Structural Steel and Fire—More Realistic Analysis

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"STRUCTURAL FIRE RESISTANCE" is the name given to the property of a building's structural elements that enables it to withstand the effects of fire. For common structural elements such as columns, beams, floors, and walls, this property is quantified by the provisions of a standard fire test.[18] This test establishes the structural fire endurance of load-bearing building components in terms of the length of time they can resist collapse.

The most significant feature of the fire test standard is that it defines a temperature-time function for the fire atmosphere to which the structure is exposed. Consequently, a standard fire test provides a yardstick by which the performance of one element may be compared with that of another under standardized conditions; it does not necessarily provide a measure of performance of an element under actual loading and fire conditions.

A great deal of expense and effort has been devoted to developing fire test data for different materials and structural elements. Most recent research in structural fire resistance has involved methods of extending (or generalizing) fire test data or calculation of fire resistance by numerical or semi-empirical methods. Thus, a good fund of knowledge for dealing with problems concerning the resistance of structural elements to standard fire conditions has become available.

The reaction of structural elements to elevated temperatures, however, depends on the severity of the fire conditions to which they are exposed. This can vary in a wide range, depending on building occupancy and design features. It is desirable, therefore, to find means by which an element's performance in a standard fire test can be used to assess performance under fire conditions more representative of those that might be expected for a particular building design.

Following a brief review of present technology for calculating the fire resistance of steel building elements, means for utilizing the results under other conditions of fire severity in order to reduce the number of standard, destructive fire tests required for structural elements and to eliminate the need for *in situ* fire tests that would be costly as well as impractical, are presented.

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CRITICAL TEMPERATURE CONCEPT

Calculation of the deformation and failure of a structural element under fire (standard or other) conditions presents an engineering problem of enormous complexity. Use of numerical methods and high speed digital computers makes such analysis feasible, but usually impractical because of the large input required. For steel structures an important simplification is possible with the use of the "critical temperature concept" which reduces the analysis to a thermal problem only.

The "critical temperature concept" rests on two major assumptions:

1. The steel core of a (protected) structural element provides the main strength of the structural unit.
2. Fire resistance is concerned only with the time of collapse of a structural element, not with its deformation history prior to collapse or its possible reusability after a fire.

The critical temperature of a structural element is defined as the cross-sectional average temperature at which the element can no longer perform its load-carrying function; it is the cross-sectional average temperature at which the factor of safety incorporated in the structural design becomes unity. The critical temperature may depend on the type of structural member (beam, column, truss, etc.), length of fire exposure, and the strength, elastic or creep properties of the steel.

Table 1. Critical Temperatures (°F) of Statically Determinate and Indeterminate I-Beams Made of Various Steels (For a Safety Factor of 1.7)

Supporting conditions	Type of Steel					
	St. 37 (from Refs. 2,3)	St. 37 (from Refs. 16,22)	CSA G40.21 44W (from Ref. 6)	ASTM A36 (from Ref. 6)	ASTM A36 Upper limit of existing data	ASTM A36 Lower limit of existing data
Statically determinate	735	890	1075	915	1110	880
Statically indeterminate	890	1020	1220	1095	1220	1075

For simple, theoretically unrestrained elements, critical temperatures can be calculated fairly readily. For example, if an axially loaded long column is designed with a safety factor of 1.92 against elastic buckling,[21] the critical temperature T_{cr} can be derived as follows:

$$\sigma_{cr} = \frac{\pi^2 E_T}{\lambda^2} \tag{1a}$$

From Ref. 21: $$\sigma_a = \frac{\pi^2 E_o}{1.92\lambda^2} \tag{1b}$$

From Ref. 12: $$E_T = E_o(1 - 2.04\theta^2) \tag{2}$$

where:

θ = $T - 68/1800$
T = temperature, °F
E_T = modulus of elasticity at temperature T, ksi
E_o = modulus of elasticity at 68°F, ksi
λ = slenderness ratio

