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A METHOD TO PREDICT THE FIRE RESISTANCE OF CIRCULAR CONCRETE FILLED HOLLOW STEEL COLUMNS

by

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SUMMARY

Experimental and theoretical studies have been carried out at the National Research Council of Canada to develop methods to predict the fire resistance of concrete filled hollow steel columns. A mathematical model that calculates temperatures, deformations and strength of columns in fire, and their fire resistance, is described in this paper. A comparison of results calculated using this model with the results of tests conducted on circular steel columns filled with non-reinforced concrete is discussed.

INTRODUCTION

Hollow structural steel sections are the most efficient of all structural sections in resisting compression loads. By filling these sections with concrete, the load-carrying capacity of such columns can be increased substantially. In addition, a high fire resistance can be obtained without the necessity of additional surface fire protection for the steel. The elimination of such surface protection in turn increases usable space in the building. Furthermore, the steel sections dispense with the need for formwork.

Research to determine the fire resistance of concrete filled hollow steel columns has been carried out in several laboratories in the world[1-15] For a number of years, the National Research Council of Canada (NRCC) has also been engaged in research to calculate the fire resistance of hollow steel columns with concrete filling. Both theoretical and experimental studies were carried out[16-18]. With the support of the North American steel industry, these studies have been expanded substantially.

Most of the studies at the NRCC have been carried out on columns with circular cross-section. These studies complement the studies carried out by other laboratories, which were dominantly on columns with square cross-section. In this paper, the fire resistance studies on circular hollow steel columns filled with siliceous aggregate concrete will be discussed.

CALCULATION PROCEDURE

The calculation of the fire resistance of the column is carried out in various steps. It involves the calculation of the temperatures of the fire to which the column is exposed, the temperatures in the column and its deformations and strength during the exposure to fire.

A procedure to calculate the fire resistance of circular hollow steel columns with concrete filling has been described in Reference 16. In the present paper, this procedure has been refined to enable faster and more precise execution of the calculations. A flow chart of the calculation procedure is shown in Figure 1.

TEMPERATURES OF COLUMNS DURING FIRE EXPOSURE

The column temperatures are calculated by a finite difference method[19]. This method has been previously applied to the calculation of temperatures of various building components exposed to fire[20,21]. Because the method of deriving the heat transfer equations and of calculating the temperatures is described in detail in those studies, it will not be discussed here; only the equations for the calculation of the column temperatures will be given.

Division of Cross-section into Layers

The cross-sectional area of the column is subdi-

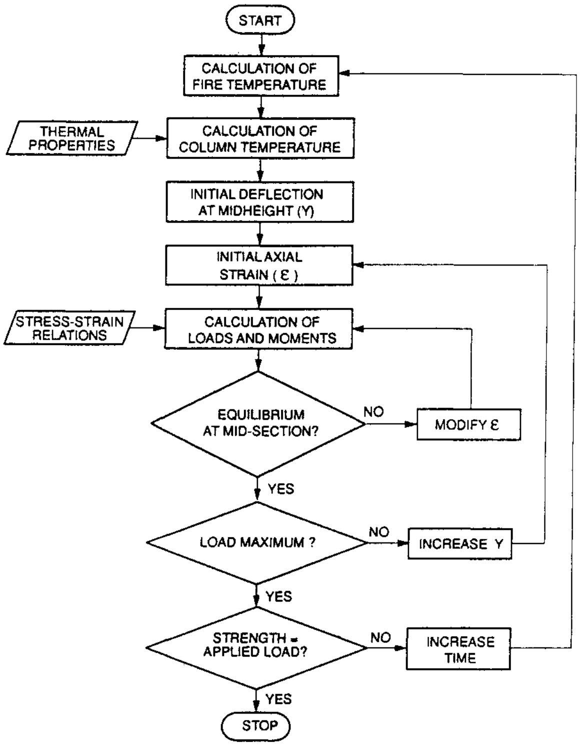

Table 1. Flow chart of calculation procedure.

vided into a number of concentric layers. There are M_1 layers in the steel and $(M_2 - M_1 + 1)$ layers in the concrete. As illustrated in Figure 2, along any radius, a point P_m representing the temperature of a layer (m), is located at a distance of $(m - 1)\Delta\xi_s$ from the fire-steel boundary when the point is in the steel, and at a distance of $(m - M_1)\Delta\xi_c$ from the concrete-steel boundary, when the point is in the concrete. The outer layer of the steel, which is exposed to fire, has a thickness of $1/2\Delta\xi_s$. The layer of steel at the boundary between steel and concrete is also $1/2\Delta\xi_s$ thick. The thickness of all other layers in the steel is $\Delta\xi_s$. The thickness of the layer of concrete at the boundary between steel and concrete, and that at the centre of the column is $1/2\Delta\xi_c$. The thickness of the other layers in the concrete is equal to $\Delta\xi_c$.

Equations for the Fire-Steel Boundary

It is assumed that the entire surface of the column is exposed to the heat of a fire whose temperature course follows that of the standard fire described in ASTM-E119[22] or CAN4-S101[23]. This temperature course can be approximately described by the following expression[20]:

$$T_f^j = 20 + 750\left[1 - \exp\left(-3.79553\sqrt{\tau}\right)\right] + 170.41\sqrt{\tau} \tag{1}$$

where τ is the time in hours and T_f^j is the fire temperature in °C at time $\tau = j\,\Delta\tau$.

The temperature rise in the layer can be derived by creating a heat balance for each layer. In the following, all calculations will be carried out for a unit length of the column. For the layer at the surface of the column, the temperature at time $\tau = (j + 1)\Delta\tau$ is given by the expression:

$$T_1^{j+1} = T_1^j + \frac{2R_s\Delta\tau}{(\rho_s c_s)_1^j\left(R_s - \frac{\Delta\xi_s}{4}\right)\Delta\xi_s}\left\{\sigma\varepsilon_s\varepsilon_f\left[\left(T_f^j + 273\right)^4 - \left(T_1^j + 273\right)^4\right]\right\} \tag{2}$$

$$- \frac{\Delta\tau\left(R_s - \frac{\Delta\xi_s}{2}\right)}{(\rho_s c_s)_1^j\left(R_s - \frac{\Delta\xi_s}{4}\right)(\Delta\xi_s)^2}\left[(k_s)_1^j + (k_s)_2^j\right]\left(T_1^j - T_2^j\right)$$

