

High Strength Concrete at High Temperature

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Synopsis: Effects of elevated temperature exposure and various factors, including water-to-cementitious material ratios (w/cm), curing conditions, heating rates, test methods, and polypropylene (PP) fibers, on (1) pore pressure buildup and potential for explosive spalling and on (2) degradation of mechanical properties in normal-strength (NSC) and high-strength concrete (HSC) are presented. Degradations of mechanical properties were measured using 100 x 200 mm cylinders, heated to temperatures of up to 600 °C at 5 °C/min, and compared with results of other studies and existing codes. Pore pressures were measured using 100 x 200 x 200 mm blocks, heated to 600 °C at 5 °C/min and 25 °C/min. Experimental evidences of the complex, temperature-dependant moisture transport process that significantly influenced pore pressure and temperature developments are described.

Keywords: elevated temperatures; explosive spalling; high-strength concrete; polypropylene fibers; pore pressure

INTRODUCTION

It is now well known that short-term exposure to elevated temperature affects properties and behavior of HSC and NSC differently, and the differences are in two main areas: (1) strength loss in the intermediate temperature range (100 °C to 400 °C); and (2) occurrence of explosive spalling in HSC at similar range of temperatures (Castillo and Durrani, 1990; Phan, 2005; Hertz, 1984). Both of these material behavioral differences have structural implications. First, the difference in relative strength loss between HSC and NSC at elevated temperature raise questions as to whether existing design rules prescribed in current codes for determining concrete strength at elevated temperatures, which were based primarily on tests of NSC, are applicable or safe for structures built with HSC. Second, the occurrence of explosive spalling at relatively low temperature, frequently observed for HSC, means HSC structural elements might sustain loss of concrete cover protecting the steel reinforcements early in a fire, causing the steel reinforcements to be exposed directly to heating and leading to premature loss of overall structural capacity.

Explosive spalling, which refers to a sudden and violent breaking away of a surface layer concrete, has been observed in many laboratory tests of HSC specimens (Diederichs et al., 1995; Castillo and Durrani, 1990; Phan, 2005; Phan and Carino, 2003, 2002; Hertz, 1984; Kodur, 2003; Kalifa et al., 2000). This phenomenon has been generally attributed to the inability of HSC, due its low permeability, to successfully mitigate the buildup of internal pressure as free water residing in the concrete pores and chemically-bound water in the concrete matrix undergo transformation from liquid to a gaseous phase, expand in volume, and transport through the concrete with increasing concrete temperature.

Reports on factors affecting the degradation of HSC's mechanical properties and tendency for explosive spalling have been inconsistent (and sometimes contradictory), and a complete understanding of how these factors influence the behavior of HSC at elevated temperature has not had the benefit of quantifiable measurements (Phan, 2005; Phan and Carino, 2002; Diederichs et al., 1995; Kalifa et al., 2000; Khoury and Algar, 1999). Given (1) the many benefits of HSC and its increased use in structural applications, and (2) the ongoing efforts to model the spalling behavior of HSC in fire, it is essential that behavior of HSC at elevated temperatures be understood to ensure that structural fire design involving HSC will be safe.

RESEARCH SIGNIFICANCE

The research results reported herein represent a unique set of experimental measurements that provides insight into the effects of high temperature exposure on compressive strength of normal and high strength concrete and on the developments of temperature and corresponding pore pressure in concrete and on the occurrence of explosive spalling. The results provide clear, quantifiable explanation for the complex heat-induced mass and moisture transport and explosive spalling phenomena, allow evaluation of factors contributing to these phenomena and of the efficacy of PP fibers for spalling mitigation. The results also provide a comparison with existing code provisions for concrete compressive strength-temperature relationships as well as a rare set of data for validation of analytical models for predicting pore pressure and spalling in concrete.

EXPERIMENTAL PROGRAM

The study includes two series of experiments. One measured concrete mechanical properties at elevated temperatures using 100 x 200 mm cylinders. The other measured the heat-induced internal pore pressures in concrete using 100 x 200 x 200 mm blocks.

In the mechanical property test series, the specimens were tested at steady-state temperature conditions using three test methods, namely *stressed*, *unstressed*, and *unstressed residual property* test methods. In the *stressed* and *unstressed* test methods, the specimens were subjected to simultaneous application of loading and heating and loaded to failure under uniaxial compression *at elevated temperatures*. In the *unstressed residual property* test method, the specimens were heated to target temperatures, allowed to cool to room temperature, and then loaded to failure *at room temperature*. In addition, *stressed* test specimens were restrained by a preload equal to 40 percent of their room-temperature compressive strength ($0.4f_{23^\circ\text{C}}$) prior to and throughout the heating process (the *unstressed* and *unstressed residual property* test specimens were heated without preload).

In the pore pressure test series, the specimens were insulated on all sides, except the face, to simulate the one-dimensional heat flow condition that exists in a concrete wall exposed to fire, and were subjected only to heating without any mechanical loading.

4 Phan

Specimen Materials

The specimens of both test series were made using the same materials and mixture proportions (ASTM Type I Portland cement, 13 mm maximum size crushed limestone aggregate, natural sand with a fineness modulus of 2.85, silica fume (in the form of a slurry with a density of 1.42 g/cm^3 and a 54 % silica fume concentration), and high range water-reducing admixture). For the mechanical property test series, the specimens were made using four mixture proportions, designated mixtures I to IV (mixtures I to III are HSC, mixture IV is NSC in [Table 1](#)). For the pore pressure test series, the specimens were made using two mixtures, I and IV (lowest and highest w/cm ratios). Some of the pore pressure specimens also contained commercially available monofilament PP fibers of two different lengths, 13 mm and 38 mm. The 13-mm long fiber has a diameter of $100 \text{ }\mu\text{m}$, and the 38-mm long fiber has a diameter of $300 \text{ }\mu\text{m}$. Both types of fibers have the same aspect ratio (125:1).

Instrumentation, Test Setup, and Heating Regimes

All specimens of the mechanical property test series were cured under water at room temperature until test time. Before testing, each specimen was instrumented with two type K thermocouples, one on the specimen's surface and one at the specimen's center. The thermocouples were used to determine whether the steady-state temperature condition has been achieved in the specimen (defined as the temperature condition where the difference between the temperatures on the specimen surface and center is within $\pm 5 \text{ }^\circ\text{C}$). [Fig. 1](#) shows the typical test setup for the mechanical property test series. The specimen was placed at the center of the electric split-tube furnace with openings at the top and bottom to allow the loading rams to transmit compressive load from the test machine. For the *unstressed residual property* tests, the cylinders were heated separately and allowed to cool down to room temperature before loading to failure (24 hours of natural cooling after heating).

[Fig. 2\(a\)](#) shows the test setup for the pore pressure test series. Prior to heating, the pore pressure test specimen was insulated on all sides with an insulation blanket except on the heated face. The specimen was then placed at the center of a programmable electric furnace and subjected to heating by convection. The insulation promoted heat flow through the specimen, from the heated face to the insulated back face, similar to the one-directional heating condition that a concrete wall or slab would experience in a fire. Pore pressure specimens were instrumented with specialized pore pressure gages and also with type K thermocouples, which were co-located at points 25 mm, 50

mm, and 75 mm deep from the specimen's heated face. In addition, thermocouples were also placed on the front and back surfaces of the specimen. The pore pressure gages were made of a porous metal disc (1 mm thick with diameter of either 6 mm or 12 mm) crimped into a stainless steel cup of the same diameter (see **Fig. 2(b)**). The stainless steel cup was welded to a stainless steel tube with an inner diameter of 1.6 mm. During concrete casting, the stainless steel cup (with the porous metal disc) was set at the desired depth in the concrete with the tube extending out of the backside of the specimen. The embedded stainless steel cup with porous metal disc allows the vapor pressure inside the concrete specimen to be transmitted through the porous metal disc and the tube. Before testing, the tube, which extends out of the specimen and the furnace, was connected to a miniature pressure transducer outside the furnace. The pressure transducers have a pressure range of 0 – 1000 psig and an uncertainty of $\pm 1\%$. The entire pore pressure assembly was filled by vacuum with high temperature silicone oil to facilitate the transfer of any vapor pressure developed inside the concrete to the miniature transducer.

Two heating profiles - both impose a maximum furnace air temperature of 600 °C but at two different heating rates: 5 °C/min and 25 °C/min - were selected for the pore pressure test series. The test is terminated after explosive spalling occurs or after 5 hours of heating exposure, whichever occurs first.

Test Variables

The mechanical property test series was designed to study the effects of (1) test conditions (stressed, unstressed, and unstressed residual property tests); (2) w/cm ratios (0.22, 0.33, and 0.57); and (3) silica fume contents (0 % or 10 % cement replacement by mass) on mechanical properties of NSC and HSC at elevated temperature.

The pore pressure test series was designed to study the effects of (1) w/cm ratios; (2) dosages of PP fiber (0, 1.5 kg/m³, and 3 kg/m³); (3) PP fiber lengths (13 mm and 38 mm); (4) heating rates (5 °C/min and 25 °C/min); and (5) curing conditions (moist-cured by submergence and air-dried) on pore pressure buildup and potential for spalling of NSC and HSC. **Table 2** shows the test matrix and key parameters of each of the pore pressure test specimen. Individual test specimens were named using the following convention:

Mixture # - Fiber Dosage - Fiber Length - Curing Condition - Heating Rate

(I or IV) (0, 1.5, or 3 kg/m³) (13 or 38 mm) (Moist- or Air-cured) (5 or 25 °C/min)

EXPERIMENTAL RESULTS AND DISCUSSIONS

Mechanical Property Test Series

Strength-Temperature Relationships

The results of the mechanical property test series are shown as relative compressive strength ($f/f_{23^\circ\text{C}}$) versus temperature relationships in **Fig. 3**. Also shown in these Figures are results of other studies for comparison purposes. In these Figures, the results of HSC are shown as solid lines, and the results of NSC are shown as dashed lines. The ranges of NIST test results are shaded for convenience.

Fig. 3(a) shows the relative strength – temperature relationships for HSC and NSC under the stressed test method. The NIST test results showed that HSC sustained an average strength loss of slightly above 25 % at 100 °C. This is followed by a minor strength recovery when concrete reaches temperature between 100 °C and 450 °C (maximum strength recovery occurs at 300 °C). At temperature between 450 °C and 600 °C, the temperature rate of strength loss becomes more significant. Also, at temperature above 450 °C, all specimens of mixtures II and III failed due to explosive spalling (while being heated to 600 °C), resulting in unavailability of data for these two mixtures at above 450 °C. The NIST test results are consistent with results by Castillo and Durrani (1990) at up to 300 °C, and with results by Khoury and Algar (1999) at 100 °C. Results by Abrams (1971), however, indicated significantly different strength-temperature relationships for NSC. For NSC with siliceous aggregate, test results by Abrams indicated that there was a slight strength gain at temperature up to 400 °C, and strength loss was not observed until concrete reaches temperature above 400 °C. For NSC with carbonate aggregate, no significant effect on strength due to temperature exposure was observed until 650 °C. Castillo and Durrani (1990) reported explosive spalling in about one third of the specimens being heated to 700 °C while Khoury and Algar (1999) did not mention explosive spalling. Further data may be found in Phan (2005, 2004) and Phan and Carino (2003, 2002).

Fig. 3(b) shows the strength-temperature relationships for HSC and NSC under the unstressed test method. These relationships shown in the NIST's unstressed test data are similar in trend with those of the NIST's stressed test data, except that the strength losses in the unstressed tests are slightly larger at each target temperature. The NIST results also followed the general trend of unstressed HSC tests reported by Hammer (1995) and Diederichs et al.

(1995), which constituted the majority of available unstressed test data for HSC. Almost all studies reported higher strength loss for HSC compared to NSC within the temperature range of 100 °C to 450 °C. At temperature above 450 °C, both HSC and NSC sustained similar rate of strength loss. More incidences of explosive spalling were observed in the NIST study under the unstressed test method. As a result, NIST mixture I has no strength data above 300 °C, and mixtures II and III have no strength data above 450 °C. Explosive spalling did not occur in any of the mixture IV specimens. Diederichs et al. (1995), Hammer (1995), and Furumura et al. (1995) also reported failure due to explosive spalling of their unstressed HSC specimens, even though some of these studies used very low heating rates (1 °C/min for Furumura and 2 °C/min for Hammer). Study by Castillo and Durrani (1990), however, indicated that explosive spalling occurred only in their stressed test specimens and none occurred under unstressed test method

Fig. 3(c) shows the strength-temperature relationships for the unstressed residual property test. NIST test data showed a wider range of strength loss between the four mixtures under this method than in the stressed or unstressed test methods. The NIST test results at less than 200 °C also differed from data for both HSC and NSC in other studies. The largest difference is at 100 °C, at which the NIST results showed a strength loss ranging between 10 % to 30 %, while data from other studies showed either a strength gain or loss of a little more than 5 %. Above 300 °C, the average difference in relative strengths between test programs appeared to be similar. Further, Explosive spalling occurred in one (out of five) NIST mixture I and one (out of four) mixture II specimens while they were being heated to 300 °C. Explosive spalling also occurred in all mixture I specimens while they were being heated to 450 °C. Again, spalling did not occur in any of the NIST mixture IV specimen. Of the referenced studies, only that of Hertz (1984), which used ultra high strength concrete, reported explosive spalling in the unstressed residual property test.

