Effect of Strength and Fiber Reinforcement on Fire Resistance of High-Strength Concrete Columns

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Abstract: In this paper, results from fire resistance experiments on five types of reinforced concrete columns are presented. The variables considered in the study include concrete strength [normal-strength concrete (NSC) and high-strength concrete (HSC)], aggregate type (siliceous and carbonate aggregate), and fiber reinforcement (steel and polypropylene fibers). Data from the study is used to determine the structural behavior of HSC columns at elevated temperatures. A comparison is made of the fire resistance performance of HSC columns with that of NSC and fiber-reinforced HSC columns. The factors that influence the thermal and structural behavior of HSC concrete columns under fire conditions are discussed. The results show that the fire resistance of a NSC column is higher than that of a HSC column. Also, the addition of polypropylene fibers and the use of carbonate aggregate improve fire resistance.

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 columns form the main load bearing components in a building and hence, the provision of appropriate fire safety measures is one of the major safety requirements in building design (NBC 1995).

In recent years, the design standards for concrete structures have been updated with detailed specifications for the design of HSC structural members under normal conditions. However, in many of the design standards (CSA 1994), there are no guidelines for the fire resistance design of HSC structural members.

The results of fire tests in a number of laboratories (Diederichs et al. 1995; Kodur and McGrath 2001) have shown that there are well-defined differences between the properties of HSC and NSC at elevated temperatures. Further, concern has developed regard-ing the occurrence of explosive spalling when HSC is subjected to rapid heating, as in the case of a fire (Diederichs et al. 1995; Phan 1996).

A collaborative research project was initiated between National Research Council of Canada (NRC) and National Chiao Tung University (NCTU), Taiwan, to develop fire resistance design guidelines for the use of high-strength concrete and for possible incorporation in codes and standards. One component of this project covered detailed fire resistance experimental studies on five different types of HSC columns. The main objective of this research was to determine the behavior 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 behavior of HSC columns at elevated temperatures.

Research Significance

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 active measures for containing the fire fail, structural integrity is the last line of defense.

The fire safety measures for structural members are measured in terms of fire resistance. Fire resistance is defined as the duration during which a structural member exhibits resistance with respect to structural integrity, stability, and temperature transmission under fire conditions. 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 fire exposure 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

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Table 1. Summary of Test Parameters and Result

| Column | Column dimensions (mm) | Concrete Strength (fc) | | Factored resistance (Cr) (kN) | Test load (C) (kN) | Load intensity (C/Cr) | Fire resistance (h:min) |
		28 day (MPa)	Test day (MPa)				
TNCl	305×305	27.8	40.2	1,728	930	0.54	4:38
THC4	305×305	60.6	99.6	3,697	2,000	0.54	3:24
THC8	305×305	60.4	72.7	2,805	2,000	0.71	5:05
THS10	305×305	63.2	89.1	3,349	1,800	0.54	3:59
THP13	305×305	51.9	86.6	3,266	1,800	0.54	4:31

under fire conditions is one of the major concerns due to the low-water-binder ratio in HSC. The spalling of concrete exposed to fire has been observed under laboratory and real fire conditions (Matsumoto 1994; Phan 1996). 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 buildup of pore pressure during heating (Diederichs et al. 1995; Kodur and Lie 1998). High-strength concrete is believed to be more susceptible to this pressure buildup because of its low permeability compared to NSC. The extremely high-water vapor pressure, generated during exposure to fire, cannot escape due to the high density of HSC and this pressure often reaches the saturation vapor 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 (Phan 1996).

Data from various studies (Diederichs et al. 1995; Phan 1996; Kodur and McGrath 2001) show that fire performance of HSC, in general, and spalling, in particular, is affected by the following factors:

- Concrete strength,
- Moisture content of concrete,
- Concrete density,
- Heating rate,
- Specimen dimensions and shape,
- Loading conditions,
- Fiber reinforcement,
- Aggregate type, and
- Reinforcement layout and configuration.