Equations for Inside the Steel

For the layers in the steel, except for the surface layer and the layer at the boundary of the steel and concrete, the temperature at time $\tau = (j + 1)\Delta\tau$ is given by:

$$T_m^{j+1} = T_m^j + \frac{\Delta\tau}{2(\rho_s c_s)_m^j\left[R_s - (m-1)\Delta\xi_s\right](\Delta\xi_s)^2} \tag{3}$$

$$\left\{\left[R_s - \left(m - \frac{3}{2}\right)\Delta\xi_s\right]\left[(k_s)_{m-1}^j + (k_s)_m^j\right]\left(T_{m-1}^j - T_m^j\right)\right.$$

$$\left.-\left[R_s - \left(m - \frac{1}{2}\right)\Delta\xi_s\right]\left[(k_s)_m^j + (k_s)_{m+1}^j\right]\left(T_m^j - T_{m+1}^j\right)\right\}$$

Equations for the Steel Boundary

For the layer at the boundary of the steel and the concrete, the temperature at time $\tau = (j + 1)\Delta\tau$ is given by:

$$T_{M_1}^{j+1} = T_{M_1}^j$$

$$+ \frac{\Delta\tau}{(\rho_s c_s)_{M_1}^j\left[R_s - \left(M_1 - \frac{5}{4}\right)\Delta\xi_s\right]\Delta\xi_s + \left[(\rho_c c_c)_{M_1}^j + \rho_w c_w \phi_{M_1}^j\right]\left(R_c - \frac{\Delta\xi_c}{4}\right)\Delta\xi_c} \tag{4}$$

$$\left\{\left[\frac{R_s - \left(M_1 - \frac{3}{2}\right)\Delta\xi_s}{\Delta\xi_s}\right]\left[(k_s)_{M_1-1}^j + (k_s)_{M_1}^j\right]\left(T_{M_1-1}^j - T_{M_1}^j\right)\right.$$

$$\left.- \frac{R_c - \frac{\Delta\xi_c}{2}}{\Delta\xi_c}\left[(k_c)_{M_1}^j + (k_c)_{M_1+1}^j\right]\left(T_{M_1}^j - T_{M_1+1}^j\right)\right\}$$

Equations for Inside the Concrete

For the layers in the concrete, except for the layer at the centre of the column and the layer at the boundary of the concrete and steel, the temperature at time $\tau = (j+1)\Delta\tau$ is given by:

$$
\begin{aligned}
T_m^{j+1} &= T_m^j \\
&+ \frac{\Delta\tau}{2\left[(\rho_c c_c)_m^j + \rho_w c_w \phi_m^j\right]\left[R_c - (m - M_1)\,\Delta\xi_c\right](\Delta\xi_c)^2} \\
&\quad \left\{\left[R_c - \left(m - M_1 - \frac{1}{2}\right)\Delta\xi_c\right]\right. \\
&\quad \left[(k_c)_{m-1}^j + (k_c)_m^j\right]\left(T_{m-1}^j - T_m^j\right) \\
&\quad \left[R_c - \left(m - M_1 + \frac{1}{2}\right)\Delta\xi_c\right] \\
&\quad \left.\left[(k_c)_m^j + (k_c)_{m+1}^j\right]\left(T_m^j - T_{m+1}^j\right)\right\}
\end{aligned}
\qquad (5)
$$

Equations for the Centre of the Concrete

For the centre layer, the temperature at time $\tau = (j+1)\Delta\tau$ is given by:

$$
\begin{aligned}
T_{M_2}^{j+1} &= T_{M_2}^j \\
&+ \frac{2\Delta\tau}{\left[(\rho_c c_c)_{M_2}^j + \rho_w c_w \phi_{M_2}^j\right](\Delta\xi_c)^2} \\
&\quad \left[(k_c)_{M_2-1}^j + (k_c)_{M_2}^j\right]\left[T_{M_2-1}^j - T_{M_2}^j\right]
\end{aligned}
\qquad (6)
$$

Effect of Moisture

The effect of moisture in the concrete on the column temperatures is taken into account by assuming that, in each layer, the moisture starts to evaporate when the temperature reaches 100°C. In the period of evaporation, all the heat supplied to a layer is used for evaporation until the layer is dry.

For the concrete layer at the boundary between steel and concrete, the initial volume of moisture is given by:

$$
V_{M_1} = \pi\left(R_c - \frac{\Delta\xi_c}{4}\right)\Delta\xi_c \phi_{M_1} \qquad (7)
$$

From a heat balance equation, it can be derived that, per unit length of the column, the volume $V_{M_1}^j$ evaporated in the time $\Delta\tau$ from the concrete layer at the boundary steel-concrete, is:

$$
\begin{aligned}
V_{M_1}^j &= \frac{\pi\Delta\tau}{\rho_w\lambda_w}\left\{\frac{1}{\Delta\xi_s}\left[R_s - \left(M_1 - \frac{3}{2}\right)\Delta\xi_s\right]\right. \\
&\quad \left[(k_s)_{M_1-1}^j + (k_s)_{M_1}^j\right]\left(T_{M_1-1}^j - T_{M_1}^j\right) \\
&\left. - \frac{1}{\Delta\xi_c}\left(R_c - \frac{\Delta\xi_c}{2}\right)\left[(k_c)_{M_1}^j + (k_c)_{M_1+1}^j\right]\left(T_{M_1}^j - T_{M_1+1}^j\right)\right\}
\end{aligned}
\qquad (8)
$$

For the concrete layers inside the column, except for the layer at the boundary between the steel and concrete and the centre layer, the initial volume of moisture is given by:

$$
V_m = 2\pi\left[R_c - (m - M_1)\,\Delta\xi_c\right]\Delta\xi_c\phi_m \qquad (9)
$$

Similarly, as for the boundary concrete layer, it can be derived that, per unit length of the column, the volume ΔV_m^j, evaporated in time $\Delta\tau$ from these layers, is:

$$
\begin{aligned}
\Delta V_m^j &= \frac{\pi\Delta\tau}{\rho_w\lambda_w\Delta\xi_c}\left\{\left[R_c - \left(m - M_1 - \frac{1}{2}\right)\Delta\xi_c\right]\right. \\
&\quad \left[(k_c)_{m-1}^j + (k_c)_m^j\right]\left(T_{m-1}^j - T_m^j\right) \\
&\quad \left[R_c - \left(m - M_1 + \frac{1}{2}\right)\Delta\xi_c\right] \\
&\quad \left.\left[(k_c)_m^j + (k_c)_{m+1}^j\right]\left(T_m^j - T_{m+1}^j\right)\right\}
\end{aligned}
\qquad (10)
$$

For the concrete centre layer, the initial volume of moisture is:

$$
V_{M_2} = \pi\frac{(\Delta\xi_c)^2}{4}\phi_{M_2} \qquad (11)
$$

From a heat balance equation, it can be derived that, per unit length of the column, the volume $\Delta V_{M_2}^j$ evaporated in time $\Delta\tau$ from the centre layer, is:.