Comparisons with Codes

The above test data are compared with the provisions for concrete strength at elevated temperature by existing codes. Among the codes which specify design rules for computing concrete strength at elevated temperature are the Comité Européen de Normalisation CEN EN 1992 (2004), the Comites Euro-International du Beton CEB (1991), and the National Building Code of Finland's RakMK B4 (1991).

8 Phan

The CEN EN 1992 and the CEB did not explicitly prescribe whether their design rules were specified for concrete in service (i.e. concrete under service load), however it is assumed that this is the case since both codes are for the design of structural concrete. Thus, CEN EN 1992 and CEB's design provisions will be compared with the data of the stressed tests.

Finland's *RakMK B4* prescribes different design rules for HSC (with 70 MPa to 100 MPa compressive strength based on 150 mm cubes, or 62 MPa to 90 MPa based on 150 x 300 mm cylinders) and NSC (concretes with 10 MPa to 70 MPa compressive strength based on 150 mm cubes, or 7 MPa to 62 MPa based on 150 x 300 mm cylinders). The *RakMK B4* also prescribes different design rules for concrete in service (stressed, $0.3 f_{23^\circ\text{C}}$) and for concrete which is not (unstressed). Thus, the applicability of *RakMK B4* will be assessed by comparing with both the *stressed* and *unstressed* test data. Also, comparisons with unstressed residual strength tests will not be made here since the code provisions were prescribed for concrete strength "at" elevated temperature and not for concrete strength at room temperature after exposure to elevated temperature.

Fig. 4(a) shows the strength-temperature relationships obtained under the stressed test method in the NIST test program and in studies by Castillo and Durrani (1990), Khoury and Algar (1999) and Abrams (1971). The design rules prescribed by the CEN EN 1992 (for Class 2 (70 MPa to 95 MPa compressive strength) and Class 3 (90 MPa to 105 MPa compressive strength) concrete) and by the CEB are superposed over the measured strength-temperature relationships for comparison.

As shown in **Fig. 4(a)**, the strength-temperature relationship for CEN EN 1992's Class 2 HSC is very consistent with results of NIST, Castillo and Durrani, and Khoury and Algar tests, while the strength-temperature relationship for CEN EN 1992's Class 3 HSC is more conservative when compared with the same test results. For the CEB, the strength-temperature provisions were based on NSC test data and found to be unconservative when used for estimating HSC compressive strength at temperatures less than 350 °C. The largest overestimation by CEB is by about 25 % at temperatures less than 200 °C. The CEB's rate of strength loss at temperature above 350 °C is consistent with data for both HSC and NSC. The unconservative estimation of HSC compressive strength by CEB at temperatures less than 350 °C is more significant since it does not address the explosive spalling problem in this

temperature range. The RakMK B4's provision for in-service concrete (stressed) with K10 to K70 strength grades (NSC) is consistent with the stressed test data for NSC up to 800 °C. The RakMK B4's provision for concrete with K70 to K100 strength grades (HSC) appears to be slightly unconservative at temperatures below 150 °C. However, this is to a much lesser degree compared with the CEB. In the intermediate temperature range (150 °C and 350 °C), the RakMK B4's provision for in-service HSC is consistent with the NIST test data and the provision for CEN EN 1992 Class 2 HSC. From 350 °C to 800 °C, the RakMK B4 provision appears to be conservative compared with all test data, and is similar to the strength estimation prescribed by CEB.

Fig. 4(b) shows the strength-temperature relationships obtained under the unstressed test method and the RakMK B4's strength-temperature relationships for HSC and NSC. Similar to the stressed test, the RakMK B4's strength provision for HSC appears to be slightly unconservative at temperatures below 150 °C with respect to the NIST's test data. However, at temperatures above 150 °C, the RakMK B4's strength predictions are consistent with the NIST test data. The RakMK B4's strength prediction for unstressed NSC prescribes a range of no strength loss between room temperature and 220 °C. This is consistent with the average results of tests by Castillo and Durrani (1990), Abrams (1971), and Diederichs et al. (1995). Above 220 °C, RakMK B4 prescribes a strength loss period that has a similar rate of strength reduction as in the case for HSC. The prescribed strength loss is on the conservative side of test results for NSC.

Pore Pressure Test Series

Figs 5 (a) to (e) show the results for a typical pore pressure specimen, in this case specimen I-1.5-13-M-5 (mix I with $w/cm = 0.22$, contained 1.5 kg/m³ of 13-mm long PP fibers, was moist cured and heated at 5°C/min). The specimen survived the heating exposure without spalling. The results shown in **Fig. 5** include: (1) pore pressures at points 25 mm and 50 mm from the heated face of the specimen plotted as functions of time (**Fig. 5 (a)**); (2) temperature histories of points inside the furnace (ambient), on the heated face of the specimen, and at 25 mm and 50 mm inside the specimen (**Fig. 5 (b)**); (3) pore pressures at points 25 mm and 50 mm from the heated face of the specimen plotted as functions of temperature (**Fig. 5 (c)**); (4) rate of temperature rise (time derivatives of temperature dT/dt , °C/min) plotted as functions of temperature (**Fig. 5 (d)**); and (5) rate of pore pressure rise (time derivatives of pore pressures dP/dt , MPa/min) plotted as functions of temperature (**Fig. 5 (e)**). These results, which

are discussed in sections that follow, provided insight on the complex heat-induced moisture transport process in concrete and the interdependency between moisture transport and the buildup of pore pressure.

Temperature and Pore Pressure Development

Figs 5 (b) and (d) show the temperature distribution and the time rate of temperature rise, respectively, inside the specimen had a complex history compared with the air temperature. As the air temperature increased, temperatures on the surface and inside the specimen also increased but at varying rates. The changes in the rates of temperature rise (or perturbations) were most pronounced when concrete reached temperatures of about 105°C, 160°C, 180°C, and 220°C. These perturbations are shown as vertical dash lines on **Fig. 5 (d)**.

As shown in **Fig. 5 (d)**, the time rate of temperature rise (dT/dt , °C/min) at point 25 mm from the heated face increased continually to a maximum rate of 1.9 °C/min when the concrete at this location reached 105 °C. At this temperature, free water in the fine concrete pores began to evaporate and diffuse. A portion of the water vapor escaped through the heated surface of the specimen, causing a sudden drop in dT/dt on the heated face (see **Fig. 5 (d)**). At the same time, the remaining gaseous mixture of water vapor was transported inward by pressure gradient. This mass transport process (vaporization and diffusion of free water) continued as the temperature inside the concrete increased to about 160 °C. Because of the vaporization and transport of water vapor in this temperature range (105 °C to 160 °C), the transmission of heat through the concrete is retarded as a result of energy being absorbed in the evaporation process, causing the time rates of temperature rise dT/dt at locations inside the concrete to decrease (from 1.9 °C/min to 1.5 °C/min for point at 25 mm deep). During this same temperature range, a noticeable increase in pore pressures, measured at points 25 mm and 50 mm deep, and their corresponding pore pressure rates (dP/dt , MPa/min) can be observed as marked by the vertical dashed lines in **Figs 5 (c) and (e)**. As the concrete temperature increased from 160 °C to about 180 °C, a more pronounced drop in dT/dt occurred (see **Fig. 5 (d)**). This is believed to be due to the release and diffusion of chemically bound water in the concrete matrix. The additional volume of water vapor in the concrete pores caused by the release and evaporation of chemically bound water caused a sharp increase in pore pressures and the rates of pore pressure rise dP/dt (which reached a maximum of 0.018 MPa/min at 180 °C for point 25 mm deep, see **Fig. 5 (e)**).

Beginning at 180 °C, there was a decrease in dT/dt on the heated face of the specimen. This decrease continued until the temperature reached 220 °C, indicating the loss of a significant amount of moisture vapor from the specimen into the furnace in this temperature range (thereby retarding the rate of temperature rise on the concrete surface (see **Fig. 5 (d)**). The escape of moisture vapor caused a sharp drop in dP/dt between 180 °C and 220 °C (from 0.018 MPa/min to 0, see **Fig. 5 (e)**). Pore pressure at point 25mm deep continued to increase in this temperature range but at a decreasing rate to a maximum value of 0.75 MPa, while pore pressure at point 50 mm deep had begun to attenuate at the beginning of this temperature range (see **Fig. 5 (c)**). Beyond 220 °C, pore pressure at point 25 mm deep dropped sharply indicating successful attenuation of pore pressure in the specimen (probably due to the presence of PP fibers and microcracks) which prevented explosive spalling from occurring.

The interdependency between temperature and pore pressure described above is consistently observed for all specimens and revealed a complex process of heat-induced moisture transformation and transport that is unique to concrete. The maximum pore pressure measured in the specimens can be found in Phan (2005).

Spalling Characteristics

Specimen I-0-0-M-5 (test #11 in **Table 2**) and its duplicate (test #12) were the control specimens. They were designed and cured in condition that would result in explosive spalling based on experience with the mechanical property test series. As expected, explosive spalling, which was characterized by the sudden, violent delamination and complete fragmentation of a 20 mm to 30 mm thick concrete layer on the heated surface of the specimen, occurred in both. This was accompanied by an instantaneous release of a large amount of energy that propels the concrete fragments at high velocity, while leaving the remaining portion of the specimen (70 mm to 80 mm thick) intact. This violent and instantaneous delamination and fragmentation is consistent with the notion that explosive spalling failure in HSC results from the build up of internal pore pressure.

Figs 6 (a) and (b) show pore pressures and the time rate of temperature rise dT/dt as functions of temperatures at points 25 mm and 50 mm deep in the control specimen I-0-0-M-5. As can be seen in **Fig. 6 (a)**, the maximum pore pressure reached just prior to explosive spalling was 2.1 MPa, which was measured at 25 mm deep from the heated face of the specimen. Typically, regardless whether explosive spalling occurred, of all points measured, the

12 Phan

maximum pore pressure occurs at about 25 mm deep in all specimens in this test series. Also, as in the case for specimen I-1.5-13-M-5 (see **Figs 5 (a) to (e)**), perturbations in the time rate of temperature rise at points 25 mm and 50 mm inside this specimen were similarly observed (**Fig. 6 (b)**). These perturbations were also marked on **Fig. 6 (b)** by the vertical dashed lines at the temperature levels where it occurred. The corresponding increases in pore pressure were also marked as shown in **Fig. 6 (a)**.

Comparisons of Pore Pressure Test Results

Fig. 7 shows pore pressure as a function of temperature in the control HSC specimen I-0-0-M-5 and two NSC specimens (IV-0-0-M-5 and IV-0-0-A-5). These specimens are differed by the w/cm ratios (between I-0-0-M-5 and IV-0-0-M-5) and curing conditions (between I-0-0-M-5 and IV-0-0-A-5). The influences of w/cm and initial moisture content (curing conditions) on pore pressure buildup can be clearly observed. Pore pressure in NSC specimens was able to attenuate gradually after reaching maximum of 1.3 MPa (IV-0-0-M-5) and 0.4 MPa (IV-0-0-A-5), while the control specimen exploded at 2.1 MPa.

Fig. 8 shows pore pressures as functions of temperature for three HSC specimens (I-0-0-M-5, I-1.5-38-M-5, and I-3-38-M-5). These specimens are differed only by the amount of PP fibers (no fibers, 1.5 kg/m³ PP fibers, and 3 kg/m³ PP fibers, respectively). The effectiveness of PP fibers for pore pressure mitigation can be clearly observed. Instead of reaching the critical pore pressure, the additions of 1.5 kg/m³ and 3.0 kg/m³ PP fibers in HSC specimens reduced their maximum pore pressure to levels comparable with those of NSC specimens (shown in **Fig. 7**).

Fig. 9 shows the effect of heating rate. Consistently, maximum pore pressures in specimens heated at 25 °C/min are lower than those heated at 5 °C/min. In the case shown, specimen I-0-0-A-5 attained a maximum pore pressure of 1.26 MPa while its counterpart (I-0-0-A-25), heated at 25 °C/min, began to attenuate after 0.84 MPa. This is probably due to the higher thermal gradient that caused microcracks to develop and thereby allowing their internal pore pressure to attenuate earlier.

