A Method To Predict the Fire Resistance of Steel Building Columns

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INTRODUCTION

Provision of structural fire resistance is one of the major fire safety requirements in building design. These requirements are intended to ensure building integrity for a certain period of time, to permit evacuation of occupants and access for fire fighters.

Unlike room temperature structural requirements, which are often determined by calculation, fire resistance requirements in building codes are generally specified. There is also a contrast between the methods of evaluation of structural resistance, which for fire has traditionally been determined by testing rather than by calculation.

Most information on structural fire resistance over the past 80 years has been gained by exposing specimens, under standardized test conditions,[1] to a fire defined by a standard temperature-time relation (Fig. 1).

The first standardized column fire tests were carried out in 1917 at Underwriters' Laboratories, Inc. in the U.S.A.[2] Results of those tests indicate that most steel columns collapsed under full design load at average steel temperatures above 1000°F (538°C). In 1947, an alternate standard fire test method for steel columns was adopted.[1] In this method, the column is tested unloaded and the failure criterion is the attainment of an average or "critical steel temperature" of 1000°F (538°C).

Since the adoption of a critical steel temperature as a criterion of failure, procedures initially based purely on thermal analysis were developed in which the fire resistance of a steel column was predicted by calculating the time it took to reach the critical steel temperature. Using these procedures, semi-empirical calculation methods were developed to predict, for a specific insulating material, the fire resistance of steel columns of given size and shape, when exposed to the standard fire.[3,4]

Since the 1940s, knowledge of the factors affecting column behavior has increased to the point where it is now possible to build mathematical models for the calculation of fire resistance in which the influence of these factors is taken into account. Calculation procedures for predicting column failure at elevated temperatures as a function of such factors as load conditions, column dimensions and fire intensity are now under development in various countries.[5–7]

A number of fire test facilities for the verification of the validity of the calculation procedures exist. One such test facility, unique in North America, was constructed a number of years ago at the Institute for Research in Construction of the National Research Council of Canada.[8] This facility, capable of testing loaded columns under fire conditions, is now used extensively for the evaluation of column performance and the validation of methods for the prediction of the fire resistance of columns. In the present paper, results of a study to predict the critical temperature and the fire resistance of steel building columns will be discussed.

The objectives of this study are to:

1. develop a method to examine the effect of temperature rise on load capacity of wide flange steel columns;
2. verify the validity of the method by comparing calculated with experimental results.

PREDICTION OF FIRE RESISTANCE

The prediction of the behavior of a steel column (or any structural assembly) in fire is a two-step process. First, the temperatures reached by the column at any time during the fire must be determined, either by test, by semi-empirical prediction formulas[9] or by thermal analysis.[10] The second step is determining the structural response of the column to elevated

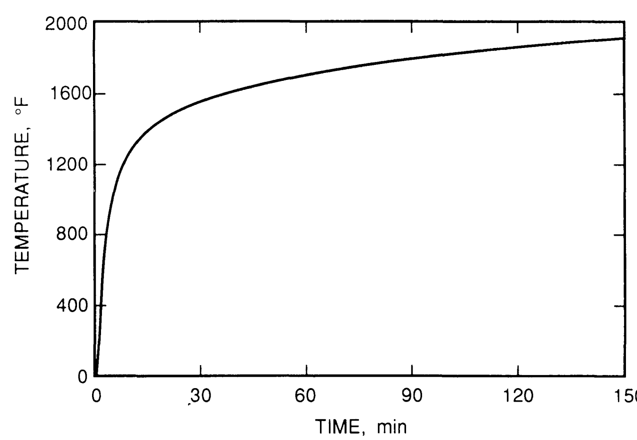

Fig. 1. Standard temperature-time curve.

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temperatures. This response can be determined in two ways, the simplest of which is the use of the critical steel temperature of 1000°F (538°C), signifying collapse, as described earlier. A more realistic assessment of column behavior at elevated temperatures is an analysis of stresses and strains in the loaded column to determine the actual critical steel temperature and hence fire resistance. In this study, the thermal and structural analysis techniques were used.

Temperatures in the Column

The temperatures in the insulation and the steel are calculated by a finite difference method. For this purpose, the insulation and steel are divided into several elements as shown in Fig. 2. For reasons of symmetry, only a one-eighth section of the insulation is considered. Note that the influence of the enclosed air space between the insulation and the steel web is neglected for the purpose of this analysis. Previous tests[10,11] have shown that this is a conservative assumption.

