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Structural Behaviour of High Strength Concrete Columns Exposed to Fire

by V.K.R. Kodur and Mohamed A. Sultan

Synopsis: The increased use of high strength concrete (HSC) in buildings has resulted in concern regarding the behaviour of such concrete in fire. In particular, spalling at elevated temperatures, as identified in studies by a number of laboratories, is of particular concern.

In this paper, the results of an experimental program are used to trace the structural behaviour of reinforced concrete columns at elevated temperatures. A comparison is made of the fire resistance performance of HSC columns with that of normal strength concrete (NSC) columns. The factors that influence the thermal and structural behaviour of HSC concrete columns under fire conditions are discussed. The results presented will generate data on the fire resistance of high performance concrete columns and contribute to identifying the difference in behaviour between HSC and NSC columns.

Keywords: fire resistance, high strength concrete, high temperature behaviour, reinforced concrete columns, spalling

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INTRODUCTION

In recent years, the construction industry has shown significant interest in the use of high strength concrete (HSC). This is due to the improvements in structural performance, such as high strength and durability, that it can provide compared to traditional, normal strength concrete (NSC). Recently, the use of high strength concrete, which was previously in applications such as bridges, off-shore structures and infrastructure projects, has been extended to high rise buildings. One of the major uses of HSC in buildings is for columns.

The specifications for fire resistance for structural members are contained in the National Building Code of Canada (NBCC)[1]. Concrete structures in Canada are to be designed in accordance with the CSA A23.3-M94 standard[2]. The most recent edition of this standard contains detailed specifications on the design of HSC structural members, however, there are no guidelines for the fire resistance design of HSC structural members either in CSA A23.3-M94[2] or the NBCC[1].

The results of fire tests in a number of laboratories[3,4], have shown that there are well-defined differences between the properties of HSC and NSC at elevated temperatures. Further, concern has developed regarding the occurrence of explosive spalling when HSC is subjected to rapid heating, as in the case of a fire[3,5].

Studies are in progress at the National Research Council of Canada (NRC) to develop fire resistance design guidelines for the use of high strength concrete and for possible incorporation in codes and standards. The main objective of this research, being undertaken in partnership with Concrete Canada and the concrete industry, is to determine the behaviour of HSC at elevated temperatures and to develop solutions to minimize spalling and thus enhance fire resistance. In this paper, the results of experiments are used to trace the structural behaviour of HSC concrete columns at elevated temperatures.

FIRE RESISTANCE OF HIGH STRENGTH CONCRETE

In buildings, HSC structural members are to be designed to satisfy the requirements for serviceability and safety limit states. One of the major safety requirements in building design is the provision of appropriate fire safety measures for structural members. The basis for this requirement can be attributed to the fact that, when other measures for containing the fire fail, structural integrity is the last line of defence.

The fire safety measures for structural members are measured using fire resistance. Fire resistance is defined as the ability of a structural element to withstand its load-bearing function under fire conditions. It is the time during which a structural member exhibits resistance with respect to structural integrity, stability and temperature transmission. The fire resistance of a structural member is dependent on the geometry, the materials used in construction, the load intensity and the characteristics of the exposing fire itself.

Generally, concrete structural members exhibit good performance under fire situations. Studies show, however, that the performance of HSC is different from that of NSC and may not exhibit good performance in fire. Further, the spalling of concrete under fire conditions is one of the major concerns due to the low water-cement ratio in HSC. The spalling of concrete exposed to fire has been observed under laboratory and real fire conditions[5,6]. Spalling, which results in the rapid loss of concrete during a fire, exposes deeper layers of concrete to fire temperatures, thereby increasing the rate of transmission of heat to the inner layers of the member, including to the reinforcement.

Spalling is theorized to be caused by the build-up of pore pressure during heating[3,7]. HSC is believed to be more susceptible to this pressure build-up because of its low permeability compared to NSC. The extremely high water vapour pressure, generated during exposure to fire, cannot escape due to the high density of HSC and this pressure often reaches the saturation vapour pressure. At 300°C, the pressure reaches about 8 MPa. Such internal pressures are often too high to be resisted by the HSC mix having a tensile strength of about 5 MPa (5).

