ASSESSMENT OF FIRE EXPOSED CONCRETE WITH FULL-FIELD STRAIN DETERMINATION

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Abstract

A concrete structure exposed to fire deteriorates when the temperature increase in the structure. An experimental study has been performed in order to evaluate the degree of degradation of concrete exposed to two different fire scenarios. As the thermal diffusivity of concrete is low, high thermal gradient is induced in the cross-section. This causes inhomogeneous mechanical properties of a concrete structure. In traditionally core testing of the elastic modulus and compressive the material is assumed to be homogeneous, this is not the case when concrete has been exposed to a real fire. By using an optical full-field strain measuring device the mechanical response at different depth, from the fire exposed surface, can be studied. In this study a typical concrete mix for civil engineer applications were used. In addition a similar concrete mix with reduced aggregate size was tested. The test samples were exposed to the standard fire curve ISO 834-1 or a temperature rise of 10 °C/min. In addition, Ultrasonic Pulse Velocity measurement and PFM Microscopy were conducted in order obtain a reliable picture of the residual mechanical properties and the durability.

1. INTRODUCTION

Concrete is one of the main building materials in society today. It is cost efficient, durable and usually performs well during fire exposure. Despite this there will always be combinations of certain constructions and fire scenarios when the concrete structure is seriously damaged and in extreme cases collapses. After a fire incident it has to be decided whether the construction can be refurbished or, in extreme cases, needs to be replaced. The choice of action will be based on an assessment of the status of the structure. This mapping of damages during the assessment needs to be accurate to ensure a good safety level but also the best solution from an economic point of view. There is several test methods available for both on site and of site measurement of fire exposed concrete [1]. The Rebound Hammer can be used to measure the surface hardness of the exposed surface, but the measure has no direct relationship to the strength. By measuring the ultrasonic pulse velocity through the fire exposed concrete the thermal damage can be assessed, since the ultrasonic pulse velocity is
mainly influenced by the dynamical elastic modulus and the density of the material [1][10]. Consequently, the decrease of the ultrasonic pulse velocity is emphasized at relatively low temperatures due to the initial free water loss [1]. An innovative method to assess fire damage is to use a specially equipped ordinary drill hammer [2]. By measuring the dissipated work per unit drilling (specific work, J/mm) the damaged areas can be located, since the concrete at deeper regions are unaffected and used as reference. This method can detect severe fire damage, such as regions with a temperature history above 500 °C which correspond to a decay of 50-70 % strength of the virgin compressive strength. One more method is to measuring the force needed to extract a metal insert from a concrete structure, a pullout test, and by this estimate the compressive strength [3]. Unlike most of the other methods this method measures an actual strength property of the material, but the method will not give any information of the depth of the damage. The strength profile can be obtained by using the Windsor probe method on surfaces cut to different depth of the exposed structure [4]. Microscopy is often used for the assessment of the temperature history of a fire damaged concrete structure. A comprehensive discussion of useful and potentially useful reactions is presented by Nijland and Larbi [5]. At increased concrete temperatures it is known that chemo-physical transformations take place. These transformations can be assessed by studying the colour changes of the components along the cross-section. Felicetti uses pictures from and ordinary low cost digital camera for colour analyses [6]. The change in colour of the aggregate and the cement past is not directly related to a change in mechanical properties but the occurrence of colour change indicates a temperature range where the mechanical properties may start to decrease. The focus of this project is the assessment of the depth and degree of deterioration of concrete properties due to fire exposure. This includes the application of a new method, a full-field strain measuring technique. By recording the deformation field with a high resolution camera system on a core sample, taken from a fire exposed concrete, during loading a picture of the degradation in the cross-section can be monitored. The results from this new method are directly coupled to the mechanical properties of the concrete in contrast to the majority of traditional methods that are indirect. This method was complimented by Ultrasonic Pulse Velocity and PFM Microscopy in the study.