For each element a heat balance is made. By solving the heat transfer equations, the temperature history in the column can be found. The methods of deriving the heat transfer equations and calculating the column temperatures are described in detail in previous papers;[10,11] only information not covered in these papers, such as the properties of the materials used (Appendix A) and dimensions of the insulation and steel will be given.

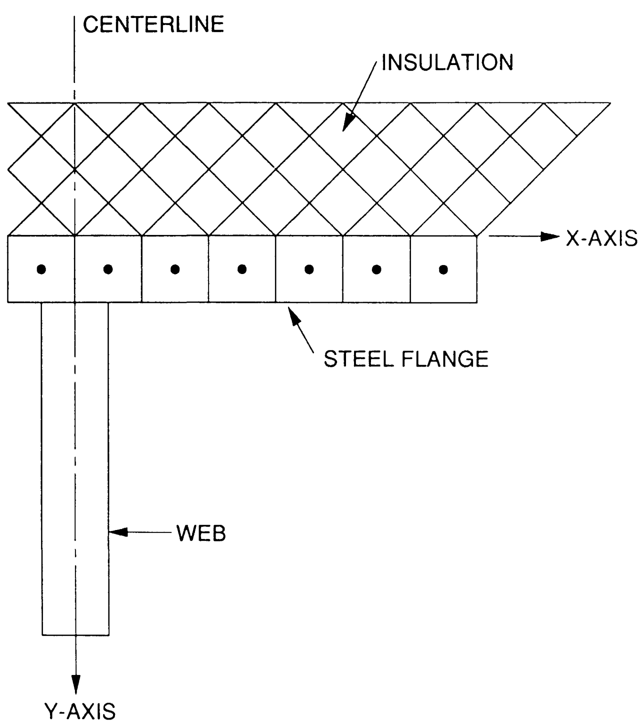

Fig. 2. Arrangement of elements in section of column. (Shown are a one-eighth section of the insulation and a quarter section of the steel.)

Structural Analysis

The fundamental principle behind all methods designed to predict the structural behavior of steel assemblies in fire is the fact that steel gradually loses strength and stiffness at elevated temperatures. These properties, characterized by the yield strength and modulus of elasticity of the steel, decrease as the temperature increases. Data concerning the dependence of these properties on temperature have been reported by several authors.[3,9,12–32] It should be noted that there is a wide spread in elevated temperature data for construction steels in the literature.[33]

In this study, the material properties used are somewhat conservative but are reasonably representative of those reported in most of the literature. The properties, which are given in Appendix A, are based on those reported in Reference 23 but have been modified to include observed increases in stress at high strains.

In room temperature design for structural steel elements, a factor of safety on strength and load is incorporated to take into account variability of material performance and the load to which the member may be subjected. If a steel member is heated in fire, however, it may reach a temperature at which its strength or stiffness decreases to the point that this safety factor is reduced to one and the member fails. This temperature, which is a function of the type of member, its end support conditions and the load applied, is the actual critical temperature of the member. This critical temperature, which for the columns considered in this study may be defined as the average steel temperature at the mid-height section at the time of failure, can be derived by analysis of the structural behavior of the member at elevated temperatures.

Calculation of Strength During Fire

The strength of the column during exposure to fire can be calculated by a method based on a load deflection analysis.[34] In this method, the columns, which are fixed at the ends during the tests, are idealized as pin-ended columns of length KL (Fig. 3). The load on the column is intended to be concentric. Due to imperfections in the column and the loading device, a small eccentricity exists. The loading system and the test columns were made with high precision, however. Therefore, in the calculations a very small initial load eccentricity will be assumed. Previous studies[38] have indicated that a value of 0.008 in. (0.2 mm) is appropriate.

The curvature of the column is assumed to vary from pin-end to mid-height according to a straight line relationship, as illustrated in Fig. 3. For such a relationship the deflection at mid-height Y, in terms of the curvature χ of the column at this height, can be given by

$$Y = \chi \frac{(KL)^2}{12} \tag{1}$$

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 given by the product

load × (deflection + eccentricity)

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 additional assumptions were made:

1. The properties of the steel are those described in Appendix A.
2. Plane sections remain plane.
3. The insulation does not contribute to carrying the load.

In the calculations, the network of elements shown in Fig. 2 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.

