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Evaluation of fire resistance of rectangular steel columns filled with fibre-reinforced concrete

V.K.R. Kodur and T.T. Lie

Abstract: Experimental and theoretical studies that have been carried out to predict the fire resistance of rectangular hollow steel columns filled with fibre-reinforced concrete are presented in this paper. A numerical model to calculate the temperatures, deformations, and the fire resistance of loaded, rectangular, hollow steel columns filled with steel-fibre-reinforced concrete is described. Predictions from the numerical model are compared with experimental data. The results indicate that the model is capable of predicting the fire resistance of square hollow steel columns filled with fibre-reinforced concrete with an accuracy that is adequate for practical purposes. The model can be used to evaluate the fire resistance of rectangular steel columns filled with steel-fibre-reinforced concrete for any value of the significant parameters, such as load, column section dimensions, column length, concrete strength, and aggregate type in concrete, without the necessity of testing.

Key words: fire resistance, numerical model, calculation, square steel columns, hollow structural section, concrete-filled columns, fibre-reinforced concrete.

Résumé : Cet article présente des études théoriques et expérimentales effectuées afin de prédire la résistance au feu de colonnes rectangulaires creuses en acier remplies de béton armé de fibres. Un modèle numérique calculant les températures, les déformations et la résistance au feu de colonnes creuses en acier remplies de béton armé de fibres et soumises à un chargement, est décrit. Les prédictions du modèle numérique sont comparées aux données expérimentales. Les résultats indiquent que le modèle est capable de prédire la résistance au feu des colonnes carrées creuses en acier remplies avec du béton armé de fibres avec une précision convenable à des fins pratiques. Le modèle peut évaluer la résistance au feu de colonnes rectangulaires en acier remplies de béton armé de fibres utilisant n'importe quelle valeur pour les paramètres significatifs, tels que la charge, les dimensions de la section de la colonne, la longueur de colonne, la résistance du béton et le type de granulats du béton, sans nécessiter des essais expérimentaux.

Mots clés : résistance au feu, modèle numérique, calculs, colonnes carrées en acier, section structurale creuse, colonnes remplies de béton, béton armé de fibres.

[Traduit par la Rédaction]

Introduction

Hollow structural steel columns are very efficient structurally in resisting compression loads and are often used in the construction of framed structures in industrial buildings. Concrete filling of these columns not only enhances their load bearing capacity, but also increases fire resistance. The use of concrete-filled hollow steel columns can lead, in an economic way, to the realization of architectural and structural design with visible steel without any restrictions on fire safety.

The National Research Council of Canada, with the support of the Canadian Steel Construction Council and the American Iron and Steel Institute, has undertaken studies to develop in-

formation on the fire resistance performance of concrete-filled hollow structural section (HSS) columns. Among the various parameters investigated were three types of concrete filling, namely, plain concrete, bar-reinforced concrete, and steel-fibre-reinforced concrete.

Research on the fire resistance of HSS columns filled with plain concrete (Lie and Chabot 1990) and with bar-reinforced concrete (Lie and Irwin 1995) was previously completed. The results showed that reductions in the loads on the columns were necessary to obtain reproducible and predictable fire resistances if the columns were filled with plain concrete. If the columns were filled with bar-reinforced concrete, the fire resistance remained predictable, even under very high loads. The reinforcement limits the minimum size of the column, however, and there is additional cost for the installation of the rebars in the column.

Fibre-reinforced concrete-filled columns, which have load limitations and costs that lie between those of plain and bar-reinforced concrete-filled columns, in many cases provide the most cost-effective fire protection (Lie and Kodur 1995c). In the present paper, experimental and numerical studies carried out to evaluate the fire resistance of rectangular HSS columns filled with steel-fibre-reinforced concrete are presented. The

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Fig. 1. Elevation and cross section of test columns.

research on HSS columns filled with fibre-reinforced concrete, presented in this paper, will enable the calculation of the fire resistance of these columns as a function of the important parameters that determine it.

Experimental studies

HSS columns

Three square HSS columns filled with steel-fibre-reinforced concrete, SQ1, SQ2, and SQ3, were tested to failure by exposing the columns to fire. Figure 1 shows elevation and cross-sectional details for the columns. The columns were 3810 mm long from end plate to end plate. The widths of columns SQ1, SQ2, and SQ3 were 152.4, 177.8, and 203.2 mm, respectively, while the corresponding shell thicknesses were 6.35, 6.35, and

12.7 mm, respectively. No external fire protection was provided for the steel.