2. EXPERIMENTAL STUDY

An experimental study has been performed in order to evaluate fire exposed concrete with a full-field strain measuring technique, Ultrasonic Pulse Velocity measurements and PFM Microscopy [7]. Two different fire scenarios and two concrete mixes were used in this study. In total ten test samples were produced and eight of them were exposed to fire conditions. The remaining test samples were used as references.

2.1 Materials and test specimens

A typical Swedish tunnel concrete was used in this study. In addition a similar concrete mix with reduced aggregate size was used. This choice of test material allows an investigation of the influence of aggregate size on the degradation of the fire exposed concrete. The concrete contained polypropylene fibres (PP-fibre) in order to avoid spalling at the fire exposed surface. To be able to add a realistic amount of super plasticizer in the concrete with reduced aggregate size the water-cement ratio (w/c) was increased compared to the original mix. The concrete mixes used are shown in Table 1. The aggregate size fraction 0-8 mm used was a natural sand composed of quartz, feldspar particles and rock fragments of granitic composition. The feldspar was often sericitic and often contained iron oxides that gave a reddish tint. Further, the biotites had a strong brown reddish colour. The granitic particles often contained strongly deformed quartz. Particles of diabase and amphibolites
were present in low numbers. The aggregate in the size fraction 8-16 mm was composed of crushed rock particles of granitic composition, mostly with equilibrium texture but also more deformed varieties. Feldspars were in some cases sericitic and contained to a lesser extent iron oxide ex-solutions. The colour of biotites was mostly green.

Table 1: Concrete mixes used in the experimental study.

<table>
<thead>
<tr>
<th>Series</th>
<th>w/c</th>
<th>Gravel 0-8 mm [kg/m³]</th>
<th>Gravel 0-16 mm [kg/m³]</th>
<th>Water [kg/m³]</th>
<th>Cement CEM [kg/m³]</th>
<th>Super-Plasticizer [kg/m³]</th>
<th>Fibre amount [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-16</td>
<td>0.45</td>
<td>898.5</td>
<td>863.3</td>
<td>180.9</td>
<td>402.8</td>
<td>0.16%</td>
<td>1.0</td>
</tr>
<tr>
<td>0-8</td>
<td>0.47</td>
<td>1637.8</td>
<td>-</td>
<td>181.1</td>
<td>385.5</td>
<td>0.72%</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Ten slabs, five of each recipe, were moulded with a size of: 600 × 500 × 200 mm³. The samples were cured indoors for approximately 6 month after moulding.

2.2 Fire exposure and external load

The fire exposure of the test samples was conducted in a small scale furnace constructed for fire resistance tests with the fire exposed area 500 × 400 mm². Detailed information concerning the construction of the furnace can be found in test method SP Fire 119 [8]. The two fire scenarios used were the standard time-temperature cure described in ISO 834-1 [9] and a slow heating temperature ramp of 10 °C/min to a maximum temperature of 1000 °C, further designated Std and Sh, respectively. Five thermocouples were pre-installed at the centre of the slabs and at a depth of 10, 30, 45, 80 and 120 mm, respectively. The duration of the two different heating scenarios were selected to reach the same temperature level at 45 mm from the fire exposed surface. After estimation by calculation of the temperature at this level the duration of standard time-temperature curve scenario was set to 90 minutes and the duration of the slow heating scenario was set to 130 minutes. This gave a temperature of approximately 310 °C at the position 45 mm at the termination of the both fire scenarios. A one dimensional compressive load was applied in the longitudinal direction of the slabs. Two rigid beams were placed at the short sides of the slabs and were pressed against the slab by two bolts to a load level of 2 MPa. Directly after the termination of the fire tests the slabs were removed from the furnace and were cool down in room temperature.

2.5 Ultrasonic pulse transmission time

The ultrasonic pulse transmission time was measured in the radial direction in the cores at different depths from the exposed side. In general the values descend away from the fire exposed side of the samples as the material becomes stiffer. At the unexposed side of the exposed cores and the virgin core the readings are almost the same. Generally, the degradation of the transmission times through the cores exposed to the standard time-temperature curve is slightly higher compared to the slow heating scenario.