The equations used in the calculation of the strength of the column during exposure to fire are given below. The strain in an element of the steel can be given as the sum of the thermal expansion of the steel $(\epsilon_T)_s$, the axial strain of the column ϵ and the strain due to bending of the column x_s/ϱ, where x_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 at the right of the y-axis in Fig. 2, the load induced strain $(\epsilon_s)_R$ is given by

$$(\epsilon_s)_R = -(\epsilon_T)_s + \epsilon + \frac{x_s}{\varrho} \tag{2}$$

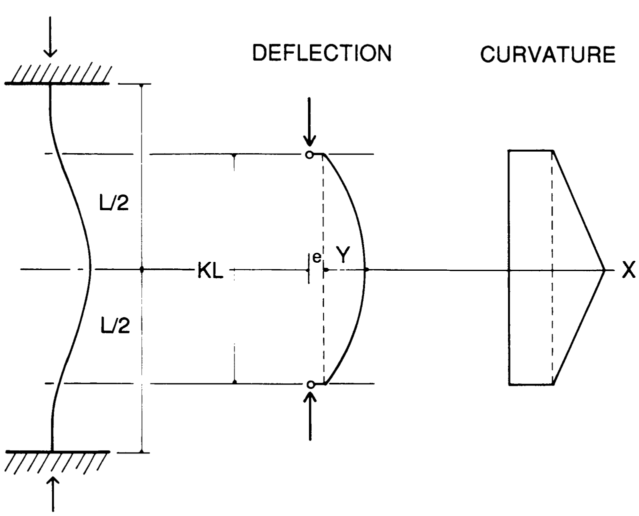

Fig. 3. Load-deflection analysis.

For the steel elements at the left of the y-axis the load induced strain $(\epsilon_s)_L$ is given by

$$(\epsilon_s)_L = -(\epsilon_T)_s + \epsilon - \frac{x_s}{\varrho} \tag{3}$$

The stresses in the elements of the network are calculated using the stress-strain relations for steel given by Eqs. 4–10 in Appendix A. These relations are illustrated in Fig. 4.

With the aid of Eqs. 1–10, the stresses in the steel elements at the mid-height section can be calculated for any value of the axial strain ϵ and curvature $1/\varrho$. From these stresses, the load that each element carried and its contribution to the internal moment at the mid-height 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, i.e., the maximum load that the column can carry, at several consecutive times during the exposure to fire. 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, and the column fails. The time to reach this failure point is the fire resistance of the column; the steel temperature at which it is reached is the critical temperature.

TEST SPECIMENS

The specimens consisted of four protected wide flange steel columns as specified in Table 1. The columns were 150 in. (3810 mm) in total length, including 21-in.×21-in.×1-in. (533-mm×533-mm×25-mm) endplates (Fig. 5). The cross-sectional dimensions of the columns and insulations are shown in Fig. 6 and are specified in Table 2. Column No. 1 was insulated by a box-type protection, whereas Columns Nos. 2–4 were insulated by a sprayed-on protection following the contours of the steel.

The temperature of the steel of the column with box-type protection was measured at three levels, one at mid-height and the other two at respectively 36 in. (914 mm) above and 36 in. (914 mm) below mid-height. The temperature of the

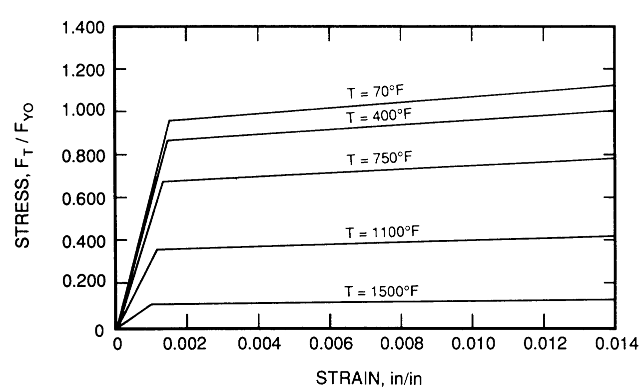

Fig. 4. Stress-strain relations for structural steel at various temperatures.