Steel hollow structural sections that meet the requirements of CSA Standard CAN3-G40.20-M81, Class H (hot formed, cold formed, stress relieved), were used (CSA 1981). The sections were made of Grade 350W steel which had a minimum yield strength of 350 MPa. The end plates were constructed of mild steel.

The HSS columns were fabricated by cutting the steel sections to appropriate lengths. The steel end plates were then welded to the column extremities, with special attention being given to the centering and perpendicular setting of the end plates. Before welding the end plates, a hole with a diameter 26 mm smaller than the inside diameter of the hollow steel section was cut in each plate. This construction provided a

Table 1. Summary of test parameters and results for rectangular HSS columns filled with steel-fibre-reinforced concrete.

Column	Dimensions (mm)	Factored resistance (kN)		Test load, C (kN)	Load intensity		Fire resistance (min)	
		C'_r	C_{rc}		C/C'_r	C/C_{rc}	Measured	Calculated
SQ1	$152.4 \times 152.4 \times 6.35$	344	1449	350	1.02	0.24	73	47
SQ2	$177.8 \times 177.8 \times 6.35$	520	1845	800	1.54	0.43	60	42
SQ3	$203.2 \times 203.2 \times 12.7$	626	3601	700	1.12	0.19	110	74

Notes: C'_r , factored compressive resistance of concrete core of the column; C_{rc} , factored compressive resistance of concrete-filled HSS column; C , test load.

Fig. 2. End plate connection details for columns.

13 mm lip to transfer the load from the steel plate to the concrete filling, as shown in Fig. 2.

Four holes, with a diameter of 13 mm, were drilled in the steel wall to provide vent holes for water vapour produced during the test. Two pairs of holes, one pair each at the bottom and top of the column, were located with one hole opposite the other at 1448 mm from the mid height of the column (Fig. 1).

Type K chromel-alumel thermocouples, with a thickness of 0.91 mm, were installed at the mid height of the column to measure the temperature at different locations in the cross section. The locations of the thermocouples and other instrumentation details are described in previous papers (Kodur and Lie 1996).

Concrete filling

Two batches of concrete mix were made. Columns SQ1 and SQ2 were cast from the first batch which had siliceous aggregate, while column SQ3 was cast from the second batch of concrete which had carbonate aggregate. The siliceous aggregate consisted predominantly of quartz, while the carbonate aggregate was made from limestone. The mixes consisted of

general purpose type 10 Portland cement, carbonate or siliceous stone, and silica-based sand. The mix proportions, per cubic metre of concrete, in both batches were as follows:

- cement, 439 kg;
- water, 161 kg;
- fine aggregate, 621 kg; and
- coarse aggregate, 1128 kg.

Steel fibres, 1.77% by mass, were added to the fresh concrete and mixed for approximately 2 min to provide uniform dispersion of the steel fibres. The 50 mm long fibres had an equivalent diameter of 0.9 mm and an aspect ratio of 57.

The concrete was poured into the column through the top opening and vibrators were used to consolidate the concrete. The 28-d average cylinder strengths of the two batches of concrete were 41.6 and 42.2 MPa, respectively. The compressive strengths of the concrete on the test day, which was 6 months or more after the pouring, were 49.5, 49.8, and 39.6 MPa for columns SQ1, SQ2, and SQ3, respectively.

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 the conditions, such as temperature, structural loads, and heat transfer, to which a member might be exposed during a fire. The hydraulic jack in the test furnace has a loading capacity of 9778 kN. Full details on the characteristics and instrumentation of the column furnace are provided by Lie (1980).

Test conditions and procedures

The tests were carried out with both ends of the columns fixed, i.e., restrained against rotation and horizontal translation. The columns were subjected to an axial load. The applied load (C) and the factored compressive resistances of the column (C_{rc}) and the concrete core (C'_r) are given in Table 1. The factored compressive resistance was computed using an effective length factor, K , of 0.65, for the fixed end condition, as recommended in CSA/CAN-S16.1-M89 (CSA 1989). The load intensity, defined as the ratio of the applied load to the factored column resistance, was 1.02 for column SQ1, 1.54 for column SQ2, and 1.12 for column SQ3.

The load was applied for approximately 45 min before the start of the fire test and was maintained until a condition was reached at which no further increase in 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 standard temperature-time curve specified in CAN/ULC-S101-M89 (ULC 1989) or ASTM E119-88 (ASTM 1988). The load was maintained constant throughout the test. The ends of the columns and the loading system were protected against heating by a well-insulated floor and roof. 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.