CONCLUSIONS

The following conclusions can be drawn:

- The transformation and mass transport of moisture in concrete have a unique influence on its temperature and pore pressure developments. Noticeable rise in pore pressure occurs when concrete reaches the temperature range of 105 °C to 160 °C (concrete temperature), which coincides with the vaporization of free water and transport of water vapor (as evidenced by the drop in the concrete's rate of temperature rise dT/dt). In the concrete's temperature range of 160 °C and 180 °C, sharp rise in pore pressure occurs. This coincides with the release of chemically-bound water (as evidenced by further drop in dT/dt). Beyond 180 °C, pore pressure continues to rise but with a decreasing rate until peaking in the concrete temperature range from 220 °C to 245 °C. After this point, either explosive spalling occurs or pore pressure attenuates. Spalling pore pressures were measured at 2.1 MPa and 2.02 MPa for the specimens in this study
- The additions of PP fibers cause a significant reduction in maximum pore pressure and prevent explosive spalling in specimens that are otherwise identical to the exploded specimens. The highest pore pressure measured for specimens with 1.5 kg/m³ of PP fibers was 1.42 MPa, a drop of about 45% compared with exploded specimen. For specimens with 3.0 kg/m³ of PP fibers, this value was 0.66 MPa, a more than 300% drop.
- Specimens with high w/cm (NSC) and with lower initial moisture contents (cured by air-dried) consistently have lower pore pressure compared with HSC and submerged specimens. Further, thermal gradient is found to be an influencing factor in the process of pore pressure buildup. Higher thermal gradient, caused by high heating rates, might cause microcracks to develop early thereby allowing pore pressure to escape and resulting in lower maximum pore pressure.
- HSC that carried a preload equal to 40 % of the room-temperature strength (stressed test) sustained, on average, the smallest strength loss due to temperatures up to 600 °C.
- The strength losses measured by the three test methods differed with exposure temperature. For temperatures of 100 °C and 200 °C, *unstressed residual property* test resulted in the lowest strength loss. For exposure to 450 °C, the strength loss was highest for the *residual property* test. These results indicate that there is a complex relationship between the strength measured at elevated temperature and room temperature strength.

14 Phan

- The effect of w/cm on relative strength was not the same for each test condition. This is indicated by the statistically significant interaction effect involving w/cm and test condition. For the unstressed test condition, there was no statistically significant difference in relative strength loss due to w/cm . For the stressed and residual property test conditions, however, mixture I with $w/cm = 0.22$ experienced less loss in relative strength.
- Overall, silica fume had no statistically significant effect on the relative strength loss.
- The CEN EN 1992's strength-temperature relationship for Class 2 HSC is consistent with available test results. The provision for Class 3 HSC is more conservative when compared with existing test data.
- CEB (1991)'s strength-temperature relationships are unconservative when applied to HSC.
- Finland's RakMK B4 (1991), which considers the differences in concrete strength grades and preload levels, is only slightly unconservative when compared with the test results.

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16 Phan

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Table 1 - Mixture proportions and properties of concrete

		Mixture I (w/cm=0.22)	Mixture II (w/cm=0.33)	Mixture III (w/cm=0.33)	Mixture IV (w/cm=0.57)
Materials	Cement (kg/m ³)	596	596	662	376
	Water (kg/m ³)	133	199	194	213
	Coarse aggregate*(kg/m ³)	846	846	846	854
	Fine aggregate* (kg/m ³)	734	734	734	868
	Silica fume	66	66	0	0
	HRWRA** (mL/m ³)	400	354	154	0
Properties	Fresh concrete				
	Slump (mm)	240	230	35	76
	Air content (%)	3.2	2.8	2.0	2.5
	Hardened concrete				
	Initial moisture content (%)	5.0	6.1	6.3	7.3
	Compressive strength (MPa)				
	28-day	75.3	66.0	53.2	40.6
	400-day	98.2	81.2	72.3	46.9
	Dynamic Modulus (GPa)				
	400-day	47.2	43.7	44.1	36.7

* Saturated surface-dry condition

**High-Range Water Reducing Admixture

Table 2 - Test matrix and specimen parameters for pore pressure test series

Test #	Specimen Name	Mixture (w/cm)	PP Fibers		Curing Condition	Heating Rate (°C/min)
			Dosage (kg/m ³)	Length (mm)		
1	I-3-38-M-5	I (0.23)	3.0	38	Submerged	5
2	I-3-38-M-25					25
3	I-3-38-A-5				Air-dried	5
4	I-3-38-A-25					25
5	I-3-13-M-5			13	Submerged	5
6	I-3-13-M-25				Air-dried	25
7	I-3-13-A-25					25
8	I-1.5-38-M-5		1.5	38	Submerged	5
9	I-1.5-13-M-5			13		5
10	I-1.5-13-M-5 (*)					5
11	I-0-0-M-5		0	0	Submerged	5
12	I-0-0-M-5 (*)					5
13	I-0-0-M-25					25
14	I-0-0-A-5				Air-dried	5
15	I-0-0-A-25					25
16	IV-0-0-M-5	IV (0.57)	0	0	Submerged	5
17	IV-0-0-M-25					25
18	IV-0-0-A-5				Air-dried	5

(*) Duplicate specimens for repeatability

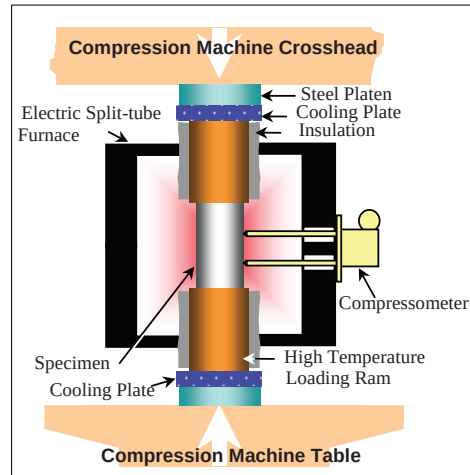


Fig. 1- Mechanical property test setup

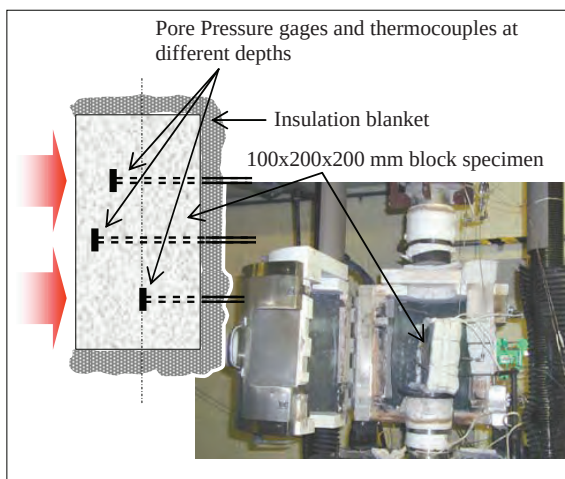


Fig. 2(a) - Pore pressure test setup



Fig. 2(b) - Pore pressure gages

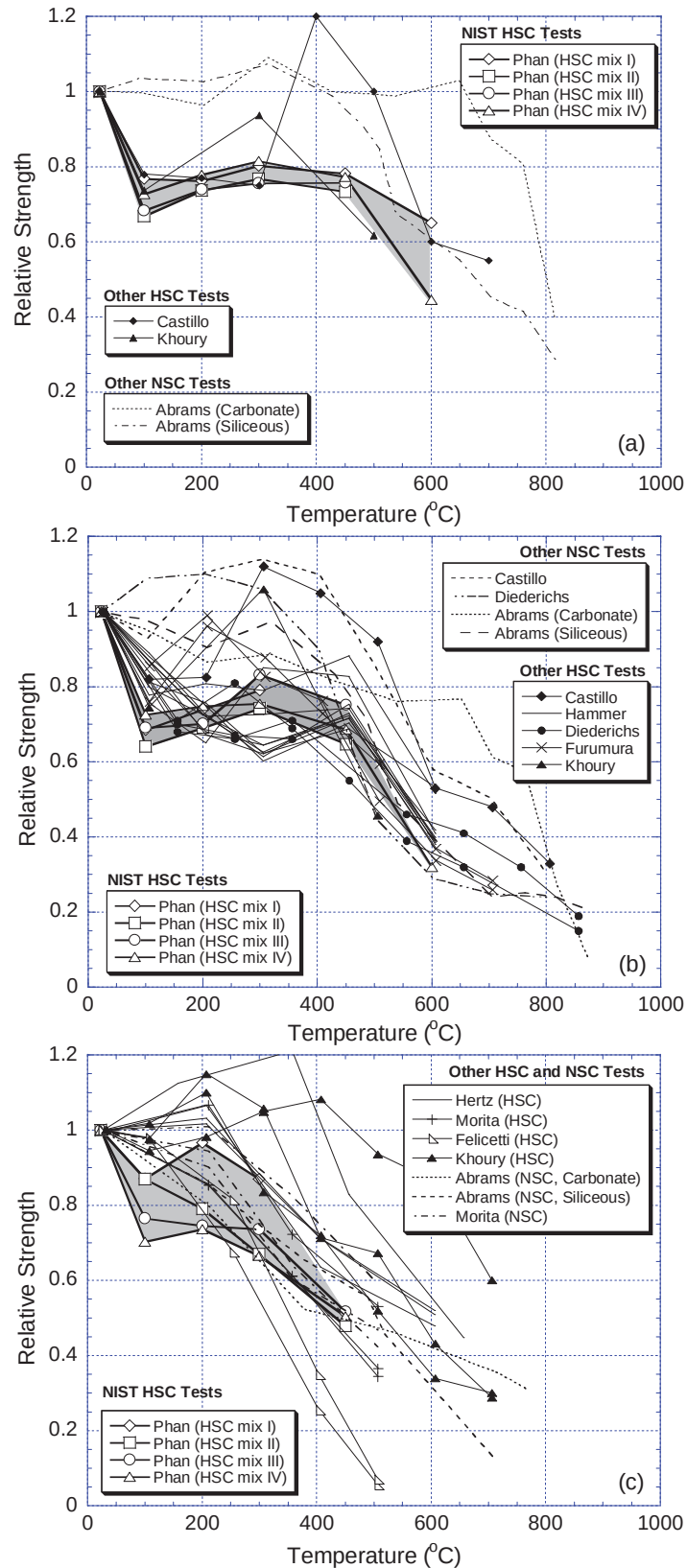


Fig. 3 - Strength-temperature relationships according to (a) Stressed; (b) Unstressed; and (c) Unstressed residual property test method

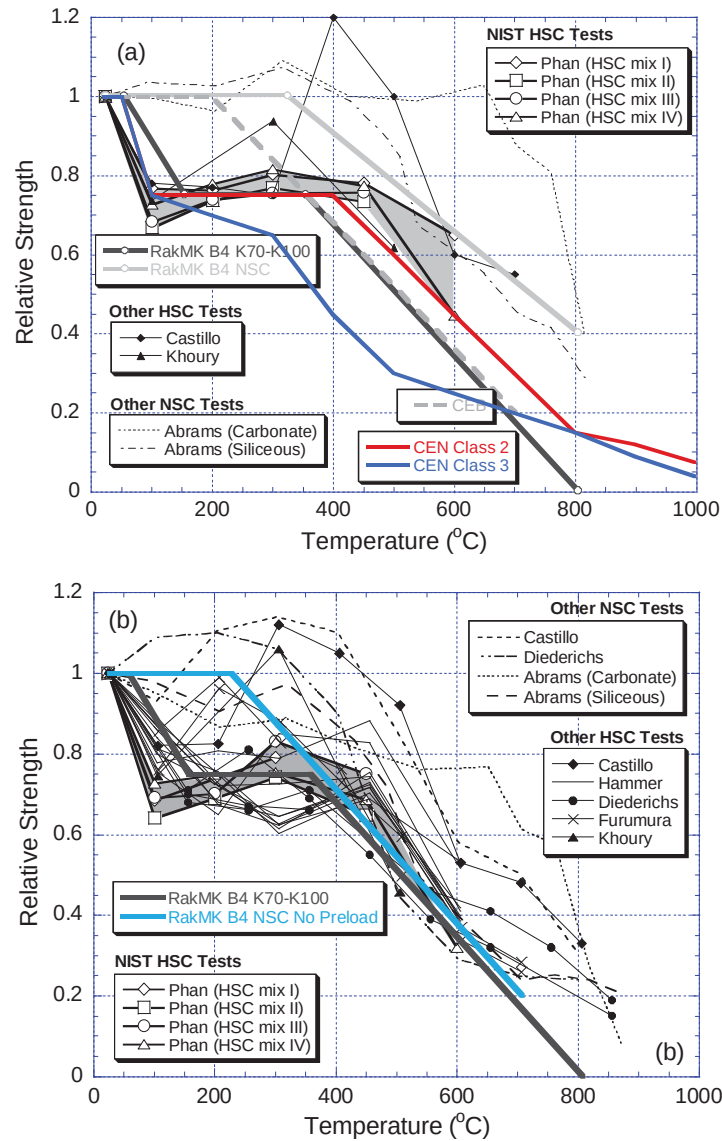


Fig. 4 - Comparisons of codes with the results of (a) stressed and (b) unstressed tests