To determine the effect of strength and the addition of fiber reinforcement on the behavior of loaded HSC columns under fire conditions, studies were undertaken on five reinforced concrete columns. In this paper, the comparative performance of HSC and NSC columns is discussed by considering the results from five reinforced concrete columns: four of HSC and one of NSC. The variables considered are strength (NSC and HSC), aggregate type (siliceous and carbonate aggregate), and fiber reinforcement (steel and polypropylene fibers).

Experimental Studies

Test Specimens

The experimental program consisted of conducting fire resistance tests on five reinforced concrete columns. Four of these columns, THC4, THC8, THS10, and THP13, were of HSC while the fifth one, TNC1, was made with NSC. All columns were 3,810 mm

long and of square cross section of 305 mm. The dimensions of the column cross section and other specifics of the columns are given in Table 1.

All columns had four, 25 mm, longitudinal bars. The bars were tied with 10 mm ties at a spacing of 75 mm in both ends and 145 mm in the middle. The main reinforcing bars and ties had a specified yield strength of 420 and 280 MPa, respectively. Fig. 1 shows the elevation and cross-sectional details of the columns together with the locations of the ties.

Five batches of concrete were used in fabricating the columns. The concrete mix for Batch 1 was of NSC, while the mixes for Batch 2–5 were of HSC. Steel and polypropylene fiber reinforcements were added to Batch 4 and Batch 5, respectively. The coarse aggregate in Batch 3 was of carbonate type, while the other batches were made with siliceous aggregate. Columns TNC1, THC4, THC8, THS10, and THP13 were fabricated from Batch 1, Batch 2, Batch 3, Batch 4, and Batch 5, respectively. All five batches of concrete were made with general purpose, Type 1 Portland cement. The mix proportions, per cubic meter of concrete, in the five batches are given in Table 2.

The average compressive cylinder strength 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 center of the column was also measured on the day of the test. The moisture conditions of Columns TNC1, THC2, THC8, THS10, and THP13 are approximately equivalent to those in equilibrium with air of 90, 78, 93, 80 and 94% relative humidity, respectively, at room temperature.

Type-K Chromel-alumel thermocouples, 0.91 mm thick, were installed at midheights in the columns for measuring concrete and rebar temperatures at different locations in the cross section. The location and numbering of the thermocouples on the cross section is shown in Fig. 2.

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 by Lie (1980).

