initial transient situations (demoulding, stocking, transportation, manipulation and thrust of the jacks). Structural fibres have proved to be an interesting solution to replace part or the total amount of the rebars. In fact, fibre reinforced concrete (FRC) has already been applied in more than fifty TBM constructed tunnels so far. However, the use of FRC is not consolidated yet in this type of tunnels due to the lack of specific design methods until the publication of the last version of the Model Code 2010 as well as the high inertia to change exhibited by the technical community.

In this scientific contribution, a multi-criteria decision-making model based on the MIVES method is proposed for assessing the sustainability index of precast concrete segments. This model is able to take into account the three main pillars of the sustainability (economic, environmental and social). By using this model, different reinforcement alternatives (rebars, fibres or the hybrid solution) can be assessed for specific boundary conditions (e.g., tunnel and segment geometry, concrete dosages, transport distances, risks during the manufacturing of the segments). Likewise, this model is designed to minimize the subjectivity of the decision and to facilitate the task of deciding which concrete reinforcement strategy is the most suitable in terms of sustainability. Finally, the model is used to assess the sustainability of different precast concrete segments (with different reinforcement solutions), which are potential alternatives to be used in a real tunnel placed in the metropolitan area of Barcelona.
1 INTRODUCTION

For economic and technical reasons, the use of fibre-reinforced concrete (FRC) to partly or entirely replace traditional passive rebar reinforcement in concrete elements has increased in classic applications, particularly as a result of the inclusion of FRC in the fib Model Code 2010 [1]. Precast concrete segments used to line TBM tunnels [2] are one of the structural elements where the use of FRC may be more interesting due to the technical and, more generally, economic advantages associated [3-4]. In terms of design, the tensile stresses generated in both transitional stages and service are usually low or, even, inexistent. Therefore, only minimal reinforcement is required to ensure adequate ductile behaviour in the event of cracking [5-7]. In such cases, the use of rebar may be reduced or entirely eliminated. The use of structural fibres is an attractive solution that can enhance concrete performance in these load states [8-10]. If the amount \((C_f)\) and type of fibre are correctly specified, it is possible to avoid spalling and to control the width of cracks that may be caused by dynamic impacts in the stages before erection and, more frequently, during the ram thrusts stage.

The objective of the present research project is to propose a multi-criteria decision making (MCDM) method based on MIVES, an Integrated Value Model for Sustainable Assessment that allows assessing different solutions for reinforcement of precast concrete segments, minimising the subjectivity in the decision making processes by using value functions [11]. MIVES was already validated in industrial buildings [12-14], underground infrastructures [15], hydraulic structures [16-17], wind towers [18], and construction projects [19-20]. Recently, the model was enhanced to include the uncertainties involved in the process of analysis [21]. The method proposed was used to analyse the sustainability of the concrete segments in the Ferrocarrils de la Generalitat (FGC) rail line extension to Terminal 1 of El Prat Airport in Barcelona. In the study, different types of concrete (conventional and self-compacting) and different reinforcement scenarios are analysed, and complete the process by presenting a sensitivity study. The resulting prioritisation of alternatives helped the technical staff in charge of the construction of the tunnel identifying the best solution.

2 METHOD FOR ASSESSING THE SUSTAINABILITY OF TUNNEL SEGMENTS

The method proposed is based on MIVES, which involves the definition of three elements: (1) the boundaries of the system, in order to establish the scope of the analysis; (2) a tree of requirements (R), criteria (C) and indicators (I) that allows decision makers to identify the important factors that must be involved in assessing the sustainability of the type of concrete and reinforcement used in the segments, and (3) the value functions used to convert the attributes or physical units associated with each indicator to unidimensional values (ranging from 0-1). These three elements were defined on seminar of experts from the public and private sectors specialised in the design and manufacture of precast lining segments. The results of the seminars were then used to define the initial requirement tree, to assign the appropriate weight to each element using the Analytic Hierarchy Process (AHP) method [22], and to provide real data from projects to define the value functions and scoring criteria for each indicator, measured in terms of attributes.

The three requirements under consideration are those that are generally associated with
sustainability: economic, environmental, and social impact. The life cycle analysis (LCA) stages considered were: (1) extraction, transportation, receiving, and in-plant processing of the materials used to fabricate tunnel linings, (2) fabrication and storage of the segments, (3) transport and installation of the segments, and (4) maintenance that may be needed to repair defects detected during the transitional stages (manufacture, transportation and installation). Based on the results of the seminars, 1 km of tunnel was considered representative of all factors involved in assessing the sustainability of the segment, without considering the infrastructure and other elements that are not crucial for the analysis (such as vertical shafts).

The requirements tree comprises 3 requirements (R), 6 criteria (C), and 9 indicators (I) (Table 1). The indicators are independent of each other to avoid overlaps in the evaluation process. Similarly, the indicators included are those considered most representative in terms of assessing the sustainability index (I) of each alternative type of segment that meets the various geometric and technical specifications, such as ring diameter and thickness, and service live and maximum loads, respectively.

Table 1: Requirements tree for the sustainability assessment of precast concrete segments for TBM tunnels.

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Criteria</th>
<th>Indicator</th>
<th>Units</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1 Economic (λR1 = 40%)</td>
<td>C1 Direct costs (λC1 = 90%)</td>
<td>I1 Total costs (λI1 = 100%)</td>
<td>ME/km</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>C2 Cost of repairs (λC2 = 10%)</td>
<td>I2 Probability of repair (λI2 = 100%)</td>
<td></td>
<td>Attributes</td>
</tr>
<tr>
<td>R2 Environmental (λR2 = 45%)</td>
<td>C3 Resources consumption (λC3 = 30%)</td>
<td>I3 Cement and aggregates (λI3 = 50%)</td>
<td>Ton/km/DCx</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I4 Water (λI4 = 20%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>I5 Reinforcing steel (λI5 = 30%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4 Emissions (λC4 = 40%)</td>
<td>I6 CO2 emissions (λI6 = 100%)</td>
<td>TonCO2-eq/km</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td>C5 Energy (λC5 = 30%)</td>
<td>I7 Embodied energy (λI7 = 100%)</td>
<td>MWh/km</td>
<td></td>
</tr>
<tr>
<td>R3 Social (λR3 = 15%)</td>
<td>C6 Labour conditions (λC6 = 100%)</td>
<td>I8 Noise pollution (λI8 = 70%)</td>
<td>Db</td>
<td>DCx</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I9 Risks during handling (λI9 = 30%)</td>
<td></td>
<td>Attributes</td>
</tr>
</tbody>
</table>

DS: decreasing S-shape; DCx: decreasing convex

Weights (λ) were assigned using the AHP method and the results were rounded to the nearest multiple of 5. These weightings allow establishing the relative importance of each element in the requirements tree (Table 1). The base scenario (E0) represents the view that economic and environmental factors are the ones that should have a greater weight (λR1 = 40%, λR2 = 45%, respectively), the latter being assigned with the greatest weighting. This approach reflects two key factors: the need to promote an environmental sensitivity, and an awareness of the impact these structures may have on future generations in terms of availability of resources and quality of life. While social aspects are taken into account, these are weighted to a lesser degree (λR3 = 15%) because it is assumed that requirements are already being met that ensure a suitable working environment and appropriate safety standards. Today, this base scenario would represent the viewpoint of an authority with a high
degree of environmental sensitivity in a developed country in the midst of a good or very good economic situation. However, it may not aptly represent certain viewpoints that might prevail in situations that differ from those described above. Therefore, a sensitivity analysis of the weightings that reflects other possible scenarios was also performed in Section 3.4.

The economic requirement (R1) is represented by two criteria: direct costs (C1) and repair costs (C2). Criterion C1 is defined by the total costs (I1), which integrates the costs associated with all the different stages of lining segment production represented in the LCA. The costs of the plant and its installation and of the amortisation of the elements associated with manufacture and handling of the segments are not considered given that they rarely are a decisive factor. The same assumption was made for the cost of the TBM. The criterion C2 is evaluated by means of the repair probability indicator (I2). This indicator qualitatively assess costs associated with the repair of any defects that might appear during any of the transitional phases taking into account the probability of such defects according to the type of reinforcement used. The assessment of the risks discussed above is conducted by defining attributes based on the experience of the seminar participants.

The three criteria in the environmental requirement (R2) are the consumption of natural resources (C3), emissions (C4) and energy consumption (C5) associated with the LCA stages. The purpose of C3 is to evaluate total consumption of materials from natural sources and identify solutions that would minimise such consumption. To this end, three indicators were defined: cement and aggregates (I3), water (I4) and steel used to reinforce the concrete (I5). The weight assigned to I5 (λR5 = 30%) is lower than the one assigned to I3 (λR3 = 50%) because the steels used to reinforce concrete, although more scarce than the aggregates used to produce the concrete, contain as much as 60% recycled steel. Criteria C4 and C5 are represented by the indicators CO2 emissions (I7) and embodied energy (I8). The value functions assigned to each indicator are intended to favour reinforcement solutions that minimise both CO2 emissions and embodied energy and are respectful of the environment and energy sources. The following items were included in the LCA to quantify both indicators: (1) extraction of materials, (2) treatment of materials, (3) segment fabrication, and (4) segment transport. The consumption associated with the installation, operation and maintenance of the TBM is not considered in the analysis since it does not discriminate between the different types of concrete or reinforcement used in the precast segments. Similarly, the consumption associated with the repair of segments damaged in the transitional phases was not taken into account either.

Finally, in social requirement (R3), the criterion labour conditions (C6) was evaluated by way of two indicators. The indicator noise pollution (I8) varies according to the type of concrete used. For instance, solutions relying on the use of self-compacting rather than traditional concrete are associated with significantly lower noise levels in the work environment because the traditional method requires strong vibration energy to ensure compaction of the material. The risks during handling (I9) of the segments, particularly the risk to workers of cuts and lesions when fibres on the surface of the segment protrude and are liable to cause injury. While surface polishing and inspection are always carried out, the risk increases with the C7 and when metal fibres are used since these are sharper and more rigid
than plastic fibres. In the absence of more precise criteria and statistical data, this indicator was evaluated on the basis of attributes that were ranked in the seminars by the technicians with experience in plants producing precast segmental linings and by others with experience in TBM operation, two situations in which the workers may have to handle or touch the segments and are thus exposed to the risks described above.

To evaluate the sustainability index ($I_s$) of each alternative solution, value functions assigned using the method previously proposed [11-14, 23] were used. The generic form of a value function is represented by eq. 1, which allows assessing the sustainability (satisfaction) associated with each indicator ($I_{ind}$) by transforming the physical units to a dimensionless value between 0.0 and 1.0.

$$I_{ind}(X) = A + B \left[ 1 - e^{-K_i \left( \frac{|X_{ind} - X_{min}|}{C_i} \right)^P_i} \right]$$

$$B = \left[ 1 - e^{-K_i \left( \frac{X_{max} - X_{min}}{C_i} \right)^P_i} \right]^{-1}$$

In eq. 1, B is the value of $I_{ind}$ for $X_{min}$; $X_{min}$ is the minimum abscissa value in the indicator interval assessed; X is the abscissa value for the indicator assessed; $P_i$ is a shape factor which defines whether the curve is concave ($P_i<1$), convex ($P_i>1$), linear ($P_i=1$) or S-shaped ($P_i>1$), see fig. 4; $C_i$ approximates the abscissa at the inflexion point; $K_i$ tends towards $I_{ind}$ at the inflexion point; B, the factor that prevents the function from exceeding the range (0, 1), is obtained by eq. 2, $X_{max}$ being the abscissa value of the indicator that gives a response value of 1 for increasing value functions. The form of the value functions assigned to each indicator (see Table 1) is a decreasing S-shape curve (DS) for $I_1$ and $I_6$-$I_7$ and a decreasing convex curve (DCx) for $I_3$-$I_5$ and $I_8$.

3 CASE STUDY: FGC EXTENSION TUNNEL TO TERMINAL 1 AT BARCELONA AIRPORT

3.1 Description of the case study

The project to connect the Prat de Llobregat FGC station with Barcelona Airport [24] includes a 2.84 km long tunnel run bored using a TBM 10.60 m in diameter. The design calls for a tunnel lining (Fig. 1a) comprising a universal ring with a mean length of 1.60 m and an internal diameter of 9.60 m. The ring is 0.32 m thick and is composed of 6 segments and 1 key. The initial project proposes concrete segments reinforced with B500SD steel bars ($f_{yk} = 500$ N/mm$^2$) and concrete with a characteristic compressive strength value $f_{ck}$ of 45 N/mm$^2$.

This value of strength ensures sufficient strength to bear the flexural compression that occurs in the service phase and subjected to the soil pressure. The designers also verified that the design forces do not exceed the crack resistance of the segment in any of the loading stages and fixed a minimum reinforcement of 13Φ12 mm on each side (Fig. 1b) to ensure adequate ductile behaviour in a hypothetical rupture situation. The concrete cover ($c$) must be
greater than 4 cm to protect the reinforcement from possible chemical attack. Notice that the layout of the tunnel passes under industrial areas where aggressive groundwater may be present.

Figure 1: (a) Ring configuration and (b) frontal section view and (c) top view of reinforcement cage.

Therefore, the original proposal specified the use of conventional reinforced concrete (CRC), however due to the economic recession affecting Spain in the past years and the publication of the new Spanish Structural Code EHE-08 [25], two new solutions for the segments using only FRC have been proposed: (1) using conventionally vibrated FRC concrete and (2) using self-compacting fibre-reinforced concrete (SC-FRC).

Table 2 shows the dosages used in the fabrication of the different types of concretes considered for the production of the segments. Two aspects of this process are of particular interest: (1) the same granular skeleton was used for the CRC and the FRC (the loss of workability is compensated with the vibration time); (2) the fine fraction (cement, sand 0/5, and fine aggregate 5/12) used in the SC-FRC is 36% greater than that of the CRC and FRC in order to guarantee the self-compactability of the SC-FRC. For the same reason, the content of superplasticiser in the SC-FRC was 50% higher than in the CRC and FRC.

Table 2: Dosages (in kg/m³) considered for the different concrete mixes.