2.6 Mechanical testing

The uniaxial compression tests of the drilled concrete cores were carried out in a GCTS servo hydraulic testing machine, see Figure 1. The load was applied with a stress rate of 12 MPa/min. The axial load was recorded by a load cell and the axial displacement of the axial actuator was recorded by an LVDT, connected to a high-speed data logger. The uncertainty of the load measurement is less than 1 %. The nominal core diameter was 60 mm and the nominal core height was 122.5 mm. The first 10 mm of the drilled cores was removed due to severe damage from the fire exposure. The core end surface closest to the fire exposure, or corresponding surface of the unexposed cores, was placed against the lower loading plate. During the loading the strain-field was monitored at the surface of the cores by means of full-field strain measurement.
Full-field strain measurement was performed on all concrete cores tested in this study. The optical full-field deformation measurement system ARAMISTM 4M (v6.2.0-6) by GOM was used. The system uses a measurement technique based on Digital Image Correlation (DIC) with a stereoscopic camera setup, consisting of two CCD-cameras with 4.0 Mega pixel resolutions. Essentially, DIC measures the displacement of the specimen under testing by tracking the deformation of a naturally occurring, or applied surface speckle pattern in a series of digital images acquired during the loading. This is done by analysing the displacement of the pattern within discretized pixel subsets or facet elements of the image. In combination with correlation based stereovision techniques the measurement of 3D shapes as well as the measurement of 3D displacements fields and surface strain field, is possible. The natural pattern at the surface of the drilled cores was used as the speckle pattern. In the tests an image pair was captured with a frequency of 1 Hz; at the same time the load and displacement, obtained from the testing machine, were recorded in the ARAMIS system. The accuracy of coordinate measurements was approximately 2 µm. The DIC measuring area was divided into nine 10 mm thick segments defined by ten equally spaced sections along the axis of the core. The first section was located approximately 5 mm from the bottom surface of the core, which should be compared to 20 mm from the fire exposed surface to the centre of the first segment. The axial displacement of each facet element along the sections was exported from the ARAMIS system. The strain in each segment $\epsilon_{cs}^{m-n}$ was then calculated as the difference between the mean values of the axial displacement $\delta_{m}^{n}$ and $\delta_{m}$ of the corresponding sections n and m, respectively, divided by the initial distance $l_0$ between the sections as:

$$\epsilon_{cs}^{m-n} = \frac{\delta_{m}^{n} - \delta_{m}}{l_0}$$

(1)

The mean axial compressive segment strain $\epsilon_{cs,m}$ evaluated according to this method is presented for a stress level of 20 MPa in Figure 2. The stress level was chosen so that all specimens were within elastic response. Some scatter was found between the segment values within the same group of specimens. This might be an effect of variations within the concrete slabs.
2.6 Microscopy

The analysis was performed using thin sections with the approximate size of 50 × 65 mm². The samples were impregnated with epoxy glue with fluorescent dye, Struers Epodye. Light optical microscopy with bright field, polarization and fluorescence, PFM, technique was applied. The thin sections covered an area about 65 mm in from the fire exposed surface. The crack counting analyses was conducted in a light microscope at 50 times magnification. The analysed thin sections were attached to a motorized table. Micro-cracks and macro-cracks were counted at 6 levels, each level was 10 mm high, and starting approximately 5 mm from the fire exposed side of the sample. The crack counting, in the cement paste and at the ITZ, was conducted in five fields per level. The analysis was conducted as a linear traverse analysis along lines perpendicular and parallel to the exposed surface, respectively, in order to assess the orientation of the cracks. The microscopic analysis shows that thermal alterations in samples from the slow heating and the standard fire are rather similar, see Table 2. For the samples containing 0-8 mm and 0-16 mm aggregates, respectively, there is a difference in the relationship between the depth to the portlandite and quartz transitions. The portlandite reaction is dependent on both temperature and water pressure. Results from the quantitative analysis of cracks indicate that thermally induced cracks are more frequent in the samples with coarser aggregate.