By combining Eqs. (1) and (2):

$$\theta = 0.486$$

$$T_{cr} = 943°\text{F}$$

Recognizing that failure takes place some time after initiation of buckling (just calculated), the 1000°F limit specified in ASTM E119[18] under "alternate test of protection for structural steel columns" seems reasonable and will be used as the critical temperature for steel columns in this paper.[15]

For the critical temperature of joists or trusses and beams, Harmathy[4] derived the following two equations:

$$T_{cr} = \frac{\Delta H/R}{\ln[37.5(s/S)\,Z]} \quad \text{(joists, trusses)} \tag{3}$$

$$T_{cr} = \frac{\Delta H/R}{\ln[(150/\pi^2)\,Z]} \quad \text{(beams)} \tag{4}$$

where:

ΔH = activation energy of creep, Btu/lb mole
R = gas constant, Btu/lb mole deg R
S = span of joist or truss, in.
s = length of key member or panel, in.
Z = Zener-Hollomon parameter, of a key member in the case of joists or trusses and of the center in the case of beams, hr^{-1}

For simply supported, unrestrained joists or trusses and beams made of ASTM A36 steel, and designed for an allowable stress of 20,000 psi, these critical temperature equations become:

$$T_{cr} = \frac{70{,}000}{46.52 - 2.3\log(S/s) - 4.23(I_d/I)} - 460 \tag{5}$$

(joists, trusses)

$$T_{cr} = \frac{70{,}000}{45.62 - 4.23(I_d/I)} - 460 \quad \text{(beams)} \tag{6}$$

where:

I_d = moment of inertia of deck, in.^{4}
I = moment of inertia of steel element, in.^{4}

Creep properties of two commonly used structural steels have been described.[6] It should be noted that the critical temperature of statically indeterminate members is considerably higher than that of statically determinate elements, as is illustrated by Table 1.[9] The critical temperature of these elements varies much more widely than does that for steel columns.[4,7,9,17] Normally, the temperature value of 1100°F that is used in ASTM E119 for structural steels is acceptable.

FORMULAS FOR STANDARD FIRE ENDURANCE

The simplification introduced by the critical temperature concept makes it possible to develop rather simple empirical and semi-empirical formulas for the fire endurance of steel columns. These may be listed as follows:

Unprotected Steel Columns[19]

$$\tau_s = 10.3(W/D_s)^{0.7} \quad \text{when } W/D_s < 10 \tag{6a}$$

$$\tau_s = 8.3(W/D_s)^{0.8} \quad \text{when } W/D_s \geq 10 \tag{6b}$$

where:

τ_s = fire resistance, minutes
W = weight of steel per ft length, lbs/ft
D_s = developed heated perimeter of steel, in.

Figure 1 shows experimental data for solid square steel columns.