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>CRC</th>
<th>FRC</th>
<th>SC-FRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I 52.5</td>
<td>315</td>
<td>315</td>
<td>381</td>
</tr>
<tr>
<td>Sand 0/5</td>
<td>817</td>
<td>817</td>
<td>1,200</td>
</tr>
<tr>
<td>Fine aggregate 5/12</td>
<td>404</td>
<td>404</td>
<td>500</td>
</tr>
<tr>
<td>Coarse aggregate 12/20</td>
<td>810</td>
<td>810</td>
<td>200</td>
</tr>
<tr>
<td>Water</td>
<td>150</td>
<td>156</td>
<td>165</td>
</tr>
<tr>
<td>Superplasticiser (% c)</td>
<td>2.80 (0.9)</td>
<td>2.80 (0.9)</td>
<td>4.60 (1.2)</td>
</tr>
<tr>
<td>Steel fibres</td>
<td>0</td>
<td>45</td>
<td>50</td>
</tr>
</tbody>
</table>

The reinforcement was different in each case, for the CRC segments (CRCS), the total amount of steel bar used was 110 kg/m³ (Fig. 1b), whereas the analysis conducted for the
FRC and SC-FRC showed that only 50 kg/m$^3$ was required for FRC and 45 kg/m$^3$ for SC-FRC to achieve strength class 4.0d. SC-FRC requires 10% less fibre material than CRC because of the better orientation of the fibres in the pouring process of the self-compacting concrete due the flow forces and boundary conditions imposed by the walls of the mould. This analysis was performed using the numerical model Analysis of Evolutionary Sections (AES) [26], considering the same design values for axial forces (N$_d$) and bending moments (M$_d$) of the original project and considering the constitutive equation of the Model Code 2010. The fibre used was MasterFiber 502 with hooked-end anchors and a length of 50 ± 5 mm, a diameter of 1.0 ± 0.1 mm, and an elastic limit of 1000 N/ mm$^2$.

In terms of workability, slump values of less than 5 cm for CRC and FRC with the Abrams cone test were obtained, being this fact an indication of very dry concrete; however, the vibration energy generated in the segment moulds is sufficient to adequately compact these concrete mixes. Likewise, values of 65 – 68 cm obtained in the slump flow testing for the SC-FRC confirmed the sufficient flowability to facilitate the self-compacting process. Finally, the mean compressive strength ($f_{cm}$) values obtained at 1, 7 and 28 days with cylinder specimens 300×Φ150 mm$^2$ were very similar for the different concrete mixes, reaching values of $f_{cm,1} = 20.2$ N/mm$^2$, $f_{cm,7} = 53.0$ N/mm$^2$ and of $f_{cm,28} = 64.5$ N/mm$^2$. These results reflect that the specified $f_{ck}$ of 45 N/mm$^2$ is reached.

### 3.2 Evaluation of indicators

The construction of the tunnel lining involves 28,322 m$^3$ of concrete. The manufacturing of the 12,425 segments will take place in an existing plant at 110 km from the TBM access shaft and will require 9 months with two 8-hour work shifts a day. The plant is expected to be in operation for a period of 16 months between the start of preparations and final shutdown. Subsequently, the information required and the criteria assumed to evaluate the indicators is presented. The results of the evaluation are presented in Table 3.

<table>
<thead>
<tr>
<th>Indicator</th>
<th>CRCS</th>
<th>FRCS</th>
<th>SC-FRCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_1$: Direct costs (M€/km)</td>
<td>2.89</td>
<td>2.60</td>
<td>2.61</td>
</tr>
<tr>
<td>$I_2$: Probability of repair</td>
<td>Moderate</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>$I_3$: Cement and aggregates (Ton/km)</td>
<td>66,444</td>
<td>66,444</td>
<td>64,603</td>
</tr>
<tr>
<td>$I_4$: Water (Ton/km)</td>
<td>15,590</td>
<td>10,863</td>
<td>11,668</td>
</tr>
<tr>
<td>$I_5$: Reinforcing steel (Ton/km)</td>
<td>1,097</td>
<td>499</td>
<td>449</td>
</tr>
<tr>
<td>$I_6$: CO$_2$ emissions (TonCO$_2$-eq/km)</td>
<td>5,305</td>
<td>4,601</td>
<td>5,083</td>
</tr>
<tr>
<td>$I_7$: Embodied energy (MWh/km)</td>
<td>12,411</td>
<td>9,375</td>
<td>9,904</td>
</tr>
<tr>
<td>$I_8$: Noise pollution (Db)</td>
<td>90</td>
<td>90</td>
<td>60</td>
</tr>
<tr>
<td>$I_9$: Risk during handling</td>
<td>Reduced</td>
<td>High</td>
<td>High</td>
</tr>
</tbody>
</table>

In the indicator $I_1$, which corresponds to the direct costs, the following aspects are assessed: (1) investment in the plant, (2) materials for the manufacture of the segmental linings, (3) personnel involved, (4) auxiliary equipment, (5) consumables, and (6) transport. The aspects that represented a difference in cost depending on the solution used to fabricate the segments are the following: (1) the cost of materials for the manufacture of concrete
(Table 2), (2) the reinforcement solution used (bars or fibres), and (3) the personnel required for the preparation of the reinforcement (2 per shift for CRCS and none for FRCS and SC-FRCS). The evaluation of indicators I3 and I5 is based on the consumption of materials shown in Table 2, taking into account that the CRCS use 110 kg/m³ of steel bars. The estimate for indicator I4 is calculated by applying the values in Table 2 for the concrete manufacture and using the values for water consumption associated with the production of steel bars and fibres from [27].

The emissions of CO₂-eq (I6) and energy (I7) involved in the LCA processes of the materials used in the concrete were calculated using the mean values listed in the Inventory of Carbon Energy version 2.0 [28]. The estimation of indicator I6 for bars and steel fibres is based on [28-29] Finally, indicators I2, I8-I9 were evaluated in the seminars, taking into account the following: (1) the information contained in [29] to establish the probability of a segment needing repairs depending on the type of reinforcement (I2); (2) the information on workplace noise pollution in precast plants and the health risks described in [20] to evaluate I8; and (3) the arguments set out in [30] to define indicator I9 and specify how it should be assessed.

Table 3 reveals that the use of FRCS (2.60 M€/km) represents a cost saving of 10.0% over CRCS (2.89 M€/km) and 0.4% over SC-FRCS (2.61 M€/km). These differences arise from differences in: (1) materials costs (concrete and steel) of 159.4, 135.9 and 136.5 €/m³ for the concrete used in CRCS, FRCS and SC-FRCS, respectively; and (2) the manufacturing costs associated with the fabrication of the segments, estimated at 67.2€/m³ for CRCS and 62.1 €/m³ for FRCS and SC-FRCS (a 7.6% reduction compared to CRCS because of labour associated with the use of steel bar reinforcement). The use of SC-FRCS (64,603 Ton/km) also results in a saving of 2.8% in the consumption of cement and aggregates in the concrete as compared to the CRCS and FRCS solutions, with 66,444 Ton/km for both.

The water consumption required in the manufacture of FRCS (10,863 Ton/km) (associated with the manufacture of the steel and the concrete) is some 30.3% lower than in the CRCS segments (15,590 Ton/km), and 6.9% lower than in the SC-FRCS solution. Finally, the manufacture of SC-FRCS (449 Ton/km) represents steel savings of 59.1% over the CRCS solution (1,097 Ton/m3) and of 10.0% compared with FRCS. FRCS (4,601 TnCO₂eq) produces 13.3% and 9.5% lower emissions compared to CRCS and SC-FRCS, respectively, due to the lower consumption of cement and steel in the reinforcement. Furthermore, FRCS (9,375 MWh/km) is the solution that requires the least energy throughout the entire LCA, some 24.5% lower than CRCS and 5.3% lower than SC-FRCS.

3.4 Sustainability indices Iₙ for each alternative

The constitutive parameters for each value function (see Table 4) were agreed during the seminars, drawing on the experience of the experts complemented by criteria presented in the literature on MIVES and the values of Xᵢ obtained for the three alternatives studied (Table 3). Subsequently, the sustainability indices Iₙ for each segment solution were calculated (Table 5) for the base scenario E₀ (λˣ₁ = 40%, λˣ₂ = 45% λˣ₃ = 15%).
Table 4: Constitutive parameters for defining the value functions.

<table>
<thead>
<tr>
<th>Indicator</th>
<th>$X_{\text{max}}$</th>
<th>$X_{\text{min}}$</th>
<th>$C$</th>
<th>$K$</th>
<th>$P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I1 Direct costs (M€/km)</td>
<td>4.00</td>
<td>2.24</td>
<td>1.00</td>
<td>1.00</td>
<td>2.50</td>
</tr>
<tr>
<td>I2 Probability of repair</td>
<td>Steel: 0.00 – 0.25 (very high); low fibre content: 0.25 – 0.50 (high); steel + low fibre content: 0.50 - 0.75 (moderate); High fibre content: 075 - 1.00 (low)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I3 Cement and aggregates (Ton/km)</td>
<td>70,000</td>
<td>65,000</td>
<td>67,000</td>
<td>0.10</td>
<td>2.50</td>
</tr>
<tr>
<td>I4 Water (Ton/km)</td>
<td>29,000</td>
<td>7,500</td>
<td>15,000</td>
<td>0.10</td>
<td>2.50</td>
</tr>
<tr>
<td>I5 Reinforcing steel (Ton/km)</td>
<td>1,350</td>
<td>450</td>
<td>800</td>
<td>1.00</td>
<td>2.50</td>
</tr>
<tr>
<td>I6 CO2 emissions (TonCO2-eq/km)</td>
<td>7,800</td>
<td>3,800</td>
<td>5,000</td>
<td>2.50</td>
<td>200</td>
</tr>
<tr>
<td>I7 Embodied energy (MWh/km)</td>
<td>18,500</td>
<td>7,500</td>
<td>10,000</td>
<td>2.50</td>
<td>2.00</td>
</tr>
<tr>
<td>I8 Noise pollution (Db)</td>
<td>150</td>
<td>0</td>
<td>80</td>
<td>3.00</td>
<td>10.00</td>
</tr>
<tr>
<td>I9 Risks during handling</td>
<td>Very high: 0.00 – 0.25; High: 0.25 – 0.50; Acceptable: 0.50 – 0.75; Reduced: 0.75 – 1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5: Values of $I_s$ and $I_R$ obtained for each alternative.

<table>
<thead>
<tr>
<th>CRCS</th>
<th>FRCS</th>
<th>SC-FRCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_s$</td>
<td>0.578</td>
<td>0.754</td>
</tr>
<tr>
<td>$I_{R1}$</td>
<td>0.703</td>
<td>0.899</td>
</tr>
<tr>
<td>$I_{R2}$</td>
<td>0.513</td>
<td>0.786</td>
</tr>
<tr>
<td>$I_{R3}$</td>
<td>0.438</td>
<td>0.326</td>
</tr>
</tbody>
</table>

The results presented in Table 5 show that the solutions that use structural fibres as an alternative to steel bars result in a higher $I_s$ value. Specifically, SC-FRCS (0.856) represents an increase of 48% in $I_s$ over CRCS (0.578) and an increase of 14% over FRCS. The better performance in terms of sustainability of the SC-FRCS solution is a result of two factors: the use of fibres rather than steel bars, a choice that reduces both overall costs and environmental impact; and the use of self-compacting concrete, which leads to a better distribution of the fibres and better mechanical performance than can be achieved with traditional FRC. To analyse the sensitivity of the results obtained (Table 5), three additional scenarios were considered as follows:

- E1 ($\lambda_{R1} = 33\%$, $\lambda_{R2} = 33\%$, $\lambda_{R3} = 33\%$) simulates a reasonable view of all the requirements involved in the analysis and represents sustainability in the strict sense.
- E2 ($\lambda_{R1} = 75\%$, $\lambda_{R2} = 10\%$, $\lambda_{R3} = 15\%$) assigns greater weight to the economic requirement $I_{R1}$ in order to consider a more entrepreneurial view or take into account a possible financial recession on the part of the authority making the investment. This scenario must be considered although it is not desirable in terms of sustainability.
- E3 ($\lambda_{R1} = 25\%$, $\lambda_{R2} = 60\%$, $\lambda_{R3} = 15\%$) gives particular weight to the environmental requirement $I_{R2}$ in order to prioritise solutions respectful of the environment based on use of available resources and respectful of society today and in the future. This scenario represents the vision of a public authority with a high environmental sensitivity and, in general terms, a situation of economic growth.

To facilitate the analysis and interpretation of the results, in these three scenarios the same weight values for the criteria ($\lambda_C$) and indicators ($\lambda_i$) as those used in scenario E0 (Table 1) were maintained. The constitutive parameters of the value functions are also maintained.
(Table 4). The resulting values of $I_R$ considered in the sensitivity analysis are shown in Table 5. It should be noted that in a more rigorous analysis of sensitivity or cases in which the ranges of values for $I_S$ and $I_R$ of the alternatives are more tight, the use of statistical techniques are recommended to ensure robust results. Table 6 shows the values of $I_S$ for each of the scenarios. The results shown in Table 6 reveal the following:

<table>
<thead>
<tr>
<th>$E_o$</th>
<th>$I_S$</th>
<th>$E_1$</th>
<th>$I_S$</th>
<th>$E_2$</th>
<th>$I_S$</th>
<th>$E_3$</th>
<th>$I_S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRCS</td>
<td>0.578</td>
<td>FRCS</td>
<td>0.762</td>
<td>SC-FRCS</td>
<td>0.856</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_1$</td>
<td>0.546</td>
<td>$E_2$</td>
<td>0.645</td>
<td>0.801</td>
<td>0.882</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_2$</td>
<td>0.550</td>
<td>$E_3$</td>
<td>0.745</td>
<td>0.845</td>
<td>0.845</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All the alternatives present the highest value for $I_S$ in the scenario that gives greatest weight to the economic requirement ($E_2$), showing that all the solutions studied should generate a high level of satisfaction in economic terms. Nevertheless, all the solutions present values of $I_S$ under 0.900, demonstrating that there is still room for improvement in the fabrication of FRC segments. SC-FRCS is the solution that presents high values for $I_S$ in all the scenarios. The ranking of alternatives is the same as that obtained for scenario $E_0$. Comparison of FRCS and CRCS shows that the total replacement of rebar with structural fibres in vibrated concrete yields values of $I_S$ between 21% ($E_1$) and 36% ($E_3$). The comparison of SC-FRCS and FRCS reveals that the use of self-compacting concrete gives rise to an increase in values of $I_S$ of between 10% ($E_2$) and 25% ($E_1$).

4 CONCLUSIONS

This paper proposes a method for assessing the sustainability of precast concrete lining segments for tunnels excavated using a TBM, taking into account economic, environmental and social factors based on MIVES. The model allows comparing and prioritising alternative solutions, minimising subjectivity in the decision process. The method has been used to assess the sustainability of three types of segments (the traditional reinforced concrete solution proposed in the preliminary plan and two alternative solutions involving total replacement of the rebar with fibres) to line a 2.84 km tunnel in Barcelona. The conclusions drawn from the sustainability indices $I_S$ obtained with the model are as follows:

The substitution of the rebar with structural fibres yields higher values of $I_S$ in all the scenarios under study. This conclusion can be extended to cases in which the risk of cracking is low in both the transitional stages (reduced tensile flexural stresses) and the service phase (ring compressed by soil pressure) and the amounts of reinforcement used are the minimum values required to ensure the ductile behaviour of the segment in case of cracking. The use of SC-FRC yields increases in the $I_S$ over FRC for the following reason: although the cost of the SC-FRC concrete mix is some 15% higher than that of FRC, the greater spatial efficiency of the fibre distribution in the case of SC-FRC reduces by 10% the quantity of fibres required to achieve mechanical characteristics equivalent to those of FRC. Likewise, the use of SC-FRC increases the useful life of the moulds and reduces noise pollution in the precast plant, aspects that can be quantified and integrated into the proposed model. The results presented were
accepted by the manufacturer of the segments and will serve as a tool for decision making on similar projects in the future.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the Spanish Ministry of Science and Innovation (MICINN) for the financial support received in the framework of the FIBHAC project (reference: IPT-2011-1613-420000). They would also like to thank the Fundació Bosch Potensa for their support.