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
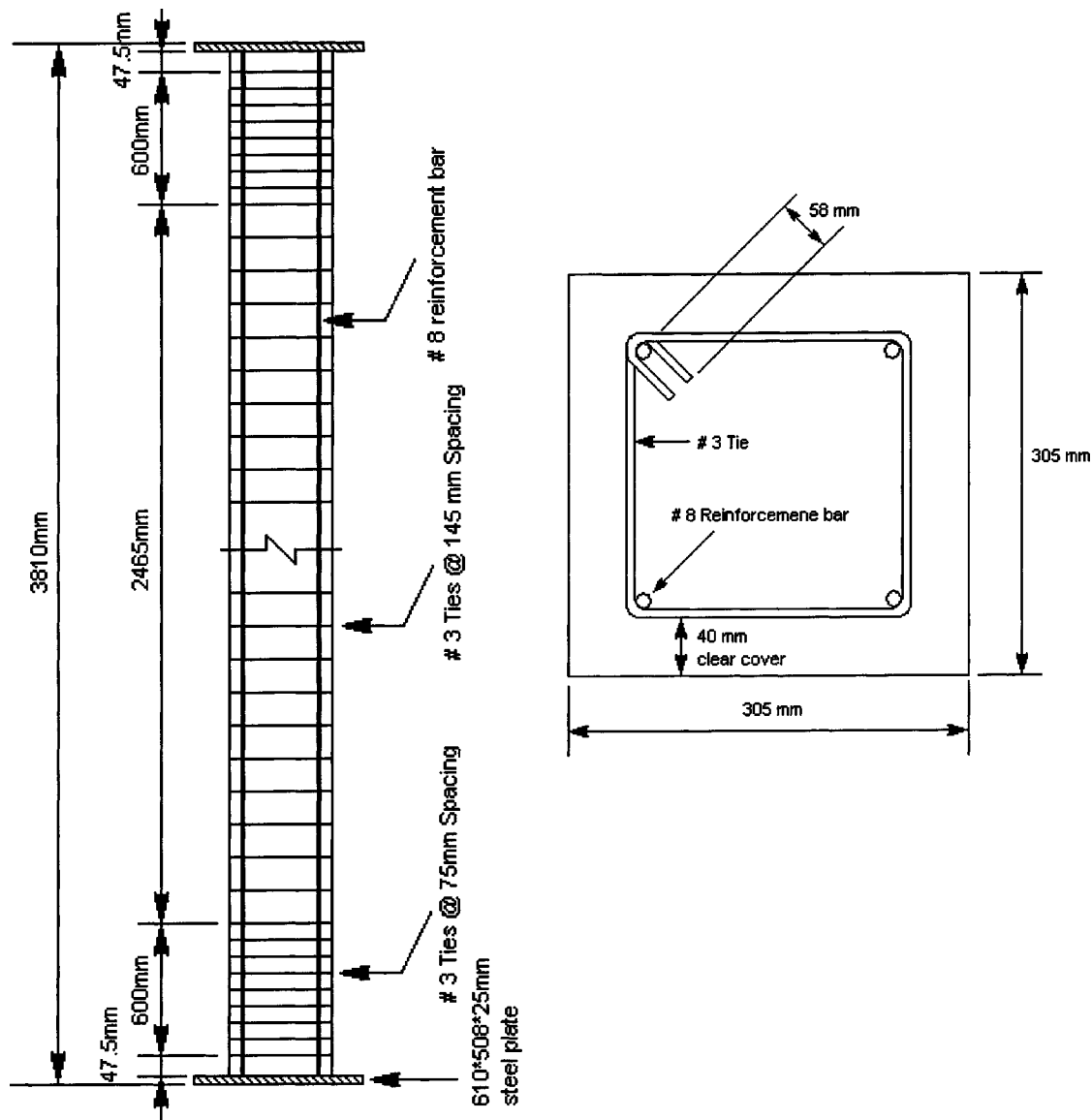
Fig. 1. Elevation and cross section of columns

bottom. The end conditions of the columns were fixed-fixed for all tests. For each column, the length exposed to fire was approximately 3,000 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 (Lie and Woollerton 1988), it was found that, for columns tested fixed at the ends, an effective length of 2,000 mm represents experimental behavior.

All columns were tested under concentric loads. Column TNC1 was subjected to a load of 930 kN, which is equal to 54% of the ultimate load according to CSA A23-3 (1994). Column THC4 was subjected a load of 2,000 kN or 54% of the ultimate load, Column THC8 was subjected a load of 2,000 kN or 71% of the ultimate load, Column THS10 was subjected a load of 1,800 kN or 54% of the ultimate load, and Column THP13 to a load of 1,800 kN or 54% of the ultimate load. The load intensity, defined as the ratio of the applied load to the column resistance, for the various columns is given in Table 1.

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 mea-

sured. This was selected as the initial condition for the axial deformation of the column. During the test, the column was exposed to heat, controlled in such a way that the average temperature in the furnace followed, as closely as possible, the ASTM E119 (ASTM 1990) or CAN/ULC-S101 (ULC 1989) 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.

