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FIRE ENDURANCE OF HIGH STRENGTH CONCRETE COLUMNS

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Abstract

In buildings, fire represents one of the most severe environmental conditions and therefore, should be properly accounted for in the design of high strength concrete (HSC) structural members. The increased use of HSC in buildings has raised concerns regarding the behaviour of such concrete in fire. In particular, spalling at elevated temperatures, as identified in studies by a number of laboratories, is a main concern.

In this paper, results from an experimental program on the fire resistance of HSC columns are presented. The factors that influence the thermal and structural behaviour of HSC concrete columns under fire conditions are discussed. Data from this study indicate that the type of aggregate, concrete strength, load intensity and detailing and spacing of ties have an influence on the fire resistance performance of HSC columns. Further, the test results show that tie configuration (bending of ties at 135° ties and provision of cross ties) and closer tie spacing has a significantly beneficial effect on the fire resistance of HSC columns. The results presented will generate data on the fire resistance of HSC columns and contribute to identifying the factors that influence the behaviour of HSC columns.

Key words: fire resistance, fire endurance, high strength concrete, high temperature behaviour, reinforced concrete columns, spalling.

1. 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). The use of HSC, which was mainly in applications such as bridges, offshore structures and infrastructure projects, is becoming more popular in 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 increased use of HSC has raised concerns regarding the behaviour of such concretes in fire. In particular, the occurrence of spalling (often explosive) at elevated temperatures, when HSC is subjected to rapid heating, as in the case of a fire, is one of the reasons for this concern [3]. Further, results of fire tests in a number of laboratories [3,4,5,6] have shown that there are well-defined differences between the properties of HSC and NSC at elevated temperatures. However, many of these studies were undertaken on small-scale specimens and also under hydrocarbon fire exposures which is different from standard building fire exposures.

Studies are in progress at the National Research Council of Canada (NRC), in partnership with the Portland Cement Association (PCA) and the Cement Association of Canada (CAC), to develop fire resistance design guidelines for the use of HSC and for possible incorporation in codes and standards [7,8]. The main objective of this study is to determine the behaviour of full-scale HSC structural columns under standard building fire exposures and to evaluate its fire endurance. As part of experimental studies full-scale fire endurance tests on HSC columns were conducted and the results from these experiments are presented in this paper.

2. Concrete under Severe Conditions - Fire

Structural members are to be designed to satisfy the requirements of serviceability and safety limit states for various environmental conditions. Fire represents one of the most severe conditions and hence the provision of appropriate fire safety measures for structural members is a major safety requirement in building design [1]. 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.

Generally, concrete structural members (traditionally constructed of NSC) exhibit good performance under fire situations. Studies show, however, that the performance of HSC is different from that of NSC and may not exhibit similar performance (as that of NSC) in fire. Further, the spalling of concrete under fire conditions is one of the major concerns due to the low porosity in HSC. The spalling of concrete exposed to fire has been observed under laboratory and real fire conditions [3,5]. 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 the reinforcement.

Spalling is theorized to be caused by the build-up of pore pressure during heating [4,5]. 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 [3].

Data from various studies [3,5,6] show that the fire behaviour of HSC, in general, and spalling, in particular, is affected by a number of factors. To determine the behaviour of loaded HSC columns under fire conditions, studies were undertaken on HSC columns. In this paper, results from these experimental studies are presented and the fire performance of HSC columns is discussed.

3. Experimental Studies

3.1 Column Specimens

The experimental program consisted of conducting fire resistance tests on six reinforced concrete columns; designated HSC1 to HSC6. All six columns were made of HSC and were of 3810 mm length. Three of the columns, HSC1 to HSC3, were of 406 mm cross-sections while the remaining three, HSC4 to HSC6, were of 305 mm. The dimensions of the column cross section and other specifics of the columns are given in Table 1.