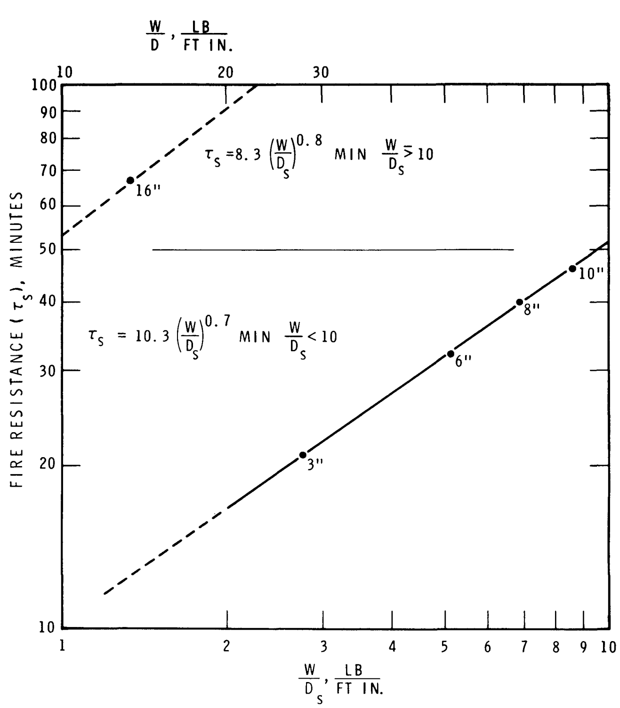

Fig. 1. Fire resistance of unprotected steel columns

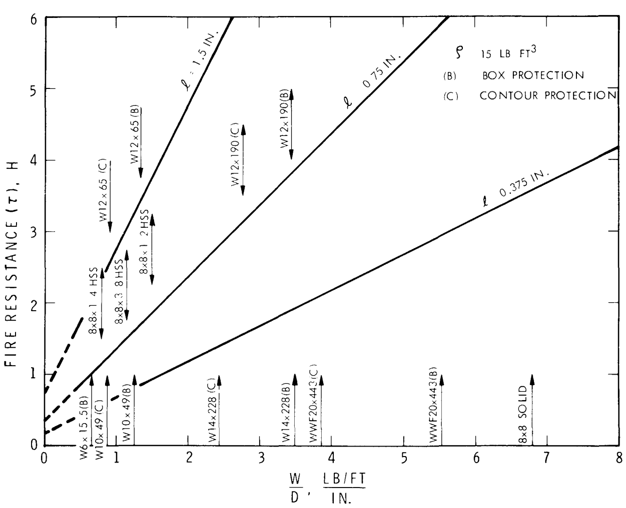

Fig. 2. Fire resistance of steel columns protected with sprayed fiber

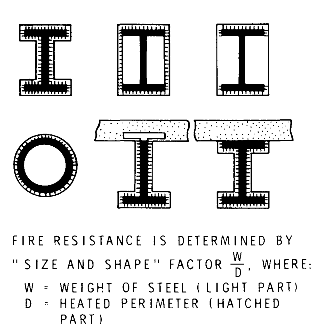

Fig. 3. "Size and shape" factor W/D for steel

Steel Columns with Light Protection[15]

For protective materials that are relatively inert (for example, sprayed fiber, see Fig. 2):

$$\tau = \left(20\frac{W}{D\rho} + 0.5\right) l\rho \leq 50 \tag{7a}$$

For materials containing cement paste or gypsum:

$$\tau = \left(20\frac{W}{D\rho} + 1.2\right) l\rho \leq 50 \text{ lb/ft}^3 \tag{7b}$$

where:

τ = fire resistance, hrs
W = weight of steel per ft length, lbs/ft
D = developed inner perimeter of protection, in.
ρ = density of insulation, lbs/ft^3
l = thickness of protection, in.

Concrete-Protected Steel Columns[14]

$$\tau = \tau_s + 15.2\frac{l^{1.6}}{k^{0.2}}\left[1 + 1.5\left(\frac{C}{\rho_c c l p}\frac{L}{L+l}\right)^{0.8}\right] \tag{8}$$

where:

τ = fire resistance, hrs
τ_s = $0.03(\rho_s A_s/p_s)^{0.7}$, hrs (9)
p = $4L$
p_s = perimeter of steel cross section, ft
L = one-fourth of inner perimeter of concrete protection, ft
l = thickness of concrete cover, ft
k = thermal conductivity of concrete at room temperature, Btu/ft hr °F
C = heat capacity of steel core of column per unit height, Btu/ft °F
c = specific heat of concrete, Btu/lb °F
A_s = area of steel per ft height, ft^2/ft
ρ_c = density of concrete, lbs/ft^3
ρ_s = density of steel, lbs/ft^3

These formulas may also be applied in calculating the fire endurance of horizontally placed steel structural members. The results are on the conservative side in that they have a higher critical temperature and, usually, a deck or other superstructure capable of absorbing heat. The equations indicate that fire endurance depends mainly on the "size and shape" factor W/D and, where present, on the thickness of protection, l. Several examples of how the developed perimeter, D, may be evaluated are shown in Fig. 3.