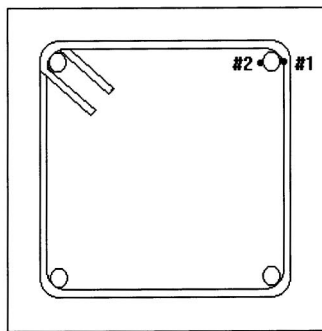## Behavior of High-Strength Concrete Columns

Variation of Temperatures

The average furnace temperature and temperature at various depths in concrete are plotted in Figs. 3–7 for Columns TNC1, THC4, THC8, THS10, and THP13, respectively. The measured temperature in the furnace followed closely the standard temperature-time curve. For all five columns, the temperatures
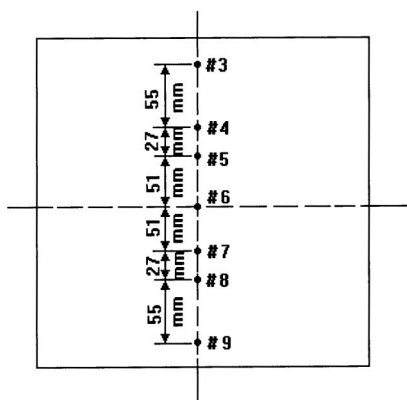
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Table 2. Batch Quantities and Properties of Concrete Mix

Property	Batch (Specimen Type)				
	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Column	TNC1	THC4	THC8	THS10	THP13
Cement content (kg/m^3)	355	483	483	483	483
Fine aggregate (kg/m^3)	949	669	667	721	721
Coarse aggregate (10 mm) (kg/m^3)	732	1,040	1,040	900	900
Aggregate type	Siliceous	Siliceous	Carbonate	Siliceous	Siliceous
Water (kg/m^3)	258	177	175	184	182
Water-binder ratio	0.73	0.38	0.37	0.39	0.39
Fiber (kg/m^3)	0	0	0	42	0.9
Fiber type	—	—	—	Steel	Polypropylene
Silica fume (kg/m^3)	0	42	42	42	42
Superplasticizer (kg/m^3)	0	20	19	18	20
28 day compressive strength (MPa)	28	61	60	63	52
90 day compressive strength (MPa)	40	100	73	89	87

inside the column rose rapidly to about 100°C and then the rate of increase of temperature decreased. Lie and Celikkol (1991) have shown that this temperature behavior is due to the thermally induced migration of moisture toward the center of the column. The influence of moisture migration is the highest at the center of the column. While there are slight differences in the temperature propagation between NSC and HSC columns, the temperature propagation for various types of HSC columns was found to be very similar.
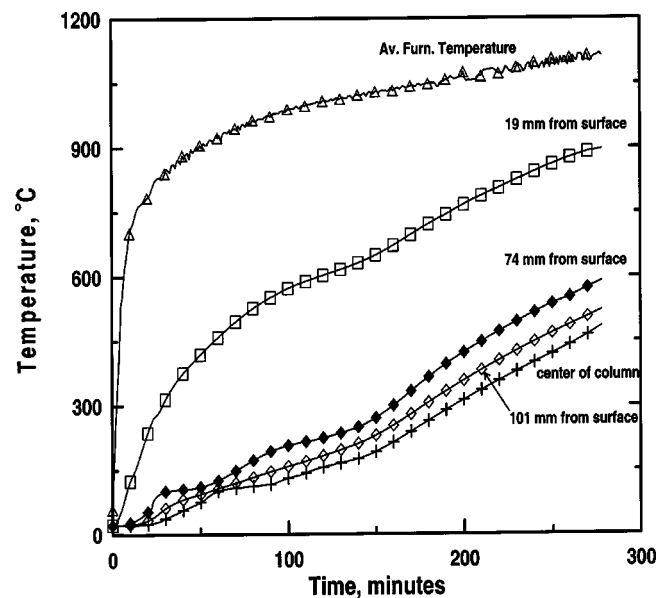
To illustrate the thermal behavior of concrete columns, with different strengths, the variation of cross-sectional temperatures for NSC and HSC columns are compared in Fig. 8 as a function of exposure time. The temperatures are shown for various depths from the surface along the centerline and at midheight of the column. Except for the strength, the NSC and HSC columns had similar characteristics and were subjected to comparable load levels. It can be seen from Fig. 8 that the temperatures in the NSC column are generally lower than the corresponding temperatures in the HSC column throughout the fire exposure. This variation can be attributed partly to the variation in thermal properties of the two concrete and to the higher compactness (lower porosity) of HSC. The lower porosity (lower-water content) of HSC affects the rate of increase of temperature until the cracks widen and spalling occurs.