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THE EFFECT OF PARTICLE SIZE DISTRIBUTION ON EARLY AGE CHEMICAL SHRINKAGE OF CEMENT PASTES

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Key words: chemical shrinkage; particle size distribution; Rietveld method; hydration process.

Abstract. The purpose of this research is to determine the effect of particle size distribution on chemical shrinkage at early ages. The term early ages corresponds to hydration times of maximum 24 hours. In this work, two different types of commercial cement were used (Type I cement, and white cement with limestone filler). Three particle size distributions (PSD) were used for each cement type. To achieve the three PSD a sieving process was performed, separating and using the particles retained in the 325 (45µm) sieve from those passing de 400 (38 µm) sieve. Thus, the three PSD used for each cement were the as-received condition, the coarser condition (particles retained in the 325 sieve) and finer condition (particles passing the 400 sieve). Cement pastes were prepared according to ASTM C305 standard. The water to cement (w/c) rate was constant for each cement type, and was determined as the w/c rate for normal consistency for the as-received condition. The chemical shrinkage was evaluated according to ASTM C1608. In order to determine the phases and their quantities in the different cements, the XRD technique was used and the Rietveld method was applied to the anhydrous cements. The chemical shrinkage test was stopped at 1, 6, 12 and 24 hours of hydration in order to determine the effect of the initial phases on the early chemical shrinkage.

The hydration process was analyzed at these four times. In most of the experiments, the hydration kinetic increased as the particle size distribution was finer. As the hydration kinetic increased, the chemical shrinkage was greater during the total test time. From the XRD and Rietveld analyses it can be concluded that the increase in the hydration kinetic is not only due to the finer PSD, but also from the change in the quantity of the anhydrous phases during the sieving process. There is a synergic effect on the hydration process and on the early age chemical shrinkage due to PSD and mineralogical composition.

Several previous works have established the effect of water to cement rate, aggregate incorporation, use of superplasticizer and the effect of mineralogical composition among other variables on early age chemical shrinkage. The contribution of this work resides on the evaluation of the effect of particle size distribution on the chemical shrinkage and hydration process at early ages. The experiments also show an effect of the sieving process on the initial mineralogical composition of anhydrous cements. Therefore this research turned into a synergic effect of particle size distribution and anhydrous phases on early age chemical shrinkage.
1 INTRODUCTION

Concrete structures are prone to crack, which means that the material loses physical and chemical resistance [1,2]. The consequence of this problem is a decrease of structure lifetime [3]. Some concrete cracks can be of non-structural type [4,5]. This type of cracks are due to volumetric changes of cement paste (expansion, shrinkage), movements at the interior of the material and effects of the environment [6,7].

Volume changes of cement paste can be presented at early or late ages. Volume changes caused during the first 24 hours of material curing are called early age volume changes. These changes include the fluid stage of cement, the transition period (skeleton formation) and initial hardening [8]. Volume changes produced after hardening of the material correspond to late age [8].

At early ages, right after mixing cement and water, cement paste has the lowest capacity to support loads, but has the greatest possibility to generate internal stresses, which are the origin of cracks [9-11]. Cracks formed during this early age or as a consequence of volume changes are internal and microscopic, but they can open and grow, thus generating big problems with time [5].

Early cracks are usually associated with autogenous deformations. Autogenous shrinkage is a macroscopic volume change occurring without moisture transfer to environmental surroundings [10,12-14]. Through further studies of autogenous shrinkage, Pichler et al. [15] completed the concept by realizing that the material should not be subjected to external forces. Autogenous deformation is the result of chemical shrinkage and capillary pressure in the material [13,19].

Chemical shrinkage is associated to the hydration process of cement particles. It is the result of chemical reactions between water and cement, producing a volume reduction [13]. In the fluid stage of the material, before skeleton formation and hardening process, autogenous shrinkage is equivalent to chemical shrinkage [8,16]. This deformation begins when water starts to mix with cement [17]. During skeleton formation, chemical shrinkage slows down and autogenous shrinkage is less composed by chemical reactions. At this stage, volume reduction is more affected by “self-desiccation” or capillary pressure mechanism [9,18]. Self-desiccation is a localized drying in internal porous of cement [8,13-15].

Early age chemical shrinkage of cementitious material is the result of complex physicochemical phenomena. These phenomena are related to hydration reactions between cement and water and the progressive hardening of mineral skeleton [20]. As cement hydrates, hydration product volume is less than the one of the original materials, including mixing water. The chemical shrinkage brings important performance implications at early ages of materials related to cement [10,13,21]. Hammer [22] and Justnes et. al [23] reported that autogenous shrinkage separates from chemical shrinkage, once the cement paste setting starts or once the skeleton starts to form.

Chemical shrinkage is also known as Le Chatelier contraction and it can be calculated using the equation (1) [24,25]

\[ V_h = V_{wc} + V_{ch} - \Delta V \]  

\( \Delta V: \) Le Chatelier contraction
This work focuses on the effect of particle size distribution on chemical shrinkage during the first 24 hours after mixing cement and water of two different Portland cement.

This paper is structured as follows: section 2 presents the experimental procedure followed. The results are presented and discussed in section 3, followed by the conclusions.

2 EXPERIMENTAL PROCEDURE

Two different types of commercial Portland cement were used in the development of this work. The chemical composition was obtained using X-ray fluorescence (XRF) in a PANalytical AXIOS spectrometer with a rhodium X-ray tube. XRF results are presented in Table 1. The cements are named CEM 1 and CEM 2.

| Table 1. Chemical composition of cements obtained by XRF |
|-----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
|                | LOI   | SiO₂ | TiO₂ | Al₂O₃ | FeO | MnO | MgO | CaO | Na₂O | K₂O | P₂O₅ | SO₃ | V₂O₅ | SrO | BaO |
| CEM 1          | 8,49  | 18,57| 0,097| 3,28  | 0,342| 0,06 | 3,28 | 0,342| 0,06 | 3,28 | 0,342| 0,06 | 3,28 | 0,342| 0,06 | 3,28 |
| CEM 2          | 2,13  | 20,29| 0,331| 4,27  | 3,441| 0,038| 2,01 | 64,12| 0,16 | 3,83 | 0,06 | 3,78 | 0,068| 2,67 | 0,012| 0,038|

The cements were sieved from their original state using 200, 325 and 400 sieves. The material retained in the 325 sieve and the one passing through the 400 sieve were selected to study the effect of the particle size distribution (PSD) on the cement hydration. The 400 passing material is named as Fine, the 325 retained is known as Ground and the original state is called Normal. Figure 1 shows the PSD for both cements. PSD for each cement was obtained using a Mastersizer 2000 of Malvern Instruments.

![Figure 1. PSD of the three cements in the "as received" conditions](image)

Pastes were prepared according to ASTM C305 standard using the different granulometries. The parameters used in this study are presented in Table 2. The w/c ratio for normal consistency was determined for each cement in its original state and this ratio was used for three different particle size distributions for both cements.
XRD analyses were performed in a PANalytical X’Pert PRO MPD, with a 6—70 (2θ) range, 0.013 step and accumulation time of 59 s.

The quantification of the phases of anhydrous cements was realized using the Rietveld method and the FullProf software.

Calorimetric tests were developed to study the evolution of cement hydration during the first day. A TAM Air calorimeter was used in this study.

The chemical shrinkage was evaluated according to ASTM C1608. The chemical shrinkage test was stopped at 1, 6, 12 and 24 hours of hydration to determine the effect of the initial phases on the early chemical shrinkage.

### Table 2. Test parameters

<table>
<thead>
<tr>
<th>Cement</th>
<th>w/c</th>
<th>Granulometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM 1</td>
<td>0.305</td>
<td>Normal (N), Fine (F), Ground (G)</td>
</tr>
<tr>
<td>CEM 2</td>
<td>0.31</td>
<td>Normal (N), Fine (F), Ground (G)</td>
</tr>
</tbody>
</table>

### 3 RESULTS AND DISCUSSION

#### 3.1 XRD and mineralogical quantification

Figure 2 shows the XRD results of both cement types. Figure 2 (a) shows the XRD pattern of three different granulometries of CEM 1. CaCO₃ was obtained in the different granulometries. As expected, there was no C₄AF in this cement type.

XRD results of CEM 2 are presented in Figure 2 (b). In this type of cement C₄AF was obtained, as expected from XRF analyses.

Table 3 presents the mineralogical quantification of CEM 1, which is a commercial white cement. As expected from the XRF, the content of CaCO₃ is high, turning CEM1 into white cement with CaCO₃ filler. The values of Rwp/Re indicate an adequate quantification of the phases, especially considering the number of phases for this cement type.

From Table 3, a similar mineralogical composition of normal and fine cements can be noticed. The main difference between them will be the finer particle size of the fine cement,
comparing to the normal one. A great amount of Gypsum can be found for normal and fine cements and a lower quantity of C$_3$A than expected. The ground cement has a high quantity of C$_3$A, 4 times greater than the present in the other two PSD cements. There is little Gypsum on ground white cement and almost the third part of CaCO$_3$ compared to other granulometries.

<table>
<thead>
<tr>
<th>Table 3. Quantification by Rietveld Method of CEM 1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fine (%)</strong></td>
</tr>
<tr>
<td>C$_3$S</td>
</tr>
<tr>
<td>C$_2$S</td>
</tr>
<tr>
<td>C$_3$A</td>
</tr>
<tr>
<td>Gypsum</td>
</tr>
<tr>
<td>CaCO$_3$</td>
</tr>
<tr>
<td>$R_{wp}/R_e$</td>
</tr>
</tbody>
</table>

Table 4 presents the mineralogical quantification of CEM 2, which is a gray commercial cement. The $R_{wp}/R_e$ ratio is also adequate and the mineralogical quantification can be accepted. A higher C$_3$A contents for normal CEM 2 than for normal CEM 1 is noticed, which can be explained from the higher contents of Al$_2$O$_3$ in normal CEM 2 than in normal CEM 1 obtained by XRF. In this case, the normal cement has higher contents of C$_3$A and C$_3$S than the fine and the ground cement. Fine cement has the highest contents of C$_4$AF, while ground cement has the highest contents of Gypsum. The Gypsum contents of fine and normal cement is lower than expected for this cement type.

From both cements, a change of mineralogical composition can be observed when a sieving process is performed. The main differences are found on C$_3$A, C$_3$S and Gypsum contents, varying among granulometries. These three phases have a great influence on hydration kinetics, especially during the first 24 hours after mixing the cement with water.

It is important to realize that CEM 2 has higher contents of C$_3$S and C$_3$A for its different granulometries than CEM 1, except for ground CEM 1, which has an unusual amount of C$_3$A (20,50 %).

<table>
<thead>
<tr>
<th>Table 4. Quantification by Rietveld Method of CEM 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fine (%)</strong></td>
</tr>
<tr>
<td>C$_3$S</td>
</tr>
<tr>
<td>C$_2$S</td>
</tr>
<tr>
<td>C$_3$A</td>
</tr>
<tr>
<td>C$_4$AF</td>
</tr>
<tr>
<td>CaCO$_3$</td>
</tr>
<tr>
<td>Gypsum</td>
</tr>
<tr>
<td>$R_{wp}/R_e$</td>
</tr>
</tbody>
</table>
3.2 Calorimetric tests

The results of the calorimetric tests are presented in Figure 3. The first peak appears between the beginning of hydration and 6000 s (1.7 hours of hydration). The second peak for all cases is analyzed between 1.7 and 24 hours (86000 s) of hydration.

Figure 3(a) represents the first peak for CEM 1 in its different granulometries. It can be seen from this graphic that the highest heat flow corresponds to the fine granulometry. The calculated area underneath this curve has a value of 113 u^2 (u=units), greater than the area underneath the curve of ground CEM 1 (93 u^2) and the normal CEM 1 (92 u^2). The highest hydration kinetics of the fine cement can be explained from the finest particle size of this material. When the particles are smaller, the hydration rate can increase, if there is no water in excess.

The area under the first peak of ground CEM 1 is larger than usual, and this can be due to the high contents of C_3A of the former granulometry. The hydration of C_3A is the main cause for the first peak in calorimetric tests.

Figure 3 (c) shows the second calorimetric peak for the three different PSDs of CEM 1. It can be seen that the greatest peak corresponds to normal granulometry, while the smallest peak represents the fine granulometry. The explanation of this phenomenon can be the particle size of each cement. As fine CEM 1 has the smallest particle size, the hydration can lower down after an initial time because of an excess of water. In this case, the water acts as a refrigerant decreasing the hydration rate. The second largest peak of normal CEM 1 compared to ground CEM 1 can be due the difference in particle size. Ground CEM 1 has the greatest contents of C_3S and C_3A, which are responsible for hydration at early ages. The high contents of these two mineralogical phases may contribute to the hydration, despite the coarser grain.

Figure 3 (b) represents first peak of calorimetric of CEM 2 for all PSD studied until the first 1.7 hours of hydration, while Figure 3 (d) shows the calorimetric second peak of these cements. It can be seen from both peaks that the cement with normal PSD has the greatest heat flow for the first 24 hours of hydration. As shown in Table 4, normal PSD has the greatest amount of C_3A and C_3S, which is in agreement with the greatest heat flow of this cement. Fine PSD of CEM 2 has a lower flow heat, for both peaks, than the normal condition, but greater than the coarser one. Fine and ground CEM 2 have similar contents of C_3A and C_3S, but the fine condition has higher contents of C_4AF, which influences also the early age hydration, according to [26, 27]. Besides having less contents of C_4AF, Ground CEM 2 has coarse grains, decreasing hydration kinetics.

Table 5 shows the Area of the calorimetric peaks for both cements. It can be noticed that peak 1 of CEM 1 has higher heat flow than the first peak of CEM 2 in all cases. For the second peak, CEM 2 has higher heat flow than CEM 1 for fine and normal conditions. The second peak of the ground condition of CEM 1 has a higher area than the ground condition of CEM 2. This phenomenon can be due to the unusually high contents of C_3A in ground CEM 1.

The higher area of peak 1 of CEM 1 can be attributed to the high contents of gypsum and the calcite present in this cement. The type of gypsum found in these samples was hemihydrate, which hydrates in presence of water forming gypsum during the first minutes of hydration. This is also an exothermic reaction, which increases the flow heat at the beginning of the hydration process, as explained by Taylor [28].
Due to its softness, calcite is easy to grind, turning into fine particles, finer than other phases [28]. This explains the greater amount of calcite in normal and fine CEM1. Calcite has chemical and physical effects on cement. With calcite, the hydration process of Celite and Alite is accelerated [28]. Calcite reacts with C₃A, producing C₄ÅCH₁₁, which competes with Gypsum.

