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# **High Strength Concrete at High Temperature**

# by L.T. Phan

<u>Synopsis:</u> Effects of elevated temperature exposure and various factors, including waterto-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.

<u>Keywords</u>: elevated temperatures; explosive spalling; high-strength concrete; polypropylene fibers; pore pressure

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#### **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  $100 \times 200$  mm cylinders. The other measured the heat-induced internal pore pressures in concrete using  $100 \times 200 \times 200$  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  $(0.4f_{23^{\circ}C})$  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 onedimensional heat flow condition that exists in a concrete wall exposed to fire, and were subjected only to heating without any mechanical loading.

#### **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  $1.42 \text{ g/cm}^3$  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  $\mu$ m, and the 38-mm long fiber has a diameter of 300  $\mu$ 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 °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

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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/m<sup>3</sup>, and 3 kg/m<sup>3</sup>); (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/m<sup>3</sup>) (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 ( $f_{\theta'} f_{23^{\circ}C}$ ) 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.

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(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 Europeen de Normalisation CEN EN 1992 (2004), the Comites Euro-International du Beton CEB (1991), and the National Building Code of Finnland's RakMK B4 (1991).

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,  $0.3 f_{23^{\circ}C}$ ) 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

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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/m<sup>3</sup> 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 from the heated face of the specime at points 25 mm and 50 mm from 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)).