As may be seen in Figs. 1 and 2, fire resistance always increases with the factor W/D. This can be useful information in interpreting fire test data where the section used in the test is specified as a "minimum size" for application. Figure 2 illustrates, for example, that a W12×190 section has a larger W/D than a W14×228 section, which is often fire tested as a "large column." It is the ratio of steel weight, W, to the area through which heat is transferred to the steel that is significant, not the nominal size and weight of the column.

As design of buildings against fire exposure becomes more directly based on factors that realistically assess fire behavior, it will be desirable to retain the use of these relatively simple methods.

FIRE LOAD CONCEPT

Data derived from standard fire tests are related to building design by use of the "fire load concept." Fire load means the weight of wood with heat content equivalent to that of the combustible material per unit floor area of a building or fire compartment. North American building codes usually specify the fire resistance required of structural elements in a building on the basis of occupancy, fire load, height, and area.

Use of the fire load concept in building design has been traditional and was experimentally justified by Ingberg,[8] who conducted fire tests in building structures that were relatively poorly ventilated. He established the long-used relations between standard fire exposure and fire severity resulting from a given fire load: 10 lb/ft^2—1 hr, 20 lb/ft^2—2 hrs, etc. Subsequent research, however, has shown that the fire load concept is untenable.[5]

FIRE SEVERITY

Fire severity can vary widely from that prescribed by standard test conditions. Several studies are now available indicating the significant parameters that determine fire severity and how to assess their influence. It is possible to estimate the temperature course of fire in compartments under various conditions by carrying out a heat balance for the compartment. Usually, part of the heat produced by combustion of the compartment contents (fire load) is absorbed by the surrounding structure as well as by the gases in the enclosure. Other heat losses result from radiation of the windows, outflow of hot gases, outflow of unburned gases (which burn outside the compartment), and outflow of suspended, unburned particles. To evaluate the temperature course, it is necessary to know the heat produced and the heat lost at all times during a fire.

Some of the factors that determine heat production and heat loss, for example, material properties, compartment and window dimensions, emissivity of flames and exposed materials, can be determined with reasonable accuracy. Others, such as heat loss due to gases that burn outside the compartment, emission of unburned particles through windows, and temperature differences in the compartment, are known only very approximately. In addition, the magnitude of several parameters cannot be predicted at all. These vary with the time of occurrence of a fire and are thus a matter of chance. They include the amount, surface area,

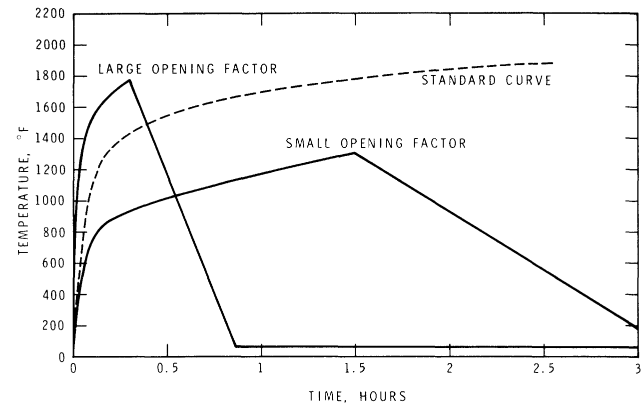

Fig. 4. Influence of opening factor on fire severity

and arrangement of the combustible contents, wind velocity, and outside temperature. Consequently, it is impossible to predict, at the design stage of a building, the temperatures to which building components might be exposed by fire during their service life.