Numerical studies

Determining fire resistance by testing full-size columns is costly and time consuming. An alternative solution is to use numerical models, in lieu of laboratory testing, to predict the fire resistance. Hence, the main objective of the experimental studies was to generate fire resistance data for the development of general methods for calculating the fire resistance of fibre-reinforced concrete-filled steel columns.

Analysis procedure

A numerical model for predicting the behaviour of fibre-reinforced concrete-filled steel columns exposed to fire was developed. The numerical procedure used in the model is similar to the one that was previously applied to the fire resistance calculations for plain concrete-filled columns (Lie and Irwin 1995). Detailed equations for the calculation of the column temperatures and strength are given by Lie and Irwin (1995).

The fire resistance calculation is performed in three steps, namely, the calculation of the temperatures of the fire to which the column is exposed, the temperatures in the column, and its deformations and strength, including an analysis of the stress and strain distribution, during the exposure to fire.

Fire temperature

In the numerical model, it is assumed that the entire surface area of the column is exposed to the heat of a fire, whose temperature follows that of the standard fire described in ASTM E119-88 (ASTM 1988) or CAN/ULC-S101-M89 (ULC 1989). This temperature course can be approximated by the following expression:

$$[1] \quad T_f = 20 + 750[1 - \exp(-3.79553\sqrt{\tau})] + 170.41\sqrt{\tau}$$

where τ is the time in hours and T_f is the fire temperature in °C at time τ .

Temperatures in column

The column temperatures are calculated by a finite difference method (Dusinbere 1961). The cross-sectional area of the column is subdivided into a number of elements, arranged in a triangular network (Fig. 3a). This network has been selected because it is more suitable than the square network for the calculation of the surface temperatures of the column, which are important and usually higher than those obtained by using the square network. The elements are square inside the steel and the concrete, and triangular at the column surface and along the boundary between the concrete and the steel. The steel elements are one half the width of the concrete elements. For the square elements, the temperature at the centre is taken as representative of the entire element. For the triangular ele-

ments, the representative points are located at the centre of each hypotenuse.

By symmetry, only one-quarter of the section needs to be considered when calculating the temperature distribution in columns with a square or rectangular cross section.

The temperature rise in a layer can be derived by creating a heat balance for each layer. By solving the heat balance equation for each layer, the temperature history of the column can be calculated, using the temperature-dependent thermal properties of the concrete and steel, of which the column is composed. In the study it is assumed that conduction is the main heat transfer mechanism in the steel and concrete, and radiation is the heat transfer from fire to the steel. Convection is assumed to be small and negligible in comparison with the radiative heat transfer. The moisture content in the concrete is assumed to be 10% by volume, and uniformly distributed in the concrete. There is no moisture migration.

The effect of moisture is taken into account by assuming that, in each element, the moisture starts to evaporate when the temperature of the element reaches 100°C. During the period of evaporation, all the heat supplied to an element is used for the evaporation of the moisture until the element is dry.

Strength of column

In order to calculate the strain and stress in the column and its strength, during fire exposure, the triangular network is transformed into a square network, which is less complicated to process than the triangular network. In Fig. 3b, a quarter section of this network, consisting of square elements, arranged parallel to the x - and y -axis of the section, is shown. The arrangement of the elements in the three other quarter sections is identical to this. The width of each steel element of this network is one half the width of a concrete element. The temperature, deformation, and stress of each element are represented by those at the centre of the element. The temperature at the centre of each element is obtained by averaging the temperatures of the elements in the triangular network.

The strain in an element of steel is given by the sum of the thermal expansion of the steel, the axial strain due to compression, and the strain due to bending of the column. A similar calculation is performed for the concrete elements.

The stresses at mid-section in the concrete elements can be calculated for any value of the axial strain, ϵ , and curvature, $1/\rho$. From these stresses, the load that each element carries and its contribution to the internal moment at mid-section can be determined. By adding the loads and moments, the load that the column carries and the total internal moment at mid-section can be calculated.

The fire resistance of the column is derived by calculating the strength, i.e., the maximum load that the column can carry at several consecutive times during the exposure to fire. To simplify the strength calculations, the following assumptions are made:

- The curvature of the column varies from pin ends to mid-height according to a straight line relation.
- Plane sections remain plane.
- Concrete has no tensile strength.
- There is no slip between the steel and the concrete at room temperature.
- There is no composite action between the steel and the concrete at elevated temperatures.