Variation of Deformations

The variation in axial deformation with time for the five columns is shown in Fig. 9. Both the NSC and HSC columns expanded

Thermocouples on Steel Rebar

Thermocouples in Concrete

Fig. 2. Location and numbering of thermocouples in Column

Fig. 3. Temperature distribution at various depths in Column TNC1

Fig. 4. Temperature distribution at various depths in Column THC4
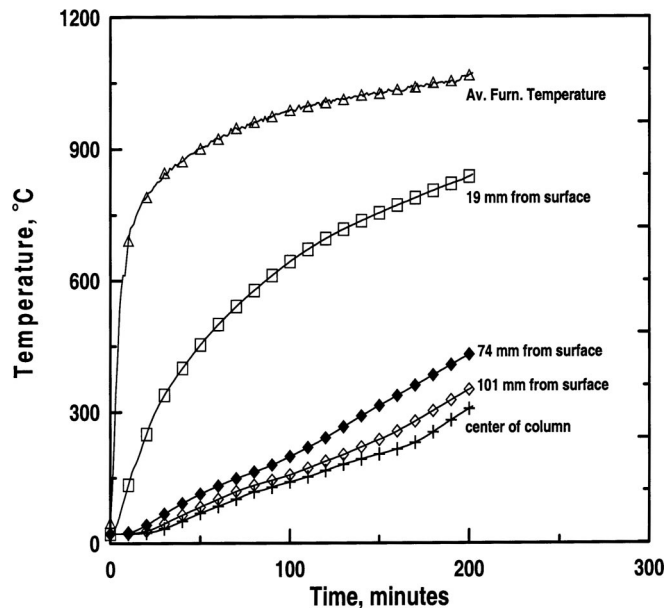

Fig. 6. Temperature distribution at various depths in Column THS10

until the reinforcement yields and then contracted leading to failure. It can be seen from the figure that the overall deformation behavior of HSC columns was similar to that of the NSC column. However, in the expansion zone, the deformation in HSC columns was 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 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.

The axial deformation in the expansion stage is very much similar for all HSC columns. THC4 failed at 8 mm axial defor-
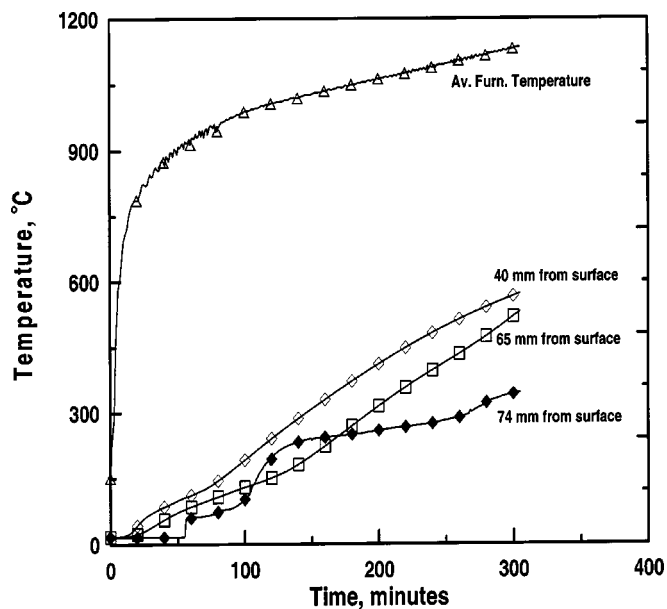
mation and all other HSC columns failed around 24 mm axial deformation. The axial deformation curves for THC4 and THS10 are similar except THS10 has larger axial deformation due to the presence of steel fiber that increased the ductility of the HSC column. The presence of polypropylene fibers in HSC Column THP13 increased the ductility of the column, as can be seen from the higher deformations attained before failure.

