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A model for evaluating the fire resistance of contour-protected steel columns

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Abstract. A numerical model, in the form of a computer program, for evaluating the fire resistance of insulated wide-flange steel columns is presented. The three stages associated with the thermal and structural analysis in the calculation of fire resistance of columns is explained. The use of the computer program for tracing the response of an insulated steel column from the initial pre-loading stage to collapse, due to fire, is demonstrated. The validity of the numerical model used in the program is established by comparing the predictions from the computer program with results from full-scale fire tests. Details of fire tests carried out on wide-flange steel columns protected with ceramic fibre insulation, together with results, are presented. The computer program can be used to evaluate the fire resistance of protected wide-flange steel columns for any value of the significant parameters, such as load, section dimensions, column length, type of insulation, and thickness of insulation without the necessity of testing.

Key words: fire resistance; computer program; high temperature; wide flange steel columns; insulated columns; contour protection.

1. Introduction

The I-shaped structural steel sections are structurally very efficient in resisting compression and bending loads and offer a number of benefits. Further, these sections have high fire resistance, compared to the other steel sections with similar unit-weight ratio. These benefits have stimulated

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the use of such steel columns in wide range of applications including buildings.

Fully developed building fires generally attain gas temperatures in the order of 1100 °C. As the mechanical properties of steel deteriorate rapidly at temperatures of about one half of this magnitude, it is necessary, with the possible exception of massive steel sections (Stanzak 1973), to provide some means of keeping steel columns relatively cool during exposure to fire. External insulation of a steel section to prevent excessive heat transfer to the steel, during the expected period of fire exposure, is the most common method of providing fire resistance. Of the various types of protection, contour protection, comprising of the application of a low density material to the profile of a section, is gaining popularity due to its cost-effectiveness.

In the past, the fire resistance of structural members could be determined only by testing, which is costly and time-consuming. However, the development of numerical techniques and an enhanced knowledge of the thermal and mechanical properties of materials at elevated temperatures, have made it possible to determine the fire resistance of various structural members by calculation. A detailed literature review (Ghani 1998) revealed the available information for determining the fire resistance of contour protected steel columns is based on the simplistic approach of limiting the temperature of steel and is applicable only for limited type of insulating materials (Lie and Stanzak 1973, BSI 1992). Further, the information, developed with limited test data, does not facilitate the use of new types of insulating materials or optimum fire protection provisions for cost-effective design under performance based codes such as the upcoming National Building Code of Canada (Richardson 1994).

To develop guidelines for the fire resistant design and construction of contour protected steel columns, a collaborative research project was undertaken between University Technology of Malaysia (UTM), Malaysia, and the Institute for Research in Construction, National Research Council (NRC), Canada. Both experimental and theoretical studies, using numerical techniques, were carried out to investigate the fire performance of contour protected steel columns. This paper deals with the development of a validated computer program for evaluating the fire resistance of wide-flange steel columns with contour protection.