Fig. 3. Idealization of HSS column filled with steel-fibre-reinforced concrete for analysis: (a) thermal network in quarter section; (b) stress-strain network in quarter section.

(f) The reduction in column length before exposure to fire, consisting of free shrinkage of the concrete, creep, and shortening of the column due to load, is negligible. This reduction can be eliminated by selecting the length of the shortened column as the initial length from which the changes during exposure to fire are determined.

Based on these assumptions, the column strength during exposure to fire was calculated. In the calculations, the network of elements shown in Fig. 3b was used. Because the strains and the stresses of the elements are not symmetrical with respect to the y -axis, the calculations were performed for both the network shown and an identical network at the left of the y -axis. The load carried by the column and the moments in the section were obtained by adding the loads carried by each

element and the moments contributed by them. The model is capable to calculate the loads on the steel and the concrete separately. Of main interest, however, is the total load that the column can carry.

Material properties

Steel

In the calculations, the thermal and mechanical properties of steel, given in the ASCE manual (Lie 1992), were used.

Steel-fibre-reinforced concrete

As part of this study, a detailed study was undertaken to establish the thermal and mechanical properties of steel-fibre-reinforced

concrete at elevated temperatures (Lie and Kodur 1995a, 1995b) for use in numerical models to predict the fire resistance of concrete-filled HSS columns. To facilitate the use of the thermal and mechanical properties as input data to the calculation of the temperatures and strengths of steel-fibre-reinforced concrete constructions, formulas that give these properties as a function of temperature in the temperature range of 0–1000°C have been derived (Lie and Kodur 1996b). These formulas were used in the computer programs. The deformations due to lateral thermal expansion, as well as the Poisson's deformations, which are small in comparison with the thermal deformations, have not been taken into account in this study. Creep has been taken into account implicitly in the stress-strain curves used. For columns SQ1 and SQ2, thermal properties corresponding to siliceous aggregate concrete mix were used; and column SQ3 was analysed with the thermal properties of the carbonate aggregate concrete mix.

All material properties used in the calculations are described in the above-mentioned references (Lie 1992; Lie and Kodur 1995a, 1995b, 1996b).

Water

The values for the thermal capacity of water and heat of vaporization, which were used in calculating the moisture vaporization when the temperature reaches 100°C, are 4.2×10^6 J/(m³·°C) and 2.3×10^6 J/kg respectively.

Specifics of columns and furnace

The combined emissivity of column furnace fire and steel is taken as 0.6. This was obtained as the product of the column furnace fire (0.75) and the emissivity of steel (0.8). The effective length of columns used in the fire resistance calculations was 2.0 m. The length of column that contributes to axial deformation was 3.5 m. The moisture concentration of concrete was 0.10 by volume.

Computer program

The numerical procedure described above was incorporated into a computer program. Figure 4 shows a flow chart of the calculation procedure associated with the computer program. For any time step, the analysis starts with the calculation of temperatures due to fire. The next stage is to determine the cross-sectional temperatures by making use of thermal properties. In the third stage, the strength of the column, during exposure to fire, is determined by successive iterations of the axial strain and curvature until the internal moment at mid height is in equilibrium with the applied moment.

For any given curvature, and thus for any given deflection at mid height (Fig. 5), the axial strain is varied until the internal moment at the mid-section is in equilibrium with the applied moment given by the product:

$$\text{load} \times (\text{deflection} + \text{eccentricity})$$

In this way, a load-deflection curve can be calculated for specific times during the exposure to fire. From these curves, the strength of the column, i.e., the maximum load that the column can carry, can be determined for each time. When the equilibrium condition is satisfied, the iteration for curvature is continued in order to make sure that the point at which the equilibrium is achieved corresponds to the maximum load condition.

The fire resistance of the column is derived by calculating the strength of the column as a function of the time of exposure to fire. Because the compressive strength of steel and that of concrete reduce with increasing temperature and thus with time, the strength of the column reduces gradually with time. The time increments continue until a certain point, at which the strength becomes so low that it is no longer sufficient to support the load. At this point, the column becomes unstable and is assumed to have failed. The time to reach this failure point is the fire resistance of the column.

Results and discussion

Data obtained from the fire tests on three rectangular fibre-reinforced concrete-filled steel columns were used to validate the numerical model described in this paper. The temperatures, axial deformations, strengths, and fire resistances of the columns measured in the tests were compared with those predicted by the model. The specifics of the columns and the furnace used in the analysis are given in Appendix 2.