The second peak is usually larger in CEM 2. This peak is governed by Alite contents and, as it can be seen from Table 3 and Table 4, Alite is always higher in CEM 2 than CEM 1. Table 5 also shows that the area underneath the second peak of ground CEM 1 is larger than that of ground CEM 2. Mineralogical compositions of these two cements are similar (Alite and Calcite content). The larger area of the second peak of ground CEM 1 might be due to its particle size and the w/c ratio.

![Calorimetric tests](image)

Figure 3. Calorimetric tests. (a) first peak of CEM 1; (b) first peak of CEM 2; (c) second peak of CEM 1; (d) second peak of CEM 2

<table>
<thead>
<tr>
<th>Peak</th>
<th>Fine</th>
<th>Normal</th>
<th>Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-CEM 1</td>
<td>113</td>
<td>92</td>
<td>93</td>
</tr>
<tr>
<td>2-CEM 1</td>
<td>729</td>
<td>1000</td>
<td>920</td>
</tr>
<tr>
<td>1-CEM 2</td>
<td>51</td>
<td>60</td>
<td>28</td>
</tr>
<tr>
<td>2-CEM 2</td>
<td>1159</td>
<td>1121</td>
<td>726</td>
</tr>
</tbody>
</table>

### 3.3 Chemical Shrinkage

Figure 4 shows the chemical shrinkage of both cements. Figure 4(a) presents the chemical shrinkage of different PSD of CEM 1, while Figure 4 (b) shows the chemical shrinkage of different PSD of CEM 2.
From both figures, a direct correlation between hydration process and chemical shrinkage can be seen. Shrinkage increases as hydration proceeds. Figure 4(a) shows that shrinkage is greater initially for fine and ground CEM 1 and less pronounced for normal CEM 1, coinciding with calorimetric results. This coincidence is also evident for a time longer than 12 hours, in which case shrinkage is greater for normal and ground CEM 1 than for fine CEM 1.

Figure 4(b) can be compared with the calorimetric test of CEM 2 and it can be seen that the shrinkage is larger for normal and fine CEM 2 for the studied time. This phenomenon is less remarked for ground CEM 2, in concordance with the calorimetric results.

When comparing Figure 4(a) and Figure 4(b), it can be noticed that shrinkage is always greater for CEM 1. This can be due to the presence of calcite, which accelerates the hydration process, especially for Alite and Celite phases.

As chemical shrinkage is induced by the hydration process, the magnitude of chemical shrinkage is directly related to the cement chemistry and the hydration process as stated by Bentz [21].

4 CONCLUSIONS

- The sieving process affects not only the particle size distribution of cements, but also their mineralogical composition. As some phases are softer than others, they become easier to grind during the milling process. With the sieving process, different cement mineralogies can be obtained.
- It can be noticed that the hydration process is affected by the particle size distribution as well as the mineralogical composition.
- Gypsum and calcite have a great influence on the hydration process of Portland cements. They both increase hydration kinetics since the beginning of the process.
- There is a direct correlation between the hydration process and the chemical shrinkage during the first 24 hours of cement hydration. The increase in hydration has an evident effect on higher chemical shrinkage.
- All chemical constituents of cement with greater hydration rate, like C₃S, can generate a high chemical shrinkage at early ages. It has been demonstrated that percentages of C₃A, C₄AF, Gypsum and CaCO₃ have a great influence on chemical...
shrinkage during the first day of hydration.

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USE OF INCINERATED SEWAGE SLUDGE ASH IN CONCRETE PRODUCTION

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Key words: waste water treatment plants, incinerated sewage sludge

Abstract. At the beginning of the twentieth century, about 10% of the world’s population lived in cities. By 2050, the global population is expected to reach between 8.3 and 10.9 billion. Rapid population growth and urbanization already has a dramatic effect on the increased demand for appropriate wastewater management. In Croatia, only about 43% of the population are connected to the public sewage network and only 23% are connected to the wastewater treatment plants with adequate level of treatment. By becoming the member state, Croatia has made the commitment to construct treatment plants of total capacities of 4,500,000 PE (250,000,000 m³ of wastewater). This implies that annually about 250,000 tons of dehydrated and stabilized sludge will be obtained and its further application needs to be investigated.

The basic objective of the presented research is to explore the possibilities of recycling of the sewage sludge for preparation of innovative products in concrete industry. Physical and chemical characteristics of incinerated sewage sludge ash (ISSA) from different treatment plants are investigated, since the possibility of recycling sewage sludge depends a great deal on its composition. Obtained results imply that ISSA may be used in concrete production, partially substituting cement or as inert filler, depending on concrete requirements.

1 INTRODUCTION

Providing of adequate water management is one of the basic preconditions of development in any area. Therefore, adequate management of wastewater belongs to priority activities of
adequate water management. In Croatia only about 43 percent of the population are connected to the public sewerage network [1], while the situation is even worse as regards connection to wastewater treatment plants. According to available data [1], only 23 percent of the population of Croatia are connected to treatment plants with adequate level of treatment.

Adequate wastewater management implies collecting, transport and treatment of wastewater, but also adequate management of waste substances generated by treatment. According to Croatian legislation, sewage sludge from wastewater treatment plants (WWTP) is considered non-hazardous waste which should be additionally processed and disposed into the environment. Incineration of sewage sludge considerably facilitates further management of the new product (incinerated sewage sludge ash - ISSA), first of all as a consequence of considerable reduction of mass and volume of the final by-product. Incineration reduces the total mass of sludge up to 85 percent [2]. For example, the central wastewater treatment plant of the city of Zagreb (Zagreb WWTP, Croatia), operating at full capacity (1,500,000 PE) will produce about 80,000 tons per annum of dehydrated and stabilized sludge, while incineration would further on produce about 18,000 tons of ISSA per annum [3].

![Figure 1](characteristics.png)

**Figure 1** Characteristics of certain forms of sewage sludge regarding its treatment [4]

### 1.1 Incineration of sewage sludge and ISSA characteristics

The possibility of recycling of sewage sludge depends a great deal on its composition, first of all its chemical composition. Lopes et al. [5] and Chen et al. [6] express a considerable influence of the origins of wastewater and types and quantities of additives on precise composition of ISSA and its properties. This stresses the importance of analysis of the quality and composition of ISSA in various circumstances, in relation to wastewater quality and technology used in sewage sludge from WWTP.

In the last two decades, different ways of recycling sewage sludge and possible applications of ISSA have been developed [7]. Numerous studies have investigated the influence of ISSA on properties of cement mortars [8] and concrete mixes [9], [10], in the production of clay products [11], as a replacement for part of the fine aggregate in mortars [12] and in asphalt mixtures [13].

Finally, the possibility of recycling of sludge and/or ash largely depends on its composition. The major chemical elements contained in ISSA as a by-product of incineration process are silicon, aluminium, calcium and phosphorus. Crystal forms of these elements are stable quartz (SiO₂), Ca₃(PO₄)₂ and hematite (Fe₂O₃). Medium size of ISSA particles ranges from 8 to 263 μm, with particle sizes up to approx. 700 μm. Exact range of ash particle sizes depends on composition of wastewater and sludge treatment technologies [7]. The ash texture
is porous, with particles of irregular shape, which make it a powdery material characterized by non-plastic properties [14]. Actual specific gravity of ash varies, according to individual authors (but not significantly) 2.3 – 3.2 g/m³ [6], 2.62 g/m³ [15], 2.86 g/m³ [16].

2 EXPERIMENTAL RESEARCH

The main aim of the presented research is to examine the possibility and feasibility of building in of ISSA, as the by-product from incineration process of sewage sludge, in concrete and concrete elements used in water management. Hereafter, results of physical and chemical characterization of ISSA from Croatian WWTP is presented together with the initial pilot study of the ISSA influence on fresh and hardened properties of mortars.

2.1 Production of ISSA for research needs

For the purposes of this study sewage sludge was collected from three WWTP (Karlovac, Koprivnica and Zagreb) and subjected to drying at 105°C to reach 90 % of dry matter. To produce ISSA, stabilized and dehydrated sludge was burned in the electric laboratory furnace at two temperatures (800°C and 1000°C), Figure 2. During exposure to high temperatures in the furnace, sewage sludge is transformed to the porous but hardened ash granules. Further grinding was required to obtain a powdery material, suitable for a cement replacement.

![Figure 2](image1.png)  
**Figure 2** a) Incandescent sludge, b) Hardened ash granulates after burning at 800 °C

![Figure 3](image2.png)  
**Figure 3** The milling process of incinerated sludge and sieving of ash particles (sieve residue highlighted in red)

The grinding was carried out by hand, using laboratory grinders with volume of 50 - 70 g and 150 - 200 g (Figure 3). For each cycle, the milling lasted 2 minutes. Presence of large granulates even after gridding required an additional sieving through a fine sieve with mesh size of 0.5 mm. The remaining clumps were returned to the mill, grinded again and re-
screened. Thus obtained ash was subjected to multiple tests to obtain detailed information on its physical and chemical composition.

2.1.1 Physical characteristics of ISSA

Physical characteristics of the ISSA, analysed as part of this study, included determination of specific gravity and particle size distribution. The density was determined according to the ASTM C-188, using representative sample of homogenous mass equal to 65 g. Determined values of specific gravities for different sources of sludge and its incineration temperatures are represented in table 1. As observed from the results, increase in the incineration temperatures resulted in the increase of the ISSA specific gravity values for all tested samples. The specific gravity of the used cement CEM II/ BM (SV) 42.5N was determined in the same manner and amounts to 2.95 g/cm³.

Table 1 Specific gravity of analysed ISSA

<table>
<thead>
<tr>
<th>Source of sludge</th>
<th>Incineration temperature</th>
<th>800°C</th>
<th>1000°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWTP Karlovac</td>
<td></td>
<td>2.67</td>
<td>2.83</td>
</tr>
<tr>
<td>WWTP Koprivnica</td>
<td></td>
<td>2.78</td>
<td>2.93</td>
</tr>
<tr>
<td>WWTP Zagreb</td>
<td></td>
<td>2.69</td>
<td>2.77</td>
</tr>
</tbody>
</table>

Particle size distribution of ISSA was determined by air jet sieving, according to the standard EN 933-10: 2009 for the classification of filler aggregates. Prior to the sieving, the ash was dried to the constant mass. Particle size distribution for different ISSA origin and incineration temperatures is presented on following figure 4.

Figure 4 Particle size distribution for ISSA from WWTP Karlovac, Koprivnica and Zagreb obtained on different incineration temperatures
No significant difference in the particle size distribution was observed for different incineration temperatures still, slight deviations are present for different sources of sewage sludge. Particle size distribution of all tested samples coincides with the literature data, according to which the particle size of ISSA ranges from 1 to 100 microns [17], [18]. It is interesting to note that ISSA from WWTP Zagreb has more than 70% of particles larger than 48 μm, while samples from other two WWTP (Karlovac and Koprivnica) have larger number of particles with smaller diameters ranging from 20 to 48 μm.

2.1.2 Chemical characteristics of ISSA

Basic minerals that make ISSA are SiO₂ and Al₂O₃ which creates good preconditions for its use in the form of mineral admixture to Portland cement-based composite materials. The share of CaO, SO₃, P₂O₅ i Fe₂O₃ is also significant [19]. Donatello et al. [7] point out that ISSA contains high shares of phosphates, usually 10% to 20% of mass share in the form of P₂O₅.

Chemical analysis of ISSA from WWTP Karlovac, Koprivnica and Zagreb, according to HRN EN ISO11885:2001 and representing three most significant oxides, is presented on figure 5. From the analysis is observed that ISSA from WWTP Koprivnica considerably differs from ISSA obtained from other sources. Significant share of CaO (which may be attributed to considerable quantities of lime added in sludge processing procedure), and considerably low share of SiO₂ imply that no pozzolanic activity can be expected from such material. Further possible applications should be considered, such as its usage as inert filler.

Pozzolanic activity can be expected from the ISSA obtained from WWTP Zagreb and Karlovac, while the highest pozzolanic activity can be expected for ISSA from WWTP Zagreb, since its chemical composition has the largest share of SiO₂, CaO and Al₂O₃; oxides important for assuring pozzolanic activity.

**Figure 5** Mean values of shares of significant individual oxides in ISSA on the basis of results of previous analyses [14]–[16], [18], [20]–[23] and comparison with results of chemical analysis of ISSA obtained by incineration of sludge from WWTP Karlovac, Koprivnica and Zagreb
2.2 Incorporation of ISSA in cement mortar

In order to investigate the possibilities of using the ISSA in the concrete industry, an investigation was conducted on samples of cement mortar as a basis for further research in this field.

2.2.1 Materials, mix design and test methods

The ISSA used in this research was obtained by incineration of stabilized and dried sludge from WWTP Karlovac in Croatia. Karlovac has a combined sewerage system, which means that WWTP along with wastewater from households and industrial wastewater also receives rain water. Maximum capacity of the wastewater treatment plant in Karlovac is 98,500 PE, and at present it operates with about 70% capacity. The plant includes the third stage of treatment. After the wastewater treatment process, primary and secondary sludge is dehydrated and anaerobically stabilized, with production of biogas which covers a part of power requirements of the plant. Dehydrated and stabilized sludge is temporarily stored at the plant location on a sheltered open spoil bank before final transport to the municipal mixed waste dump. For the purpose of this paper, dehydrated and stabilized sludge four months old from the temporary spoil bank was used. The sludge was additionally dried in laboratory drier at the temperature of 105°C, to reach the level of dry matter content above 90%. Dried sludge was then incinerated in laboratory conditions at different temperatures: 800°C and 1000°C. Furthermore, for preparing of cement mortar, cement CEM II/B-M (S-V) 42.5N was used, and dolomite sand 0/4 mm.

For the purpose of this research, 8 different mixes (water : cement : sand = 0.5 : 1 : 3) were prepared. Reference mix (without added ISSA) was marked as M0. Furthermore, a separate mix was prepared for each of the combinations of incineration temperatures (800°C and 1000°C) and percentages of cement substituted by ISSA (5%, 10%, 20% and 30% of cement mass). Table 2 shows the marks and characteristics of all obtained mixtures. The reference mixture is marked M0, and all others as Mn, where n represents the mass percentage of cement replaced by ISSA (5%, 10%, 20% and 30% respectively). Different incineration temperatures of ISSA used are represented by asterisk (*) with mixture mark.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>ISSA share [%]</th>
<th>w/b ratio</th>
<th>Incineration temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>M0</td>
<td>0</td>
<td>0.50</td>
<td>-</td>
</tr>
<tr>
<td>M5</td>
<td>5</td>
<td>0.50</td>
<td>800 °C</td>
</tr>
<tr>
<td>M10</td>
<td>10</td>
<td>0.50</td>
<td>800 °C</td>
</tr>
<tr>
<td>M20</td>
<td>20</td>
<td>0.50</td>
<td>800 °C</td>
</tr>
<tr>
<td>M30</td>
<td>30</td>
<td>0.50</td>
<td>800 °C</td>
</tr>
<tr>
<td>M5 *</td>
<td>5</td>
<td>0.50</td>
<td>1000 °C</td>
</tr>
<tr>
<td>M10 *</td>
<td>10</td>
<td>0.50</td>
<td>1000 °C</td>
</tr>
<tr>
<td>M20 *</td>
<td>20</td>
<td>0.50</td>
<td>1000 °C</td>
</tr>
</tbody>
</table>
The following tests were carried out in fresh and hardened state of mortar: air content, workability and mechanical properties (compressive and flexural strength at the age of 1, 7 and 28 days). Test specimens were prisms with dimensions 4x4x16 cm.