Table 1 - Summary of Test Parameters and Results

Column Designation	Size mm (square c/s)	Reinf. Qty – Size	Tie-conf.- spacing	Agg. Type	Factored Resistance kN	Comp. strength (test day) MPa	R H %	Test Load kN	Test Load ----- Full Serv. Load	Fire End. minutes
HSC1	406	8-25M	90°@ h	S	6436	100	69	3,080	0.63	239
HSC2	406	8-25M	90°@ h	S	5900	86	86	2,406	0.54	224
HSC3	406	8-25M	90°@ h	C	7400	96	57	4,919	0.90	104
HSC4	305	8-15M	135°@ h/2	C	3145	120	50	1,979	0.82	266
HSC5	305	8-15M	135°@ h/4	C	3145	120	68	2,363	0.98	290
HSC6	305	8-15M	135°@ h/4	C	3145	120	64	2,954	1.23	266

h: width of the column S: Siliceous C: Carbonate RH: Relative humidity

The longitudinal reinforcement in columns was symmetrical arrangements of 25mm (designated as No. 8 or 25M) bars for Columns HSC1 to HSC3 and 16 mm (designated as No. 5 or 15M) bars for Columns HSC4 to HSC6. The ties were of 8 mm (No. 3) diameter bars for Columns HSC1 to HSC3 while for columns HSC4 to HSC6 they were of 6 mm (1/4 in) diameter bars. The spacing of ties for Columns HSC1 to HSC3 was 406 mm. For Columns HSC4 to HSC6, the tie spacing was varied, in terms of the width of the column (h), as indicated in Table 2. Deformed bars meeting requirements of ASTM A615-80 were used as the reinforcement and these bars had specified yield strength of 414 MPa.

The ties in columns HSC1 to HSC3 were lapped with 90° bends at the ends. In columns HSC4 to HSC6, the ends of all ties and cross-ties were bent 135° back into the core of the column. The confinement effect in columns HSC6 were enhanced by providing additional two cross-ties, in addition to 6 mm (1/4 in) ties at 76 mm spacing, respectively. The clear concrete cover of the ties is 38 mm for Columns HSC1 to HSC3 and 41 mm for Columns HSC4 to HSC6. Full details of ties, including the spacing and configuration, are given in Table 1 and Reference 7. Figure 1 shows the elevation and cross-sectional details of the columns together with the locations of the ties.

Four batches of concrete mixes were used for fabricating the columns to obtain required test variables, such as concrete strength, aggregate type and size. Columns HSC1 and HSC2 were fabricated with siliceous aggregate, while the remaining batches were made with carbonate aggregate. All four batches of concrete were made with general purpose, Type 10 Portland cement (7). The mix proportions, per cubic metre of concrete in the three batches, are given in

Table 2. The 28-day compressive strengths of concrete batches, as determined from tests on 152 mm x 305 cylinders, ranged from 81 MPa to 107 MPa and are given in Table 2.

Table 2 - Batch quantities and properties of concrete mix

Property	Batch (specimen type)			
	HSC1	HSC2	HSC3	HSC4-HSC6
Cement content – Type 1 (kg/m ³)	372	296	363	373
Silica fume - dry weight (kg)	36.3	29.5	-	-
Slag	-	-	-	164
Fine aggregate (kg/m ³)	517.6	697.8	522.1	740
Coarse aggregate (kg/m ³) (10 mm)	817	710	844	1120
Aggregate type	Siliceous	Siliceous	Carbonate	Carbonate
Water (kg/m ³)	119	117	114	72
Water : Cement ratio	0.32	0.395	0.313	0.223
Water : Cementitious ratio	0.291	0.359	0.359	0.313
HRWR Type F, L	8.6	5.1	7.5	10
Retarder Type D, L	0.7	0.6	0.7	2
28 day compressive strength (MPa)	83.7	81	84	107

Type-K Chromel-alumel thermocouples, 0.91 mm thick, were installed at mid-height in the columns for measuring concrete temperatures at different locations in the cross section. To measure the concrete temperatures, thermocouples were mounted on supplemental support, while steel temperatures were measured by mounting the thermocouples on rebars and ties.

Full details on fabrication of these columns, including rebar arrangement, concrete placement, curing and instrumentation is given by Kodur et al. [7].

3.2 Experimental Set-up

The experimental studies were carried out by exposing the columns to heat in a propane-fired 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, lined with fiber-proux on ceramic bricks, 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 9.