$$
\begin{aligned}
\Delta V_{M_2}^j &= \frac{\pi\Delta\tau}{2\rho_w\lambda_w} \\
&\quad \left[(k_c)_{M_2-1}^j + (k_c)_{M_2}^j\right]\left[T_{M_2-1}^j - T_{M_2}^j\right]
\end{aligned}
\qquad (12)
$$

Stability Criteria

In order to ensure that any error existing in the solution at some time will not be amplified in subsequent calculations, a stability criterion has to be satisfied. For a selected value of $\Delta\xi$, this limits the maximum time step $\Delta\tau$. Following the method described in Reference 19, it can be derived that, for the fire-exposed columns, the criterion of stability is given by the smallest of the following three criteria of stability:

$$\Delta\tau_1 = \frac{(\rho_s c_s)_{\min}\,(\Delta\xi_s)^2}{2\left[h_{\max}\Delta\xi_s + (k_s)_{\max}\right]}, \tag{13}$$

at boundary fire-steel

$$\Delta\tau_2 = \left\{(\rho_s c_s)_{\min}\left[R_s - \left(M_1 - \frac{5}{4}\right)\Delta\xi_s\right]\Delta\xi_s + (\rho_c c_c)_{\min}\left(R_c - \frac{\Delta\xi_c}{4}\right)\Delta\xi_c\right\} \Big/ 2\left\{\frac{\left[R_s - \left(M_1 - \frac{3}{2}\right)\Delta\xi_s\right]}{\Delta\xi_s}(k_s)_{\max} + \frac{\left(R_c - \frac{\Delta\xi_c}{2}\right)}{\Delta\xi_c}(k_c)_{\max}\right\}, \tag{14}$$

at boundary steel-concrete

$$\Delta\tau_3 = \frac{(\rho_c c_c)_{\min}\,(\Delta\xi_c)^2}{4\,(k_c)_{\max}} \tag{15}$$

at the centre

In these equations $(\rho_s c_s)_{\min}$ and $(\rho_c c_c)_{\min}$ are the minimum values of the heat capacity of the steel and concrete, $(k_s)_{\max}$ and $(k_c)_{\max}$ the maximum values of the thermal conductivity of steel and concrete and $h_{\max}$ the maximum value of the coefficient of heat transfer to be expected during the exposure to fire. For exposure to the standard fire, the maximum value of the coefficient of heat transfer $h_{\max}$ is approximately 675 W/m^2 °C (Reference 16).

Procedure for Calculation of Column Temperatures

With the aid of Equations 1 - 15, and the relevant material properties given in the Appendix, the temperature distribution in the column and on its surface can be calculated for any time, $\tau = (j + 1)\Delta\tau$, if the temperature distribution at the time $j\Delta\tau$ is known. Starting from an initial temperature of 20°C, the temperature history of the column can be calculated by repeated application of Equations 1 - 15.

Strength of Columns During Fire Exposure

Division of Cross-section into Annular Elements

To calculate the deformations and stresses in the column and its strength, the cross-sectional area of the column is subdivided into a number of annular elements. In Figure 3, the arrangement of the elements is shown in a quarter section of the column. The arrangement of the elements in the three other quarter sections is

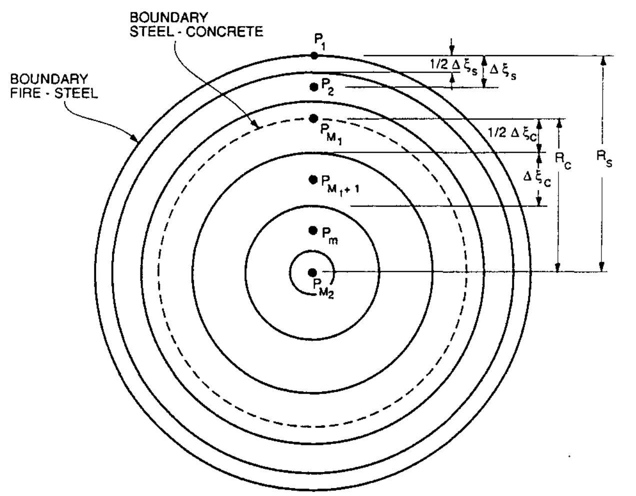

Figure 2. Arrangement of layers in section of concrete filled hollow steel column.

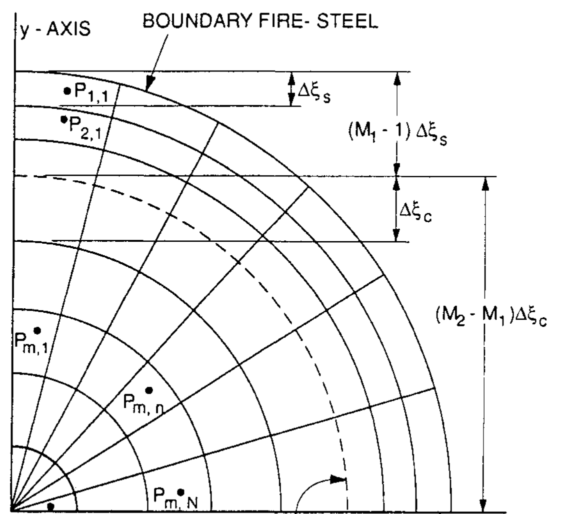

Figure 3. Arrangement of elements in quarter section of concrete filled hollow steel column.

identical to this. In the radial direction, the subdivision is the same as that shown in Figure 2, where the cross-section is divided into concentric layers. In the tangential direction, each quarter section layer is divided into N elements. The temperature representative of that of an element is assumed to be equal to the temperature at its centre. It is obtained by taking the average of the temperatures at the tangential boundaries of each element, previously calculated with the aid of Equations 1 - 15.

Thus for an element $P_{m,n}$, the representative temperature is:

$$\left(T^j_{m,n}\right)_{\text{annular}} = \left(\frac{T^j_m + T^j_{m+1}}{2}\right)_{\text{layer}} \qquad (16)$$

where the subscripts, annular and layer, refer to the annular elements shown in Figure 3 and the element layers shown in Figure 2, respectively.

Similarly, it is assumed that the stresses and deformations at the centre of an element are representative of those of the whole element.

Assumptions in the Calculation of Strength During Fire

During exposure to fire, the strength of the column decreases with the duration of exposure. The strength of the column can be calculated by a method based on a load deflection or stability analysis[24].

In this method, the columns are idealized as pin-ended columns of effective length KL (Figure 4). The load on the column is intended to be concentric. Due to imperfections of the columns and the loading device, some eccentricity exists. The loading system and the test columns were made with high precision. Therefore, in the calculations, a very small arbitrary load eccentricity of 0.2 mm, reflecting a nearly concentric load, has been selected for the initial eccentricity.