2.2.2 Analysis of the results and discussion

Fresh state properties are important for future application of developed materials and have major influence of the behaviour of cementitious materials in hardened state. Analysis of the measured air content values for the mixes with different ISSA share, implicates that increase in the ISSA content causes an increase of air content if compared to the reference mix, Figure 6. Independently, increase of incineration temperatures has positive influence on the amount of air inside of the mix, i.e. air content is reduced. This is in line with determined physical characteristics of ISSA (Table 1), where was demonstrated that an increase of incineration temperatures increases the specific density of the ISSA. Higher specific density assures better packing of particles inside of the composite. However, all ISSA mixes have higher air content compared to the reference mix.

![Figure 6](image-url) Influece of ISSA share and incineration temperatures on the porosity of mixes

It is well known, that incorporation of large percentage of very fine particles causes higher water demand and without additional corrections of mix design implicates lower consistency of mix [24]. Partial substitution of Portland cement by ISSA influences workability [7], [14], [15], [21], [22], [25] and this was further demonstrated using the consistency test within this research. No chemical admixtures were used during mixing, accordingly ISSA shares higher than 20% on the total binder content caused problems during compaction.

Analysis of the results implicates a correlation between the ISSA share and incineration temperatures, Figure 7. Increase in the ISSA share, for both incineration temperatures, is followed by the reduction of the consistency. For the same ISSA share, improved workability is obtained by using ISSA incinerated at higher temperatures (1000°C). This is in line with determined physical characteristics of ISSA, where was demonstrated that ISSA obtained at higher incineration temperatures has reduced water absorption [26].
The influence of the ISSA share and its incineration temperatures on the values of compressive strength of mortar are presented at Figure 8. Testing was performed 1, 7 and 28 day after casting. Increase of ISSA content caused decrease of mortars compressive strength. For ISSA incinerated at 1000°C, a decrease from 5 – 13.8 % compared to the reference mix was determined. Mixes with the ISSA incinerated at 800°C and with same ISSA share, established decrease of compressive strength below 3 %. This may be explained by structural changes of ash when incinerated at temperatures above 900 - 950°C, when depending on sources, increased crystallization and weakening of pozzolanic properties of the ash are present [2], [16], [20]. This reduction of pozzolanic properties is considered responsible for higher reduction of strength values for mixes with ISSA incinerated at 1000°C although, fresh state results may imply otherwise.

Investigation of the ISSA influence on flexural strength showed similar trend as determined from compressive strength tests, figure 9. Higher reductions of strength values are
associated with ISSA contents higher than 10 %, which may be considered as confirmation of up to date research [23]. Further investigation is still needed to identify admixtures which would, together with ISSA potentially achieve better mechanical properties than the results of research carried out so far.

![Figure 9 Influence of ISSA share, incineration temperatures and the maturity of mortar on the flexural strength values](image)

3 CONCLUSIONS

Performed initial study on the use of incinerated sludge ash in concrete production showed great potential of this waste material. By analysing physical and chemical characteristics of Croatian ISSA, it was confirmed that all available amounts can be used in concrete industry, either as cement replacement or inert filler. When used as cement replacement, ISSA’s content should be kept below 20%, if no chemical admixtures are used to improve workability of mix. Higher ISSA contents have strong influence on fresh and hardened state properties of mortar. However, it should be highlighted that all tested mixes, except mix with ISSA share of 30 %, have obtained equal or similar properties as the reference mix without ISSA.

Here was demonstrated that the incineration temperature and the source of the sewage sludge has influence on both, physical and chemical characteristics of ISSA, and following mortar properties. The optimum incineration temperature of the sewage sludge is determined as 800°C. Although, workability of mortar may be improved with ISSA obtained at higher incineration temperatures (1000°C), due to the lower pozzolanic activity of ISSA obtained in such manner the strength of mortars is decreased. Eventually when evaluating all possible benefits from usage of ISSA, lower energy consumption when sludge is incinerated at lower temperatures should also be considered as an additional prerequisite for achieving economic and environmental sustainability.
ACKNOWLEDGEMENT

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USE OF PHOTOCATALYTIC CEMENTS FOR HEAVY DUTY URBAN ROADS

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Key words: Grout, Depollution, Photocatalytic Cement, Roads, Sustainability

Summary. Among the different photocatalytic solutions available on the market, i.active® cements by Italcementi Group are the most common raw materials currently utilized for the manufacturing of building products possessing an added value in terms of sustainability and durability.

One of the main purposes for their use is to reduce pollution, in order to improve the air quality in urban environments. Another environmental advantage in their use is in terms of mitigation of urban heat island effects.

Currently, i.active® cements are commercially available in Europe, North America, North Africa and India. Several million square meters of photocatalytic cement-based surfaces, have already been applied, and this number is expected to grow substantially in the coming years.

In this framework, photocatalytic cementitious roads and pavements represent the widest application for the improvement of air quality in urban environment.

This paper gives an overview of the product performances to help engineers to choose and design sustainable and durable paved surfaces that typically represent from 30% to 60% of developed urban areas.

1 INTRODUCTION

Air pollution is a prominent critical issue of vehicles. Toxic pollutants (NOx, VOC, ammonia and other hydrocarbons) are harmful to the environment, hazardous to human health and difficult to degrade by natural means.

For example, a very recent study published by NASA laboratories explains the still high
level of NO$_2$ in several areas of our planet (Europe, East Coast of USA and China, above all), particularly concentrated in large urban areas, Figure 1 [1].

![Figure 1: NO$_2$ concentration – A global view in 2014 year (NASA) [1]](image)

Therefore a significant amount of efforts have been made, especially in this last decade, in order to reduce air pollution, adopting national rules and recommendations for a better management of the environment, strictly connected with traffic, industrial production and urban heating sources.

Some solutions have been adopted in terms of limited industrial emissions and an increasing number of more ecological vehicles (hybrid or electric). However, the percentage of pollution due to traffic is still relevant, and some local solutions could be considered at ground level. Among them, the wide application of photocatalytic pavements that directly interact with vehicles seems to be a promising solution.

Thanks to the synergetic effect of cement and titanium dioxide discovered in recent years [2-5], new cementitious pavements were developed and launched on the market, thanks to their ability to accelerate the natural reaction of oxidation [6-9].

This paper deals with the development of several types of photocatalytic cementitious road pavements, such as paving blocks, special asphalts filled up with a photocatalytic grout and concrete roads. Indeed, other photocatalytic solutions could be adopted in our urban open situations for a higher amount of “active” surfaces, such as concrete infrastructures, urban furniture, sidewalks, roundabouts, pedestrian areas,…

First of all, a description of the testing methods for the characterization of these photocatalytic materials will be briefly given (NO$_x$ depollution tests). These tests are very important for the better understanding of the materials performance, for the chosen solution.

Large laboratory and in-field testing programs – PICADA [10] and PhotoPAQ [11] EU
projects, for example – have been conducted, measuring the environmental effectiveness by means of a specific set-up and in-situ sources, respectively. Due to the complex local situation in terms of (sometimes extremely) variable environmental conditions (e.g. temperature, relative humidity, sunlight, wind speed and pollution level), it is not easy to demonstrate with open-air monitoring campaign what can be determined in a controlled condition, by means of a laboratory test.

During the development of a specific road solution we must consider the structural performance (mechanical, under traffic loads), some functional properties (such as the surface roughness for safety reasons) and the durability performance (in terms of service life of the pavement and from the photocatalytic point of view). Indeed, road pavements have to guarantee both de-polluting effects and performance usually required to a road pavement wearing course, according to the current standards.

The extensive research activity conducted till now clearly showed the significant contribution of such innovative materials to the improvement of air quality, reducing the nitrogen oxides concentration level up to 60% in some local weather conditions. They can represent a new frontier of the research aimed at reducing air pollutants, for environmental friendly solutions.

In some urban conditions, the NO$_2$ reduction below the official limit of emissions imposed can represent an effective result for a better air quality in our cities.

Of course, the efficiency of the photocatalytic solution on the global urban situation is strictly connected with a large use of high performance depolluting pavements and manufactures: the limited amount of applied surface cannot modify the global values, but can be only a better improvement of local conditions (that is, only for the area of influence where active surfaces are applied).

Among possible photocatalytic solutions, the cement-based materials based on TX Active® technology represent a very effective and widely demonstrated technology. In comparison with other photocatalytic building materials, these special products are able to absorb the NOx on the surface, to transform them in non-noxious ions and to block them in form of salts (nitrates), Figure 2. These substances can be easily removed from the surface by means of the rain (lixiviation) or by washing.

The use of TiO$_2$ as a photocatalyst, in combination with the cementitious bulk (TX Active® technology, by Italcementi), is also effective for the transformation of other pollutants (aromatics, ammonia and aldehydes) with different mechanism, as observed in [5, 12 - 13].

Finally, in the particular case of NO$_x$ reduction, as an indirect consequence, it was demonstrated that the production of ozone is strongly inhibited in the atmosphere and a further environmental advantage in warmer periods of the year is evident.
This study focuses on the nitrogen oxides reduction, the most critical harmful substances with their high concentration in urban air, due to the increasing traffic volumes. Nitrogen dioxide (NO₂) is the gas considered as a reference parameter in urban emissions, as well as PMs (particulate matters).

In this paper, some road applications are described, where TX Active® technology can be effectively applied.

2 LABORATORY TESTS

The depollution effectiveness of photocatalytic materials is measured by means of a NOx test equipment, able to reproduce plausible in-service conditions in terms of gas concentration, gas flow rate in controlled conditions of UV light, temperature and air humidity. Most tests are carried out in NOx flow-through conditions, conducted with a fixed concentration of NOx (i.e., in the Italian standard UNI 11247-2010 [14] the NOx content is set to 0.55 ppm, of which 0.15 ppm of NO₂ and 0.4 ppm of NO) in nitrogen gas, corresponding to a possible atmospheric pollution in urban condition. The result of the test according to UNI 11247 standard is expressed as the NOx photocatalytic decomposition percentage of a sample under a determined UV radiation level. The degradation rate (NOx abatement per unit of surface and unit of time) can also be derived.

Furthermore, the CEN/TS 00386023 [15] has recently been approved (and it will soon be published) in the framework of the CEN TC 386 committee “Photocatalysis”. Based on the concept of CSTR reactor (Continuous Stirred Tank Reactor, Figure 3) theory [16] and following the recently published Italian Standard 11484-2013 [17], this test can be used for the evaluation of all photocatalytic materials – including cement-based products, coated asphalts, paints, bituminous membranes and industrial ceramics. Some round-robin tests are still in progress at CEN level, to verify the reproducibility and repeatability of this method, with the participation of several European laboratories. On the grounds of this testing approach, the degradation rate of the material can be determined.
Figure 3: Continuous Stirred Tank Reactor for photocatalytic NO\textsubscript{x} test – top view (left) and side view (right)

Using photocatalytic cement-based pavements, the typical values of NO\textsubscript{x} abatement range between 10 and 30 gNO\textsubscript{x}/year/m\textsuperscript{2}, so that we can calculate a mean NO\textsubscript{x} removal of 100-300 kgNO\textsubscript{x}/year for an active surface of 10,000 m\textsuperscript{2} (i.e., a surface area equivalent to a 2-lanes road 5m x 2000m). Considering the NO\textsubscript{x} reference emissions values for Euro 5 and Euro 6 gasoline cars (0.06 g/km) whose annual average distance is of about 12,000 km, this is equivalent to the elimination of NO\textsubscript{x} pollution annually produced by about 416 cars.

3 FIELD TESTS

For several years, the photocatalytic cementitious products have been widely used for building applications all over the world, in order to obtain depolluting and self-cleaning actions [3, 5, 7, 10].

Materials applied for depollution were previously developed and validated at laboratory scale. However, for the first field applications the attempt to demonstrate the in-situ performances was necessary by adopting dedicated experimental test protocols and using specific analysis equipment. One of the first significant examples was represented by the European project PICADA (Photocatalytic Innovative Coverings Applications for Depollution Assessment), where an experimental activity for the validation of the photocatalytic process by means of a street canyon was carried out to evaluate the effects of wall surface-coated with a photocatalytic coating [10]. In this project, the effect of photocatalytic materials on air quality was demonstrated thanks to the correlation between in situ results and the developed model [18].

On the laboratory promising findings, the effectiveness of the photocatalytic surface layers has been evaluated on large scale, proving the ability of the photocatalytic concrete to reduce NO and NO\textsubscript{2} content in the air.

After the PICADA project, other in-situ monitoring campaigns were carried out in some experimental projects, with positive results, such as:
- Borgo Palazzo street in Bergamo, Italy (a city road built with paving blocks) [6, 19]
- Rue Jean Bleuzen in Vanves (Paris), France (concrete city road) [20]
- Umberto I city road tunnel in Rome, Italy (vault coated with a photocatalytic paint,
A reliable estimate of photocatalytic surfaces obtained with cement-based materials, is equal to several million square meters. Horizontal surface applications (paving blocks, industrial pavements, whitetopping, etc.) comprise approximately 50% of the total. Most of the projects were developed in Europe, especially Italy, France, Belgium, Germany, Spain and Greece.

The monitoring the effectiveness of photocatalytic surface in real conditions has also been carried out in other pilot projects (e.g. within the framework of Life+ PhotoPAQ project [11]), but results are not really positive, due to the complexity of the measurements (due to the influence of local atmospheric conditions) and the limited quantity of data collected, for an acceptable statistical evaluation (on the basis of the laboratory tests, the material applied was active).

4 INNOVATIVE PHOTOCATALYTIC CEMENTITIOUS ROAD MATERIALS

Currently, there are several solutions for the construction of photocatalytic roads and pavements, using cement-based materials (Figure 4):

1. Interlocking paving blocks for roads, parking areas, shopping areas, sidewalks, crossroads, traffic calming zones and bike routes;
2. Pre-cast or ready-mix concrete pavements;
3. Slurry-infiltrated open-graded asphalt

Paving blocks have always found wide application, as represent a good, economical solution for the peculiarity of the manufactures (two-layers, the top is photocatalytic). They are an aesthetic solution in urban applications (roads, pavements, parking lots, sidewalks, roundabout, …) as well as other similar products (slabs, plates and curbs) [7, 8, 20].