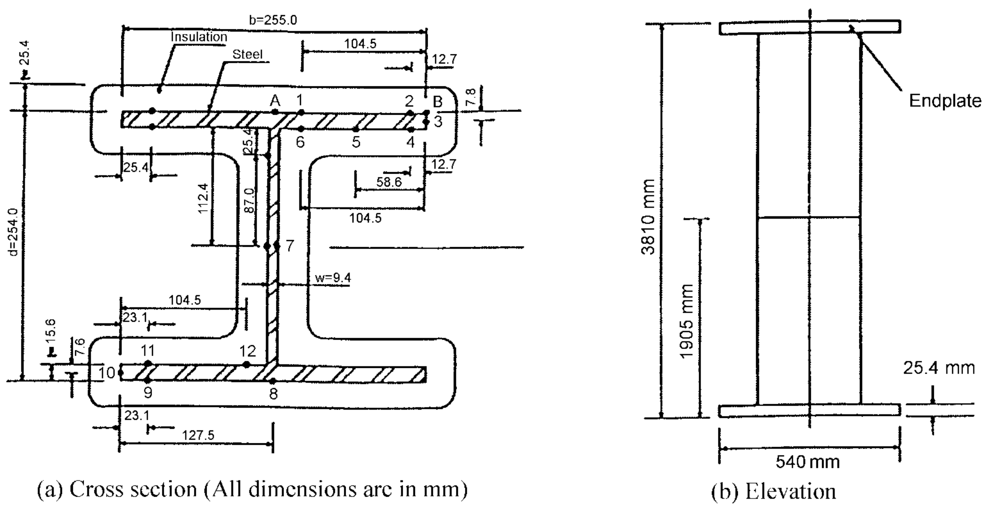


Fig. 1 Elevation and cross section of a typical contour protected steel column

2. Numerical model

A numerical model for predicting the behaviour of contour protected wide-flange steel section columns exposed to fire was developed. The numerical procedure used in the model is similar to the one which was previously applied to the fire resistance calculation for other structural elements (Lie and Almond 1990, Kodur and Lie 1997). Detailed equations for the calculation of the column temperatures and strength are given in Ghani (1998). Fig. 1 shows the elevation and cross-sectional details of a typical wide-flange steel section column protected by ceramic fiber insulation.

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

2.1 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 CAN/ULC (1989) which is similar to ASTM (1990). This temperature course can be approximated by the following expression:

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

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

2.2 Thermal analysis

The column temperatures are calculated by a finite difference method (Dusinberre 1961), where the temperature rise in the column can be derived by creating a heat balance for each element. By solving the heat balance equation for each element, the temperature history of the column can be calculated. This is achieved through using the temperature-dependent thermal properties of the insulation and steel of which the column is composed.

2.2.1 Assumptions

To calculate the temperature history, the following assumptions are made in order to reduce the complexity and increase its utility:

1. The temperature along the length of the column is equally distributed. This assumes that the heat distribution along the column length is equal and each plane of the cross-section experiences equal heat flow from the ceramic wall surface to the inner steel core, thus the mathematical model will be 2-dimensional.
2. It is assumed that heat transfer by convection is negligible, as the flow by convection will result in minimal increase in the rate of heat energy reaching the column core.

2.2.2 Cross section idealisation

For reasons of symmetry, only one-quarter of the section needs to be considered when calculating the temperature distribution in a square or rectangular column cross-section. The cross-section of the column is subdivided into meshes of elements and nodes, arranged in a network sloped at 45^o to the horizontal. The elements are square inside the column and triangular at the surface. For the square elements, the temperature at the centre node is representative of the temperature of the whole element. For the triangular surface elements, the representative nodes are located on the centre of each hypotenuse. The typical idealisation of a quarter section is shown in Fig. 2.

The cross-section is located on a set of X-Y co-ordinate axes with a virtual matrix axes M, N representing nodal points and the origin 0,0 at X-Y axes will start with (1,1). Each (m,n) point is located at $x = (m-1)/\sqrt{2}$ and $y = (n-1)\Delta\xi/\sqrt{2}$ where $\Delta\xi$ is the width of each square element. The maximum distance that can be attained along the X and Y axes is m=M and n=N.

There are nine boundary conditions for the possible flow of heat in the cross-section. These boundaries are located at m=Ma, Mb, Mc, M and at n=Na, Nb, Nc, N as shown in Figs. 2 and 3. They represent the following nine boundary conditions that have to be analysed individually along the X and the Y axes:

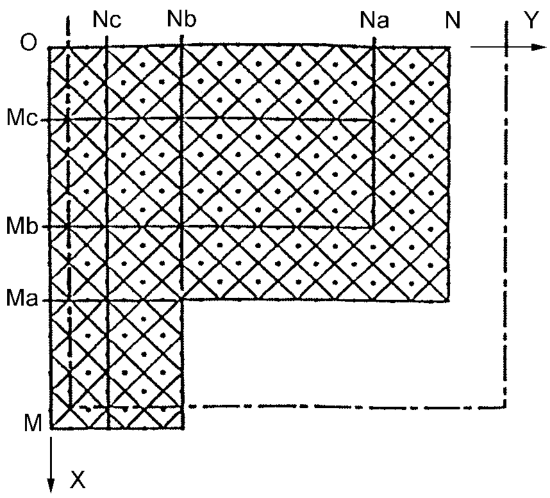


Fig. 2 Idealization of column for cross-section for analysis

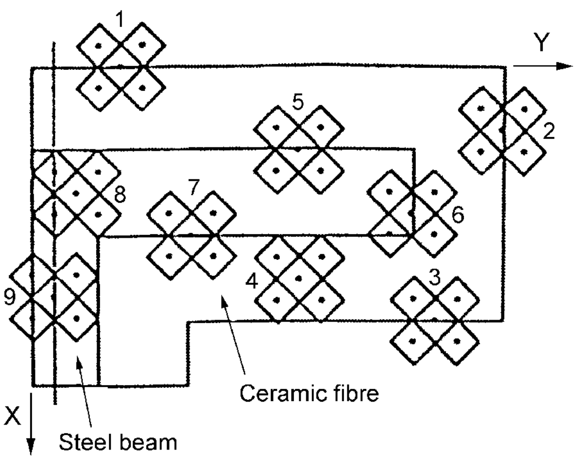


Fig. 3 Boundary conditions for thermal analysis of a contour protected steel column

Case 1: Fire-Insulation Boundary
Case 2: Fire-Insulation Boundary
Case 3: Fire-Insulation Boundary
Case 4: Insulation Region
Case 5: Insulation-Steel Boundary
Case 6: Insulation-Steel Boundary
Case 7: Insulation-Steel Boundary
Case 8: Steel Region
Case 9: Auxiliary Region

2.2.3 Governing heat transfer equations

In Case 1, involving a fire-insulation boundary, heat is absorbed into the column from fire through the insulation. Because of very low thermal conductivity of insulation, heat is transferred at slow rate across the depth of the insulation. The heat is transferred by radiation from the fire to the insulation surface and by conduction in the column. Thus, the governing heat balance expression is:

$$q_r = A_{fs}\sigma\varepsilon_f\ \varepsilon_c\left[(T_f^j+273)^4-(T_{m,n}^j+273)^4\right] \tag{2}$$

where q_r: heat transferred by radiation, (J/m.sec); σ = Stefan-Boltzman Constant (W/m^{2}K^4); ε_f = fire emissivity; ε_c=ceramic emissivity; T= temperature; superscript j = current time step; subscripts m and n = mesh points; A_{fs} = surface area of the element exposed to fire =2(Δh_g)(1.0)

The heat transfer equations employed by the model have been derived largely from the following two-dimensional energy equation:

$$\left[\frac{\partial}{\partial x}K\frac{\partial T}{\partial x}\right]+\left[\frac{\partial}{\partial y}K\frac{\partial T}{\partial y}\right]=\left[\rho C\frac{\partial T}{\partial t}\right] \tag{3}$$

where K = thermal conductivity; ρ = density; C = specific heat

By applying the finite difference technique, and combining all the terms in the sensible heat equation, $q_1-q_2=\partial E/\partial t$, the following expression for the temperature rise in the fire-insulation boundary element was obtained:

$$\begin{aligned} T_{m,n}^{j+1} = T_{m,n}^{j}+\frac{\Delta t}{[\rho_c C_c]_{m,n}^{j}\Delta h_g^2}\cdot &\left\{2\sqrt{2}\Delta h_g\sigma\varepsilon_f\ \varepsilon_c\left[\left(T_f^j+273\right)^4-\left(T_{m,n}^j+273\right)^4\right]\right. \\ &\left.+\left[\left(K_{C_{m+1,n-1}}^j+K_{C_{m,n}}^j\right)\left(T_{m+1,n-1}^j-T_{m,n}^j\right)+\left(K_{C_{m+1,n+1}}^j+K_{C_{m,n}}^j\right)\left(T_{m+1,n+1}^j-T_{m,n}^j\right)\right]\right\} \end{aligned} \tag{4}$$

where Δt = time interval; Δh_g = mesh width; subscript C = coating (insulation).

Following a similar approach, remaining equations were derived for the other eight boundaries cases and the full derivation of these equations is given in Ghani (1998).