The curvature of the column is assumed to vary from pin-ends to midheight according to a straight line relation, as illustrated in Figure 4. For such a relation, the deflection at midheight Y, in terms of the curvature X of the column at this height, can be given by:

$$Y = X\frac{(KL)^2}{12} \qquad (17)$$

For any given curvature, and thus for any given deflection at midheight, the axial strain is varied until the internal moment at the mid-section is in equilibrium with the applied moment, i .e.,

$$\sum_{m,n} f_{m,n} A_{m,n} x_{m,n} = \sum_{m,n} f_{m,n} A_{m,n} (Y + e) \qquad (18)$$

In this way, a load-deflection curve can be calculated for any specific time 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. In the calculation of column strength, the following assumptions were further made:

1. The properties of the steel and concrete are those described in the Appendix.
2. Concrete has no tensile strength.
3. Plane sections remain plane.
4. There is no slip between steel and concrete.
5. There is no composite action between the steel and concrete.
6. 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

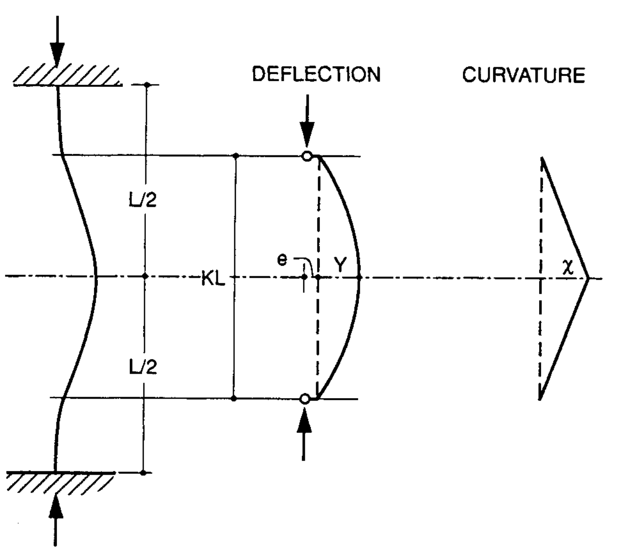

Figure 4. Load-deflection analysis.

shortened column as 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 annular elements shown in Figure 3 was used. Because the strains and 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 that the column can carry and the moments in the section were obtained by adding the loads carried by each element and the moments contributed by them.

Equations for the Steel

The strain in an element of the steel can be given as the sum of the thermal expansion of the steel $(\varepsilon_s)_T$, the axial strain of the column ε and the strain due to bending of the column χ_s/ρ, where χ_s is the horizontal distance of the steel element to the vertical plane through the y-axis of the column section and ρ is the radius of curvature. For the steel to the right of the y-axis (Figure 3), the strain $(\varepsilon_s)_R$ is given by:

$$(\varepsilon_s)_R = -(\varepsilon_s)_T + \varepsilon + \frac{x_s}{\rho} \qquad (19)$$

For the steel elements to the left of the y-axis, the strain $(\epsilon_s)_L$ is given by:

$$(\varepsilon_s)_L = -(\varepsilon_s)_T + \varepsilon - \frac{x_s}{\rho} \qquad (20)$$

The stresses in the steel are calculated using the same stress-strain relations for steel, given in Reference 16. These relations are given by Equations 23 - 25 in the Appendix.

Equations for the Concrete

The strain in the concrete for the elements to the right of the y-axis can be given by:

$$(\varepsilon_c)_R = -(\varepsilon_c)_T + \varepsilon + \frac{x_c}{\rho} \qquad (21)$$

and for the elements to the left of the y-axis by:

$$(\varepsilon_c)_L = -(\varepsilon_c)_T + \varepsilon - \frac{x_c}{\rho} \qquad (22)$$

The stresses in the elements are calculated using the same stress-strain relations for concrete, given by Equations 29 - 31 in the Appendix.

Procedure for the Calculation of Column Strength

With the aid of Equations 19 - 22, Equations 23 - 25 and Equations 29 - 31, the stresses at mid-section in the steel and 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 derived. 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 of the column as a function of time of fire exposure. This strength reduces gradually with time. At a certain point, 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.

TEST SPECIMENS

Thirty test specimens consisting of cylindrical hollow steel columns filled with siliceous aggregate concrete were constructed for the purpose of verifying the validity of the mathematical model described in this paper. The test specimens are described in detail in Reference 17 and are illustrated in Figure 5.

All columns were 3810 mm long from end plate to end plate. The columns had an outside diameter ranging from 141 mm to 406 mm. The wall thickness of the columns varied from 4.8 mm to 12.7 mm.

The plate thickness and dimensions varied with the cross-section size. The thicknesses were 19, 25 and 38 mm and the plate dimensions were 356 x 356 mm and 508 x 610 mm. They are given for each column section size in Reference 17.

The steel columns were fabricated by cutting the steel to appropriate lengths. Steel end plates were then welded at the column extremities. Accurate centering and perpendicularity of the end plates were given special attention. Before welding the end plates, a hole with a diameter approximately 26 mm smaller than the inner diameter of the hollow steel section was cut in each plate. The smaller diameter of the holes created, after welding, a lip of 13 mm, as shown in Figure 5.

Four small holes were also drilled in the steel wall to provide vent holes for water vapour produced during the experiment. Two of the holes were located opposite one another at 1448 mm above midheight of the column. The other two were also located opposite to one another at 1448 mm below midheight of the column (Figure 5).

The steel of the columns was manufactured according to CSA Standard CAN3-G40.20-M77[25], Class H (hot formed or cold formed, stress relieved) and had a specified yield strength of 350 MPa.

The concrete was poured in the column through the top opening. Its composition, per cubic metre of concrete mix, was as follows:

Cement	366 kg
Water	189 kg
Sand (quartz)	660 kg
Stone (quartz)	1120 kg

The average 28-day concrete cylinder strength was approximately 25 MPa. The average cylinder strength at time of testing was approximately 30 MPa.

Chromel-alumel thermocouples with a thickness of 0.91 mm were installed at midheight of the column for measuring temperatures of the steel and concrete at different locations in the cross-section. The locations of the thermocouples are described in detail in Reference 17.

Test Apparatus

These tests were carried out by exposing the columns to heat in a column test furnace. The test furnace was designed to produce the conditions to which a member might be subjected during a fire. It consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework. The characteristics and instrumentation of the furnace, which has a loading capacity of 1000 t, are described in detail in a previous paper[26].

Test Conditions and Procedure

The columns were installed in the furnace by bolting the end plates to a loading head at the top and a hydraulic jack at the bottom. Rotation of the end plates was prevented. The conditions of the columns were fixed-fixed in all tests. For each column, the length that was exposed to fire was approximately 3000 mm. At high temperature, the stiffness of the unheated column ends, which is great in comparison with that of the heated portion of the column, contributes to a reduction in the column effective length. In previous tests[27], it was found that for columns tested fixed at the ends, an effective length of 2000 mm represents experimental behaviour.