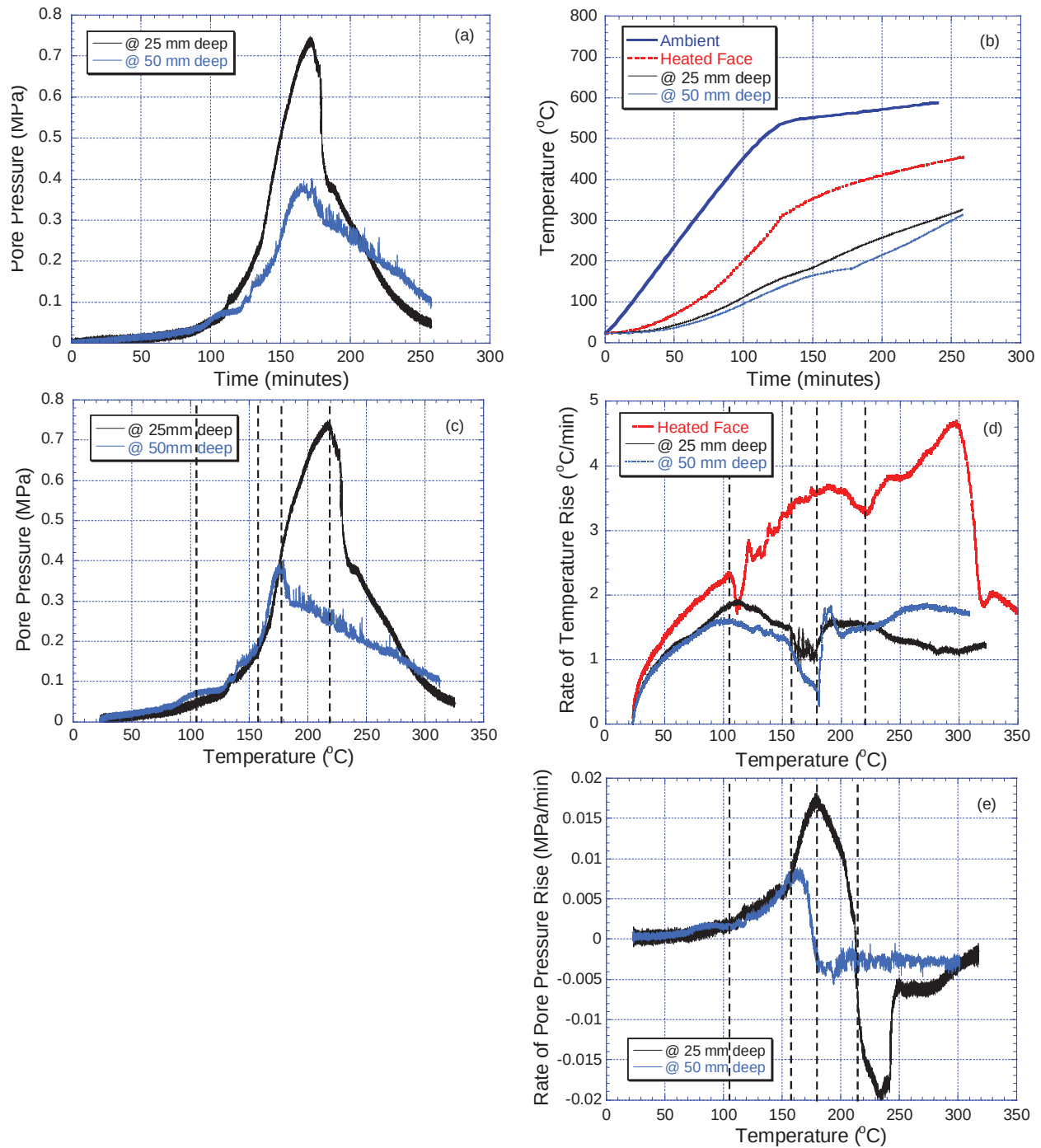


Fig. 5(a) to (e) - Test results for I-1.5-13-M-5

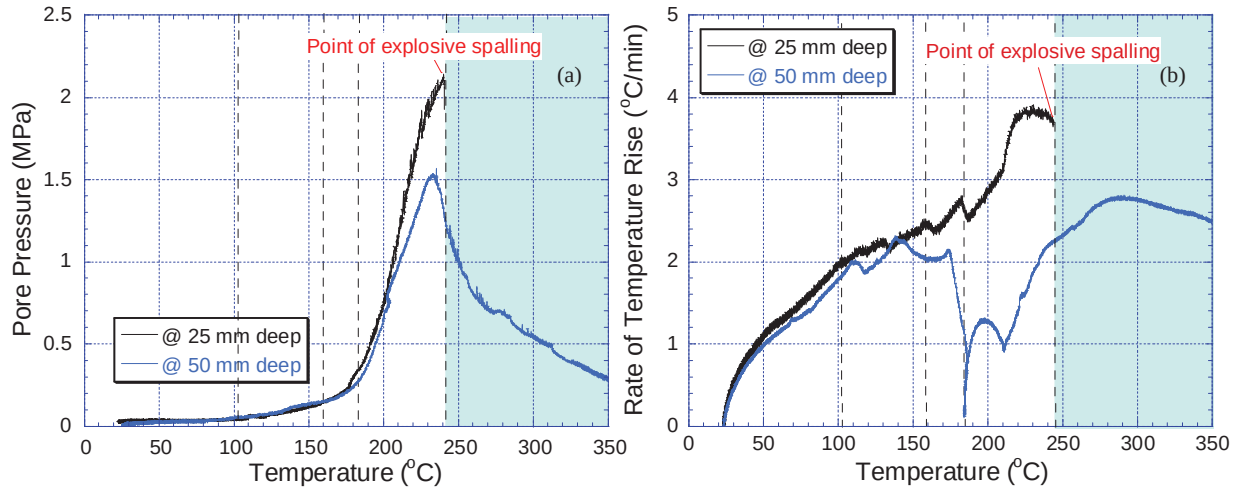


Fig. 6 - Pore pressure (a) and rate of temperature rise (b) of I-0-0-M-5 vs temperature

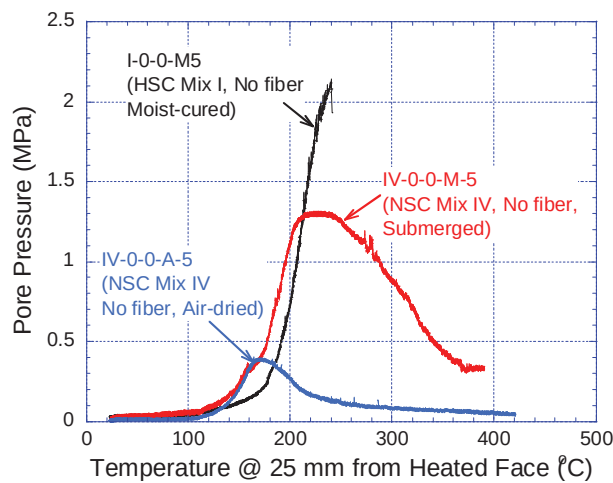


Fig. 7 - Effects of w/cm and curing conditions

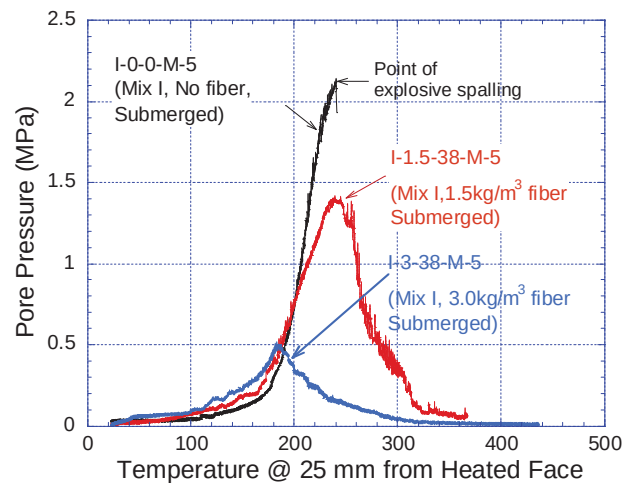


Fig. 8 - Effects of fiber dosages

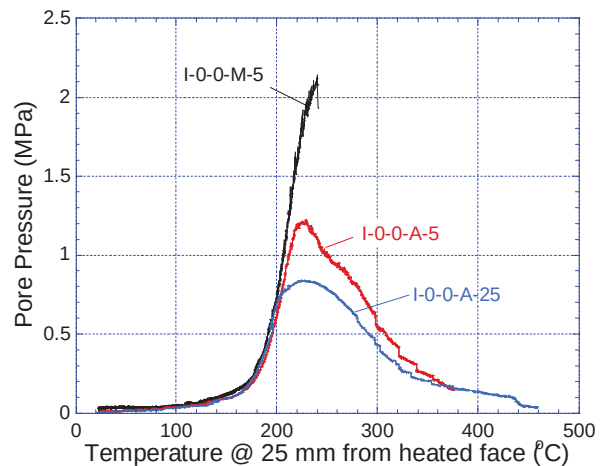


Fig. 9 - Effects of heating rate

Effect of Slag on the Performance of Concretes in Hydrocarbon Fire

by M. Guerrieri, J. Sanjayan, and F. Collins

Synopsis: A hydrocarbon fire test was conducted on nine concrete slabs incorporating three different types of binders: 100% ordinary portland cement (OPC), 50% OPC, and 50% ground-granulated blast-furnace slag (GGBFS), and alkali-activated slag (AAS). The specimens (780 mm [30.71 in.] x 360 mm [14.17 in.]) were made with three different thicknesses (100 mm [3.94 in.], 200 mm [7.87 in.] and 400 mm [15.75 in.]). Specimens were tested at an age of six months when the strengths were about 75 MPa (10,877 psi). The specimens were exposed to the hydrocarbon fire on one side. Explosive spalling only occurred in the 400 mm (15.75 in.) AAS concrete specimen that had a lower moisture content and higher permeability than the OPC and OPC/slag concretes. This suggests that the well-renowned moisture clog theory is unlikely to be a predominant mechanism of spalling in AAS concrete. It is speculated that high thermal gradients caused explosive spalling in the AAS concrete specimen.

Keywords: activated slag; hydrocarbon fire; moisture clog; spalling; thermal gradient

Biography

Maurice Guerrieri is a PhD candidate at Monash University, Melbourne, Victoria. He received his BE (Hons) from the same university. His current research interest is in the material and structural behaviour of alkali activated slag and fly ash concrete when exposed to fire.

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INTRODUCTION

When exposed to fire, concrete deteriorates, evidenced by progressive breakdown of the cement gel structure and therefore consequential loss in its load bearing capacity¹. The threat of a fire in buildings and other infrastructures is one of the risks that must be considered during the design stages. Typically, for buildings, the designer is expected to demonstrate that the structure can withstand a standard fire (e.g. ASTM E119-07a, ISO 834-1)²⁻³. Furthermore, certain infrastructures can be exposed to hydrocarbon fires which are a more severe form of fire, due to its rapid rate of temperature rise at the beginning. For example, in a standard fire (ISO 834-1)³, the temperature rise in the first minute is 349°C (660°F), compared to 743°C (1369°F) in a hydrocarbon fire. The time versus temperature curves of standard and hydrocarbon fires recommended by ASTM E119-07a² are shown in **Fig. 1**.

Concrete structures exposed to fire can collapse due to thermal warping (bending due to thermal gradients), strength loss and loss of section⁴. One of the major concerns of concretes exposed to fire is spalling⁵. However, this is dependent on a variety of factors such as concrete strength, loading conditions and rate of temperature rise. The

appropriate dosages of polypropylene fibers in concrete have been shown to reduce the risk of spalling of concrete in fire⁶.

Due to the rapid rate of temperature rise, hydrocarbon fire exposure significantly increases the risk of concrete spalling. Road and rail tunnels are vulnerable to hydrocarbon fire exposure due to the conveyance of volatile bulk liquids. Past fire accidents in tunnels have caused spalling of concrete, even though the fires were not necessarily as severe as a standard hydrocarbon fire test. Three significant fires have occurred in the Great Belt Tunnel 1994, Channel Tunnel in 1996 and the Mount Blanc Tunnel in 1999⁷. The fire which occurred in the Great Belt Tunnel caused severe spalling in some sections of the tunnel. The Channel Tunnel fire reached temperatures as high as 700°C (1292°F) and reduced the 400 mm (15.75 in.) concrete lining thickness, at the worst located locations, to 20 mm (0.79 in.). This caused the tunnel to be closed for 6 months at a loss of approximately US\$ 1.5 million per day⁸, excluding repair costs. The fire in the Mount Blanc Tunnel burned for 53 hours and caused the death of 39 people and destroyed 23 trucks and 11 cars⁹.