The deformation-time response of the HSC columns is different from that of the NSC column. In the case of HSC columns, the deformation is significantly lower than that of the NSC column in the expansion zone. This can be attributed partly to the lower-thermal expansion and higher-elastic modulus of HSC. When the steel reinforcement in the column gradually yields, because of increasing temperatures, the column contracts. At this stage, the column behavior is dependent on the strength of the concrete. Another significant difference that can be observed from Fig. 9 is that the NSC column maintained the expansion plateau
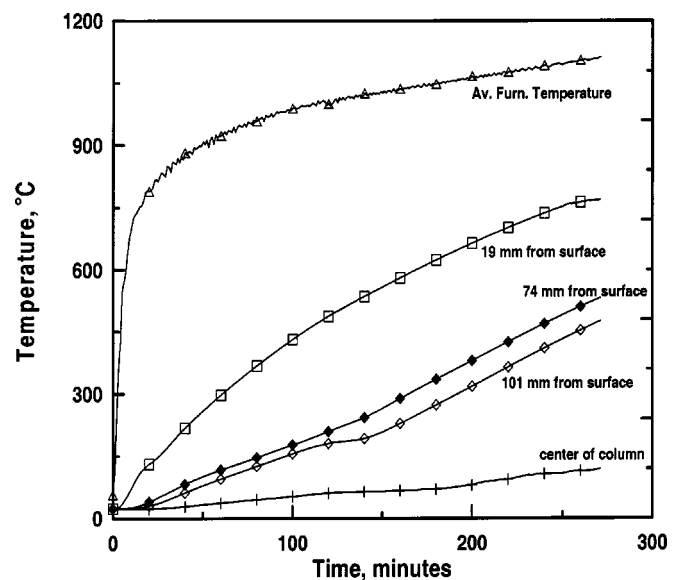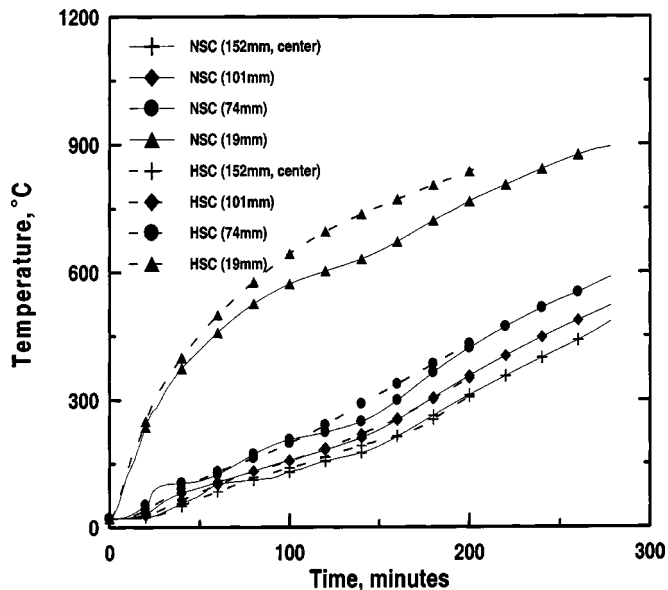

Fig. 5. Temperature distribution at various depths in Column THC8

Fig. 7. Temperature distribution at various depths in Column THP13

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Fig. 8. Temperature distribution at various depths in Columns TNC1 and THC4

for a considerable duration before contracting, while the contraction starts much earlier in the HSC columns. There is significant contraction in the NSC column leading to gradual ductile failure.