Data from various studies[3,4,5] show that fire performance of HSC, in general, and spalling, in particular, is affected by the following factors:

- original compressive strength
- moisture content of concrete
- concrete density
- heating rate
- specimen dimensions and shape
- loading conditions

To determine the behaviour of loaded HSC columns under fire conditions, studies were undertaken on two types of concrete columns; namely NSC and HSC. In this paper, the comparative performance of HSC and NSC columns is discussed by considering the results from three reinforced concrete columns: two of HSC and one of NSC.

EXPERIMENTAL STUDIES

Test Specimens

The experimental program consisted of conducting fire resistance tests on three reinforced concrete columns. Two of these columns, HSC1 and HSC2, were of high strength concrete while the third one, NSC1, was made with normal strength concrete. All columns were 3810 mm long and were of square cross section of 305 mm size. The dimensions of the column cross section and other specifics of the columns are given in Table 1.

Columns HSC1 and HSC2 had eight, 20 mm, longitudinal bars, while Column NSC1 had four, 20 mm, longitudinal bars. In the HSC columns, the bars were tied with 10 mm ties at a spacing of 225 mm; while in the NSC columns, the bars were tied with 8 mm ties at a spacing of 305 mm. The main reinforcing bars and ties had a specified yield strength of 400 MPa. Figure 1 shows the elevation and cross-sectional details of the columns together with the locations of the ties.

Three batches of concrete were used in fabricating the columns. The mix for Batch 1 and Batch 2 used HSC, while the mix for Batch 3 used NSC. The coarse aggregate in Batches 1 and 3 was carbonate while in Batch 2 it was siliceous aggregate. Columns HSC1, HSC2 and NSC1 were fabricated from Batch 1, Batch 2 and Batch 3, respectively. All three batches of concrete were made with general purpose, Type 10 portland cement. The mix proportions, per cubic metre of concrete, in the three batches are given in Table 2.

The average compressive cylinder strengths of the concrete, measured 28 days after pouring and on the day of the testing, are given in Table 1. The moisture condition at the centre of the column was also measured on the day of the test. The moisture conditions of Columns HSC1, HSC2 and NSC1 are approximately equivalent to those in equilibrium with air of 69% relative humidity (RH), 63% RH and 75% RH, respectively, at room temperature.

Type-K Chromel-alumel thermocouples, 0.91 mm thick, were installed at midheights in the columns for measuring concrete temperatures at different locations in the cross section.

Test Apparatus

The tests were carried out by exposing the columns to heat in a furnace specially built for testing loaded columns. The furnace consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework. The test furnace was designed to produce conditions, such as temperature, structural loads and heat transfer, to which a member might be exposed during a fire. The furnace has a loading capacity of 1,000 t. Full details on the characteristics and instrumentation of the column furnace are provided in Reference 8.

Test Conditions and Procedure

The columns were installed in the furnace by bolting the endplates to a loading head at the top and to a hydraulic jack at the bottom. The conditions of the columns were fixed-fixed for all tests. For each column, the length exposed to fire was approximately 3000 mm. At high temperature, the stiffness of the unheated column ends, which is high in comparison to that of the heated portion of the column, contributes to a reduction in the column effective length. In previous studies[9], it was found that, for columns tested fixed at the ends, an effective length of 2000 mm represents experimental behaviour.

All columns were tested under concentric loads. Column HSC1 was subjected to a load of 2000 kN, which is equal to 60% of the ultimate load according to CSA-A23.3-M94[2]. Column HSC2 was subjected a load of 1700 kN or 50% of the ultimate load, and Column NSC1 to a load of 1067 kN or 60% of the ultimate load. The load intensity, defined as the ratio of the applied load to the column resistance, varied slightly from Column HSC1 to HSC2 to determine the influence of load on fire resistance.

The load was applied approximately 45 min before the start of the fire test and was maintained until a condition was reached at which no further increase of the axial deformation could be measured. This was selected as the initial condition for the axial deformation of the column. During the test, the column was exposed to heating controlled in such a way that the average temperature in the furnace followed, as closely as possible, the ASTM E119-88[10] or CAN/ULC-S101[11] standard temperature-time curve. The load was maintained constant throughout the test. The columns were considered to have failed and the tests were terminated when the hydraulic jack, which has a maximum speed of 76 mm/min, could no longer maintain the load.