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. All columns were tested with both ends fixed, i.e., restrained against rotation and horizontal translation. For this purpose, eight 19 mm (3/4-in.) diameter bolts, spaced regularly around the column, were used at each end to bolt the end plate to the loading head at the top and to the hydraulic jack at the bottom. 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 [10], 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 a concentric load and the applied load on the columns ranged from 54% to 123% of full service load (factored compressive resistance of the column) determined according to the CSA Standard CSA-A23.3-M94 [2]. The factored compressive resistance of each column, as well as the applied loads, are given in Table 1. The factored compressive resistance of the columns were calculated using the effective length factor, K , 0.65 for fixed ends and is based on compressive strength of cylinders on the day of the test. The factored resistance, computed using the PCACOL computer program [11], together with applied loads on the column are given in Table 1. The load intensity, defined as the ratio of the applied load to the full service load, varied slightly to determine the influence of load on fire resistance.

3.3 Experimental Procedure

Since relative humidity has significant influence on the fire performance of concrete columns a year or more elapsed between the time a column was fabricated and the time it was tested [5]. The moisture condition in the concrete core of the columns was measured, at about 50.8 mm deep, on the day of the test by inserting a Vaisala* moisture sensor into a hole drilled in the concrete. In general, a moisture content, corresponding to approximately 50 to 86% relative humidity, at room temperature, was measured. The relative humidity (RH) of each column is given in Table 1. It should be noted that the measured RH are approximate values as it depends on a number of factors including measuring devices and techniques [7].

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 [12] or CAN/ULC-S101 [13] 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.

The furnace, concrete and steel temperatures, as well as axial deformations of the columns were recorded at two-minute intervals. Also, during fire tests, visual observations were made through window ports to record spalling, as well as crack propagation, in the columns.

4. Results and Discussion

The results of the six column tests are summarized in Table 1, in which the column characteristics, test conditions, and fire endurance are given for each column. Full results from experiments, including the furnace, concrete and steel temperatures, as well as the axial deformations of the column specimens recorded during the tests, are given by Kodur et al. [7].

* Certain commercial products are identified in this paper in order to adequately specify the experimental procedure. In no case does such identification imply recommendations or endorsement by the National Research Council, nor does it imply that the product or material identified is the best available for the purpose.

During the tests, special attention was paid to make visual observations and to record spalling, as well as crack propagation in the columns. Also, after the completion of fire tests, post-test observations were made to analyse the failure pattern, extent and nature of spalling and condition of rebars and ties. The following are some of the observations that were recorded:

- The cracks in the columns progressed with time, at the corners of the cross-section and led to spalling of chunks of concrete. While minute hairline cracks could be noticed at corners at about 20 to 30 minutes, the widening of these cracks occurred after about 60 minutes. These cracks would widen with time and run diagonally across the corner of the column section, reflecting on the adjacent face. Though these cracks would widen to approximately 15 mm at the 1 hr 30 minute mark, they did not intersect the reinforced core of the column, which might partly be due to spalling of concrete.
- In most columns the rebars yielded before failure occurred. A post-failure examination of ties showed that the ties in columns HSC4 to HSC6 performed better than those in Columns HSC1 to HSC3. In Columns HSC1 to HSC3, some ties, close the failure section, opened up at the corners where they were lapped with 90° bent ends. However, in Columns HSC4 to HSC6, the opening up of ties was not evident mainly due to the fact that the ties were bent 135° back into the core of the column, with an extension of 6 times the diameter of bars [2]. In addition, the rebars in columns HSC1 to HSC3 had buckled significantly, as compared to those in columns HSC4 to HSC6.
- Generally, there was no significant spalling in the first few minutes. In the column with high relative humidity (Column HSC2), some spalling was observed about 10–20 minutes into the fire endurance test. In columns where there were wider cracks, significant spalling at the corners was observed as the test progressed (after about 1 hour or so and towards the end of the test). Further, prior to the failure of the column, significant spalling was noticed in all columns. The spalling of chunks of concrete occurred when the large cracks progressed at the corners. This spalling was significant at about mid-height. In Columns HSC1 and HSC2 the extent of spalling, before the failure of the column, was quite high and this could be attributed to high relative humidity of these columns and the use of siliceous aggregate in the concrete mix [14]. Generally, the extent of spalling in columns HSC4 to HSC6 was relatively less compared to that in Columns HSC1 to HSC3. This could be attributed to tie configuration (bent 135° at the corners) and also to closer tie spacing in these columns. Columns containing only 90° ties would typically lose a significant portion of the columns section upon failure. Columns using 135° ties would exhibit the classic pyramid compression failure section with the failed section being confined locally to one or two tie spacing. Figures 2 and 3 show photographs of the column specimens, with significant spalling, after the fire tests.