It becomes necessary to resort to a probabilistic approach in establishing temperature-time curves suitable as a basis for specifying the building fire resistance. Curves have been chosen such that the probability of exceeding their heating effect during the lifetime of a building is very low. Such curves are generated by ventilation-controlled fires, for which formulas characterizing the temperature course are available.[11]

Two major parameters determine the severity of ventilation-controlled fires. The first is related to the dimensions of the window opening and is generally known as the "opening factor"; the second is the fire load.* Figure 4 illustrates how the opening factor affects the fire temperature course (with the same available fire load): a large opening factor results in a fire with relatively high temperatures but short duration; a small opening factor produces a fire with relatively low temperatures and long duration. This clearly illustrates why the fire load concept, as previously discussed, is insufficient for rational fire resistance design of building structures.

Figure 5 shows the temperature course of a fire in a compartment with an intermediate opening factor, illustrating the effect of fire load on fire severity. The opening factor affects the intensity and duration of the fire, but the fire load affects only duration; that is, the higher the fire load, the longer the time that elapses before the fire starts to decay. The effect of this more realistic fire representation on the behavior of structural steel elements can now be examined.

* *Here and in subsequent sections, fire load means the weight of wood with heat content equivalent to that of the combustible material per unit area of the bounding surfaces of the fire compartment.*

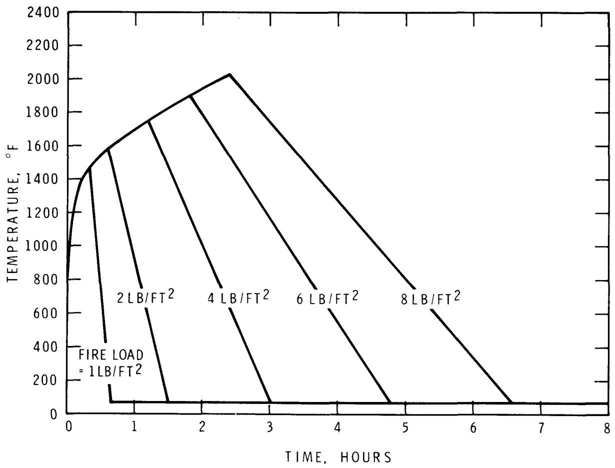

Fig. 5. Characteristic temperature curves for various fire loads

STEEL TEMPERATURES

Temperature rise in protected steel columns exposed to fires of varying severity was determined by both calculation and experiment. Calculations were based on a numerical procedure.[13] Columns were W12X190 sections protected by 1-in. pressed vermiculite board (density 27 lbs/ft^3, specific heat 0.3 Btu/lb °F, thermal conductivity 0.108 Btu/ft hr °F) with a standard fire resistance of 3.15 hrs according to Eq. (7a).

Results are shown in Fig. 6, with standard experimental data taken from a previous study[20] superimposed for comparison. Curves 1 and 2 are the steel temperatures measured during exposure at two levels on the column. Note that at about 1½ hrs, when the fire had started to decay, the temperature of the steel continued to rise until it reached the critical temperature at about 3 hrs. Curve 3 was obtained from a numerical analysis of the experimental situation. It accurately approximates the measured data, illustrating the excellent results that can now be obtained by calculation.