Table 2: The estimated depth in mm from the fire exposed surface to different changes

<table>
<thead>
<tr>
<th></th>
<th>Portlandite ((\text{Ca(OH)}_2))</th>
<th>Quartz (\alpha) to (\beta) transition</th>
<th>Reddish cement paste</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8 SH-4</td>
<td>19-20 mm</td>
<td>21 mm</td>
<td>4-12 mm</td>
</tr>
<tr>
<td>0-8 Std-1</td>
<td>11-14 mm</td>
<td>13-16 mm</td>
<td>0.5-12 mm</td>
</tr>
<tr>
<td>0-16 SH-1</td>
<td>15-18 mm</td>
<td>26 mm</td>
<td>3-12 mm</td>
</tr>
<tr>
<td>0-16 Std-1</td>
<td>14-17 mm</td>
<td>25 mm</td>
<td>2-13 mm</td>
</tr>
<tr>
<td>0-16 Std-8</td>
<td>13-15 mm</td>
<td>25 mm</td>
<td>1-14 mm</td>
</tr>
</tbody>
</table>

The quantification of crack frequencies gives a similar pattern in all samples. There is a high crack frequency near the surface, a minimum at about 15 mm depth and a maximum again at about 25 mm. This pattern is specific for the combination of material and fire scenario and cannot be transferred to a different situation. Moreover, the results indicate that thermally induced cracks are more frequent in the samples with coarser aggregate.

3. DISCUSSION AND CONCLUSIONS

Both the full-field strain measurement and the ultrasonic pulse transmission time measurement are showing a significant degradation at the fire exposed side of the tested
To relate the residual stiffness of the fire exposed specimens to the unexposed concrete a stiffness reduction factor was introduced. As the mean axial stress level is the same in all specimens, the stiffness reduction factor of each segment can directly be related to the segment strain as:

\[
k_{c,s} = \frac{\varepsilon_{c,sm}^{\text{unexposed}}}{\varepsilon_{c,sm}^{\text{exposed}}}
\]

where \(\varepsilon_{c,sm}^{\text{unexposed}}\) and \(\varepsilon_{c,sm}^{\text{exposed}}\) are the mean values of the segment section strain of the specimens from the unexposed and fire exposed slabs, respectively. The mean stiffness reduction factors for the fire exposed slabs are presented in Figure 3. The stiffness reduction factor gives a picture of the damage level within the concrete slabs as a result of a temperature profile caused by the fire heating scenario. The stiffness degradation of 0-8 Std and 0-8 Sh coincides well, while there is a rather big difference between 0-16 Std and 0-16 Sh with increased distance to the fire exposed surface. The explanation for the latter difference has not yet been found, but one explanation could be that the scatter in strain values for each segment is larger for the 0-16 specimens than for the 0-8 specimens. The slabs with 0-16 mm aggregates exhibit larger stiffness reduction close to the surface for both fire scenarios, compared to the slabs with 0-8 mm aggregates. At a depth of 100 mm the reduction factor is around 0.6 to 0.7 for 0-16 Std, 0-8 Std and 0-8 Sh, while it is above 1.0 for 0-16 Sh. Values above 1.0 should not be physically possible and are probably explained by natural scatter and measurement uncertainties. The dynamic modulus of elasticity is a function of the density, the ultrasonic pulse velocity and the Poisson’s ratio [10]. To estimate the reduction of the dynamic modulus of elasticity a reduction factor was introduced analogous to the stiffness reduction factor.