Some tunnel operators (e.g. Citylink in Melbourne Australia) prohibit trucks carrying hydrocarbon fuels or inflammable liquids from traveling into tunnels. Hence, the tunnels are only required to have the minimal fire resistance of 25 to 50 MW (33500 - 67000 hp) fire⁹. However, it is still possible for a hydrocarbon type fire to commence in a tunnel from ordinary legal transportation materials. This is evident in a recent experiment in Norway where a truck carrying wooden pallets was burnt in a tunnel and the temperatures reached 1100°C (2012°F), similar risk to a standard hydrocarbon fire test⁹.

To date, a significant amount of research has been carried out in order to understand the spalling phenomenon to ensure the safety of concrete structures exposed to extremely high temperatures and rapid temperature rise⁵. Explosive concrete spalling is the most severe form of spalling and represents the catastrophic failure of concrete breaking into smaller pieces and also delamination of large sheets of concrete. Although there is no coherent consensus towards the governing spalling mechanism, several theories are dominant and well supported in literature, namely, moisture clog mechanism¹⁰, thermal-stress mechanism¹¹⁻¹⁴, thermal incompatibilities mechanism¹¹ and physicochemical changes¹⁵. The moisture clog mechanism describes that the spalling is caused by the buildup of

26 Guerrieri et al.

water vapour pressure which, when prevented from escaping from dense concrete, leads to spalling. The thermal-stress mechanism describes that the spalling is caused by internal stresses developing due to the thermal gradient within the concrete. The thermal incompatibilities and physicochemical changes mechanisms attribute the reason for spalling to the differences in material properties of the concrete constituents, i.e., differences in thermal and hygral expansion coefficients that can create differential gradients.

Recent consensus¹⁶ is that spalling is believed to be due to different co-existing and coupled processes such as thermal (thermal dilatation), chemical (dehydration of cement paste), hygral (evaporation of the liquid pore water, associated by the vapour pressure increase) and mechanical damage (cracking) processes. These mechanisms arise from factors associated with temperature variation inside the concrete element, as well as the profile of the cross section (size, shape, permeability, mean pore radius, concrete strength, and moisture content)¹⁷⁻²⁰.

To date, the majority of the literature on concrete spalling has concentrated on concrete with Portland cement as the sole binder (OPCC). Studies on the addition of silica fume to the binder in concrete have demonstrated increase risk of spalling²¹. The National Institute of Standards and Technology²² recently published a thorough literature review on the fire performance of OPCC cylinders and measured that almost no residual strength following exposure to temperatures reaching 800°C (1472°F) due to the decomposition of calcium hydroxide (Ca(OH)_2) and other physiochemical changes. Between 400-500°C (752-932°F) calcium hydroxide dehydrates to calcium oxide (CaO) which can rehydrate back to (Ca(OH)_2) during cooling in the moist air. This process is accompanied by a volume expansion which can disrupt a concrete that had previously withstood a fire without disintegration²³.

Concrete made with slag blended cements as the binder are likely to perform better in a fire than OPCC, since these concretes contain less free lime (Ca(OH)_2). Further, concretes made with alkali activated slag contain no Ca(OH)_2 and are therefore expected to provide even superior performance in fire.

Alkali Activated Slag Concrete (AASC) is a special type of concrete which is manufactured by the alkali activation of GGBFS. Unlike Portland cement/slag binder systems, no Portland cement is used in alkali activated slag concrete. Instead, sodium silicate and hydrated lime is added as the activator for the hydration reaction between slag and water.

Many comprehensive research works on AASC have been carried out²⁴⁻²⁸ however none have reported fire resistance. AASC is considered an environmentally friendly alternative to OPCC as it utilizes by-products of the iron industry and therefore requires minimal raw materials compared to OPCC. AASC can achieve equivalent and higher strength than OPCC²⁷, and is considered a better performing concrete in applications that require high chemical resistance²⁶.

In this paper, three different types of concretes were investigated: (1) Concrete made with OPC as the only binder, and denoted as OPCC. This concrete is used as the reference concrete. (2) Concrete made with 50% slag and 50% OPC blended as the binder and denoted as 50/50. (3) Concrete made with 100% slag as the binder which is activated by powdered sodium silicate and hydrated lime, denoted by AASC.

RESEARCH SIGNIFICANCE

Concrete containing OPC/slag blended cements are very commonly used, especially where long term durability of concrete is essential in aggressive environments. However, these infrastructures must also be designed for exposure to extreme elevated service temperatures, and therefore, their fire resistance and performance must be well understood. This research adds new data on the fire performance of AASC and concretes made with slag blended cements. To the best of the authors' knowledge, there have not been any previous studies reported on the fire performance and spalling of AASC.

EXPERIMENTAL INVESTIGATION

Concrete slab specimens incorporating three different types of binders; 100% OPC, blended OPC and Slag (50/50), and AAS were made. The slab specimens (780 mm [30.71 in.] in length and 360 mm [14.17 in.] in width) were made with three different thicknesses; (100 mm [3.94 in.], 200 mm [7.87 in.] and 400 mm [15.75 in.]). The geometry and layout of the specimens are shown in **Fig. 2**. All the concretes investigated were approximately the same strength of 75 MPa (10877 psi) at the time of the test date which is considered a high strength concrete¹⁴. The specimens were exposed to hydrocarbon fire on one side (underside).

28 Guerrieri et al.

Cementitious materials

The chemical composition and physical properties of the cementitious materials (binders) used are shown in **Table 1**. The binders used were slag and OPC which were produced in Australia. The slag material has a basicity coefficient $K_b = (\text{CaO} + \text{MgO})/(\text{SiO}_2 + \text{Al}_2\text{O}_3)$ equal to 1.05 and therefore considered basic²⁹. Slags with a $K_b > 1.0$ are the most active and provide the highest strength²⁹. The slag is preblended with 2% gypsum during the manufacturing process, and the slag is comprised of 90% of glass, gehlenite and merwinite etc which are amorphous components³⁰. From hereon, the conventional water/cement ratio term is replaced with water/binder to incorporate all the binders' used³¹.

Activators

Industrial grade dry powdered sodium metasilicate, consisting of 29% Na_2O , 28% SiO_2 , and 43% H_2O , $M_s \approx 1.0$; and industrial grade dry hydrated lime consisting of 72.5% CaO , 1.3% SiO_2 , 0.2% ($\text{Al}_2\text{O}_3 + \text{TiO}_2$), 0.4% Fe_2O_3 , 0.9% MgO and the loss of ignition 24.2% (of the lime) were used.

Mix design, specimen geometry, moisture content determination and in-situ compressive strength measurements

The fine aggregate consisted of river sand, locally produced at Lynhurst with a specific gravity of 2.65, 24 h water absorption of 0.5%, and a fineness modulus of 2.19. The coarse aggregate consisted of 14 mm (0.55 in.) maximum size basalt with a specific gravity of 2.95 and 24 h water absorption of 1.2%. The mix proportions are given in **Table 2** which are the same as those conducted previously³¹.

The materials used for the production of the concrete and the methods of preparing the concrete mixes in the laboratory were same as those conducted previously³¹. For the blended OPC/Slag concrete, the cementitious materials were pre-blended in a dry form for a period of 10 minutes. For the AASC mixture, the powdered slag was pre-blended with the sodium silicate powder for a period of ten minutes before the other constituents were added. The reinforcement consisted of 100 x 100 mm mesh (3.94 x 3.94 in.) which was 6 mm (0.24 in.) in diameter.

The slab specimens were stripped at 24 hours and moist cured in a fog room at 23°C (73°F) and 100% RH for 28 days. Until the test date (approximately six months), the slab specimens were removed from the fog room and kept

exposed to laboratory air at room temperature (approximately 23°C (73°F) and 50% RH). The specimens were instrumented with Type K thermocouples (which were operational up to 1500°C (2732°F)) to monitor the insitu temperature profile and distribution. **Fig. 2** shows the layouts of the slabs, reinforcement and the thermocouple locations and **Figs. 3 - 4** shows the positioning of the slabs for the fire test.

To determine the moisture content of the slab specimens on the day of the fire test, six standard concrete cylinders (200 mm [7.87 in.] high and 100 mm [3.94 in.] diameter) were also cast together with each concrete pour and were cured in a similar manner to the slabs. The cylinders were sealed along their longitudinal cross sectional area with water-proof paint to replicate the similar drying conditions of the slab specimens; i.e. - drying through the top and bottom faces only. These were also tested for compressive strength at 28 days in addition to the time of the test date.

The moisture content was considered as an average for the entire depth of the slab and was obtained by drying the cylinders in an oven after the curing period until stable weights were achieved and then allowing the cylinders to absorb moisture from the air until the test date. Insitu residual compressive strength profiles of the slabs after the hydrocarbon fire exposure tests were determined by extracting cores (82 mm [3.23 in.] length by 41 mm [1.61 in.] diameter) and measuring the compressive strength. The averages of three cores taken along two heights, position A and B as shown in **Fig. 5** for each specimen were used as a representation of the post-fire residual compressive strengths.

Spalling Test Setup

The spalling test was conducted in a gas firing furnace that was programmed to follow the hydrocarbon fire curve³, as shown in **Fig. 1**. The specimens were unloaded during testing and the undersides of two slab specimens were exposed simultaneously to the hydrocarbon fire curve conditions for duration of one hour. Spalling of concrete generally occurs around 30 minutes after the commencement of fire²¹ and well within the first 60 minutes. Therefore all the tests were terminated at the end of 60 minutes. To determine the intensity and degree of spalling during the test, the weight of each specimen was independently monitored whilst the test was in progress. This was done by suspending the specimens with load cells and positioned into the furnace as shown in **Figs. 3 and 4** respectively. Further, the times at which significant spalling noises occurred were recorded. At the conclusion of the test, the

specimens were inspected after cooling and the type of spalling which occurred (surface spalling, corner separation, explosive spalling etc) was recorded.

EXPERIMENTAL RESULTS AND DISCUSSION

Compressive strengths of the concretes (cylinders) at the age of 28 days and 6 months (test age), in addition to the insitu strengths of the extracted cores taken from the specimens after the hydrocarbon fire exposure, are shown in **Table 3**. **Table 3** also presents the moisture contents of the cylinders which had sealed sides to represent the similar drying condition as the slab specimens. **Table 4** shows the percentage weight loss of the specimens. **Table 5** shows the spalling characterization and spalling depth of the specimens. **Fig. 6** shows the thermocouple data of the specimens. **Fig. 7** shows the weight loss of the 400 mm (15.75 in.) slab specimens as a function of time as recorded by the load cells. **Fig. 8** shows the temperature gradients of the 400 mm (15.75 in.) slab specimens. **Fig. 9-11** shows the photographs of the underside of the 400 mm (15.75 in.) specimens for the OPCC, blended and AASC concretes.

Results from 100 mm (3.94 in.) Specimens

The 100 mm (3.94 in.) thick specimens did not show any signs of distress or spalling. At this thickness level, the thermal stresses due to thermal gradients and steam pressure build up are not sufficiently large to cause any spalling of concrete.

Results from 200 mm (7.87 in.) Specimens

The extracted cores from the 200 mm (7.87 in.) thick specimens showed that the AASC retained the highest residual strength at both position A and B where the cores were extracted. At position A, AASC had a residual strength of (58%) followed by the 50/50 (44%) and then the OPCC (34%). At positioned B, AASC had no strength loss whilst minimal strength loss occurred for the 50/50 (4%) and the OPCC (8%) respectively. Minimal aggregate spitting and corner separation occurred for the OPCC and blended specimens; however, no spalling noise during the test was detected.

The representative moisture contents for the OPCC and 50/50 concretes were 4.70 and 4.11% and the weight loss of the specimens were 5.11 and 6.24% respectively. The weight loss includes both the moisture loss and concrete

section loss. In both of these specimens, significant amounts of steam came out of the specimens from 22 minutes into the test for the OPCC and 27 minutes into the test for the 50/50.

In the case of the AASC specimen, continuous popping and blistering noises were noted from 19 minutes into the test. These noises were due to the significant surface spalling and surface spitting which occurred on the exposed side of the AASC specimen. The representative moisture content for the AASC was 3.50% and the weight loss was 6.91%. No steam was seen migrating out of the specimen.