Table 1. Specifics of Test Specimens					
	Steel		Insulation		
Column No.	Section	Weight lb/ft (kg/m)	Type	Thickness in. (mm)	Density lb/ft^3 (kg/m^3)
1	W150x37, Ref. 35	25 (37)	Vermiculite Board	1.00 (25)	26.5 (425)
2	W10x60, Ref. 36	60 (89)	Sprayed Mineral Fiber	2.20 (55)	12.5 (200)
3	W10x49, Ref. 36	49 (73)	Cementitious Mixture	1.56 (39)	56.2 (900)
4	W10x49, Ref. 36	49 (73)	Sprayed Mineral Fiber	1.20 (30)	12.5 (200)

Table 2. Dimensions of Columns and Insulation					
Column No.	Depth, *d* in. (mm)	Width, *b* in. (mm)	Thickness Flange, *t* in. (mm)	Thickness Web, *w* in. (mm)	Thickness Insulation, *i* in. (mm)
1	6.38 (162)	6.08 (154)	0.455 (11.6)	0.320 (8.1)	1.00 (25)
2	10.25 (260)	10.08 (256)	0.683 (17.3)	0.415 (10.5)	2.20 (56)
3	10.00 (254)	10.00 (254)	0.559 (14.2)	0.340 (8.6)	1.56 (40)
4	10.00 (254)	10.00 (254)	0.559 (14.2)	0.340 (8.6)	1.20 (30)

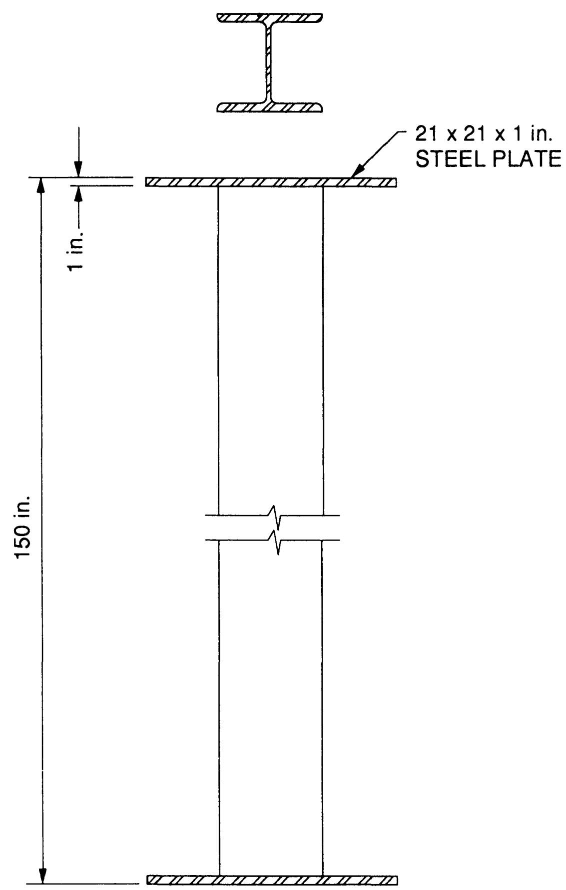

Fig. 5. Elevation and cross section of test columns.

steel of the other columns was measured with thermocouples at four levels. The location of the thermocouples are described in detail in an earlier publication.[37]

TEST APPARATUS

The test was carried out by exposing the column to heat in the column furnace. The test furnace was designed to produce conditions in accordance with the requirements of the standard fire resistance test. 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 2200 kips (1000 t), are described in detail in a previous paper.[8]

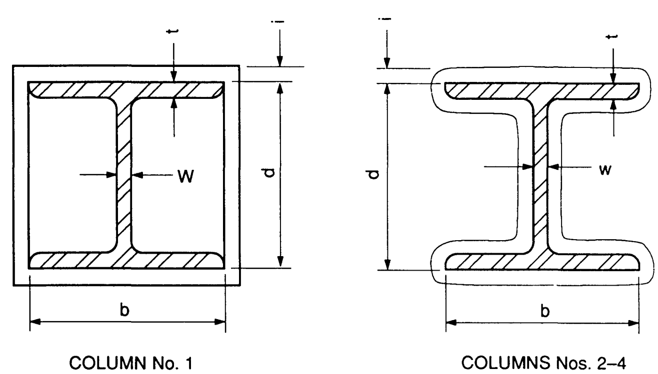

Fig. 6. Cross section of columns and insulation.

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. The end conditions of the columns were partially fixed for all four tests. The length of the columns that was exposed to fire was approximately 120 in. (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[38] it was found that for columns, tested fixed at the ends, an effective length of 80 in. (2000 mm) represents experimental behavior.

The columns were tested under nominally concentric loads. Column No. 1 was subjected to a load of 167 kips (742 kN), which is equal to 0.79 of the factored resistance according to the Standard S16.1.[39] Column No. 2 was subjected to a load of 396 kips (1760 kN) and Columns Nos. 3 and 4 to a load of 320 kips (1424 kN). The loads on the latter columns were equal to 0.64 of the factored resistance of the columns.