Typical temperature-time values, measured during the test, in the furnace, reinforcement, and at various depths in concrete are plotted in Figs. 4 to 5 for columns HSC2 and HSC6 respectively.

These plots illustrate the typical variation of temperatures in a siliceous aggregate concrete column (HSC2) and a carbonate aggregate concrete column (HSC6). The measured temperature in the furnace followed closely the standard temperature-time curve through out the duration of the test. The temperatures inside the concrete rose rapidly to about 100°C and then the rate of increase of temperature decreased. Lie and Celikkol [15] have 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 Column HSC2 and HSC6 is shown in Figs. 6 and 7 respectively. The columns expanded until the reinforcement yielded (at higher temperatures) and then contracted leading to failure. 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.

The expansion range and its magnitude is significantly higher in Column HSC2 than in Column HSC6. This is because, due to relatively lower load level in Column HSC2, contribution from thermal expansion dominated the deformations. Further, since there was lesser confinement, Column HSC2 failed in a brittle manner just after reaching the expansion plateau, without undergoing significant contraction. However, in Column HSC6, due to a relatively higher load intensity and higher confinement level, the load and creep effects contributed significantly to deformations (contractions). The higher confinement level in HSC6 also helped to achieve gradual failure (due to creep effects), as opposed to brittle failure in Column HSC2.

All six columns failed in compression mode. In all columns, there was noticeable spalling at the end of the test. This is in contrast to the earlier studies on NSC columns, where only slight significant spalling occurred at failure of the columns [10]. In Table 1, a comparison of the fire resistance values for the columns is shown. In the fire tests, the time to reach failure is defined as the fire resistance for the column. The higher fire resistance for Columns HSC4-HSC6, as compared to the Columns HSC1-HSC3, can be attributed to the higher confinement. Further, the higher spalling, which resulted in the decrease in the cross-section of the column, also contributed to lowering the fire resistance in the columns HSC1-HSC3. The lower fire resistance of Column HSC3, as compared to that of Column HSC1 and HSC2, can be attributed to higher load intensity on this column.

5. Fire Endurance of HSC Columns

Based on the analysis of test data and the visual observations made during and after the fire tests, some of the factors that influence the fire endurance of HSC columns are briefly discussed below.

5.1 Effect of Concrete Strength

Data from these experimental studies indicate that good fire endurance can be obtained for HSC columns. However, a comparison of the fire endurance in Table 1 with that of normal strength concrete columns [6,10] indicate that the HSC columns have the lower fire endurance of the two.

While for NSC columns a fire endurance of about 6 hours was obtained in Reference 14, for HSC columns, with similar confinement, fire endurance of about 4 hours was obtained. Data from the Table shows that high fire endurance (three hours or more) can be obtained for HSC columns even under full service loads. However, HSC columns must be reinforced with increased levels of confinement reinforcement (as prescribed in CSA-A23.3-M94 [2]) over that used in NSC columns if the same fire endurance ratings are to be achieved by both types of columns.