Results from 400 mm (15.75 in.) Specimens

At position A, AASC had a residual strength of (55%) followed by the 50/50 (46%) and then the OPCC (38%). At positioned B, AASC had no strength loss whilst minimal strength loss occurred for the 50/50 (4%) and the OPCC (8%) respectively.

Both the OPCC and 50/50 suffered surface spalling and moderate levels of surface spitting. During the tests, continuous popping and blistering noises were recorded from these specimens. The photographs of the fire exposed side of all the 400 mm specimens are shown in **Fig. 9-10**.

The weight loss of the OPCC and 50/50 specimens were 3.11 and 4.41%. In both of these specimens, significant steam came out of the specimens from 33 minutes into the test for the OPCC specimen and 34 minutes into the test for the 50/50 specimen. However, in both of these instances, the amount of steam was significantly less than in the 200 mm (7.87 in.) specimens. In the case of the AASC specimen, explosive spalling occurred, as recorded by a loud noise 23 minutes into the test. The spalling which occurred was found to be due to dislodgement of a concrete piece of 1.7 kg (3.75 lbs) in weight. The total weight loss of the specimen was 5.19%, which was composed mainly of dislodged concrete as no steam was seen escaping. The spalling damage which occurred in the 400 mm AASC specimen can be seen in **Fig. 11**.

Discussion

The residual strengths of the cores were within 4% of each other when comparing between the same core location and concrete type. This is because residual strength is dependent on the insitu temperatures and thermal gradients which were similar between identical concretes of varying thickness. The centerline of Row A was located 50 mm (1.97 in.) from the heated surface and respectively 150 mm (5.91 in.) for Row B. From the insitu temperature profile data (**Fig. 6**), the insitu temperatures of OPCC, 50/50 and AASC specimens reached temperatures in the order of 400°C (752°F) at the end of the fire test at Row A. Although the final temperatures were very similar, the thermal gradients were more severe for the AASC during the fire test. At its maximum; the magnitudes of the thermal gradients were 10 and 20% higher than the 50/50 and OPCC specimens respectively. The temperatures were minimal for all the concrete types at the location of Row B (50°C [122°F]) and therefore explains the minimal strength loss.

The higher loss in residual strength of OPCC is associated with the decomposition of calcium hydroxide which was accompanied by a volume expansion that damaged the concrete and resulted in its strength loss²³. The strength loss was less for the 50/50 concrete and even smaller for the AASC due to the reduction in calcium hydroxide content³².

The moisture content of the OPCC specimen was in good agreement with existing literature²², however, there is no literature regarding the moisture content of 50/50 and AASC specimens. The moisture content for the AASC (3.50%) specimen is less than that of the 50/50 specimen (4.11%) and significantly less than that of the OPCC specimen (4.70%). Previous study³³ showed that drying shrinkage of AASC can be more than twice the amount of OPCC shrinkage. This causes significant microcracks on the surface due to restrained shrinkage (drying of the surface causes significant shrinkage while the inner core of the concrete restrains this shrinkage causing microcracks to form on the surface.) The additional drying of the specimen caused by shrinkage microcracks is the likely reason for the lower moisture content of AASC. The moisture contents were measured within $\pm 0.3\%$ accuracy, and were considered as an average throughout the slab.

The degree and depth of spalling shown in **Table 5** illustrates that spalling tendency is only of concern between the heated surface and approximately 40 mm (1.57 in.) away from the heated face. This is in good agreement with the

fact that the temperature gradients were at a maximum between 0 – 25 mm (0 – 0.98 in.) and were at a maximum for the 400 mm (15.75 in.) AASC specimen at 22 minutes. This coincided with the time when explosive spalling was observed for the 400 mm (15.75 in.) AASC specimen. It has been reported that the AASC is more permeable than OPCC based on sorptivity and mercury intrusion porosimetry tests³⁴ which investigated identical concrete mixtures to the ones reported in this paper. The higher permeability of the AASC in addition to the lower moisture content compared to the 50/50 and OPC concretes suggests that explosive spalling which occurred in AASC is the result of another mechanism than the well renowned “moisture clog” theory. It is speculated that high thermal gradients caused explosive spalling in the AAS concrete specimen.

The intensity of spalling during the fire test was monitored by measuring the weight of the specimens as a function of time as shown in **Fig. 7**. This test setup had the potential to pinpoint the time of spalling as well as both the insitu and furnace temperatures and provide key insights into the factors that cause spalling. It can be seen from **Fig. 7**, the various envelopes of spalling and clearly, spalling was more severe for the AASC specimen.

CONCLUSIONS

The main findings of this paper are summarized as follows:

1. AASC retained the highest residual strength after the hydrocarbon fire exposure. This was followed by the 50/50 and OPC concrete specimens. This is despite the fact the insitu temperatures were similar at the locations where the strength tests were carried out. This is likely to be caused by reduction in Ca(OH)_2 present in the system which causes damage to concrete temperatures between 400-500°C (752-932°F) due to volumetric changes associated with dehydration and rehydration phases. Replacement of Portland cement with slag reduces the Ca(OH)_2 content. However, replacement beyond 65% would require an additional alkali activator to obtain relative strength as in the case for AASC.
2. Alkali activated slag concretes are more prone to spalling than the Portland cement and blended cement concretes despite the considerably lower calcium hydroxide. AASC was found to have a lower moisture content and higher permeability than the OPC and OPC/slag concretes. This suggests that the moisture clog is unlikely

to be the predominant mechanism of spalling in AASC. Rather, it is speculated that high thermal gradients caused explosive spalling in the AAS concrete specimen.

3. Increasing thickness of concrete slab increases the spalling tendency in a hydrocarbon fire.

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36 Guerrieri et al.

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Table 1 – Physical and chemical compositions of cement and slag

Constituent/property %	OPC	Slag
SiO ₂	19.90	32.50
Al ₂ O ₃	4.70	13.00
Fe ₂ O ₃	3.38	0.22
MgO	1.30	5.47
CaO	63.93	42.10
Na ₂ O	0.17	0.21
TiO ₂	0.25	1.08
K ₂ O	0.45	0.25
MnO	0.08	5.47
P ₂ O ₅	0.06	-
SO ₃	2.54	4.10
LOI	2.97	0.35
Fineness, m ² /kg	360.00	435.00
Specific gravity	3.15	2.92
Basicity coefficient, Kb	-	1.05

– = not measured items

Table 2 – Mix proportions of OPCC, 50/50 and AASC concretes

Constituent	OPCC	50/50	AASC
Cement, kg/m ³	360.00	180.00	-
Slag, kg/m ³	-	180.00	360.00
Coarse aggregate, SSD,* kg/m ³	1130.00	1130.00	1130.00
Fine aggregate, SSD,* kg/m ³	830.00	830.00	830.00
Powdered Sodium Metasilicate	-	-	66.90
Hydrated Lime	-	-	14.40
Water to add to Lime	-	-	43.20
Free Water	180.00	180.00	108.03
w/b	0.50	0.50	0.50
% Na as slag	-	-	4.00
% Lime as slag	-	-	4.00

– = not measured items

* SSD = Saturated Surface Dry Condition

Table 3 – Concrete compressive strengths (MPa) and moisture contents (%)

Specimen type		OPCC	50/50	AASC
Moisture content (%)		4.7	4.11	3.5
Strength (MPa)	28 Days	73.2	73.9	72.8
	6 Months	77.0	76.2	74.3
Residual strength after fire test (MPa)				
200 mm specimen	Row A	26.5	33.3	43.2
	Row B	70.8	73.2	74.0
400 mm specimen	Row A	29.0	35.0	41.0
	Row B	71.1	72.9	74.1

Table 4 – Specimen weight loss after fire test (%)

Specimen	200 mm	400 mm
OPCC	5.11	3.11
50/50	6.24	4.41
AASC	6.91	5.19

Table 5 – Spalling characterization and spalling depth (mm)

Specimen	Spalling depth (mm)	Spalling type
OPCC (200 mm)	0	minimal aggregate spitting and minimal corner separation
50/50 (200 mm)	10	moderate aggregate spitting and minimal corner separation
AASC (200 mm)	22	moderate surface spalling and moderate surface spitting
OPCC (400 mm)	28	moderate surface spalling and moderate surface spitting
50/50 (400 mm)	32	moderate surface spalling and moderate surface spitting
AASC (400 mm)	33	explosive spalling

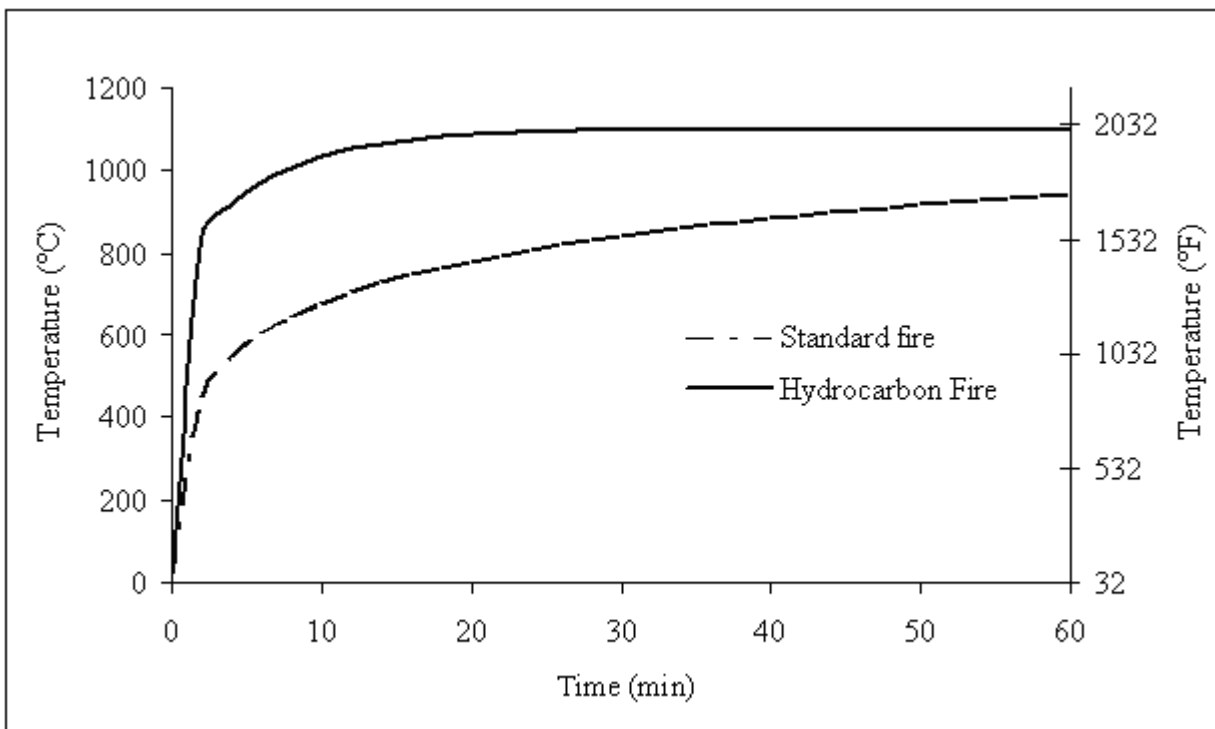


Fig. 1 - Temperature versus time of standard and hydrocarbon fires (ASTM E119).

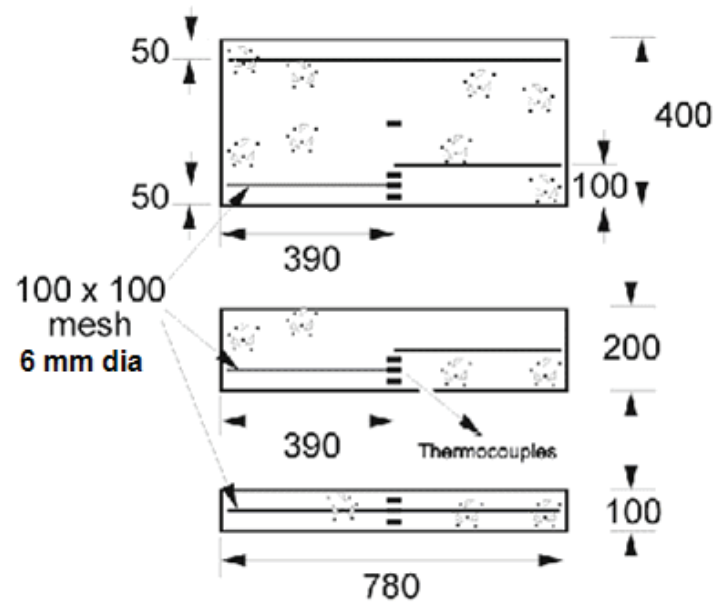


Fig. 2 – Layouts of the slabs, reinforcement and the thermocouple locations (all dimensions are in mm).