The calculated temperatures were compared with the temperatures measured at the external surface of the steel section and at various depths in the concrete, in Figs. 6, 7, and 8 for columns SQ1, SQ2, and SQ3, respectively. It can be seen that, with the exception of the temperatures measured at an early stage deeper in the specimen, there is a good agreement between the calculated and measured column temperatures. In general, there is a reasonably good agreement between the calculated and measured steel temperatures. The temperatures measured in the concrete are, on the average, somewhat lower than the calculated temperatures. The differences reduce, especially for columns SQ1 and SQ3, with increasing temperatures and duration of fire exposure.

For all three columns, the temperatures, measured deeper inside the column, rise rapidly to about 100°C and then the rate of increase of temperature decreases in the early stages of fire exposure. This temperature behaviour is due to the thermally induced migration of moisture toward the centre of the column (Chabot and Lie 1992). The influence of moisture migration is highest at the centre of the column.

Although the model takes into account evaporation of moisture, it does not take into account its migration towards the centre. That migration appears to account for the deviation between the calculated and measured temperatures at the earlier stages of fire exposure. At later stages, however, which are more important from the point of view of predicting the fire resistance of the columns, there is reasonable agreement between the calculated and measured temperatures.

The calculated and measured axial deformations of columns SQ1, SQ2, and SQ3, during exposure to fire, are compared in Figs. 9–11. As expected, the columns expand in the initial stage of the test, due to rapid temperature rise and expansion of the steel. At this stage most of the load is carried by the steel. Afterwards, when the steel gradually loses strength, the concrete takes over in carrying the load. The columns then contract until failure. There is reasonably good agreement between the calculated and measured results; however, there are some differences. These differences can be attributed to factors such as load, thermal expansion, bending, and creep, which have not been completely accounted for in the numerical calculations.

Fig. 4. Flowchart showing the numerical procedure for fire resistance calculations. **Fig. 5.** Load–deflection analysis for a column.

While the effects of load and thermal expansion are significant in early stages, the effect of creep becomes pronounced in the later stages. Since the axial deformations are of the order of 30 mm for columns with a length of about 3800 mm, small inaccuracies in these factors may cause noticeable differences between the calculated and measured axial deformations. A difference of 10% between the theoretical and actual coefficients of thermal expansion of steel, for example, will cause a difference of approximately 5 mm in the axial deformations.

The model defines the failure point as the point at which the column can no longer support the applied load and assumes that failure at this point is instantaneous. During the tests, failure was not instantaneous but the columns contracted considerably, apparently as a result of continued loss of strength and creep, before they were crushed. Column SQ3 failed by compression and columns SQ1 and SQ2 failed by buckling.

For all three columns, the calculated deformations near the point of failure were smaller than the measured deformations. The main reason for this difference is that the creep of the steel

Fig. 6. Calculated and measured temperatures of column SQ1 as a function of exposure time.

Fig. 7. Calculated and measured temperatures of column SQ2 as a function of exposure time.

Fig. 8. Calculated and measured temperatures of column SQ3 as a function of exposure time.

Fig. 9. Calculated and measured axial deformations of column SQ1 as a function of exposure time.

Fig. 10. Calculated and measured axial deformations of column SQ2 as a function of exposure time.

Fig. 11. Calculated and measured axial deformations of column SQ3 as a function of exposure time.

and concrete becomes more pronounced at higher temperatures. However, a part of the creep is implicitly taken into account in describing the stress-strain relations of the concrete and steel. The effect of creep, which is more pronounced at a later stage of the fire exposure, may be even greater. This was particularly the case with column SQ3. Further, there is uncertainty in load sharing between the steel and concrete. This is most likely the main reason for the difference between the calculated and measured axial deformations for the columns in the 15–40 min time interval.

The calculated and measured fire resistances, corresponding to the test loads, are given in Table 1. The time to reach a point at which the column can no longer support the applied load is the fire resistance of the column. It can be seen that the calculated fire resistances are about 30% lower than those measured. The differences are due to considerable contraction of the columns, which the model can only partly take into account. However, in the area of fire resistance prediction, which is influenced by a large number of factors, a difference of 30%, which lies on the safe side, is usually regarded as acceptable for practical applications.

Design implications

Generally, filling the HSS column with plain concrete without any steel reinforcement offers the most economical arrangement from the point of view of fire resistance. However, experimental and numerical studies have indicated that the fire resistance of plain concrete-filled columns is limited to 2 h (Lie and Stringer 1994).