5.2 Effect of Lateral Reinforcement

Results from experimental studies can be used to study the influence of confinement on the fire performance of HSC columns. The higher fire endurance in columns HSC4 to HSC6, as compared to columns HSC1 to HSC3, can be attributed to the closer spacing and better detailing of the column ties (ties were bent 135° back into the core of the column and increased lateral reinforcement). This is in spite of the fact that the 90° ties enclosed larger columns, which characteristically exhibit better fire resistance than smaller column sections. Also, despite the smaller section size for columns HSC4 to HSC6, these columns with higher levels of confinement, demonstrated better fire performance than the larger columns with 90° ties at a larger spacing. It should be noted that for column HSC5, where additional confinement was provided, fire endurance as high as 290 minutes was obtained even under full service load. Further, Column HSC6 has attained a high fire resistance of 266 minutes under an increased load level, as a result of enhanced confinement provided by the provision of cross-ties.

5.3 Effect of Load Intensity

The influence of load on the fire endurance of HSC columns can approximately be gauged by comparing the performance of columns HSC5 and HSC6. The fire endurance of a column increases with decreasing load. The fire endurance of Column HSC6 with a load intensity of 1.23 is 266 minutes as compared to 290 minutes for Column HSC5 with a load intensity of 0.98. This lower fire endurance occurred in spite of Column HSC6 having a higher confinement (additional cross-ties) than Column HSC5. Also, the lower fire resistance of Column HSC3, as compared to that of Column HSC1 and HSC2, is due to higher load intensity on this column.

5.4 Effect of Aggregate Type

The effect of the aggregate type on the fire endurance of HSC columns is illustrated by comparing the performance of Columns HSC1 and HSC4. While there are other differences in the configuration of these two columns, such as the concrete strength and load level, one of the major variables is the type of aggregate. The fire endurance of column HSC4 made with carbonate aggregate concrete is 10% higher than Column HSC1 made with siliceous aggregate concrete. This occurs mainly because carbonate aggregate has a substantially higher heat capacity (specific heat) than siliceous aggregate due to an endothermic reaction that takes place in carbonate aggregate at about 700°C [14]. It could be argued that the contribution to higher fire endurance in Column HSC4 is mainly due to carbonate aggregate as any positive contribution from better tie configuration goes towards counteracting the effects from a higher load level and higher concrete strength (compared to that for Column HSC1) [7].

The aggregate type also influences the spalling phenomenon. In Columns HSC1 and HSC2, the extent of spalling, before the failure of the column, was quite high as compared to the other

columns. This could be partly attributed to the effect of aggregate type used in the concrete mix. Of the two concrete types, carbonate aggregate concrete provides better spalling resistance than siliceous aggregate concrete [5,6,10]. The higher specific heat and lower thermal expansion of carbonate aggregate concrete at higher temperatures contributes to this spalling resistance [5]. The specific heat of carbonate aggregate concrete, above 600°C temperature, is generally much higher than that of siliceous aggregate concrete. This specific 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 the spalling of the concrete [14].

6. 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 [16,17]. These calculation methods are far less costly and time consuming. For these calculation methods to be used with assurance, however, the material properties of HSC at elevated temperatures are required.

At present, studies are in progress at NRC to develop thermal and mechanical properties of HSC at elevated temperatures [18]. 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. These computer programs are being used to develop guidelines for fire resistance design of HSC columns [19].

7. Summary

Based on the experimental studies the fire endurance of HSC columns with equivalent levels of confinement is lower than that of NSC columns. However, fire endurance up to 4 hours can be obtained with HSC columns if sufficient confinement is provided.

The type of aggregate, porosity (concrete strength), load intensity, detailing and spacing of ties have an influence on the fire resistance performance of HSC columns. The tie configuration (bending of ties at 135° ties and provision of cross ties) and closer tie spacing has a significantly beneficial effect on the fire resistance of HSC columns.

No significant spalling was observed in the initial stages of fire exposure. Generally, the spalling in HSC columns was significant only towards the end of the tests. The type of aggregate, relative humidity, porosity, load intensity, detailing and spacing of ties has an influence on spalling.

These fire tests were carried out in order to develop design guidelines for predicting the fire endurance of HSC columns. However, further data are needed to quantify the influence of various parameters on the fire performance of HSC columns. The studies currently in progress at NRC, with the collaboration of PCA, will generate additional data on the fire endurance of HSC columns and will identify the conditions under which these columns can be safely used.

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Fig. 2 View of Column HSC2 after Fire Test

Fig. 3 View of Column HSC6 after Fire Test