Spalling Pattern and Failure Mode

All five columns failed in compression mode with slight bending. While there was no spalling in the NSC column, significant spalling at the corners was observed before failure occurred in the HSC columns. There was slight random local spalling in HSC columns in the first 30 min after fire exposure. In HSC Column
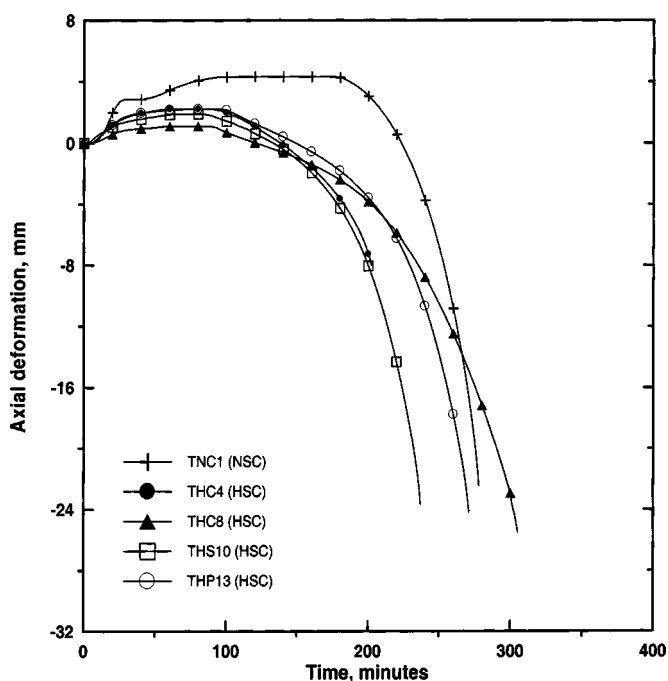
Fig. 9. Axial deformation for Columns TNC1, THC4, THC8, THS10, and THP13

THP13, the presence of polypropylene fibers reduced spalling, both at early stages and just prior to failure.

The spalling resulted from the crack propagation in the columns. The cracks in the HSC columns progressed at the corners of the cross section, with time, and led to spalling of chunks of concrete just before failure of the columns. This spalling was significant at about midheight. While minute (hair line) cracks could be noticed in about 20 to 30 min the widening of these cracks occurred after about 60 min or so. Large cracks occurred in HSC columns after about 3 h of fire exposure. In the case of Column THC8 (carbonate aggregate concrete), the spalling, as noticed at the end of the test, was less compared to Column THC4 (siliceous aggregate). This could be attributed to the effect of aggregate 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. This increase in specific heat is caused by the dissociation of the dolomite in the carbonate concrete and is beneficial in preventing spalling of the concrete (Kodur 2001).

When steel reinforcement 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 behavior 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 (Phan 1996). This is especially applicable to the descending portion of the stress-strain curve of HSC at elevated temperatures (Phan 1996).

Fire Resistance

A comparison of fire resistance of the five columns is given in Table 1. The time to reach failure is defined as the fire resistance for the column. For the NSC Column TNC1, the fire resistance was approximately 278 min while, for plain HSC Columns THC4 and THC8, it was 202 and 305 min, respectively. For the HSC column with steel fibers, THS10, the fire resistance was approximately 239 min and for the HSC column with polypropylene fibers, THP13, the fire resistance was 271 min. 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, spalling which results in the decrease in the cross section at later stages of fire exposure, also contributed to lowering the fire resistance in the HSC columns. The higher-fire resistance of Column THC8, despite higher-load intensity, as compared to that of Column THC4, can be attributed to the type of aggregate used in the concrete mix. Columns made of siliceous aggregate concrete typically have lower-fire resistance compared to those made with carbonate aggregate concrete (Kodur and Sultan 1998; Kodur 2001).