The columns were tested under loads that varied from about 60 to 140% of the factored

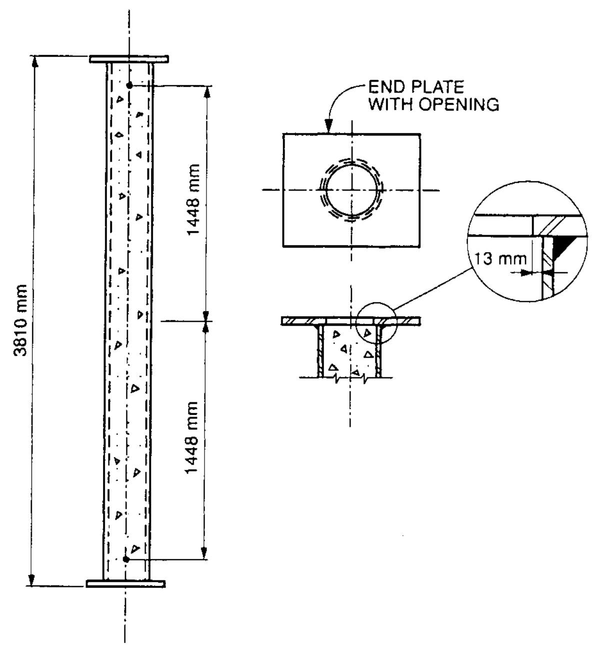

Figure 5. Elevation and cross section of test specimens.

compressive resistance of the concrete section and about 10 to 45% of the factored compressive resistance of the composite column, according to the CAN3-S16.1-M84 Standard[28]. The effective length used in the calculation of the factored resistances was that recommended in CAN3-S16.1-M84 for columns with both ends fixed, i.e., 0.65 times the column unsupported length.

The columns were exposed to heating controlled in such a way that the average temperature in the furnace closely followed the ASTM-E119[22] or the CAN4-S101[23] standard temperature-time curve.

The columns were considered to have failed and the tests terminated when the load, which was applied with a hydraulic jack, could no longer be maintained. The hydraulic jack has a maximum speed of 76 mm/min.

Results and Discussion

In the following, the results obtained for five column sections, representative of the range of sections tested, will be discussed. Information on the column dimensions, fire resistances and test loads are given in Table 1.

Using the mathematical model described in this paper, the temperatures, deformations and fire resistances of the five columns were calculated. The results are shown in Figures 6 - 15.

In Figures 6 - 10, calculated temperatures are compared with those measured in the steel and at various depths in the concrete for the five columns under consideration. In general, there is reasonably good agreement between calculated and measured steel temperatures. The temperatures measured in the concrete, relatively close to the steel, are on the average somewhat lower than the calculated temperatures. The differences reduce, however, with increasing temperatures and duration of fire exposure.

For locations deeper in the concrete, there is initially a rapid rise in the measured temperatures, followed by a period of nearly constant temperatures in the earlier stages of the fire exposure. This temperature behaviour may be the result of thermally induced migration of moisture towards the centre of the column. Although the model takes into account evaporation of the moisture, it does not take into account the migration of the moisture towards the centre. That migration appears to account for the deviation between calculated and measured temperatures at the earlier stages of fire exposure. At a later stage, however, which is the important stage from the point of view of predicting the fire resistance of the columns, there is good agreement between calculated and measured temperatures.

In Figures 11-15, the calculated and measured axial deformations during the exposure to fire are shown for the five columns. In general, there is reasonably good agreement in the trend of deformations between calculated and measured results. There are some differences, however, in the actual values of the calculated and measured deformations. These differences are caused by several factors, namely, load, thermal expansion, bending and creep, which cannot be completely taken into account in the calculations. There is also uncertainty in load sharing between the steel and concrete. In particular in the case of Column No. 1 (Figure 11), this is

TABLE 1. COLUMN DIMENSIONS, FIRE RESISTANCES AND TEST LOADS

Column No.	Outer Diameter (mm)	Steel Wall Thickness (mm)	Test Load (kN)	Load Intensity % of Factored Compressive Resistance of: Concrete Section	Composite Column	Fire Resistance (min) Measured	Calculated
1	141.3	6.55	110	77	12	55	48
2	168.3	4.78	218	87	23	56	48
3	219.1	4.78	492	121	35	80	65
4	273.1	5.56	525	74	23	133	117
5	355.6	12.70	1050	103	18	170	144

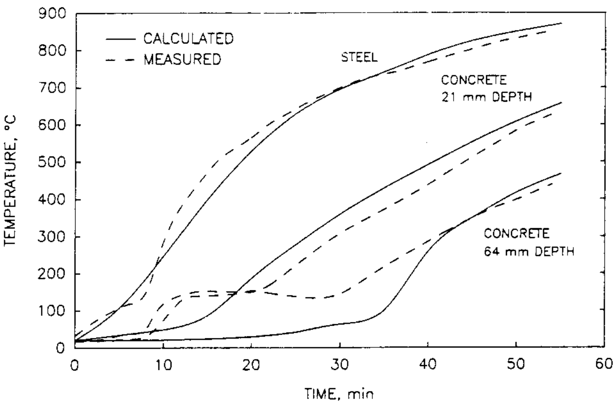

Figure 6. Calculated and measured temperatures in steel and at various depths in the concrete (column diameter=141.3 mm, wall thickness=6.55 mm).

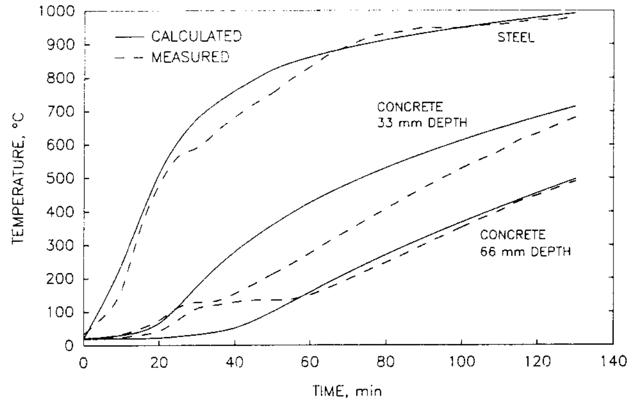

Figure 9. Calculated and measured temperatures in steel and at various depths in the concrete (column diameter=273.1 mm, wall thickness=5.56 mm).