The material cost of this solution is higher than conventional elements, but the incidence of their cost on the whole construction costs of the pavement is only slightly higher (3-4%) than...
conventional concrete.

Ready mix concrete pavements have also been studied and applied, in order to optimize structural performance, architectural needs and photocatalytic properties for the ideal cost/performances ratio.

Rue Jan Bleuzen in Paris- Porte de Vanves is one of the first demonstrative project [20], where the technology of one-lift paving (slipforming) was adopted.

Another important demonstrative project were completed in St. Louis (USA) in the framework of a national showcase sponsored by FHWA (Federal Highway Administration) and MoDOT (Missouri Department of Transportation) [22]. In that occasion, a special equipment was used to cast/extrude a fresh photocatalytic concrete layer (about 12 cm of thickness) on a traditional concrete layer, for a total of about 30 cm.

Other photocatalytic concrete pavements have also been completed in Italy. However, the total number of applications has been limited, due to the high photocatalytic concrete thickness and to the corresponding construction costs.

In terms of cost-effective solution, the technology of thin and ultra-thin whitetopping has been sometimes adopted, still ensuring the photocatalytic performance. This technology can be adopted by applying a very thin top layer (from 2-3 mm up to 10-15 mm) over a traditional concrete pavement (fresh on fresh technology). The main characteristics of this layer are photocatalytic properties and abrasion resistance.

Pervious concrete is a promising ‘green’ solution. Its mix design must incorporate the correct blend of aggregates that allows adequate drainage of rainwater. From the photocatalytic point of view, this solution really offers high specific surfaces and corresponding high depolluting performance. However, pervious concrete does provide a lower compressive strength than that of conventional concrete, so that its use can be limited for sidewalks, parking lots, pedestrian areas and other special pavement applications.

4.1 Asphalt pavement infiltrated with a cementitious slurry

Combining an open-graded porous asphalt with a photocatalytic cementitious slurry (or grout), high performance road pavements can be obtained, Figure 5.

This hybrid technology which includes the use of two different binder materials (cement and bituminous asphalt) allows to construct a special concrete (jointless) pavement with very interesting structural performance (from a high punching strength in case of industrial pavements to semi-flexible performance, for roads), Figures 5 and 6.
The photocatalytic grout is firstly prepared by mixing a photocatalytic binder in powder form (i.active CARGO®, patented by Italcementi) with water and immediately applied to obtain the expected composite pavement [23], Figure 6.

Thanks to its rheological performance (high flowability), this cement grout can percolate through highly porous asphalt pavements. As a result, it fills the voids and forms a monolithic structure that can withstand heavy loads both under static and dynamic conditions (by combining enhanced physical and chemical resistance with high strength).

The main phases of the correct construction of a pavement are the following:

a) Preparation of the open-graded asphalt for the slurry infiltration.
   - Preliminary rutting of the old pavement, if necessary
   - Application of the base (asphalt)
   - Application of a bitumen-based primer
   - Application of the open-graded asphalt (wearing course) – thickness: 4-5 cm
   Sometimes, the resurfacing can involve only the application of an open-graded asphalt on an existing substrate in good conditions.

b) Preparation, application of the i.active cargo and other operations.
   - Preparation of the i.active CARGO® slurry (mixing with water)
   - Complete percolation (infiltration) of the open-graded asphalt, using the slurry
- Curing
- (possible) Post-finishing treatment of the surface – e.g. shot peeling (steel blasting), bush hammering or similar.

The post-treatment step can be necessary in order to obtain the desired surface roughness finishing (design skid values) or only for obtaining a better colour homogeneity.

This solution is particularly suitable for heavy duty traffic loads and can be applied in urban environment, where pollution is really a big issue, so that can give an active contribution for a better air quality.

Furtherly, the road surface results more resistant to high temperatures than traditional asphalt-based solutions (hot weather) and contributes to promoting safety, as it does not undergo the traditional heat deformation phenomena. Fire resistance is high, as well.

Another positive aspect which can be highlighted for this technology is the speed of construction:
- With the proper organization of the construction site, you can complete up to 2.500-3.000 square meters in 8 working hours.
- 24-36 hours to get the opening of the road, instead of the “usual” 28 days required for the solution with concrete. This can be really important in case of urban works: saving time, means saving exercise costs.

Finally, thanks to its bright colored surface, the use of i.active CARGO® reduces the pavement’s temperature during hot periods and improves comfort for people and vehicles (reduction of heat island effect).

This technology has already been recently applied in Italy, France, Spain and Greece. Figure 7 shows the work recently completed in Malaga (Spain), for the construction of a bus-lane. Figure 8 shows another urban application recently completed in Athens (Greece).

5 CONCLUSIONS

In this paper, an overview on photocatalytic concrete solutions which can be used for the construction of sustainable, high resistant and durable pavements was given.

Photocatalytic road materials could affect significally air quality, reducing pollutant concentrations, through the beneficial presence of adequate photocatalysts in the cementitious matrix. Another relevant advantage deriving from the use of cementitious pavement is the reduction of the heat island effect.

Among different solutions, the use of i.active CARGO® for the percolation of open-graded asphalts seems to be the most suitable one for semi-flexible pavements in presence of heavy traffic loads, with added values (heat resistance, absence of joints and speed of construction).
Figure 7: Bus lane in Alameda Principal, Malaga (Spain)

Figure 8: Urban street in Athens (Greece)

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VALORISATION OF GRANITE CHIPPINGS IN THE DESIGN OF NEW CEMENT MATRICES

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Keywords: granite quarrying, reuse, cement

Abstract
The use of natural stone in construction is varied and becoming increasingly widespread, despite the presence of other materials such as concrete or fired clay brick and similar. Granite quarrying and hewing entail the generation of large volumes of waste whose stockpiling in uncontrolled landfills constitutes a significant environmental problem. This research attempted to valorise such waste in the cement industry, which is characterised by a demand for large volumes of raw materials and a wide spectrum of end products. The waste was first characterised physically and chemically to assess its potential for use in the cement industry. Subsequent determination of the physical and mechanical properties of the new waste-bearing cement showed it to be standard-compliant.
1 INTRODUCTION

Given the geological characteristics of the Iberian Peninsula, Spain is one of the world’s major producers of ornamental stone such as marble and granite; the decline in output in recent years (Figure 1) due to the slump in domestic demand has made the industry increasingly dependent on exports. Granite production was highest in the country’s northwestern region of Galicia in 2012, the first year that Madrid out-produced southwestern Extremadura, which slid to third place [1, 2].

Granite quarrying and transformation generate large volumes of waste (sludge), which in present uncontrolled practice is removed to landfills [3]. These fine and hence readily airborne particles have adverse consequences for both the environment and human health.

Valorisation of this type of waste is a highly topical issue in societies seeking to change from the present linear to a circular economic model in which waste is reused to produce new products or raw materials, thereby ensuring intelligent, sustainable and integrated growth [4].

Research along these lines has been and is underway to determine the feasibility of valorising quarry chippings as a raw material in the design and manufacture of products such as new floor, wall and porcelain tiles, subsequently applicable in building construction [5-8].

Similarly, a number of recent papers have analysed the possibility of including granite industry waste as a filler or fine aggregate in the design of new mortars and concretes [3, 9, 10].

This study aimed to valorise such waste as an addition in new eco-efficient binders to reduce the volume of natural raw materials needed to manufacture cement. To that end, the waste was characterised and its possible pozzolanicity explored, after which the physical, mechanical and microstructural properties of the new cements were assessed.

2 EXPERIMENTAL

The chemical composition of the cement used (EN 197-1 [11] class CEM I 42.5 R) was determined on powder samples with a Bruker S8 Tiger wavelength dispersive X-ray
fluorescence (XRF) spectrometer fitted with standardless Spectraplus Quant Express software.

Samples of the granite industry waste, in turn, were carbon-coated and secured to a metallic holder with a two-sided graphic adhesive to ensure conductivity and detection of all the signals sought during characterisation on a Bruker SEM/EDX scanning electron microscope.

Samples of the anhydrous CEM I 42.5 R containing 10 wt% of granite industry waste were characterised by FTIR, 1 mg of the material was pressed into pellets with 300 mg of KBr for analysis on a Nicolet 6700FT-IR (Fourier transform infrared) spectrometer operating in a range of 4000 to 400 cm\(^{-1}\).

The total porosity and mean pore diameter were found as prescribed in ASTM standard D 4404, using a Micromeritics Autopore IV 9500 mercury porosimeter designed to operate at pressures of up to 33 000 psi (227.5 MPa) and measure pore diameters of 0.006–175 \(\mu\)m.

The pozzolanic activity of the granite industry waste was found chemically [12] by soaking 1 g of the material in 75 mL of a saturated calcium hydroxide solution ([Ca(OH)\(_2\)] = 17.68 mM/L) at 40 °C for 3 or 28 days. The CaO concentration was analysed after each test age essentially as described in European standard EN 196-5 [13]: i.e., the amount of lime fixed was calculated as the difference between the initial concentration in the saturated calcium hydroxide solution (17.68 mM/L) and the amount of CaO remaining in the solution in which the (pozzolan) sample was soaked.

Prismatic specimens measuring 40x40x160 mm were prepared with cement containing 0 and 10 wt% of granite waste using a water/cement ratio of 1:2 and a cement/aggregate ratio of 3:1. The drying shrinkage and normal consistency of these cement pastes were determined as laid down in European standard EN 196-3 [13]. Compressive and flexural strength of the mortars at 2 and 28 days were also found, as described in EN 196-1 [14].

3 RESULTS AND DISCUSSION

Further to the XRF findings, the chemical composition of the cement in (weight) per cent of oxides was as follows: 20.29 % SiO\(_2\), 59.46 % CaO, 4.62 % Al\(_2\)O\(_3\), 4.14 % SO\(_3\), 3.35 % MgO, 2.25 % Na\(_2\)O+K\(_2\)O and 0.39 % other oxides. Loss on ignition at 1000 °C was 3.04 %.

The granite industry waste studied exhibited a particle size of under 100 \(\mu\)m (see the SEM micrograph reproduced in Figure 2). The chemical composition found for the waste with scanning electron microscope/energy-dispersive spectroscopy (SEM/EDX) included a high percentage of SiO\(_2\) (~65 %), Al\(_2\)O\(_3\) (~16 %) and alkaline oxides (~8 % Na\(_2\)O+K\(_2\)O) and small amounts of CaO (~1.8 %). In other words, mineralogically it was silicate- and aluminate-high (quartz, feldspars, micas and clay) and calcium carbonate-low (dolomite or calcite). The presence of Fe\(_2\)O\(_3\) (~2.9 %) denoted the existence of chlorite group mineral phases. All these findings are characteristic of the existing varieties of granite [6, 15]. The total reactive amorphous silicon was around 22-23 % was identified as reactive amorphous silicon (as per the procedure laid down in Spanish standard UNE 80225 [16]).
The FTIR findings for the cement with 10 wt% granite industry waste are shown in Figure 3. In addition to the most characteristic cement bands (alite, belite, C₄AF, C₃A, gypsum and basanite), the specifics for which are listed in Table 1, the spectrum exhibited other bands attributable to the granite waste.

### Table 1: FTIR spectrum band assignments for cement bearing 10 wt% granite industry waste

<table>
<thead>
<tr>
<th>Band number</th>
<th>Wavenumber (cm⁻¹)</th>
<th>Attributed to</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3641</td>
<td>(\nu) Al(VI)-OH (\rightarrow) muscovite and kaolinite</td>
<td>[17-20]</td>
</tr>
<tr>
<td>2</td>
<td>3432</td>
<td>(\nu) O-H (H₂O) (\rightarrow) clays and micas such as muscovite and kaolinite</td>
<td>[17-20]</td>
</tr>
<tr>
<td>3</td>
<td>1150</td>
<td>(\nu) S-O (\rightarrow) basanite and gypsum; (\nu) Si-O (\rightarrow) muscovite</td>
<td>[21-23]</td>
</tr>
<tr>
<td>4</td>
<td>1118</td>
<td>(\nu) S-O (\rightarrow) basanite</td>
<td>[21, 22]</td>
</tr>
<tr>
<td>5</td>
<td>1096</td>
<td>(\nu) S-O (\rightarrow) basanite; (\nu) Si-O (\rightarrow) quartz and chlorite</td>
<td>[21, 22, 24, 25]</td>
</tr>
<tr>
<td>6</td>
<td>1048</td>
<td>(\nu) S-O (\rightarrow) gypsum and (\nu) Si-O y/o Al-O (\rightarrow) clays such as kaolinite</td>
<td>[17, 18, 26]</td>
</tr>
<tr>
<td>7</td>
<td>939-925</td>
<td>(\nu) Si-O (\rightarrow) alite + belite</td>
<td>[21, 27, 28]</td>
</tr>
<tr>
<td>8</td>
<td>876</td>
<td>(\nu) Si-O and (\nu) C-O (\rightarrow) silicates + carbonates</td>
<td>[2, 29]</td>
</tr>
<tr>
<td>9</td>
<td>845</td>
<td>(\nu) (\delta) Si-O (\rightarrow) belite</td>
<td>[2, 21]</td>
</tr>
<tr>
<td>10</td>
<td>714</td>
<td>(\nu) C-O, (CO(_2))^2⁺ (\rightarrow) carbonates</td>
<td>[29]</td>
</tr>
<tr>
<td>11</td>
<td>668</td>
<td>(\delta) O-S-O (\rightarrow) gypsum</td>
<td>[21]</td>
</tr>
<tr>
<td>12</td>
<td>659</td>
<td>(\delta) O-S-O (\rightarrow) basanite</td>
<td>[21]</td>
</tr>
<tr>
<td>13</td>
<td>599-600</td>
<td>(\delta) O-S-O (\rightarrow) basanite, gypsum and Fe-O (FeO(_4)) (\rightarrow) C₄AF</td>
<td>[21]</td>
</tr>
<tr>
<td>14</td>
<td>522</td>
<td>(\delta) O-Si-O (\rightarrow) alite and belite; (\delta) Al-O (\rightarrow) C₃A</td>
<td>[21, 27, 28]</td>
</tr>
<tr>
<td>15</td>
<td>463</td>
<td>(\delta) O-Si-O (\rightarrow) alite, belite, kaolinite and orthoclase</td>
<td>[21, 30, 31]</td>
</tr>
<tr>
<td>16</td>
<td>443</td>
<td>Al-O (AlO(_3)) (\rightarrow) C₄AF</td>
<td>[21]</td>
</tr>
<tr>
<td>17</td>
<td>426</td>
<td>albite</td>
<td>[31]</td>
</tr>
</tbody>
</table>
Three and 28 days granite pozzolanicity is depicted in Figure 4, where the values are compared to the activity for other previously studied new (coal mining waste, calcined paper sludge, sanitary ware waste) and conventional (fly ash) pozzolanic additions [32-34]. The figure shows that this new addition exhibited low 3-day pozzolanicity (capacity to fix lime), with a value approximately double fly ash activity but much lower than found for other new additions with a high SiO₂ content (75 and 66 % less than coal mining and sanitary ware waste, respectively). At 28 days, pozzolanicity rose to approximately 38 %: i.e., higher than for fly ash and lower than for the aforementioned new pozzolans.