Results and Discussion

The temperature-time curves for the external surface and for various depths in concrete are plotted in Figs. 2 to 4 for columns HSC1, HSC2 and NSC1 respectively. The measured temperature in the furnace followed the standard temperature-time curve, as shown in the figures. For all three columns, the temperatures inside the column rose rapidly to about 100°C and then the rate of increase of temperature decreased. Lie[12] has shown that this temperature behaviour is due to the thermally-induced migration of moisture toward the centre of the column. The influence of moisture migration is the highest at the centre of the column.

The variation in axial deformation with time for the three columns is shown in Figs. 5 to 7. Both the NSC and HSC columns expanded until the reinforcement yielded and then contracted leading to failure. It can be seen from the Figure that the deformation behaviour of HSC columns was similar to that of the NSC column, in the expansion zone and the deformation in the HSC columns was slightly less than that for the NSC column. The deformation in the columns resulted from several factors, such as load, thermal expansion and creep. The initial deformation of the column was mainly due to the thermal expansion of concrete and steel. While the effect of load and thermal expansion is significant in the intermediate stages, the effect of creep becomes pronounced in the later stages due to the high fire temperature. This is one of the main reasons that the deformations are quite large before the failure of the columns.

All three columns failed in compression mode. In the NSC column, no significant spalling occurred until the failure of the column. In the case of HSC columns, there was noticeable spalling at the end of the test. The cracks in these HSC columns progressed at the corners of the cross section, with the increase in time, and led to spalling of chunks of concrete. This spalling was significant at about mid-height. While minute cracks could be noticed in about 20 to 30 min, the widening of these cracks occurred after about 60 min or so. In the case of Column HSC1 (carbonate aggregate concrete), the spalling was less compared to Column HSC2 (siliceous aggregate). This could be attributed to the effect of aggregate used in the concrete mix. The specific heat of carbonate aggregate concrete, above 600°C temperature, is generally much higher than that of siliceous aggregate concrete. This heat is approximately ten times the heat needed to produce the same temperature rise in siliceous aggregate concrete. The increase in specific heat is likely caused by the dissociation of the dolomite in the carbonate concrete and is beneficial in preventing spalling of the concrete[13].

In Table 1, a comparison of the fire resistance values for the three columns is shown. In the fire tests, the time to reach failure is defined as the fire resistance for the column. For the NSC column, the fire resistance was approximately 366 min while, for Columns HSC1 and HSC2, it was 225 and 189 min, respectively. The decreased fire resistance for HSC columns, as compared to the NSC column, can be attributed to the thermal and mechanical properties of HSC. Further, the spalling phenomenon, which resulted in the decrease in the cross-section of the column, also contributed to lowering the fire resistance in the HSC columns. The lower fire resistance of Column HSC2, as compared to that of Column HSC1, can be attributed to the type of aggregate used in the concrete mix, as explained above, and also to the lower load intensity on this column. Columns made of siliceous aggregate concrete typically have lower fire resistance compared to those made with carbonate aggregate concrete[13,14].

RELATIVE PERFORMANCE OF HSC

To further illustrate the behaviour of HSC columns, data from the tests was used to compare the relative performance of the HSC1 column to that of the NSC1 column. The comparative performance of NSC and HSC columns under fire conditions is illustrated in Figs. 8 and 9. Except for the concrete strength, the NSC and HSC columns had similar characteristics and were subjected to comparable load levels. The variation of cross-sectional temperatures for NSC and HSC columns is shown in Fig. 8 as a function of exposure time. These temperatures, measured during the fire tests, are shown for various depths from the surface along the centreline and at mid-height of the column. It can be seen from Fig. 8 that the temperatures in the HSC column are generally lower than the corresponding temperatures in the NSC column throughout the fire exposure. This variation can be attributed partly to the variation in thermal properties of the two concretes and to the higher compactness (lower porosity) of HSC. The lower porosity of HSC affects the rate of increase of temperature until the cracks widen and spalling occurs. Large cracks occurred in the HSC column after about 3 h of fire exposure. While there was no spalling in the NSC column, significant spalling at the corners was observed just before failure occurred in the HSC column.

The variation of axial deformation with time is compared for NSC and HSC columns in Fig. 9. It can be seen from the figure that the behaviour of the HSC column was different from that of the NSC column. In the case of the HSC column, the deformation is significantly lower than that of the NSC column. This can be attributed partly to the lower thermal expansion of HSC and the slower rise of temperature in the HSC column during the initial stages due to the high compactness of HSC. When the steel reinforcement in the column gradually yields, because of increasing temperatures, the column contracts.