2.2.4 Stability criterion

In order to ensure that any error existing in the solution at some time point will not be amplified in subsequent calculations, a stability criterion has to be satisfied which, for a selected value of Δh, limits the maximum of time step Δt. Following the method described by Dusinberre (1961), it can be derived that, for the fire-exposed column, the criterion of stability is most restrictive along the boundary between fire and insulation. It is given by the condition:

$$\Delta t \le \frac{(\Delta h)^2}{\dfrac{4K_{\max}+2\sqrt{2}\cdot\Delta h\cdot h_{\max}}{(\rho_c C_c)_{\min}}} \tag{5}$$

where the maximum value of the coefficient of heat transfer during exposure to the standard fire (h_{max}) is approximately 3×10^6 J/m^{2}K (147 Btu/ft^{2}h°F).

2.2.5 Calculation procedure

With the aid of the above equations and the relevant material properties given in Ghani (1998) and Lie (1992), the temperature distribution in the column and on its surface can be calculated for any time $t = (j+1)\Delta t$, if the temperature distribution at the time $j\Delta t$ is known. Therefore, starting from an initial temperature of 20°C, the temperature history of the column can be calculated by repeated application of the derived equations.

2.3 Strength analysis

The strength of the column during exposure to fire can be calculated by a method based on a load deflection analysis. In this method, an iterative procedure is used to carry out the strength analysis at each time step. In the calculations, the network of elements shown in Fig. 2 was used. Because the strains and stresses in the elements are not symmetrical with respect to the *y*-axis, the calculations were performed for both the network shown and for an identical network at the left of the *x*-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.

2.3.1 Assumptions

To simplify the strength calculations, the following assumptions are made:

1. The curvature of the column varies from pin ends to mid-height according to a straight-line relation;
2. Plane sections remain plane even after bending; The insulation does not contribute to the load carrying capacity;
3. The reduction in column length before exposure to fire, consisting of 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.

2.3.2 Governing strength equations

The load on the column is intended to be concentric. Due to imperfections of the column and loading device, a small eccentricity exists. Therefore in the calculations a very small initial load eccentricity of 0.002 m was assumed.

The curvature of the column is assumed to vary from pin-ended to mid-height according to a straight-line relation, as illustrated in Fig. 4. For such a relation the deflection at mid-height y, in terms of the curvature χ of the column at this height, is given by:

$$y = \chi \cdot \frac{(KL)^2}{12} \tag{6}$$

where KL is the effective length of the column.

Strains: The strain in an element of steel is given as the sum of the axial strain of the column (ε_A), the strain due to bending of the column (ε_B), and the thermal expansion of the steel (ε_T). For any given curvature ($1/\rho$), and thus for any given deflection at mid-height, the axial strain (ε_A) is varied until the internal moment at the mid-section is in equilibrium with the applied moment given by:

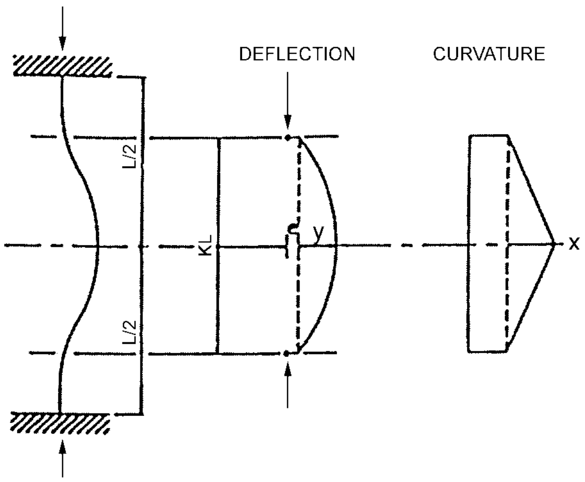


Fig. 4 Load-deflection analysis

load x (deflection + eccentricity)