Respect to the physical properties, with a 35 mm slump and 1 mm drying shrinkage [11], the cement pastes containing granite waste proved to be European standard EN 197-1-compliant.

The effect of this waste on mortar compressive and bending (or flexural) strength is shown in Figure 5. Replacing 10 wt% of cement with granite waste induced slight decline of less than 14 % in compressive and 10 % in flexural strength, irrespective of mortar age. These
mortars were nonetheless European standard EN 197-1-compliant [11], with 2- and 28-day compressive strength values upper to 20 and 42.5 MPa, respectively.

![Figure 5](image)

**Figure 5**: Comparison of a) compressive and b) flexural strength in granite waste-bearing and waste-free mortar (strength ratio = \{blended mortar strength/waste-free mortar strength\} x 100)

Total porosity and mean pore diameter for the mortars at 28 days are listed in Table 2. Note that the blended material with 10% granite waste has higher total porosity and mean pore size than the reference mortar with 0% granite waste.

<table>
<thead>
<tr>
<th>Property</th>
<th>With 0% granite waste</th>
<th>With 10% granite waste</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total porosity (vol.%)</td>
<td>12.52</td>
<td>12.74</td>
</tr>
<tr>
<td>Mean pore diameter (µm)</td>
<td>0.0726</td>
<td>0.0735</td>
</tr>
</tbody>
</table>

Figure 6 shows the DTA findings for the 28-day mortars with and without 10 wt% granite waste. The differential thermal analysis curve exhibited four endothermal signals. The first (a), asymmetrical and located at 90-170 °C, with peaks at 96 and 135 °C, was attributed primarily to interstitial water and structurally bound water in C-S-H gel, in keeping with reports by authors studying tobermorite- and jennite-like C-S-H gel [32], but also to ettringite decomposition [35]. The second (b), a higher intensity band at around 400-475 °C, was associated with dehydroxylation of the portlandite forming during silicate (alite and belite) phase hydration. The third (c), observed at around 571 °C and associated with no weight loss, was generated essentially by the conversion of the α to the β quartz polymorph [36]. The fourth signal (d) was attributed to decarbonation (loss of CO₂) of the more or less crystalline carbonates formed from portlandite exposed to the air. Lastly, the exothermal signal at around 900 °C may have been the result of quartz recrystallisation into cristobalite or the crystallisation of mullite-like phases or cubic spinels deriving from clay minerals such as kaolinite present in the waste.
Figure 6: Differential thermal analysis for mortars with: (a) 10 % and (b) 0 % granite industry waste

4 CONCLUSIONS

The conclusions that can be drawn from the present study are set out below.

- The moderate pozzolanicity in the 28-day mortars containing granite industry waste was greater than the activity reported for some types of fly ash.

- The mortars containing 10 wt% granite industry waste, while exhibiting slightly lower mechanical strength than the unadditioned mortars, were European standard EN 197-1-compliant.

- Inclusion of the granite industry waste induced an increase in total porosity and mean pore diameter, with concomitant pore system refinement.

5 ACKNOWLEDGEMENTS

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REFERENCES

VALORIZATION OF A WASTE INTO CEMENTITIOUS MATERIAL: DREDGED SEDIMENT FOR PRODUCTION OF SELF COMPACTING CONCRETE

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Key words: Protocol, waste treatment, cementitious material, dredged sediment, self-compacting concrete

Abstract. It is remarkable that no general rules, as established protocols, are found in the bibliography related with including wastes into concrete. In this way, there are no previous efforts in order to systematically evaluate and decide on the feasibility for including a waste into a cementitious matrix. Filling this gap is the objective undertaken in this paper, which presents general guidelines to be followed in order to analyse the feasibility of including a waste material in the production of one of the most widely used building materials: concrete. The procedure is illustrated with a full example, obtaining a self compacting concrete (SCC) including dredged sediment taken from a Spanish harbour.

1  INTRODUCTION

In the last years, lot of research has been done in including wastes into concrete, mainly for secondary uses but also trying to accomplish with structural performance. Even though there are a lot of contributions in this area, and society concerns about the reutilization of waste in concrete fabrication, the research is mainly focused on the reuse of several wastes, as industrial, demolition or glass, for most abundant, but is rare to found research works including dredged sediments as a waste to be use in concrete fabrication. On the other hand, it is remarkable that no general rules, as established protocols, are found in the bibliography. In this way, there are no previous efforts in order to systematically evaluate and decide on the feasibility for including a waste into a cementitious matrix.

The objective undertaken in this paper is to check the compatibility of the waste with a cementitious material. If necessary, a decontamination step will be proposed. As a last step, mechanical, environmental and durability properties have to be evaluated.

The procedure is illustrated with a full example, obtaining a self compacting concrete (SCC) including dredged sediment taken from a Spanish harbour. The reason for choosing SCC as the product to be constructed has been derived from the characteristics of the waste, taking into account its special characteristics in relation with their intrinsic environmental friendly technology: is able to flow by its own weight, eliminating the need of vibration, which can be translated into substantial reduction in energy, labour cost and construction time. Additionally, it contributes to a better working environment by eliminating the impact of noise and vibration as well as a making more comfortable the periods of works for the affected society.

It is remarkable that the research in the area of reusing materials for self compacting concrete
(SCC) started in the last decade, and has increased very fast in the last years with interesting contributions [1]-[3], so, the necessity of some guidelines to assess the inclusion of these wastes in SCC is clear. These guidelines have been illustrated on the basis of using dredged material in performing SCC. This is an interesting case study, as no bibliography has been found on it. Dredged sediment has been chosen because dredging is necessary to create and maintain navigation channels in naval facilities, and despite several hundred million cubic meters of sediment dredged each year just in U.S. most of this dredged material is disposed in open water, confined disposal facilities, and upland disposal facilities, not having so much research dedicated to, as reflected in the bibliographic results showed in Fig. 1. In addition, in U.S., consistent with the National Dredging Policy, EPA encourages the “beneficial reuse” of dredged material. In Europe, having the same problem, according to new European Union Directives, mainly the EU Water Framework Directive, it is necessary to find an environmental solution to these materials. Dredged sediments have to be seen as a valuable resource better than a waste.

The reason for choosing SCC as the product to be constructed has been derived from the characteristics of the waste, taking into account its special characteristics in relation with their intrinsic environmental friendly technology: is able to flow by its own weight, eliminating the need of vibration, which can be translated into substantial reduction in energy, labour cost and construction time. Additionally, it contributes to a better working environment by eliminating the impact of noise and vibration as well as a making more comfortable the periods of works for the affected society.

2 GUIDELINES FOR ASSESSING THE VALORISATION OF THE WASTE

The scheme proposed and designed to assess the feasibility of a waste material to be incorporated in a cementitious material is given in Figure 1.

- **Step 1:** First of all, the waste material has to be subjected to a thorough chemical and mineralogical analysis in order to establish their elemental composition compatibility with a cementitious material. This is a very important step in the process, as if compatibility is not clear, it will be discarded for this type of valorisation.

- **Step 2:** If the composition of the waste is compatible with cement, hazardous characterisation of the waste has to be carried out. According to the definition, characteristic hazardous or toxic wastes are determined by evaluating their ignitability, corrosivity, reactivity, and toxicity. If they do not accomplish with these requirements, the following question is: it is possible to remediate it? This is quite important in the case of toxicity, with wastes having heavy metals and organic pollutants, which is the most frequent case. Different techniques to remediate contaminated wastes can be found in literature, for example, electrokinetic remediation [4]-[6]. The same apply for components non hazardous but deleterious for the use: for example chlorides for reinforced concrete [7]-[9].

- **Step 3:** As a third step, it is necessary to carry out a thoroughly granulometric characterisation.
• Step 4: Having material and hazardous compatibility assured, in function of composition and granulometry characteristics, the most suitable material to be produced, that includes it, has to be established to be decided.

• Step 5: The following step is the design of the material chosen with traditional components and including the waste, trying always to introduce the highest amount of waste compatible with the expected use.

• Step 6: After having designed the initial mix, according to the results obtained in the previous steps, the specimens have to be analysed both in fresh and in hardened state, where mechanical and environmental characteristics of the concrete, together with its durability properties have to be evaluated.

If results of step 6 are satisfactory, the material designed is feasible. If not, it is necessary to go again to step 4 and reconsider the type of material to fabricate.

Figure 1 Schema of the protocol to be followed to investigate the feasibility of reusing a waste material as a component of a cementitious material.

3. EXPERIMENTAL PROCEDURE: APPLICATION TO THE DREDGED MATERIAL.

3.1 Characterization of the dredged materials

The dredged material used in this research has been real non polluted marine dredged sediment taken from a Spanish Mediterranean harbour. After dredging, the material was stored under controlled conditions. Previous works about the characterization of these materials had been taken into account [10]-[13]

The composition of the dredged sediment used in this investigation was characterized on elemental composition by chemical analysis. The mineralogical determination was carried out
by X-Ray diffraction.

All the analysis for sediment physical and chemical characterization were made according to Spanish recommendations for dredged materials and following the recommended protocols [10]. The dry weight fraction was determined by weight loss at 105 °C. For the rest of analyses, sediments were dried at 40 °C for 24 h. Additionally, physical characterization (granulometric analyses) and determination of the total organic content (TOC) was carried out. Grain size distribution followed UNE 103 101 and total organic carbon (TOC) content was estimated by loss of ignition (LOI, determined heating the sample during 15 minutes at the temperature of 950°C ± 25°C) and gravimetric determination as recommended for small dredged volumes and applying the following expression to express the results as total organic carbon.

\[
\text{TOC (g kg}^{-1}) = 0.35 \times \text{LOI (g kg}^{-1})
\]

It was analysed in order to assess the presence of heavy metals, polycyclic aromatic hydrocarbons (PAHs), organophosphorous pesticides and other organic chlorinated compounds. These analyses are made using the fraction with particle size lower than 63µm.

### 3.2 Fabrication of the concrete specimens.

Concrete was cast with cement type I 42.5R without additions. Other components, apart from the sediment were water, sand and coarse aggregates. Three different mixes were tested. Its composition is shown in Table 1

<table>
<thead>
<tr>
<th>Mix</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement I 42.5 R</td>
<td>420</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>Dredged material</td>
<td>73</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>water</td>
<td>205</td>
<td>202</td>
<td>202</td>
</tr>
<tr>
<td>w/c</td>
<td>0.49</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>Silicious coarse aggregate (6/12)</td>
<td>717</td>
<td>676</td>
<td>667</td>
</tr>
<tr>
<td>Silicious sand (0/6)</td>
<td>850</td>
<td>870</td>
<td>859</td>
</tr>
<tr>
<td>Admixture I</td>
<td>9.24</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>Admixture II</td>
<td></td>
<td></td>
<td>7.425</td>
</tr>
</tbody>
</table>

### 3.3 Characterization of the concrete.

In fresh state, the fluency of the concrete was characterized through the slump test using the Abram’s cone and the consistency of the concrete was measured following the UNE EN 12350-2 standard by measuring the spread of the concrete on a flat plate subjected to a shocking movement. It was also followed the UNE EN 12350-8 standard, suitable for SCC. The density of the fresh concrete was measured according to the UNE EN 12350-6 standard. Another important parameter is the entrapped air of the concrete, which was determined through by means of the standard UNE EN 12350-7.

In hardened state it was analyzed from the point of view of its mechanical, environmental and durable properties: From the mechanical point of view, compressive strength tests were carried out following the standard UNE-EN 12390-3.

From an environmental point of view, leaching tests were carried out following the standard EN 12457-2:2002. This test involves immersing the sample at a specified leachant volume in
order to extract the components of the material. The solid residue is separated by filtration and an aliquot of the leachate is taken and analysed following the standards UNE ENV 12506 and UNE ENV 13370.

The durability properties were evaluated by analysis of porosity, pore size distribution and density in two specimens by mercury intrusion porosimetry (MIP). Also, the crystalline phases of the concrete have been analysed in duplicate powdered samples by X-Ray diffraction using a diffractometer model D8 Advance of Bruker AXS. Other indicator of durability is the electrical resistivity, which was measured by means of a commercial resistivimeter. Additionally, two different experiments to evaluate transport properties through the matrix were carried out: the first one was the measurement of the steady and non-steady state chloride diffusion coefficients and the other was capillary absorption. The steady (Ds) and non-steady (Dns) state chloride diffusion coefficient were determined using the multi-regime method [30] standardized as UNE 83987-2009. The water take-up of hardened concrete was determined by capillary absorption. This method was standardized as UNE 83982:2008.

All the procedures followed can be found in the mentioned standards.

4. RESULTS AND DISCUSSION

Step 1: Chemical and mineralogical characterization of the dredged material.
Elemental composition of the original sediment, and on the fractions smaller than 2 mm and smaller than 63 µm (sand + fine and fine fractions) are given in Table 2, where it can be seen that the major component is silicon oxide followed by calcium carbonate in all the different fractions analysed. Loss of material by calcination (LOI, lost of ignition) was also determined heating the sample during 15 minutes at the temperature of 950ºC ± 25ºC.

This composition has been confirmed by X-ray diffraction, where presence of quartz and calcite are the main components. Silicate of Fe and Mg are also present, dolomite, iron titanium sulphate, complex chlorides including different species, iron, and some clay (chamosite) in small proportions have also been identified. Expansive clays of the group of philsolicates (montmorillonite, saponite, nontronite, bentonite sepiolite and palygorskite), have been looked for specifically, not having been identified in the sediment. Therefore, the sediment is compatible with cementitious materials.

Concerning pH of the sediment, 1 gr of sediment suspended in 100 ml deionized water, gives a pH in the solution of 8.5, which, as expected, imply alkaline pH compatible with cementitious materials.

Therefore, it is possible to go ahead to step 2.

Step 2: Hazardous characterisation of the waste.
The dredged sediment is a mineral waste that is not ignitable, corrosive or reactive. So, the point to analyse from a hazardous point of view is the toxicity. There are different recommendations and or environmental limits concerning discharge of dredged sediments to the sea. According to the Spanish recommendations for the management of dredged material in the Spanish harbours [31], toxicologically, the dredged sediments can be divided into three categories in function of their relative values concerning two levels of action: level 1 and level 2:
Category I: their effects on the marine flora and fauna are null or negligible. Therefore, these sediments can be discharged to the sea considering only mechanical effects. For this category, the level of contaminants is smaller than the level of action 1.

Category II: Moderated amount of contaminants. They could be discharged into the sea in a controlled way. For this category, the level of contaminants is higher than the level of action 1 but smaller than the level of action 2.

Category III: Dredged materials with highly contaminated. They must be isolated from the sea or remediated accordingly. For this category, the level of contaminants is higher than the level of action 2.