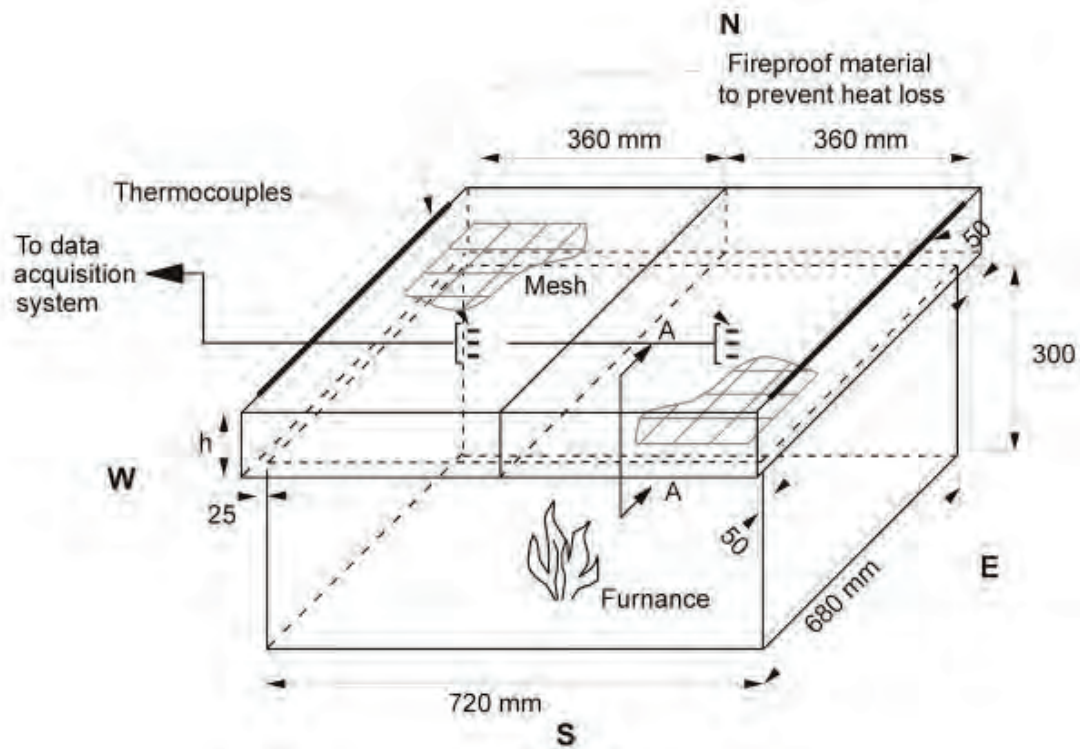


Fig. 3 – Experimental setup.

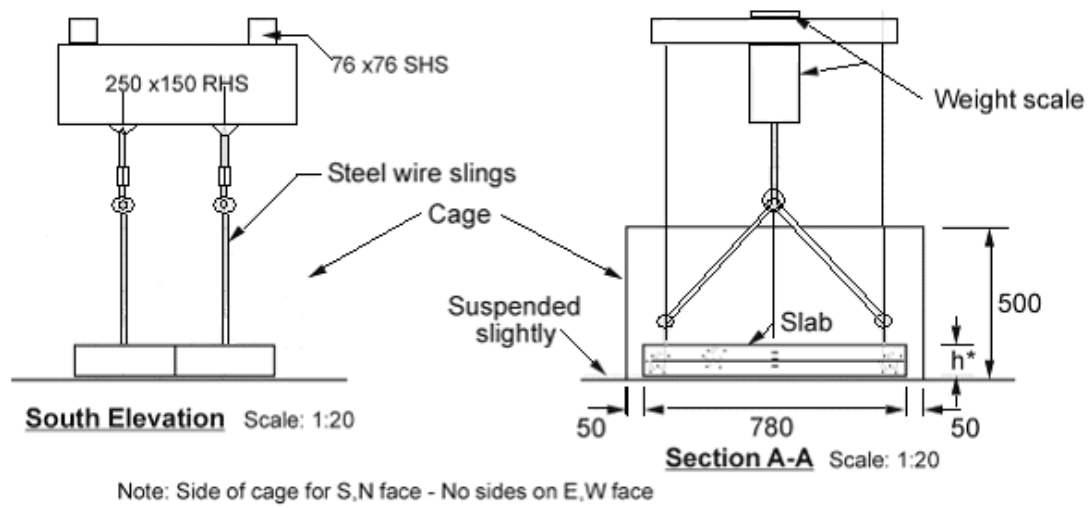


Fig. 4 – Spalling test setup.

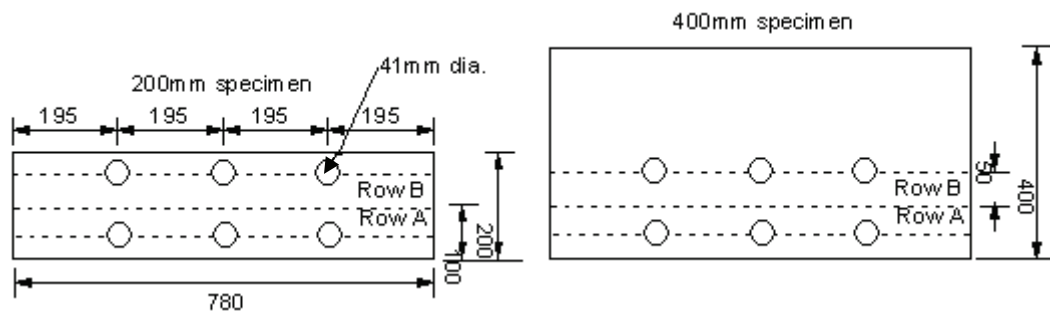


Fig. 5 – Schematic diagram of the location of the extracted cores.

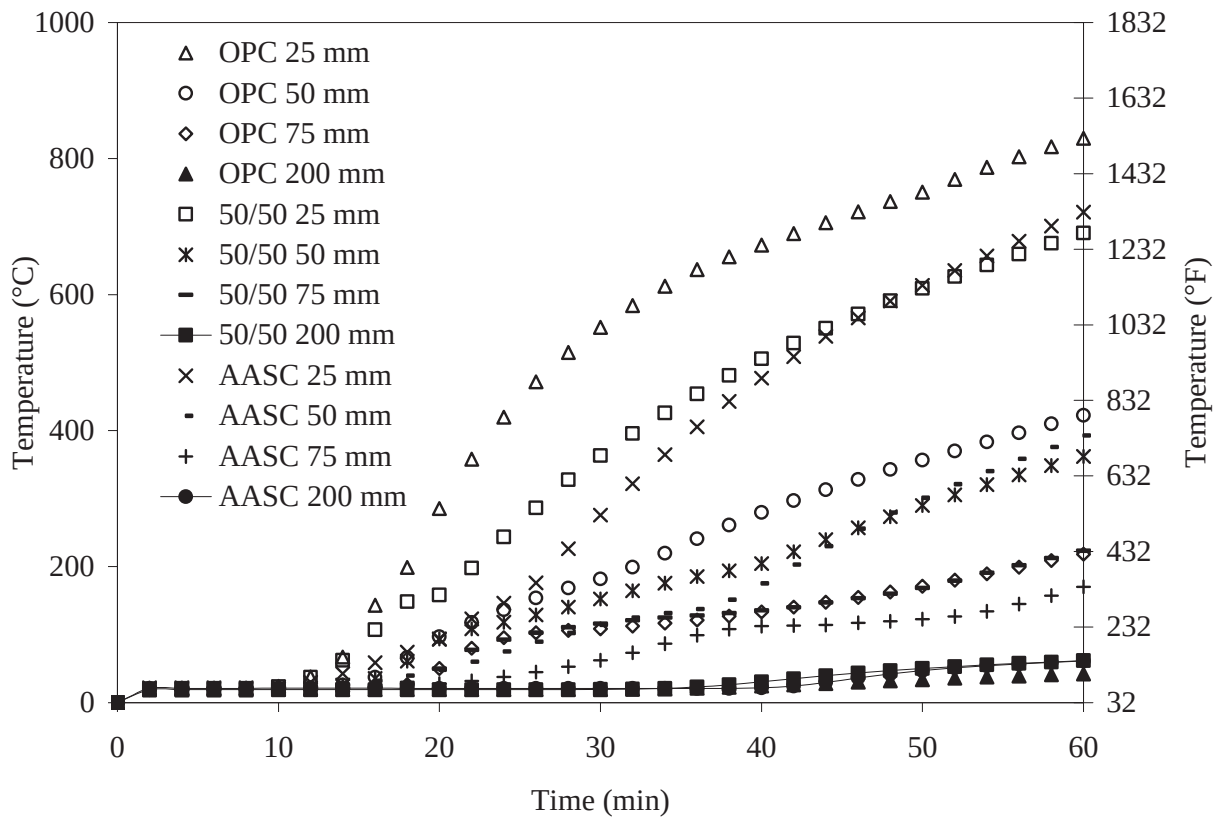


Fig. 6 – Measured temperatures for the OPCC, 50/50 and AASC, 400 mm concrete slabs. Distance is from the heated surface

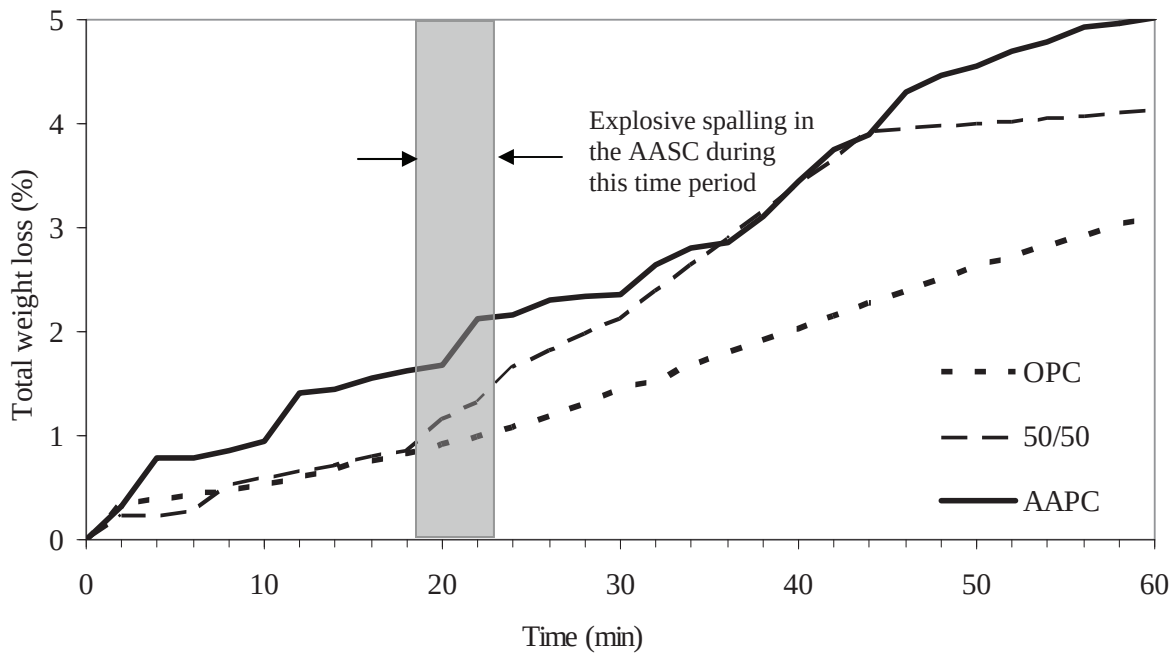


Fig. 7 – Total weight loss of 400 mm slab specimens as recorded by load cells.