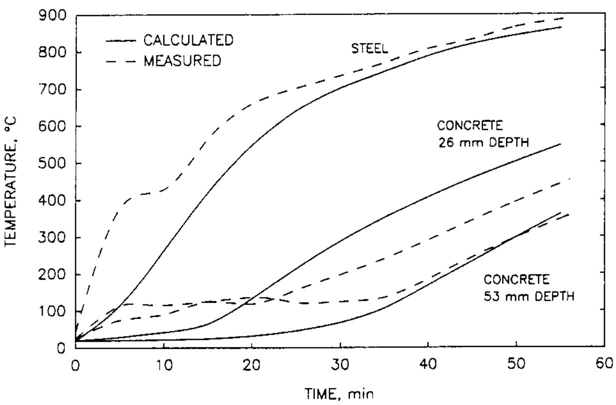

Figure 7. Calculated and measured temperatures in steel and at various depths in the concrete (column diameter=168.3 mm, wall thickness=4.78 mm).

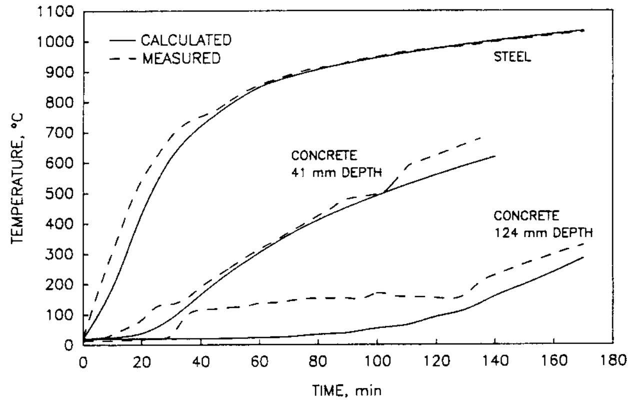

Figure 10. Calculated and measured temperatures in steel and at various depths in the concrete (column diameter=355.6 mm, wall thickness=12.7 mm).

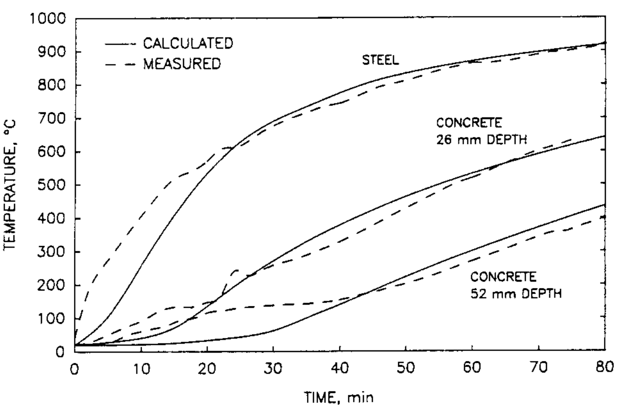

Figure 8. Calculated and measured temperatures in steel and at various depths in the concrete (column diameter=219.1 mm, wall thickness=4.78 mm).

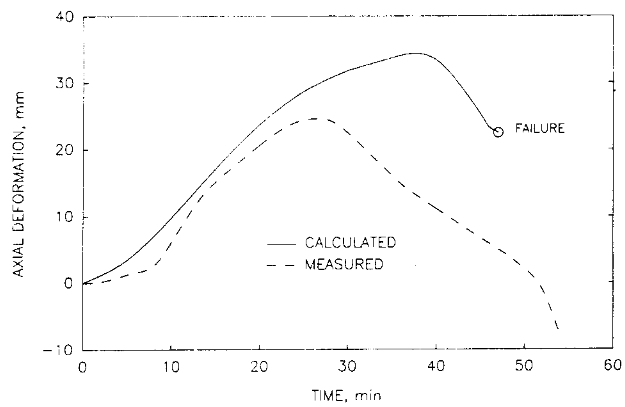

Figure 11. Calculated and measured column axial deformations (column diameter=141.3 mm, wall thickness=6.55 mm).

probably the main cause for the difference between calculated and measured axial deformations at approximately 40 minutes.

In all cases, the calculated deformations near the point of failure are smaller than the measured deformations. It is likely that the main cause of the difference is creep of the steel and concrete, which becomes more pronounced at higher temperatures. A part of the creep, however, is implicitly taken into account in describing the mechanical properties of the materials used.

In Table 1, the calculated and measured fire resistances, i.e., failure times of the columns, are given. In general, measured fire resistances are higher than calculated fire resistances. The differences vary from about 10 to 20% of the measured fire resistances. One reason for the difference is the definition of the failure point. At present, there are no generally accepted failure criteria for columns. In the calculations, the failure point has been defined as the point at which the column can no longer support the applied load. At this point, the column is assumed to fail instantaneously. In tests, failure generally does not occur instantaneously. At an advanced stage of the test, the column starts to contract axially at a rate that increases with time. Both the contraction and rate of contraction can reach very high values.

A current failure criterion, based on the contraction and the rate of contraction, is that proposed in the ISO 834 Standard[29]. According to this Standard, a column is considered to have failed if the column has contracted axially by 0.01 L mm and the rate of contraction has reached 0.003 L mm/min, where L is the length of the column. For the columns under consideration these criteria correspond to a maximum contraction of 38 mm and a rate of contraction of 11.4 mm/min. In the tests by the National Research Council of Canada (NRCC), the columns were considered to have failed when the load, applied by a hydraulic jack that has a maximum speed of 76 mm/min, could no longer be maintained. Because both the ISO and the NRCC criteria for the rate of contraction are high, there is virtually no difference in the time between reaching the ISO rate of contraction or the NRCC rate of contraction, as can be seen in Figures 11 - 15.

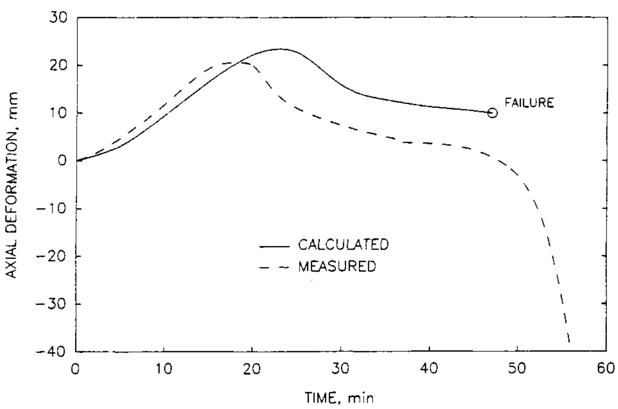

Figure 12. Calculated and measured column axial deformations (column diameter=168.3 mm, wall thickness=4.78 mm).