For a radius of curvature (ρ), and horizontal distance, (Z_s), from steel elements to the x-axis, the bending strain is given by:

$$\varepsilon_b = \frac{Z_s}{\rho} \tag{7}$$

Thermal strain due to expansion is given by $\varepsilon_T = \alpha_s \cdot \Delta T$
where α_s – coefficient of thermal expansion of steel
ΔT – temperature rise in the element

For the steel at the right of the x-axis, the strain $(\varepsilon_s)_R$ is given by:

$$(\varepsilon_S)_R = -(\varepsilon_T)_S + \varepsilon + z_s/\rho \tag{8}$$

For the steel elements at the left of the x-axis the strain $(\varepsilon_s)_L$ is given by:

$$(\varepsilon_S)_L = -(\varepsilon_T)_S + \varepsilon - z_s/\rho \tag{9}$$

Load in Steel: The total load in the steel column is given by:

$$2\left\{\sum_{n=1}^{N} (f_{TR})_n \cdot A_S + \sum_{n=1}^{N} (f_{TL})_n \cdot A_S\right\} \tag{10}$$

where f_{TR} = stress in the steel element at right side of x-axis, f_{TL} = stress in the steel element at left side of x-axis, A_s = area of steel element, N = total number of steel elements and n = nth element.
Moments in Steel: The total moment in the steel column is given by:

$$2\left\{\sum_{n=1}^{N} (f_{TR})_n \cdot A_S \cdot Z_n + \sum_{n=1}^{N} (f_{TL})_n \cdot A_S \cdot (-Z_n)\right\} \tag{11}$$

where Z_n = distance of steel element from x-axis

The full derivation of the strength equations, comprising strains, forces and moments, from the first principles is given in Ghani (1998).

2.4 Calculation procedure

Using Eqs. (6) to (11), the stresses at mid-section in the steel elements can be calculated for any value of the axial strain and curvature. 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 moment, the load that the column carries and the total internal moment at mid-section can be calculated. In this way, a load deflection curve can be calculated for specific times during the exposure to fire. At each time step, the iteration continues until the equilibrium, compatibility and convergence criteria are satisfied. From these curves, the strength of the column, i.e., the maximum load that the column can carry, can be determined for each time. The above procedure is repeated for various times till the failure of the column.

2.5 Computer implementation

The numerical procedure described above was incorporated into a 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 the thermal properties of the column materials. 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, the axial strain is varied until the internal moment at the mid-section is in equilibrium with the applied moment. 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 step. 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. This strength 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.

3. Experimental studies

For the purpose of verification of the computer program developed above, fire resistance tests were conducted on three full-scale wide-flange steel columns with contour protection. The variables included cross-section size, insulation thickness and load intensity.

3.1 Test specimens

The test specimens consisted of three protected steel columns as specified in Table 1. The

Table 1 Summary of test parameters and results

Column	Section dimensions (mm)	Insulation thickness (mm)	Factored strength (Cr) (kN)	Max allowable load (kN)	Applied load (C) (kN)	Load intensity (C/Cr)	Fire resistance	
							Test (min)	Model (min)
FSC1	W250 × 80	25	2550	1908	1750	0.69	89	79
FSC2	W310 × 158	25	5098	1908	3000	0.59	120	120
FSC3	W310 × 158	50	5098	3856	3800	0.75	194	170

columns designated as FSC1, FSC2 and FSC3 were fabricated with structural steel, meeting the requirements of wide-flange steel sections as per the CSA G40.21-M81 (1981). Column FSC1 was made of W250-80 section, while columns FSC2 and FSC3 were made with W315-58 section. The endplates welded to the steel columns were of 533 × 533 mm size and 25 mm thick. All three columns were of 3,810 mm long from endplate to endplate. The specified yield strength of steel was 300 MPa.

To protect the columns against fire, steel sections were covered in ceramic blanket, which follows the contour of the I section. The thickness of the insulation was 25 mm for FSC1 and FSC2, and 38 mm for column FSC3. It is assumed that the insulation does not contribute to the strength of the column. A typical elevation and cross-section of a test column is illustrated in Fig. 1. The location of thermocouples is also shown on the cross section. The full dimensions of the column cross section and other specifics of the columns are given in Table 1.