The columns were exposed to heating controlled in such a way that the average temperature in the furnace closely followed the ASTM-E119[1] standard temperature-time curve, which can be approximated by:

$$T_f = 20 + 750[1 - \exp(-0.49\sqrt{\tau}] + 22\sqrt{\tau}$$

where:

T_f = temperature in °C
τ = time in minutes

or by:

$$T_f = 68 + 1350[1 - \exp(-0.49\sqrt{\tau}] + 39.6\sqrt{\tau}$$

where:

T_f = temperature in °F

The tests were terminated when the load, which was applied by a hydraulic jack, could no longer be maintained. The hydraulic jack has a maximum speed of 3 in./min (76 mm/min).

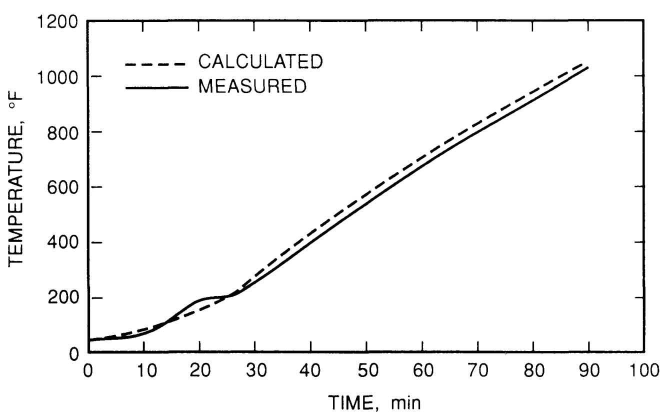

Fig. 7. Calculated and measured average steel temperatures of Column No. 1 as a function of exposure time.

RESULTS

Using the mathematical model described earlier,[10] the temperatures of the steel of Column No. 1 were calculated. In the calculation, the thermal properties of the steel, vermiculite board insulation and the specifics of the column and furnace, given in Appendix A, were used.

Because the thermal properties of the insulation of the other columns were not known, no temperature calculations were made for these columns. The results for Column No. 1 are given in Fig. 7 where the calculated average steel temperatures are compared with those measured. As also shown in previous studies,[8,10,11] it is possible, using the mathematical model, to predict the steel temperatures with high accuracy, if the thermal properties of the insulation are known. In this case the thermal properties of the insulation were determined at the National Fire Laboratory, Institute for Research in Construction, NRCC.[40]

The temperatures are to some extent also dependent on the heat transfer characteristics of the furnace. If the heat transfer is high, however (which is the case for the furnace in which the column was tested), variations in heat transfer have only a small effect on the temperature of the steel.[41,42]

With the calculated steel temperatures as input data, the axial deformation of Column No. 1 during the exposure to fire was calculated. The calculation was carried out using the mathematical model described earlier in the paper and the relevant material properties given in Appendix A. Similar calculations were made for Columns Nos. 2–4. Because the material properties of the insulation of these columns were not known, the measured average steel temperatures of the column, which are given in Appendix B, were used as input data.

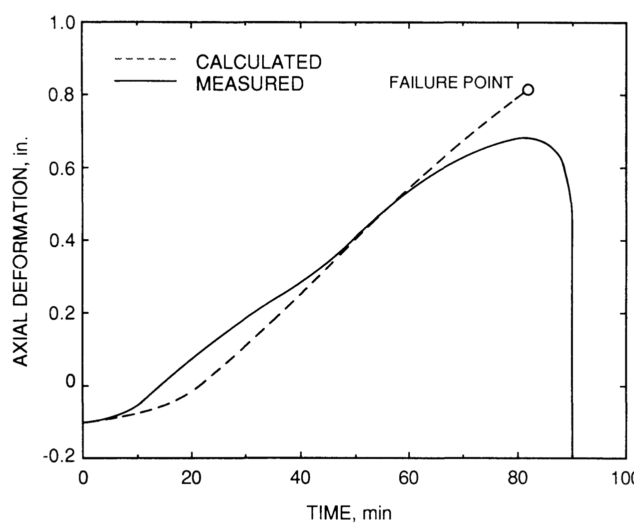

Fig. 8. Calculated and measured axial deformations of Column No. 1 as a function of exposure time.