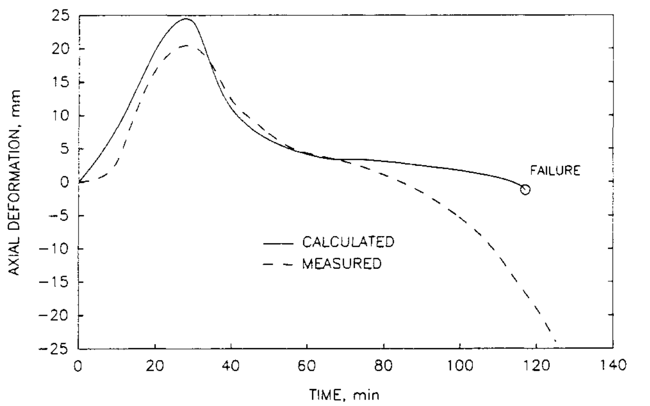

Figure 14. Calculated and measured column axial deformations (column diameter=273.1 mm, wall thickness=5.56 mm).

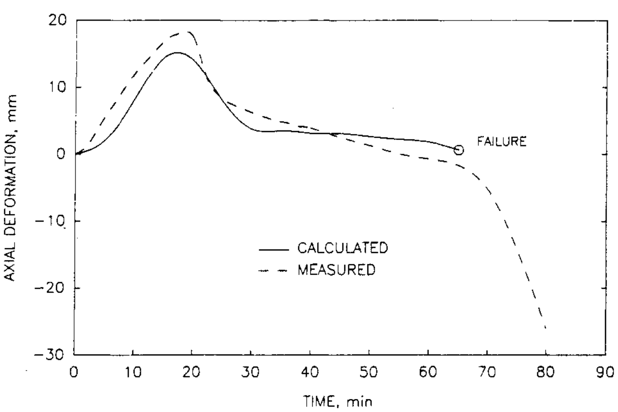

Figure 13. Calculated and measured column axial deformations (column diameter=219.1 mm, wall thickness=4.78 mm).

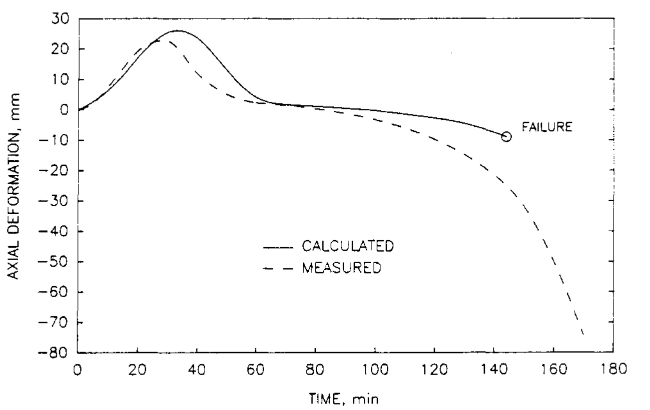

Figure 15. Calculated and measured column axial deformations (column diameter=355.6 mm, wall thickness=12.7 mm).

Since there are no specific failure criteria for columns in North American test standards and ISO Standards are widely used in the world, it is proposed to adopt these criteria in this study. It implies that, for concrete filled columns, calculated fire resistances should be increased by an amount to correct for the difference in failure criteria between the model and the ISO Standard.

As shown in Reference 17, more columns were tested than those considered in the present study. Some of the columns were tested at a relatively high load, namely well over 100% of the factored compressive resistance of the concrete section, according to the CAN3-S16.1M84 Standard[28]. In some of these tests, especially when the diameter of the column was large (323 mm and more), the column failed considerably earlier than expected and the behaviour of the column could not be predicted. A possible reason for the early failure of these columns is that, due to transfer of the load from the steel to the concrete during the exposure to fire, the concrete is subjected to local excessive stresses.

Until further studies have been carried out and other existing results[18] have been analyzed, it is recommended to restrict, for non-reinforced concrete filling, the use of the model to loads not greater than 80% of the factored compressive resistance of the concrete section which, as indicated by the test results[17], appears to be a conservative limit.

With steel fibre or bar reinforcement in the concrete, however, much higher loads can be applied without the occurrence of unexpected early failure[2-9,12,13,15]. Tests on reinforced concrete columns[30] have shown that the failure time or fire resistance of the columns remains predictable, even when the applied load is approximately 1.5 times the maximum factored axial load resistance of the column calculated according to the CAN3-A23.3-M84[31] Standard or the ACI Standard 318-83[32].

Conclusions

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

1. The mathematical model employed in this study is capable of predicting the fire resistance of concentrically loaded circular concrete filled steel columns with an accuracy that is adequate for practical purposes.

2. To avoid premature failure of the column, an upper limit for the load has to be set. A conservatively estimated value for columns filled with plain concrete is 80% of the factored compressive resistance of the concrete section of the composite column as defined in CAN3-S16.1-M84. Further studies are needed to more precisely establish this limit.

3. The model can be used for the evaluation of the fire resistance of circular concrete filled hollow steel columns for any value of the significant parameters, such as load, column section dimensions, column length and concrete strength, without the necessity of testing. The results produced by the model are expected to be conservative because of the use of a conservative failure criterion in the model in comparison with those in current test standards, and conservative heat transfer coefficients. Further studies, using mainly existing data, are needed to enable more precise evaluation of the fire resistance of the columns as a function of the significant parameters.

4. The model can also be used for the calculation of the fire resistance of circular steel columns filled with concretes other than those investigated in this study; for example, carbonate aggregate or lightweight concretes, if the relevant material properties are known.

NOMENCLATURE

A	area of element
c	specific heat [J/kg °C]
e	eccentricity [m]
f	stress [MPa]
f'	cylinder strength of concrete at temperature T [MPa]
f'_{c0}	cylinder strength of concrete at room temperature [MPa]
f_y	strength of steel at temperature T [MPa]
h	coefficient of heat transfer at fire exposed surface [W/m^2 °C]
k	thermal conductivity [W/m °C]
K	effective length factor [-]
L	unsupported length of the column [m]
l	length of column that contributes to axial deformation [m]
M_1	number of points P in the steel section in radial direction [-]
M_2	total number of points P in the column section in radial direction [-]
N_1	number of elements in tangential direction
P	point [-]
R_c	radius of concrete core [m]
R_s	radius of steel column [m]
T	temperature [°C]
V	volume of moisture in an element [m^3]
x	coordinate [m]
y	coordinate [m]
Y	lateral deflection of column at mid-height [m]
α	coefficient of thermal expansion [1/°C]
Δ	increment or difference [-]
$\Delta\xi$	mesh width in radial direction [m]
ε	emissivity [-], strain [m/m]
λ	heat of vaporization [J/kg]
ρ	density [kg/m^3], radius of curvature [m]
σ	Stefan-Boltzmann constant [W/m^{2}K^4]
τ	time [h]
ϕ	concentration of moisture [-]
χ	curvature of column at mid-height [1/m]

Subscripts

0	at room temperature
c	of concrete
f	of fire
L	left of the x-axis
m, M_1, M_2	at the points m, M_1, M_2 in radial direction
max	maximum
min	minimum
n, N_1	at the points n, N_1 in tangential direction
p	pertaining to proportional stress-strain relation
R	right of the x-axis
s	of steel
T	pertaining to temperature
w	of water