The steel for the columns was cut to appropriate lengths and then the end plates were welded to the steel at column extremities. The centering and perpendicularity of the end plates were given special attention to ensure a high degree of accuracy. After welding the end plates, twelve holes each with a diameter of 22 mm were drilled in each endplate. The holes were created for facilitating the bolting to the load head at the top and hydraulic jack at the bottom.

Type-K Chromel-alumel thermocouples, 0.91 mm thick, were installed at mid-height in the columns for measuring temperatures at different locations in the cross section (Fig. 1b).

3.2 Test apparatus

The tests were carried out by exposing the columns to heat in a specially-built furnace for testing loaded columns. The furnace consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework. The test furnace was designed to produce conditions, such as temperature, structural loads and heat transfer, to which a member might be exposed during a fire. The furnace has a loading capacity of 1,000 t. Full details on the characteristics and instrumentation of the column furnace are provided in Lie (1980).

3.3 Test conditions and procedure

The columns were installed in the furnace by bolting the endplates to a loading head at the top and a hydraulic jack at the bottom. The conditions of the columns were fixed-fixed for all tests. For each column, the length exposed to fire was approximately 3000 mm. At high temperature, the stiffness of the unheated column ends, which is high in comparison to that of the heated portion of the column, contributes to a reduction in the column effective length. In previous studies (Lie and Irwin 1993), it was found that, for columns tested fixed at the ends, an effective length of 2000 mm represents experimental behaviour.

All columns were tested under concentric loads. The applied loads on the columns varied from 59% and 75% of the factored compressive resistance of the columns (Cr) as determined according to the Canadian Standards Association CSA/CAN-S16.1-M89 (1989). The factored resistance of each column (at ambient temperature), as well as the applied load, are given in Table 1. The effective length factor K, used in the calculation of the factored compressive resistance was that recommended in CSA/CAN-S16.1-M89 (1989) for the given end condition, i.e., 0.65.

The load was applied approximately 45 min before the start of the fire test and was maintained until a condition was reached at which no further increase of the axial deformation could be measured. This was selected as the initial condition for the axial deformation of the column. During the test, the column was exposed to heating controlled in such a way that the average temperature in the furnace followed, as closely as possible, the CAN/ULC-S101 (1989) or ASTM E119-88 (1990) 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.

4. Computer program validation

Data obtained from fire tests was used to validate the computer program described above. This was done by analyzing the behaviour of three columns tested, using the computer program, and comparing the predictions with test results. In the analysis, the temperatures, axial deformations, and strength of the columns were calculated.

4.1 Material properties

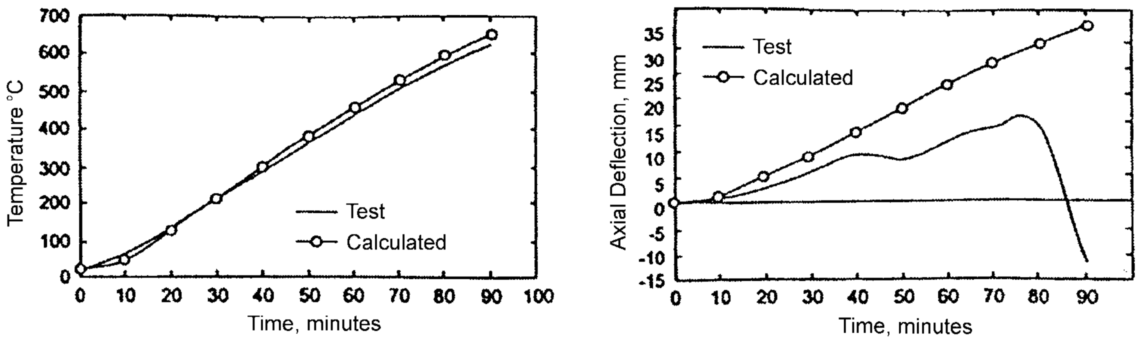
In the analysis, the thermal and mechanical properties of the steel at elevated temperature, given in the ASCE Manual (Lie 1992), were used. These properties are incorporated into the program and the only material property the user has to specify within the data file is the yield strength of steel, which is taken as 300 MPa for the present analysis. For the ceramic fibre the thermal properties provided by the *Unifrax Corporation,* manufacturer of ceramic fibre, were used (Ghani 1998). In the analysis the emissivity of ceramic fiber is taken as 0.6 and that of fire as 0.9.