Column No.	Failure Time: Theoretical	Experimental: Max. axial deformation	Experimental: 0.08 in. (2 mm) axial compression from point of maximum expansion	Experimental: Load can no longer be maintained
1	81	84	89	90
2	137	135	141	150
3	137	145	151	160
4	78	83	88	92

Table 3.
Failure Time for Various Failure Criteria

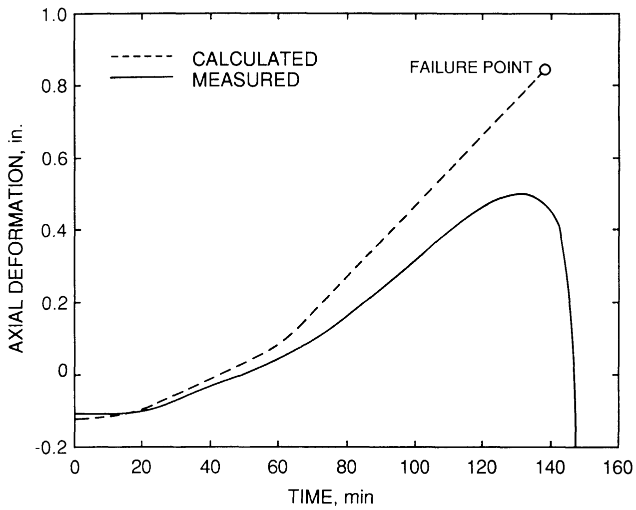

Fig. 9. Calculated and measured axial deformations of Column No. 2 as a function of exposure time.

The calculated axial deformations for the four columns are compared with the measured axial deformations in Figs. 8–11. It can be seen that at the earlier stages of the exposure to fire, there is good agreement between calculated and measured deformations. Closer to the failure point, calculated deformations are somewhat higher than those measured. The differences, on the order of 0.2 in. (5 mm) for a column length of 150 in. (3800 mm), may be regarded as small, however.

Fire Test Failure Criteria

In Table 3, the calculated and measured failure times of the columns are given for various failure criteria. At present, there are no generally accepted fire test failure criteria for columns. It has been customary to consider attainment of an arbitrary lateral deflection of the column during a fire test as failure. This deflection varies from country to country but lies in the range of 2–6 in. (50–150 mm), corresponding to an axial compression of about 0.08–0.8 in. (2–15 mm) from

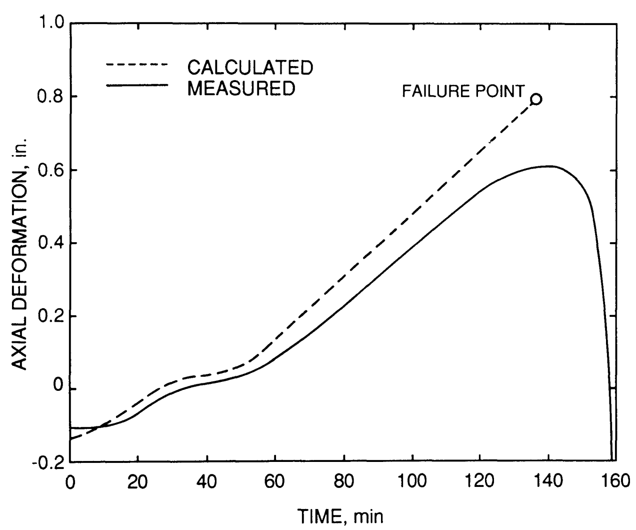

Fig. 10. Calculated and measured axial deformations of Column No. 3 as a function of exposure time.

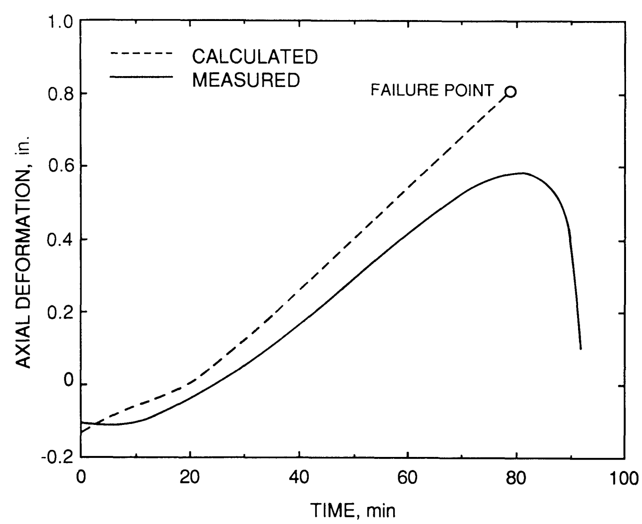

Fig. 11. Calculated and measured axial deformations of Column No. 4 as a function of exposure time.

the point of maximum expansion for a column length of 120 in. (3000 mm).