When the steel yields, the concrete carries a progressively increasing portion of the load. The strength of the concrete also decreases with time and, ultimately, when the column can no longer support the load, failure occurs. At this stage, the column behaviour is dependent on the strength of the concrete. There is significant contraction in the NSC column leading to gradual ductile failure. The contraction in the HSC column is much lower. This can be attributed to the fact that HSC becomes brittle at elevated temperatures and the strain attained at any stress level is lower than that attained in NSC for any given temperature. This is especially applicable to the descending portion of the stress-strain curve of HSC at elevated temperatures.

CURRENT RESEARCH

The main objective of the experimental studies reported above was to obtain test data for the development of computer programs that can model the behaviour of HSC columns under fire conditions. In the past, the fire resistance of structural members could be determined only by testing. In recent years, however, the use of numerical methods for the calculation of the fire resistance of various structural members has been gaining acceptance[14,15]. These calculation methods are far less costly and time consuming. For these calculation methods to be used with assurance, however, the material properties at elevated temperatures are required.

At present, studies are in progress at NRC to develop the mechanical properties of HSC at elevated temperatures[16]. The data on thermal and mechanical properties is being used to develop thermal and mechanical relationships, as a function of temperature. These relationships can be used as input to numerical models which in turn can be used to determine the behaviour of HSC structural members at high temperatures. The development of computer programs for the calculation of the fire resistance of HSC columns is also in progress at NRC[15]. Simultaneously, fire tests are being conducted on full-size HSC columns, fabricated with and without fibre-reinforcement, since data from various studies[3,4,6] show that the presence of fibre-reinforcement minimizes spalling in HSC.

The computer programs will be used to carry out detailed parametric studies to investigate the influence of the various parameters, such as concrete strength and load intensity on the fire resistance. Data from the parametric studies will be used to develop design guidelines for predicting the fire resistance of HSC columns and to overcome the problem of spalling.

CONCLUDING REMARKS

Based on the studies completed so far, it was found that:

1. The behaviour of HSC columns at high temperatures is significantly different from that of NSC columns. The fire resistance of HSC columns is lower than that of NSC columns.
2. The type of aggregate and load intensity have an influence on the performance of HSC columns at elevated temperatures. The presence of carbonate aggregate in HSC increases fire resistance.
3. The studies, currently in progress at NRC, will generate data on the fire resistance of HSC columns and will identify the conditions under which these columns can be safely used.

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Table 1 - Summary of Test Parameters and Result

Column	Column Dimensions (mm)	Concrete Strength (f'c) 28 day (MPa)	test day (MPa)	Factored Resistance (Cr) (kN)	Test Load (C) (kN)	Load Intensity (C/Cr)	Fire Resistance (hr:min)
HSC1	305 x 305	83	97	3270	2000	0.61	3:45
HSC2	305 x 305	81	86	3346	1700	0.5	3:09
NSC1	305 x 305	34	37	1764	1067	0.60	6:06

Table 2 - Batch quantities and properties of concrete mix

Property	Batch (specimen type) Mix 1	Mix 2	Mix 3
Cement content (kg/m^3)	500	500	346
Fine aggregate (kg/m^3)	700	700	816
Coarse aggregate (kg/m^3) (10 mm)	1100	1100	1065
Aggregate type	Carbonate	Siliceous	Carbonate
Water (kg/m^3)	140	140	193
Water - cement ratio	0.28	0.28	0.55
Retarding admixture (mL/m^3)	1450	1450	-
Silica fume (kg/m^3)	50	50	-
Superplasticizer (mL/m^3)	9000	13500	-
28 day compressive strength (MPa)	83	81	34
90 day compressive strength (MPa)	97	86	37

LIST OF FIGURES

Fig. 1 Elevation and Cross Section of Columns

Fig. 2 Measured Temperature Distribution for Column HSC1

Fig. 3 Measured Temperature Distribution for Column HSC2

Fig. 4 Measured Temperature Distribution for Column NSC1

Fig. 5 Measured Axial Deformation for Column HSC1

Fig. 6 Measured Axial Deformation for Column HSC2

Fig. 7 Measured Axial Deformation for Column NSC1

Fig. 8 Temperature Distribution at Various Depths in NSC and HSC Columns

Fig. 9 Axial Deformation for HSC and NSC Columns