The above problem can be overcome by using bar-reinforced filling in HSS columns (Lie and Kodur 1996a). The fire resistance of such columns improved significantly and the load bearing capacity of the column increased. However, there is the additional cost of steel and of installation of the rebars in the column. Furthermore, bar-reinforced concrete filling cannot be used for columns less than 219 mm in diameter because of the practical problems associated with placement of reinforcement (Lie and Kodur 1995c).

The fire resistance of fibre-reinforced concrete-filled columns lies in between those of plain concrete-filled and bar-reinforced concrete-filled steel columns (Kodur and Lie 1995a, 1995b). The results from fire tests and numerical studies have shown that filling HSS columns with steel-fibre-reinforced concrete filling offers many advantages over plain and bar-reinforced concrete filling. Test results (Kodur and Lie 1996) indicate that the use of fibre-reinforced concrete filling in HSS columns prevents early cracking and indicates fire resistance values that are comparable to those of bar-reinforced concrete-filled HSS columns. The additional cost in the case of fibre-reinforced concrete-filled columns, over the cost of concrete, is the cost of the steel fibres. Since there is no installation cost, as with the bar-reinforced concrete, this can offer an economical solution for obtaining a required fire resistance rating where fire resistance governs the design.

At present, studies are in progress to develop simple design equations, suitable for incorporation in design standards and the National Building Code of Canada, for calculating the fire resistance of steel-fibre-reinforced concrete HSS columns. In the meantime, computer programs, such as the ones presented

above, can be used for predicting the fire resistance of rectangular HSS columns filled with steel-fibre-reinforced concrete.

Conclusions

Based on the results of this study, the following conclusions can be drawn:

1. The computer programs presented in this study are capable of predicting the fire resistances of rectangular columns, made of hollow structural steel filled with steel-fibre-reinforced concrete, with an accuracy that is adequate for practical purposes.
2. The computer programs can be used to further expand data on the fire resistances of rectangular concrete-filled steel columns. At present, these data consist predominantly of information on columns filled with plain concrete and bar-reinforced concrete. Expansion of the data for columns filled with steel-fibre-reinforced concrete is under way.
3. Using the model, the fire resistances of rectangular concrete-filled steel columns can be evaluated for any value of the significant parameters, such as load, column section dimensions, column length, and concrete strength, without the necessity of testing.
4. The use of steel-fibre-reinforced concrete filling in hollow steel section columns can, in many cases, provide adequate fire resistance and, in design situations where the load-carrying capacity of a column is not as critical a factor, it can provide an economical alternative to bar-reinforced concrete filling.

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Appendix 1. List of symbols

c	specific heat, J/(kg·°C)
C	load intensity
C'_r	factored compressive resistance of concrete core
C_{rc}	factored compressive resistance of concrete-filled steel columns
D	outside dimension of the HSS section, mm
f	stress, MPa
f'_c	cylinder strength of concrete at temperature T , MPa
f'_{co}	cylinder strength of concrete at room temperature, MPa

f_y	strength of steel at temperature T , MPa
k	thermal conductivity, W/(m·°C)
K	effective length factor
l	length of column that contributes to axial deformation, m
L	unsupported length of the column, m
t	wall thickness of HSS section, mm
T	temperature, °C
α	coefficient of thermal expansion, 1/°C
ϵ	emissivity, strain, m/m
λ	heat of vaporization, J/kg
ρ	density, kg/m ³
τ	time, h
ϕ	concentration of moisture

Subscripts

c	concrete
f	fire
max	maximum
o	at room temperature
p	pertaining to proportional stress strain relation
s	steel
T	pertaining to temperature
w	water

Appendix 2

The properties of water and the specifics of columns and the furnace used in the analysis are given below:

Water properties

Thermal capacity:

$$\rho_w c_w = 4.2 \times 10^6 \text{ J/(m}^3 \cdot \text{°C)}$$

Heat of vaporization:

$$\lambda_w = 2.3 \times 10^6 \text{ J/kg}$$

Specifics of columns and furnace

$\epsilon_f = 0.75$	(emissivity of column furnace fire)
$\epsilon_s = 0.8$	(emissivity of steel)
$KL = 2.0 \text{ m}$	(effective length of columns for fire resistance calculations)
$l = 3.5 \text{ m}$	(length of column that contributes to axial deformation)
$\phi = 0.10$	(concentration of moisture in concrete by volume)