\[
R_{E_{\text{dyn}}} = \frac{E_{\text{dy,after fire}}}{E_{\text{dy,before fire}}} = \frac{\rho_{\text{after fire}}}{\rho_{\text{before fire}}} \left(\frac{t_{\text{before fire}}}{t_{\text{after fire}}}\right)^2 \cdot \frac{(1 + \nu_{\text{after fire}}) \cdot (1 - 2\nu_{\text{after fire}}) \cdot (1 - \nu_{\text{before fire}})}{(1 - \nu_{\text{after fire}}) \cdot (1 + \nu_{\text{before fire}}) \cdot (1 - 2\nu_{\text{before fire}})}
\]

In this study only the change in ultrasonic pulse velocity was considered when calculating this reduction factor. A loss in density will lower the relative dynamic modulus of elasticity. At low temperatures the reduction of the stiffness is overestimated due to the initial free water loss [1]. The curves may therefore deviate in regions where the temperature rise was low and the moisture content was considerably affected by the fire. Since a change in Poisson’s ratio of the concrete has low influence on the relative dynamic modulus of elasticity this factor was not considered either. In Figure 3 the dynamic modulus of elasticity reduction factor (R), considering only the change in ultrasonic pulse velocity, are presented together with the stiffness reduction factor obtained from the DIC measurements. The reduction factor, R, for the 0-8 specimens coincides better than the 0-16 specimens, as discussed previously; one explanation can be the larger scatter in the 0-16 strain measurement than the 0-8 strain measurements. The depth away from the fire exposed surface where the crack frequency decreases agrees with a strong change in mechanical properties shown by the DIC measurements and the ultrasonic pulse velocity decay. The change in mechanical properties cannot be explained only by the formation of cracks. This is shown by the crack distribution in the 0-8 mm samples where the crack frequency at 55 mm depth is comparable to the background values for samples that have not been exposed to a fire while the DIC results indicate a reduction in stiffness of about 50% at this level.
Decomposition of portlandite occurs closer to the surface and therefore at higher temperature than the quartz transition which is reversed to the actual reaction temperatures. Furthermore, the portlandite decomposition occurs further away from the exposed surface in the slow heated samples. This may be due either to the kinetics of the reaction or its water pressure dependence, i.e. if the water vapour pressure is higher this decomposition reaction will occur at a higher temperature. The quartz transition on the other hand is pressure independent and is a rapid transition that takes place as soon as the reaction temperature is reached. In the present study the level for the portlandite decomposition given is where the portlandite is entirely consumed. The results imply that it may be more appropriate to identify the level for the onset of the reaction.

A large variation of experimentally determined stiffness reduction of heated concrete is found in the literature. Most of the results were found in the region between two simple linear models where the stiffness reduction factor goes from 1 to 0 in the temperature range of 20 – 600°C and 20 – 800°C, respectively. In this study there is a better agreement between the test results and the 20 – 600°C model, see Figure 4. However, at lower temperatures the model seems to underestimate the stiffness degradation for 0-16 Std, 0-16 Sh and 0-8 Sh. The reason for this might be the development of a crack system in the virgin part of the cross-section caused by extensive thermal expansion closer to the surface. This leads to a bowing of the structure with associated crack formation. In general during fire tests on slabs, cracks open up on the non-fire exposed side of the specimen, often followed by water pouring out in the cracks. During tests of E-modulus at high temperatures the test specimens are heated slowly to reduce effects from high thermal gradients (RILEM TC 129 MHT, 2004).
Ultrasonic pulse velocity measurements on core samples provide information concerning the depth of damage. On site measurements allow the operator to increase the number of core samples or cancel planned drilling depending on the from previous cores results. By using the DIC measuring technique on the cores samples a reliable picture of the damage is obtained. To determine the maximum temperature occurred at the reinforcement the depth of the quartz transition can be studied in microscope. This transition is rapid and pressure independent. The structure may deteriorate after fire by frost or corrosion of the reinforcement if the fire has caused an increased porosity and a high amount of cracks at the surface layer. An estimation of the durability can be done by counting the cracks along the cross-section in microscope. Combining these test methods an accurate picture of the fire damage is obtained. The structural engineer can determine required strengthening actions to retrieve the load carrying capacity and the durability of the concrete structure.

REFERENCES