Table 2. Characterization of the main components of the sediment. Loss of material by calcination/ignition (LOI - 15 minutes at the temperature of 950ºC ± 25ºC).

<table>
<thead>
<tr>
<th></th>
<th>Original sediment</th>
<th>&lt;2 mm</th>
<th>&lt;63µm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>70.6</td>
<td>71.9</td>
<td>68.5</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.1</td>
<td>4.7</td>
<td>5</td>
</tr>
<tr>
<td>FeO</td>
<td>2.3</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td>CaO</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>CaCO₃ %</td>
<td>26.3</td>
<td>27.4</td>
<td>34.2</td>
</tr>
<tr>
<td>MgO</td>
<td>2.3</td>
<td>2.6</td>
<td>2.1</td>
</tr>
<tr>
<td>Na₂O</td>
<td>1.5</td>
<td>1.2</td>
<td>0.93</td>
</tr>
<tr>
<td>K₂O</td>
<td>2.0</td>
<td>1.1</td>
<td>1.6</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.24</td>
<td>0.24</td>
<td>0.43</td>
</tr>
<tr>
<td>SO₂</td>
<td>0.11</td>
<td>0.07</td>
<td>0.11</td>
</tr>
</tbody>
</table>
| Loss of material by calcination | 16         | 16    | 20
| Humidity         | 21.9              |       |       |
| Solid concentration | 78.1            |       |       |
| TOC (%)          |                   | 4.1   |       |

In terms of toxic substances contained in the material, the chemical characterization on pollutants was done over the fraction smaller than 63 µm, where they accumulate. The results obtained by ICP-AES, as well as the level of action 1 and 2 specified in [31], are given in Table 3, were it can be seen that the sediment corresponds to Category I, except in the case of cadmium, with a concentration corresponding to the category of slightly polluted. Additionally, bacteriological analysis on the water supernatant on the sediment gave a total count of Coli-form bacteria: 0 ufc/100ml and also for Faecal streptococcus: 0 ufc/100ml. Therefore, the material can be considered as is not hazardous and does not need to be decontaminated as a previous step in its reutilization procedure.

Step 3: Granulometric characterization.
The granulometric distribution of the dredged sediment has been carried out according to UNE 103-102, by sedimentation. The deposits belong preferably to the sand fraction between 2 mm and 63 µm (94%). The clay and silt fraction reaches 5%. In this figure, the values obtained in the granulometric analysis of the sediment are also compared with the limit values of the fine aggregates for concrete [14] and that of the filler for SCC concretes.
Step 4: Decision on the most suitable material, including the waste, to be produced.
Considering the composition of the sediment, but mainly their granulometric characteristics (beyond the limits of fine aggregates) it has been chosen as a first option, trying to make self compacting concrete, SCC. This decision has also been motivated for the novelty, as seen in the bibliographic search, and for the high added value that it would suppose.

Step 5: Design of the material
First of all, it has to be pointed out that the intention of this step is not to optimise the mix of SCC, but to demonstrate the feasibility of their production with this waste on the basis of their properties. Therefore, provided the granulometry of the sediment, it could be used as a partial substitution of fine aggregates or as quasi-filler, with addition of more fines.

- Trials using the sediment as substitution of fine aggregate.

The design of the sample started replacing 8% of the fine aggregate with dredged sediment, in relation with a conventional concrete (mix A in table 2). The cement type used was I 42.5 R without additions. The particle size was 6/12 for the coarse aggregate and 0/6 for the sand. After several trials for optimization, the concrete obtained contained 420 Kg/m³ of cement with a water/cement ratio of 0.49 and a consistency of 12. In fresh state the behaviour of this concrete was good, however, owing to the high content of fine arid and therefore its high demand of water, it was not so cohesive and not very fluid. This behaviour was confirmed with the data obtained in the compressive strength measured at the age of 28 days. These data show an average value of 34 MPa. Without using the sediment this value is close to 40 MPa.

- Trials using the sediment as filler

<table>
<thead>
<tr>
<th>mg/Kg</th>
<th>&lt;63µm</th>
<th>Level 1</th>
<th>Level 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mercury</td>
<td>0.21</td>
<td>0.6</td>
<td>3.0</td>
</tr>
<tr>
<td>Cadmium</td>
<td>1.5</td>
<td>1.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Lead</td>
<td>11.0</td>
<td>120</td>
<td>600</td>
</tr>
<tr>
<td>Copper</td>
<td>8.5</td>
<td>100</td>
<td>400</td>
</tr>
<tr>
<td>Zinc</td>
<td>150</td>
<td>500</td>
<td>3000</td>
</tr>
<tr>
<td>Chromium</td>
<td>30.5</td>
<td>200</td>
<td>1000</td>
</tr>
<tr>
<td>Nickel</td>
<td>9.5</td>
<td>100</td>
<td>400</td>
</tr>
<tr>
<td>PCB (28, 52, 101 118, 138, 153 y 180 IUPAC)</td>
<td>&lt;0.03</td>
<td>0.03</td>
<td>0.1</td>
</tr>
<tr>
<td>Arsénico</td>
<td>6.6</td>
<td>80</td>
<td>200</td>
</tr>
<tr>
<td>Total PCB</td>
<td>&lt;0.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oils and fats</td>
<td>502</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chlorinated organic pesticides</td>
<td>&lt;0.05 (each)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic extractable compounds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total polycyclic aromatic hydrocarbons (PAHs)</td>
<td>&lt;0.05 (each)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organophosphorus pesticides</td>
<td>&lt;0.5  (each)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tin organic compounds</td>
<td>&lt;0.01 (each)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total petroleum hydrocarbons</td>
<td>&lt;5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
According to the results obtained in the preliminary tests, the possibility or reusing the sediment as a part of the filler instead of using it in the replacement of fines was evaluated. Several tests were done with the same siliceous aggregate previously used, 450 Kg of cement with a water/cement ratio of 0.45 and 80 Kg of dredged material (mix B in Table 1). The behaviour observed for this mixture was very close to the fluidity, with the consistency of a self compacting concrete but without achieving the typical characteristics of this material since the spread diameter was 33 cm and 22 cm using the Abrams cone. Additionally, more tests were done using 450 Kg of cement with a water/cement ratio of 0.45 and 100 Kg of dredged material (mix C in Table 1). The percentage of arid was 56% and 44% of gravel. This third mix contains 1.7% of different admixture so the concrete obtained was extremely cohesive and self-levelling with a spread diameter of 56 cm. So, a self compacting concrete (SCC) was reached.

**Step 6: Characterization of the SCC produced.**

*Fresh state.*

The characterization of the fresh state of the SCC concrete produced was performed by the determination of the consistency, air content, density and fluency. Results are given in Table 4.

<table>
<thead>
<tr>
<th>Air content (%)</th>
<th>3</th>
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<tbody>
<tr>
<td>Density (g/m³)</td>
<td>2.23</td>
</tr>
<tr>
<td>Spread diameter (cm)</td>
<td>56</td>
</tr>
</tbody>
</table>

*Hardened state.*

The SCC produced with dredged sediments were tested after 28 days of curing in a humid chamber from a mechanical, environmental and durability point of view.

**Mechanical tests:** Compressive strength was measured over three concrete samples at the age of 28 days, values obtained are those expected for a SCC developed with standard siliceous filler and they are given in Table 5.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Compressive strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>43.8</td>
</tr>
<tr>
<td>2</td>
<td>45.1</td>
</tr>
<tr>
<td>3</td>
<td>43.7</td>
</tr>
<tr>
<td>Average</td>
<td>44.2</td>
</tr>
</tbody>
</table>

**Environmental tests:** Even though the sediment was previously analysed and classified as Category I [31], leaching of the concrete samples, as concrete is the final material fabricated with the sediment, were carried out according EN 12457-2:2002. This test is important in order to assure no leaching of hazardous metals in the final product. The results obtained, concerning heavy metals, pH and conductivity in the leachate, are given in Table 6, where it can be seen that all data except nickel are below the detection limit of the analytical technique.
Table 6. Results of the leaching tests

<table>
<thead>
<tr>
<th>Leaching test</th>
</tr>
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<tbody>
<tr>
<td>% weight dry material</td>
</tr>
<tr>
<td>Conductivity after the filtration (µS/cm)</td>
</tr>
<tr>
<td>Average temperature (ºC)</td>
</tr>
<tr>
<td>pH</td>
</tr>
<tr>
<td>Leaching L/S (mL/g)</td>
</tr>
<tr>
<td>Cadmium (mg/Kg)</td>
</tr>
<tr>
<td>Chromium (mg/Kg)</td>
</tr>
<tr>
<td>Lead (mg/Kg)</td>
</tr>
<tr>
<td>Copper (mg/Kg)</td>
</tr>
<tr>
<td>Nickel (mg/Kg)</td>
</tr>
<tr>
<td>Zinc (mg/Kg)</td>
</tr>
</tbody>
</table>

All the results of metal leaching expressed in Table 6 are under the limits established in the RD 1481/2001, by which regulates waste disposal by landfill.

Durability tests: Parameters obtained by mercury intrusion porosimetry (total porosity, average pore diameter and bulk density) are given in Table 7, where it can be noticed that the concrete presents an average of total porosity is 9.19 (%vol.), with a small average pore diameter (4V/A) and typical value of density for a SCC concrete.

Table 7. Microstructural characteristics of the SCC including dredged sediment (Mix C).

<table>
<thead>
<tr>
<th>Sample</th>
<th>Total porosity (% vol.)</th>
<th>Average pore diameter (4V/V) (µm)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>8,74</td>
<td>0,0437</td>
<td>2,269</td>
</tr>
<tr>
<td>Sample 2</td>
<td>9,64</td>
<td>0,0394</td>
<td>2,203</td>
</tr>
<tr>
<td>Average</td>
<td>9,19</td>
<td>0,041</td>
<td>2,236</td>
</tr>
<tr>
<td>COV (%)</td>
<td>6,92</td>
<td>7,32</td>
<td>2,08</td>
</tr>
</tbody>
</table>

X-ray diffraction tests were carried out on grounded samples of concrete. The diffraction patterns obtained indicated that with the exception of the positive identification of a silicate of Fe and Mg, and a sulphate of Fe and Ti, no differences with a conventional concrete designed with normal silica filler are found with no evidence of any harmful phase for this material. Other indicator of concrete durability is the electrical resistivity, which was measured using a commercial resistivimeter. Values obtained in two specimens of concrete were, respectively, 49.4 KΩcm and 49.15 KΩcm, corresponding to a high durability concrete.

The transport properties through the matrix were measured by means of the determination of the steady and non-steady state chloride diffusion coefficients and the capillary absorption. The determination of the steady and non-steady state chloride diffusion coefficients is based on the accelerated measurement of the amount of chlorides arriving to the anolyte because of a voltage drop applied to the system.
In Figure 2-a, schematic representation of the set up of the test is presented. In Figure 2-b the evolution of the accumulated Cl- in the anolyte passing through the concrete is given. From the representation in Figure 2-b, the effective and apparent diffusion coefficients can be determined, and from them, the binding factor of the material (alpha), supposed a linear binding. These results are given in Table 8.

![Figure 2](image)

**Table 8.** Chloride diffusion coefficients and binding factor of the SCC (Mix C).

<table>
<thead>
<tr>
<th></th>
<th>Ds (cm²/s)</th>
<th>Dns (cm²/s)</th>
<th>alpha</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 1</td>
<td>1,21E-08</td>
<td>8,28E-08</td>
<td>0,146</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>1,03E-08</td>
<td>1,09E-07</td>
<td>0,094</td>
</tr>
<tr>
<td>Average</td>
<td>1,12E-08</td>
<td>9,61E-08</td>
<td>0,12</td>
</tr>
<tr>
<td>Desv.</td>
<td>1,24E-09</td>
<td>1,89E-08</td>
<td>0,0365</td>
</tr>
<tr>
<td>COV(%)</td>
<td>11,073</td>
<td>19,623</td>
<td>30,366</td>
</tr>
</tbody>
</table>

The water take-up of hardened concrete was determined by capillary absorption. The water absorption of the two specimens of concrete, in percentage in weight, is around 1.7% in weight of sample, which is quite low. The summary of the parameters describing the capillary process is given in Table 9. Values obtained for the resistant of water penetration, capillary suction and effective porosity are similar for both samples, and are indicative of materials with good resistance to the penetration of water.

**Table 9.** Parameters describing the capillary process (Mix C).

<table>
<thead>
<tr>
<th>Capillary absorption</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Average</th>
<th>COV(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>m(s/m²)</td>
<td>7,14 x 10⁸</td>
<td>7,17 x 10⁸</td>
<td>7,15 x 10⁸</td>
<td>0,4</td>
</tr>
<tr>
<td>k(Kg/m² min⁻⁰.⁵)</td>
<td>1,46 x 10⁻³</td>
<td>1,42 x 10⁻³</td>
<td>1,44 x 10⁻³</td>
<td>2,3</td>
</tr>
<tr>
<td>(Kg/m³)</td>
<td>3,91 x 10⁻²</td>
<td>3,80 x 10⁻²</td>
<td>3,85 x 10⁻²</td>
<td>2,1</td>
</tr>
</tbody>
</table>
Results obtained in the durability tests have been evaluated according to the durability indicators described in “Concrete design for a given structure service life” [14]. They are presented in Table 10.

Table 10. Potential durability according to “Concrete design for a given structure service life” [14].

<table>
<thead>
<tr>
<th>Potential durability indicator</th>
<th>Experimental SCC</th>
<th>Average values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total porosity</td>
<td>9,19</td>
<td>High-medium</td>
</tr>
<tr>
<td>Electrical resistivity (KΩcm)</td>
<td>49,3</td>
<td>High</td>
</tr>
<tr>
<td>Coefficient of diffusion (Ds) (cm²/s)</td>
<td>1,12E-08</td>
<td>Medium</td>
</tr>
<tr>
<td>Coefficient of diffusion (Dns) (cm²/s)</td>
<td>9,60E-08</td>
<td>Medium</td>
</tr>
</tbody>
</table>

The low porosity of the concrete and its high electrical resistivity are indicators of a high durability. However, values obtained in the measurement of the steady and non-steady diffusion coefficients indicate a moderated durability. Therefore, concerning environmental compatibility and durability, the main characteristics of the self compacting concrete developed are completely in agreement with those expected for a conventional concrete designed with normal silica filler.

5. CONCLUSIONS

In this paper, a feasibility protocol to assess the possibility of including a waste material in the production of a structural cementitious material has been designed. Then the protocol has been illustrated with the application of it to a real case: a dredged sediment of an Spanish harbour that has resulted feasible to be part of a self compacting concrete (SCC).

6. ACKNOWLEDGMENTS

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The authors also thank the Ministry of Economy and Competitivity the funding provided through the project BIA 2011-25653 “TELEPASSCLOR” granted within the Spanish National Plan of R+D+i.

7. REFERENCES


[14] Spanish Code on Structural Concrete EHE-08.
<table>
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<th>Page</th>
</tr>
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