At the Institute for Research in Construction, the column is considered to have failed when the hydraulic loading system can no longer apply the contemplated load. This criterion, although well defined because of the fixed maximum ram speed, is dependent on the capabilities of the loading system. Since the maximum ram speed is high (3 in./min, 76 mm/min) the loading system will be able to apply the load until very large deformations occur, as can be seen in Figs. 8–11. Therefore, the use of this criterion might overestimate failure times. For steel columns, the point at which the column reaches maximum expansion appears to be the best criterion.

CONCLUSION

The mathematical model employed in this study is capable of predicting the critical steel temperature and fire resistance of protected steel columns within the range of those tested with an accuracy that is adequate for practical purposes.

If the thermal properties of the insulation are not known, the fire resistance of the column can be derived by calculating, for a given load, the critical steel temperature of the column, using the mathematical model, and determining by testing of an unloaded column the time it takes for the steel to reach the critical temperature. If the thermal properties of the protecting material are known, the temperature course of the steel can be calculated using existing validated mathematical models for the determination of the steel temperature of fire-exposed protected steel columns, and the fire resistance of the column can be determined entirely by calculation.

APPENDIX A

STEEL PROPERTIES

The yield strength and modulus of elasticity of steel at elevated temperatures are based on the formulas for these properties given in Ref. 23. The stress-strain relations for stresses and strains up to the proportional limit are given by Eqs. 4, 6, 9 and 10. For greater stresses and strains, it was conservatively assumed, based on information in the literature, that the stress-strain relations are straight lines with a positive slope above the proportional limit. These stress-strain relations are given by the lines through the 0.002 offset points and the points defined by a strain equal to 0.01 offset with regard to the proportional limit and a stress equal to 1.1 of the yield strength. The stress-strain relations for various temperatures are illustrated in Fig. 4.

Stress-Strain Relations

for $\epsilon_s \leq \epsilon_p$

$$f_T = E_T \epsilon_s \tag{4}$$

for $\epsilon_s > \epsilon_p$

$$f_T = 12.5 f_{YT} \epsilon_s + 0.975 f_{YT} - 12.5 \frac{(f_{YT}{}^2)}{E_T} \tag{5}$$

where the proportional limit

$$\epsilon_p = \frac{0.975 f_{YT} - 12.5 \dfrac{(f_{YT})^2}{E_T}}{E_T - 12.5 f_{YT}} \tag{6}$$

the yield strength

for $0 < T \leq 600°C$

$$f_{YT} = \left[1.0 + \frac{T}{900 \ln\left(\dfrac{T}{1750}\right)} \right] f_{YO} \tag{7}$$

for $600 < T < 1000°C$

$$f_{YT} = \frac{340 - 0.34T}{-240 + T} f_{YO} \tag{8}$$

and the modulus of elasticity

for $0 < T \leq 600°C$

$$E_T = \left[1.0 + \frac{T}{2000 \ln\left(\dfrac{T}{1100}\right)} \right] E_O \tag{9}$$

for $600 < T < 1000°C$

$$E_T = \frac{690 - 0.69T}{T - 53.5} E_O \tag{10}$$

In the calculations, the following values for f_{YO} and E_O have been used:

$f_{YO} = 300$ MPa
$E_O = 200{,}000$ MPa

Thermal Capacity

for $0°C \leq T \leq 650°C$

$$\varrho_s c_s = (0.004T + 3.3) \times 10^6 \text{ J m}^{-3} °C^{-1}$$

for $650°C < T \leq 725°C$

$$\varrho_s c_s = (0.068T - 38.3) \times 10^6 \text{ J m}^{-3} °C^{-1}$$

for $725°C < T \leq 800°C$

$$\varrho_s c_s = (-0.086T + 73.35) \times 10^6 \text{ J m}^{-3} °C^{-1}$$

for $T > 800°C$

$$\varrho_s c_s = 4.55 \times 10^6 \text{ J m}^{-3} °C^{-1}$$

Thermal Conductivity

for $0°C \leq T \leq 900°C$

$$k_s = -0.022T + 48 \text{ W m}^{-1} °C^{-1}$$

for $T > 900°C$

$$k_s = 28.2 \text{ W m}^{-1} °C^{-1}$$

Coefficient of Thermal Expansion

for $T < 1000°C$

$$\alpha_s = (0.004T + 12) \times 10^{-6} °C^{-1}$$

for $T \geq 1000°C$

$$\alpha_s = 16 \times 10^{-6} °C^{-1}$$

VERMICULITE BOARD PROPERTIES

Thermal Capacity

for $0°C \leq T \leq 50°C$

$$\varrho_i c_i = 4200T + 425000 \text{ J m}^{-3} °C^{-1}$$

for $50°C < T \leq 100°C$

$$\varrho_i c_i = -4250T + 847500 \text{ J m}^{-3} °C^{-1}$$

for $100°C < T \leq 1000°C$

$$\varrho_i c_i = 335T + 389000 \text{ J m}^{-3} °C^{-1}$$

for $T > 1000°C$

$$\varrho_i c_i = 724000 \text{ J m}^{-3} °C^{-1}$$

Thermal Conductivity

for $0°C \leq T \leq 1000°C$

$$k = 0.000192T + 0.135 \text{ W m}^{-1} °C^{-1}$$

for $T > 1000°C$

$$k = 0.327 \text{ W m}^{-1} °C^{-1}$$

WATER PROPERTIES

Thermal Capacity

$$\varrho_w c_w = 4.2 \times 10^6 \text{ J m}^{-3} °C^{-1}$$

Heat of Vaporization

$$\lambda_w = 2.3 \times 10^6 \text{ J kg}^{-1}$$

SPECIFICS OF COLUMNS AND FURNACE

α = fraction of heat transferred by conduction from insulation to steel (Col. No. 1): 0.5

ϵ_{fi} = combined emissivity of column furnace fire and insulation: 0.6

ϵ_s = emissivity of steel: 0.8

KL = effective length of columns: 2000 mm for fire resistance calculations

ϕ = concentration of moisture in insulation by volume: 0.05

These values were determined based on previous test and modeling results.

APPENDIX B

EQUATIONS FOR THE AVERAGE TEMPERATURE OF THE COLUMNS NOS. 2–4 AT MIDHEIGHT DURING FIRE TESTS (FOR USE AS INPUT DATA)

Column No. 2

for $0 \leq \tau \leq 16.7$

$$T_{as} = 5.89 + 0.57\tau$$

for $16.7 < \tau \leq 59.27$

$$T_{as} = -24.21 + 2.41\tau$$

for $\tau > 59.27$

$$T_{as} = -186.49 + 5.15\tau$$

Column No. 3

for $0 \leq \tau \leq 9.85$

$$T_{as} = 16.48 + 0.55\tau$$

for $9.85 \leq \tau \leq 29.63$

$$T_{as} = -15.14 + 3.76\tau$$

for $29.63 \leq \tau \leq 49.17$

$$T_{as} = 77.85 + 0.62\tau$$

for $\tau > 49.17$

$$T_{as} = -117.65 + 4.60\tau$$

Column No. 4

for $0 \leq \tau \leq 19.97$

$$T_{as} = 5.94 + 3.66\tau$$

for $\tau > 19.97$

$$T_{as} = -68.69 + 7.39\tau$$

NOMENCLATURE

Notations

c specific heat (J kg^{-1} °C^{-1})
E modulus of elasticity (MPa)
f_T stress of steel at temperature T (MPa)
f_{YT} yield strength of steel at temperature T (MPa)
f_{YO} yield strength of steel at room temperature (MPa)
k thermal conductivity (W m^{-1} °C^{-1})
K effective length factor
L unsupported length of column (m)
T temperature (°C)
x coordinate (m)
y coordinate (m)
Y lateral deflection of column at midheight (m)
z coordinate (m)

Greek Letters

α coefficient of thermal expansion (°C^{-1}), fraction of heat transferred by conduction from insulation to steel
ϵ emissivity, strain (m m^{-1})
λ heat vaporization (J kg^{-1})
ϱ density (kg m^{-3}), radius of curvature (m)
τ time (min)
ϕ concentration of moisture (fraction of volume)
χ curvature of column at midheight (m^{-1})

Subscripts

a average
f of the fire
i of insulation
o at room temperature
p pertaining to proportional stress-strain relation
s of steel
T pertaining to temperature
w of water

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