Superscripts

j	at $\tau = j\Delta\tau$

REFERENCES

1. Grandjean, G. and Lelong, C., "Tenue au Feu de Profils Creux Carrés Remplis de Béton," *Construction Metallique*, No. **80**, Institut Technique du Bâtiment et des Travaux Publics, 1977.
2. Giddings, T.W., "Fire Resistant Construction in SHS—Today and Tomorrow," British Steel Corporation, Tubes Division, U.K., 1979.
3. Flemington, R.A., "Fire Protection of Hollow Structural Sections," Technical Bulletin 21, Stelco Inc, Hamilton, Ontario, 1980.
4. Grandjean, G., Grimault, J.P. and Petit, L., "Détermination de la Durée au Feu des Profils Creux Remplis de Béton," Rapport Final, Recherche Technique Acier, Commission des Communautés Européennes, Luxembourg, 1981.
5. Grimault, J.P., "Calcul à l'Incendie des Profils Creux Remplis de Béton, Manuel Pratique," Notice 1092 de la Chambre Syndicale des Fabricants de Tubes d'Acier, Paris, France, 1982.
6. Kordina, E.H.K. and Klingsch, W.,

"Brandverhalten von Stahlstützen im Verbund mit Beton und von massiven Stahlstützen ohne Beton," Teil 1: Bericht, Institut für Baustoffe, Massivbau und Brandschutz Lehrstuhl für Massivbau, Technische Universität Braunschweig, West Germany, 1983.

7. *Design Manual for SHS Concrete Filled Columns*, British Steel Corporation, Corby Technical Centre, U.K., 1984.
8. Schleich, J.B., "Fire Safety Design of Composite Columns," *International Conference Fire Safe Steel Construction: Practical Design*, ARBED-Recherche, Luxembourg, 1984.
9. Grimault, J.P. and Mouty, J., "Prévision de la Stabilité au Feu des Poteux en Profils Creux Remplis de Béton,Æ *Construction Métallique*, No. **3**, 1984.
10. Suzuki, T., Kimura, M., Kodaira, A. and Aburakawa, M., "Experimental Study on Fire Resistance of Concrete-Stiffened Steel Structures," Takenaka Technical Research Report No. 32, Tokyo, Japan, 1984.
11. Rudolph, K., Richter, E., Hass, R. and Quast, U., "Principles for Calculation of Load Bearing and Deformation Behaviour of Composite Structural Elements under Fire Action," *Fire Safety Science, Proceedings of the First International Symposium*, Hemisphere Publishing Corporation, Washington, D.C., 1985, pp. 301-310.
12. Twilt, L., "Design Charts for the Fire Resistance of Concrete Filled HSS Columns under Centric Loading," Instituut TNO voor Bouwmaterialen en Bouwconstructies, Delft, Holland, 1988.
13. Kruppa, J., "Méthode de Prévision par le Calcul du Comportement au Feu des Poteaux Mixtes (Acier+Béton)," *Construction Métallique*, No. **3**, 1988.
14. Ikeda, K., Kumagai, T., Nakamura, K. and Saito, H., "Fire Resistance Tests of Unbonded Tubular Steel Concrete Columns," Fire Research Laboratory of the Institute of Technology, Shimizu Corporation, Tokyo, Japan.
15. Klingsch, W., and Wittbecker, F.W., "Fire Resistance of Hollow Section Composite Columns of Small Cross Sections," Bergische Universität, Wuppertal, West Germany, 1988.
16. Lie, T.T., "A Procedure to Calculate Fire Resistance of Structural Members," *Fire and Materials*, Vol. **8**, No. 1, 1984, pp.40-48.
17. Lie, T.T. and Caron, S.E., "Fire Resistance of Circular Hollow Steel Columns Filled With Siliceous Aggregate Concrete: Test Results," Internal Report No. 570, Institute for Research in Construction, National Research Council of Canada, Ottawa, 1988.
18. Lie, T.T. and Caron, S.E., "Fire Resistance of Circular Hollow Steel Columns Filled With Carbonate Aggregate Concrete: Test Results," Internal Report No. 573, Institute for Research in Construction, National Research Council of Canada, Ottawa, 1988.
19. Dusinberre, G.M., *Heat Transfer Calculations by Finite Differences*, International Textbook Company, Scranton, Pennsylvania, 1961.
20. Lie, T.T. and Harmathy, T.Z., "A Numerical Procedure to Calculate the Temperature of Protected Steel Columns Exposed to Fire," Fire Study No. 28, Division of Building Research, National Research Council of Canada, NRCC 12535, Ottawa, Ontario, 1972.
21. Lie, T.T., "Temperature Distributions in Fire-exposed Building Columns," Journal of Heat Transfer, Vol. **99**, Series C, No.l, 1977.
22. *Standard Methods of Fire Tests of Building Construction and Materials*, ASTM-E11983, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1983.
23. *Standard Methods of Fire Endurance Tests of Building Construction and Materials*, CAN4-SlOl-M82, Underwriters' Laboratories of Canada, Scarborough, Ontario, 1982.
24. Allen, D.E. and Lie, T.T., "Further Studies of the Fire Resistance of Reinforced Concrete Columns," National Research Council of Canada, Division of Building Research, NRCC 14047, Ottawa, Ontario, 1974.
25. *General Requirements for Rolled or Welded Structural Quality Steel*, CAN3-G40.20M77, Canadian Standards Association, Toronto, Ontario, 1977.
26. Lie, T.T., "New Facility to Determine Fire Resistance of Columns," *Canadian Journal of Civil Engineering*, Vol. **7**, No. 3, 1980, pp. 551-558.
27. Lie, T.T., Lin, T.D., Allen, D.E. and Abrams, M.S., "Fire Resistance of Reinforced Concrete Columns," National Research Council of Canada, Division of Building Research, NRCC 23065, Ottawa, Ontario, 1984.
28. *Steel Structures for Buildings* (Limit States

Design), CAN3-S16.1-M84, Canadian Standards Association, Toronto, Ontario, 1984.

29. *ISO 834, Fire Resistance Tests—Elements of Building Construction*, Part I, General Requirements for Fire Resistance Testing (proposed revision), Underwriters' Laboratories of Canada, Scarborough, Ontario, 1989.

30. Lie, T.T., "Fire Resistance of Reinforced Concrete Columns: A Parametric Study," *Journal of Fire Protection Engineering*, Vol. 1, No. 4, 1989, pp. 121-130.

31. *Design of Concrete Structures for Buildings*, CAN3-A23.3-M84, Canadian Standard Association, Rexdale, Ontario, 1984.

32. *Building Code Requirement for Reinforced Concrete*, ACI Standard 318-83, American Concrete Institute, Detroit, MI, 1983.