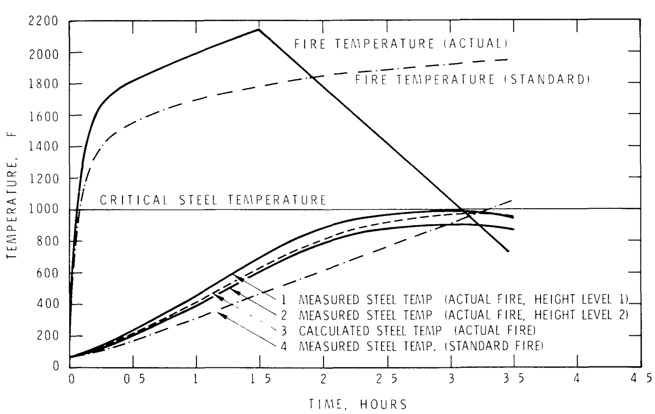

Fig. 6. Calculated and experimental temperature rise of protected steel column

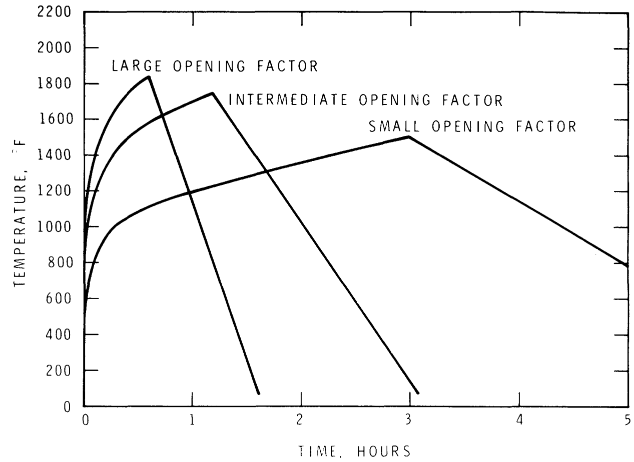

Fig. 7. Characteristic temperature curves for various opening factors (fire load 4.1 lbs/ft^2)

Having demonstrated the reliability of the calculation procedure, analyses were carried out to determine the temperature rise in a W10X49 steel column for a fire load of 4.1 lbs/ft^2 (20 kg/m^2) for three opening factors*: large (F = 0.1), intermediate (F = 0.05), and low (F = 0.02), as shown in Figs. 7 and 8.

With a large opening factor, structural failure cannot occur; with an intermediate opening factor, it occurs at 1½ hr; and with a low opening factor, at 2⅓ hr. It is clear that the temperature rise in steel resulting from fires of different severity differs markedly from that anticipated by the fire load concept.

* *Opening factor is defined as:*

$$F = \frac{A\sqrt{H}}{A_T}$$

where A = *window area*
H = *window height*
A_T = *area of walls and floors*

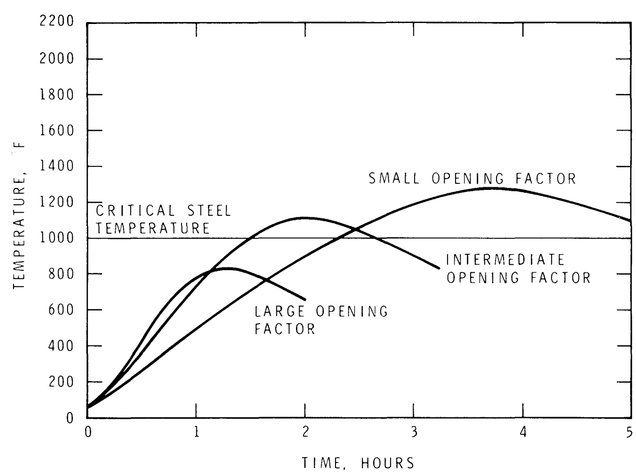

Fig. 8. Effect of opening factor on steel temperature (fire load 4.1 lbs/ft)

APPLICATION IN DESIGN

It has been shown that the opening factor has a significant effect on the steel temperature of protected columns: the higher the opening factor, the lower the steel temperature. An attempt will now be made to interpret this finding with respect to standard fire endurance and application in building design. Because of the infinite possibilities involved, however, any approach must be undertaken on a probabilistic basis.

For tall or otherwise large buildings, the probability of a structural failure as a result of burn-out of the contents should be kept very low, from both the point of view of occupant safety and preservation of property. The latter concept was first investigated by Lie,[10] who established design fire load factors on the basis of loss expectation. To determine the probability of structural failure, it is necessary to establish the fire load that will cause failure of structural elements, i.e., the fire load that will raise the steel temperature to the critical level.

Such critical fire loads have been calculated by methods described elsewhere.[11,13] The corresponding fire and steel temperature courses for the W10X49 column are shown in Figs. 9, 10 and 11. Critical fire loads are as in Table 2.