4.2 Analysis

The three columns were idealized as shown in Fig. 1(a) and the analysis was carried out using the material properties stated above. Analysis was carried out until the failure of the column occurred. The results obtained from the computer program were then used to trace the response of the columns from the initial preloading stage to the collapse. Temperatures, axial deformations and fire resistance predicted by the computer program, were then compared to the test data.

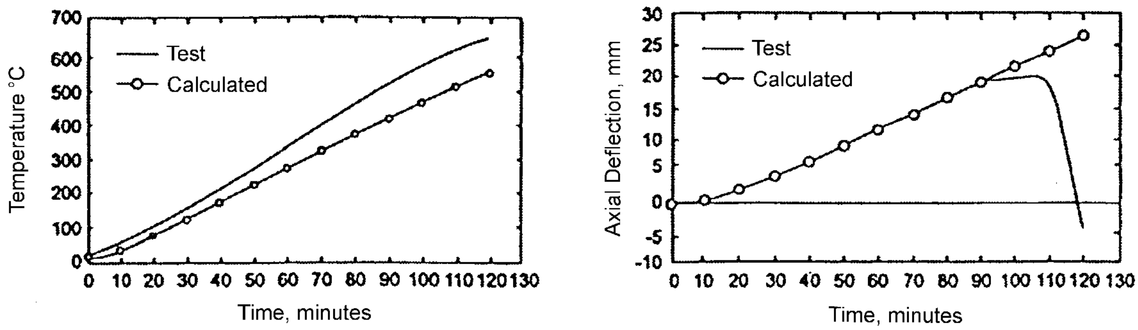
4.3 Results and discussion

In Figs. 5(a), 6(a) and 7(a) the calculated average temperatures are compared with the average



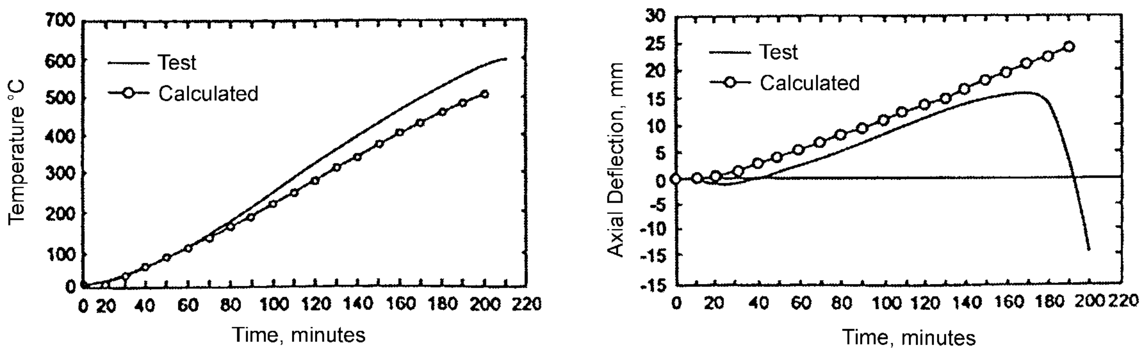
(a) Temperatures (b) Axial Deformations

Fig. 5 Comparison of temperatures and axial deformations for Column FSC1



(a) Temperatures (b) Axial Deformations

Fig. 6 Comparison of temperatures and axial deformations for Column FSC2



(a) Temperatures (b) Axial Deformations

Fig. 7 Comparison of temperatures and axial deformations for Column FSC3

temperatures measured at the external surface of the steel section for Columns FSC1, FSC2 and FSC3 respectively. The temperatures measured initially showed a relatively slow rise up to about 50°C, followed by a period of a relatively faster rate of temperature rise. This temperature behaviour may be the result of the steel section having reached some equilibrium for a particular time step after the initial cold start. While there is good agreement between the calculated and the measured temperatures in the initial stages, the calculated temperatures are slightly lower than the measured ones at later stages of fire for Columns FSC2 and FSC3. These differences, which are within an acceptable range, could be attributed mainly to variation in the thermal properties, especially in higher temperature ranges. Even a 5% difference in thermal properties can result in a temperature variation of more than 10% (Ghani 1998).