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 behavior 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 (Kodur and Lie 1996; Sullivan et al. 1997). 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 and NCTU to develop the mechanical properties of HSC at elevated temperatures. The data on thermal and mechanical properties is being used to develop thermal and mechanical relationships, as a function of temperature (Kodur and Sultan 1998). These relationships can be used as input to numerical models which in turn can be used to determine the behavior 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 (Kodur and Lie 1996).

Concluding Remarks

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

1. The behavior 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 has a visible influence on the performance of HSC columns at elevated temperatures. The presence of carbonate aggregate in HSC increases fire resistance;
3. The addition of steel and polypropylene fibers in HSC can improve the ductility of HSC columns and increase their fire endurance;
4. The presence of polypropylene fibers in HSC columns can reduce spalling and enhance their fire resistance; and
5. The studies, currently in progress, will generate data on the fire resistance of HSC columns and will identify the conditions under which these columns can be safely used.

Acknowledgments

References

American Society for Testing and Materials (ASTM). (1990). "Standard methods of fire endurance tests of building construction and materials." *ASTM E119-88*, Philadelphia.

Canadian Standards Association (CSA). (1994). "Code for the design of concrete structures for buildings." *CAN3-A23.3-M94*, Rexdale, Ont., Canada.

Diederichs, U., Jumppanen, U. M., and Schneider, U. (1995). "High temperature properties and spalling behaviour of high strength concrete." *Proc., 4th Weimar Workshop on High Performance Concrete*, HAB Weimar, Germany, 219–235.

Kodur, V. K. R. (2001). "Fibre-reinforced concrete for enhancing the structural fire resistance of columns." *ACI Special Publ.,* in press.

Kodur, V. K. R., and Lie, T. T. (1996). "A computer program to calculate the fire resistance of rectangular reinforced concrete columns." *Proc., 3rd Canadian Conf. on Computing in Civil and Building Engineering*, Canadian Society of Civil Engineering, Montreal, 11–20.

Kodur, V. K. R., and Lie, T. T. (1998). "Fire resistance of fibre-reinforced concrete." *Fibre reinforced concrete: present and the future*, Canadian Society of Civil Engineering, Canada, 189–213.

Kodur, V. K. R., and McGrath, R. (2001). "Performance of high strength concrete columns under severe fire conditions," *Proc., 3rd Int. Conf. on Concrete Under Severe Conditions*, Vancouver B.C., Canada, 254–268.

Kodur, V. K. R., and Sultan, M. (1998). "Thermal properties of high strength concrete at elevated temperatures." *CANMET-ACI-JCI Int. Conf. on Recent Advances in Concrete Technology*, American Concrete Institute, SP179, Tokushima, Japan, 467–480.

Lie, T. T. (1980). "New facility to determine fire resistance of columns." *Can. J. Civ. Eng.,* 7(3), 551–558.

Lie, T. T., and Celikkol, B. (1991). "Method to calculate the fire resistance of circular reinforced concrete columns." *ACI Mater. J.,* 88(1), 84–91.

Lie, T. T., and Woollerton, J. L. (1988). "Fire resistance of reinforced concrete columns: Test results." *Institute for Research in Construction Internal Rep. No. 569*, National Research Council of Canada, Ottawa, 302.

Matsumoto, Y., et al., (1944). "A study on spalling resistance of LMF fibre-reinforced high strength concrete." Institute of Technology, Tokyo.

National Research Council of Canada (1995). *National building code (NBC) of Canada*, Ottawa.

Phan, L. T. (1996). "Fire performance of high-strength concrete: A report of the state-of-the-art." *National Institute of Standards and Technology*, Gaithersburg, Md.

Sullivan, P. J. E., Terro, M. J., and Morris, W. A. (1997). "Critical review of fire-dedicated thermal and structural computer programs." *J. Appl. Fire Sci.,* 3(2), 113–135.

Underwriters' Laboratories of Canada (ULC). "Standard methods of fire endurance tests of building construction and materials." *CAN/ULC-S101-M89*, Scarborough, Ont. Canada, 49.