Each fire load has a certain probability of occurrence and a certain probability of being exceeded. A typical frequency of fire loads for offices, derived from preliminary surveys by the U. S. National Bureau of Standards[1] is shown in Fig. 12. (These data are used by way of illustration and should not be considered applicable to all office buildings.) Figure 12 indicates that per 10,000 there are about 2000 compartments with a fire load of 1 lb/ft^2; 700 compartments with a fire load of 2 lbs/ft^2; 30 compartments with a fire load of 3 lbs/ft^2; two compartments with a load of 4 lbs/ft^2; and a very small number with a higher fire load. According to the distribution shown (mean 1 lb/ft^2, standard deviation 0.7 lb/ft^2), the probability of exceeding the critical fire loads (previously calculated) are shown in Table 3.

TABLE 2

Opening Factor	Critical Fire Load
Large (0.1)	5.3 lbs/ft^2
Intermediate (0.05)	3.5 lbs/ft^2
Small (0.02)	2.7 lbs/ft^2

TABLE 3

Opening Factor	Critical Fire Load	Probability[a]
Large (0.1)	5.3 lbs/ft^2	less than one per million
Intermediate (0.05)	3.5 lbs/ft^2	two per ten thousand
Small (0.02)	2.7 lbs/ft^2	eighty per ten thousand

[a] Probability of exceeding the critical fire load is same as the probability of collapse.

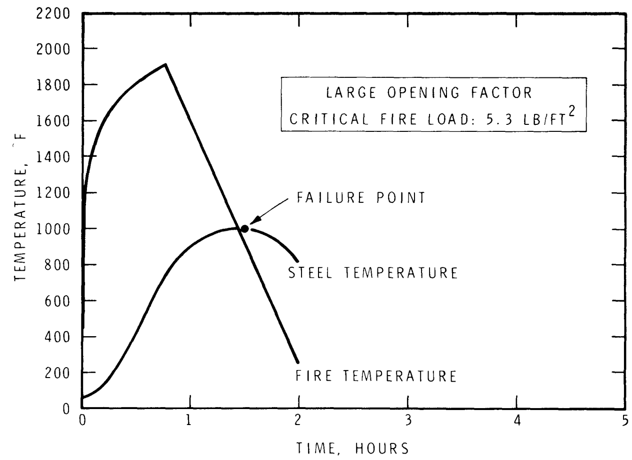

Fig. 9. Critical condition for large opening factor ($F = 0.1$)

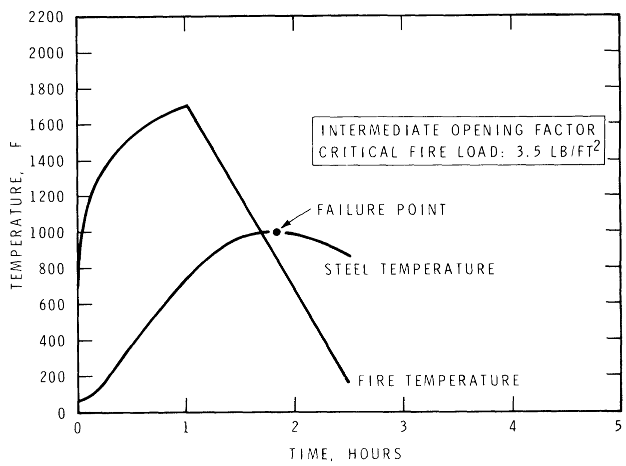

Fig. 10. Critical condition for intermediate opening factor ($F = 0.05$)