In Figs. 5(b), 6(b) and 7(b), the calculated and measured axial deformations of the column during exposure to fire are shown. In the initial stages, there is reasonably good agreement in the trend of deformations between calculated and measured results. However, at later stages, the predictions from the program differ from those of the tests. The variation, which near the failure points are in the order of 5-10 mm, may be regarded as small, if it is taken into account that these are the differences between calculated and measured deformations for a column length of about 3800 mm.

It must be noted that the column deforms axially as a result of several factors namely, thermal expansion, load, bending and creep. These deformations, which contribute to the contraction of columns, increase significantly as the column approaches failure. The effect of creep, which is more pronounced at the later stages of fire exposure, cannot be completely taken into account in the calculations (Stanzak 1973). A part of the creep, however, is implicitly taken into account in the mechanical properties of steel. Further, a difference of 10% between the theoretical and actual coefficients of thermal expansion of steels, for example, will cause a difference of approximately 5 mm in the axial deformations (Ghani 1998).

In Table 1, the calculated and measured fire exposure times are tabulated. The fire resistance of Column FSC2, as obtained from tests and the computer program, is higher than that FSC1. This is mainly due to the massive steel section (higher weight) and lower load intensity on Column FSC2. The higher fire resistance of Column FSC3, as compared to FSC2, is due to thicker insulation.

The results also show that the calculated fire resistance of Columns FSC1 and FSC3 are about 12-14 % less than that of measured fire resistance. For Column FSC2, the calculated and measured fire resistance is in good agreement. The possible reasons for the difference in results are the considerable contraction of the columns that the model can only partly take into account and any approximation in thermal properties of insulation and steel.

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 the columns were crushed. In the tests, the first failure when the columns became unstable (i.e. reached maximum expansion and began to deflect), occurred 5-10 min earlier than the final failure (Ghani 1998). If this is taken as measured fire resistance, then the predicted results are in closer agreement. In total, the predicted fire resistance from the computer program is conservative and within 10-15% of the measured values and this is considered adequate for practical purposes.

5. Design implications

Determining fire resistance by testing full-size columns is costly and expensive. An alternative is to use computer-programmed numerical models to predict the fire resistance. Computer programs, such as the one described above, can be used to trace the response of a insulated wide-flange steel column from the initial pre-loading stage to the collapse of the column, due to fire exposure.

Using these computer programs, a designer can arrive at a desired fire resistance value by varying different parameters, such as length, load, thickness and type of insulation. The use of these programs leads to an optimum design that is not only economical but is also based on rational design principles. Further, it facilitates the integration of the fire resistance design with the structural design.

The required fire resistance ratings in many design codes are prescriptive. Further, the existing fire resistance design guidelines for insulated steel columns are only for particular types of insulation and steel section. The computer program described above can be used for predicting the behaviour of any type of wide-flange steel column with any type of insulation provided that the relevant thermal properties of insulation are available. Further, the computer program can be used to develop simplified design guidelines for providing optimum fire protection in contour protected steel columns. Such studies are currently in progress at the National Research Council of Canada.

6. 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 protected wide-flange steel columns with an accuracy that is adequate for practical purposes. The results indicate that the model is conservative in its predictions.
2. Using the computer programs, the fire resistance of wide-flange steel columns can be evaluated for any value of the significant parameters, such as load, column-section dimensions, thickness and type of insulation, without the necessity of testing.
3. The model can be used for the calculation of the fire resistance of contour protected steel columns other than those investigated in this study, for example other types of steel sections and other types of insulation which were not tested, if the relevant material properties are known.
4. Computer programs, such as the one described in this paper, offer a convenient way to integrate fire resistance design with structural design and lead to an optimum design that is not only economical, but is also based on rational design principles.

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