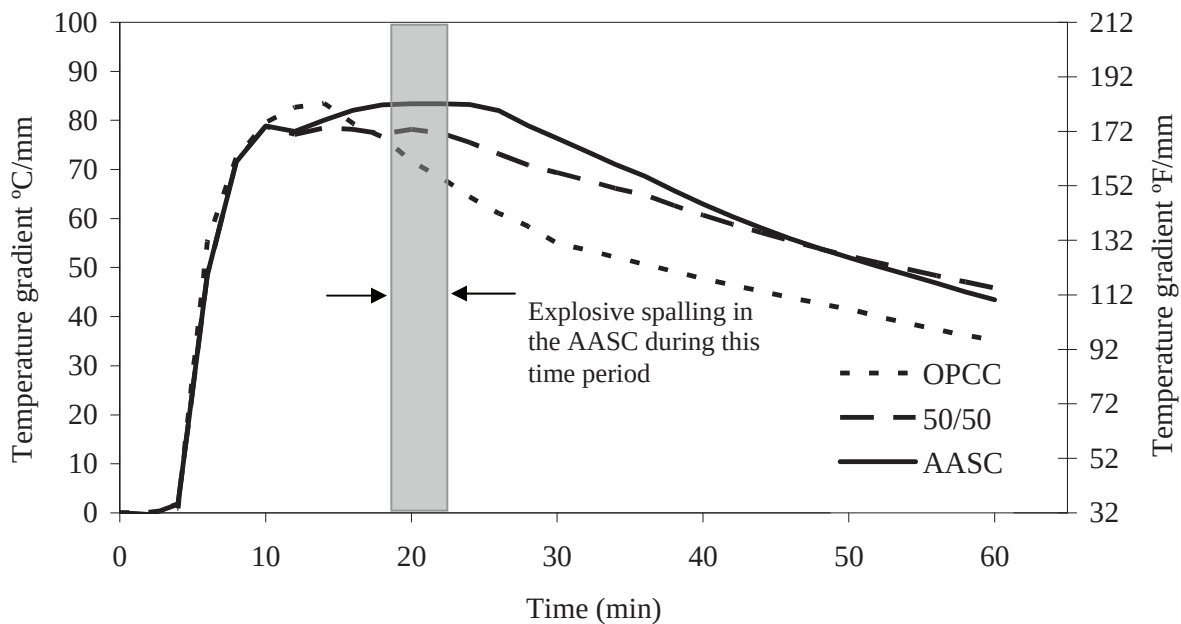


Fig. 8 – Temperature gradients of 400 mm specimens in the 0-25 mm region.

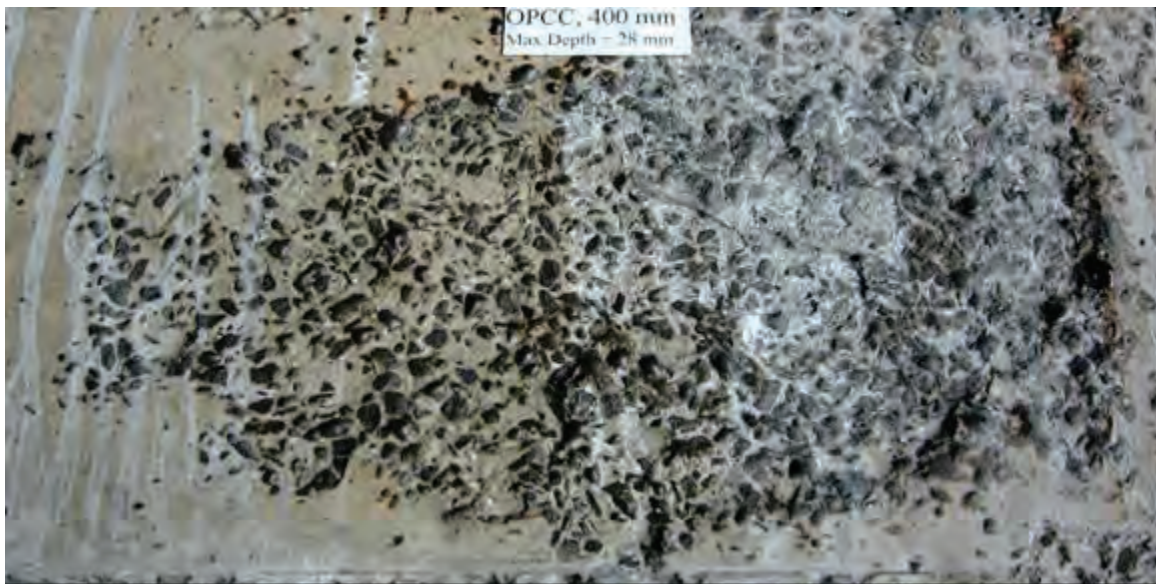


Fig. 9 – Underside of OPCC, 400 mm specimen.



Fig. 10 – Underside of 50/50, 400 mm specimen.

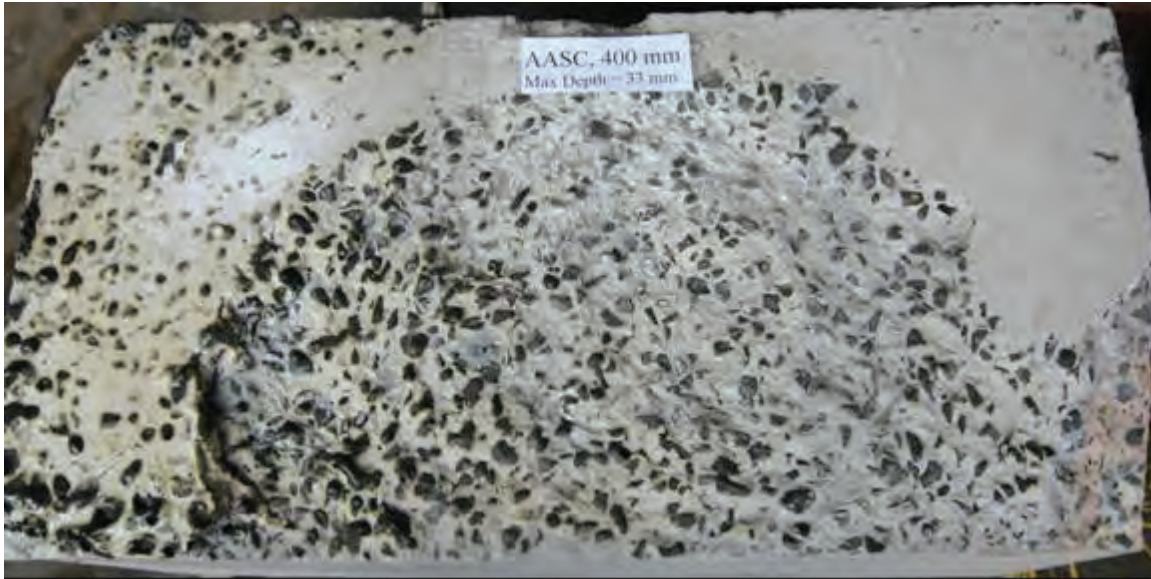


Fig. 11 – Underside of AASC, 400 mm specimen.

Flexural Behavior of Protected Concrete Slabs after Fire Exposure

by M.A. Youssef, S.F. El-Fitiany, and M.A. Elfeki

Synopsis: Fire is one of the common events that might occur during the lifetime of any concrete structure. At elevated temperatures, mechanical properties of concrete and reinforcing bars experience significant deterioration. Following a fire event, these properties improve with time toward their original values. The paper focuses on the flexural behavior of unreinforced or lightly reinforced siliceous concrete slabs after exposure to elevated temperatures. Such behavior is controlled by the concrete tensile behavior. Models to predict related concrete and steel mechanical properties during and after exposure to elevated temperatures are presented. When needed, new models are developed based on available experiments data. A case study involving flexural testing of 11 concrete slabs after 85 days from exposure to fire is presented. The slabs were protected by a thin sprayed liner (TSL). The case study allowed evaluating the presented models and assessing the effect of the TSL layer on the slabs' behavior.

Keywords: elevated temperatures; fire; mechanical properties; post-cooling; slab; strength regain; stress-strain relationship; tensile behavior

Biography:

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INTRODUCTION

Designing concrete structures for fire exposure and evaluating the condition of fire-damaged concrete structures are research topics that received significant attention by many researchers¹⁻⁴. Fire affects concrete structures by generating a heat flow that initiates at the exposed surfaces and produces high temperatures and pore pressure gradients within the concrete mass. When concrete is subjected to such a condition, its properties including compressive strength, tensile strength, and initial modulus of elasticity deteriorate¹⁻⁴. The level of deterioration is influenced by aggregate type, temperature level, heating rate, applied loading, and external sealing^{5, 6}. After extinguishing a fire, concrete properties improve with time toward their original values⁶⁻⁹. Fire temperatures also result in marked changes in concrete strains. It increases strains required for equilibrium by shifting the value of the strain corresponding to peak stress². It also introduces new strains, transient and thermal^{2, 3}. Changes in concrete properties due to fire temperatures are dependent on the level of preloading^{2, 3, 10}.

The flexural behavior of unreinforced or lightly reinforced concrete slabs is mainly controlled by the concrete tensile behavior. The most common method for measuring the tensile strength of concrete involves testing unreinforced concrete beams according to ASTM C78¹¹. This paper reviews the available work on siliceous concrete properties affecting its tensile behavior during heating, cooling, and post-cooling stages. These properties are used to develop

estimates for the concrete tensile behavior at each of these stages. Based on these estimates, a tensile stress strain relationship is proposed.

As a case study, eleven siliceous concrete slabs sprayed with a TSL layer were tested in flexure after 85 days from fire exposure. These slabs were constructed and exposed to fire at the side coated with the TSL layer. Flexural tests were conducted on two groups of slabs with the TSL layer either in compression or in tension. The case study allowed examining the proposed material models and assessing the effect of the TSL layer on the behavior of the slabs.

RESEARCH SIGNIFICANCE

Fire-damaged concrete elements are usually assessed using destructive and/or non-destructive field tests. However, there are no design guidelines to help designers predict the overall behavior of a fire-damaged structure. Few experimental investigations can be found in the literature addressing concrete residual compressive capacity after fire exposure. A review of the analytical and experimental work related to the concrete tensile behavior is presented in this paper. Formulas to predict the concrete tensile stress-strain relationships are proposed. The paper provides essential knowledge for structural engineers to judge on the safety of unreinforced or lightly reinforced fire-damaged concrete members.

CONCRETE COMPRESSIVE STRENGTH

Concrete compressive strength at elevated temperatures, $(f'_{cT})_{t=0}$, can be evaluated experimentally by heating unloaded or loaded concrete specimens and testing the hot concrete in compression. The time t is measured from the day concrete was exposed to fire. To evaluate the residual compressive strength, $(f'_{cT})_t$, specimens are cooled-down before testing. The following sections trace changes in $(f'_{cT})_t$ as the time t increases.

Heating Stage

Concrete compressive strength experiences a large reduction at temperatures above 300 °C [572 °F]⁴, **Fig. 1a** at $t = 0$. This reduction is caused by the complete evaporation of chemically bound water, the decomposition of the calcium hydroxide crystals CH at 400-600 °C [752-1112 °F], and the full destruction of the gel structure C-S-H at

900 °C [1652 °F]^{4, 7}. Several numerical models were developed by different researchers to estimate $(f'_{cT})_{t=0}$. A review of these models is given by Youssef and Moftah² and resulted in recommending Hertz model⁴, Eq. (1).

$$(f'_{cT})_{t=0} = \frac{f'_c}{1 + \frac{T}{15,000} + \left(\frac{T}{800}\right)^2 + \left(\frac{T}{570}\right)^8 + \left(\frac{T}{100,000}\right)^{64}} \quad (1)$$

Where T is the elevated temperature in degree Celsius [$T\text{ °C} = (T\text{ °F} - 32) \times 5/9$] and f'_c is the siliceous concrete compressive strength at ambient temperature.

Heating loaded concrete specimens reduces the effect of fire temperature on its compressive strength. Applied external loads act as crack-development inhibitors. Hertz⁴ reported that preloading siliceous concrete specimens with a stress level ($\lambda_L = f_c / f'_c$) of 0.25 to 0.30 results in 25% increase in $(f'_{cT})_{t=0}$ for temperatures between 500 °C and 700 °C [932 °F and 1292 °F].

Cooling Stage

$(f'_{cT})_t$ is greatly affected by the type of the cooling process. Rapid cooling, such as quenching in water, result in filling the micro-cracks formed during the heating stage by new hydration products and thus $(f'_{cT})_t$ increases upon cooling. On the other hand, slow cooling results in widening the micro-cracks⁴ due to the relative hydration expansion between the moist outer layer and the dry inner core⁶ and thus $(f'_{cT})_t$ decreases upon cooling.

Post-Cooling Stage

With time, the induced cracks during the heating and cooling stages start to be refilled with the new hydration products. Poon et al.⁷ and Crook and Murray⁹ have immersed concrete specimens in water after heating to 600 °C [1112 °F] and 500 °C [932 °F], respectively. These specimens experienced an accelerated re-curing that enhanced the strength regain. Sarshar and Khoury⁶ used thermally stable firebrick aggregates to isolate the effect of aggregate and the cement paste. Harada et al.⁸ used concrete mixes with siliceous aggregates and natural air drying where the air moisture was the only source affecting the re-curing process (relative humidity $\cong$ 65%). This experimental work is considered the most appropriate to simulate the behavior of fire damaged concrete in reality. Moreover, Harada et al.⁸ reported $(f'_{cT})_t$ values at temperatures up to 500 °C [932 °F] and for a relatively long period of re-curing, one year, as shown in **Fig. 1a**. The observed initial reduction in $(f'_{cT})_t$ after cooling is due to widening of the micro-