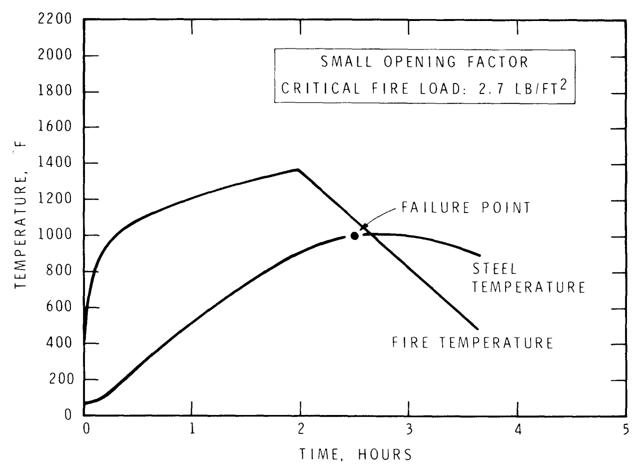

Fig. 11. Critical condition for small opening factor ($F = 0.02$)

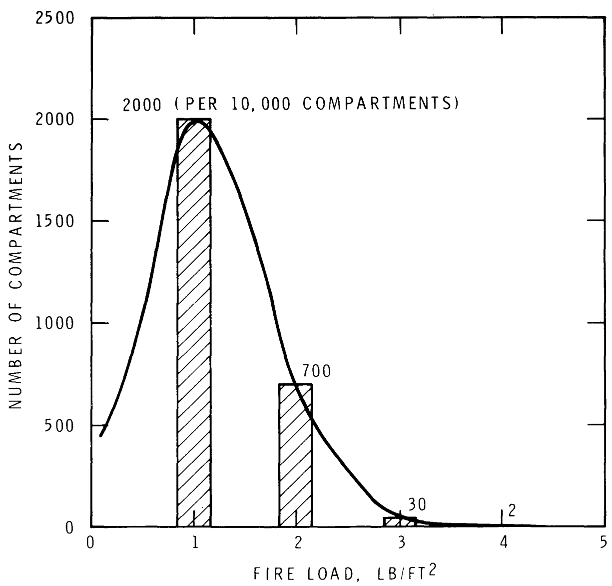

Fig. 12. Typical frequency distribution of fire load in offices

It is now possible to assess the value of protected columns with a standard fire endurance of 1½ hrs. With a small opening factor, collapse will occur in 80 of 10,000 fires; with an intermediate opening factor, in two of 10,000 fires; and with a large opening factor the possibility of collapse is negligible. Thus, if the required probability of failure is known, the critical fire load can be determined for each situation. From this, the required fire resistance, and protection (if needed), can be derived for each member or group of similar members.

CONCLUSION

Analysis based on a realistic assessment of fire behavior can form a rational basis for the design of structural fire protection of buildings, provided a design probability for failure is specified — by building codes, for example. The acceptable probability of failure depends on the value at risk and community standards (particularly for tall buildings where collapse could result in multiple deaths).

Given the required regulatory information, it is possible to proceed with design based entirely on scientific and engineering methods. These methods enable interpretation of the performance of steel members under standard test conditions for other conditions met with in practice, so that good use can be made of the considerable standard fire test data concerning protective materials that have been accumulated over the years.

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APPENDIX: NOMENCLATURE

A = area, ft^2/ft
C = heat capacity of steel core of column per unit height, Btu/ft °F
c = specific heat; without subscript that of concrete, Btu/lb °F
D = developed perimeter; without subscript inner perimeter of protection, in.
E = modulus of elasticity, ksi
ΔH = activation energy of creep, Btu/lb mole
I = moment of inertia of steel elements, $in.^4$
L = one-fourth of inner perimeter of concrete protection, ft
l = thickness of protection, in.; also thickness of concrete cover, ft
k = thermal conductivity of concrete at room temperature, Btu/ft hr °F
p = perimeter; without subscript: inner perimeter of concrete protection, ft
R = gas constant, Btu/lb mole °F
S = span of joist or truss, in.
s = length of key member or panel, in.
T = temperature, °F
W = weight of steel per ft length, lbs/ft
Z = Zener-Hollomon parameter, of a key member in the case of trusses, and of the centre in the case of beams, hr^{-1}
$\theta = T - 68/1800$
λ = slenderness ratio
ρ = density, lbs/ft^3
σ = stress, ksi
τ = fire resistance, min. or hr

Subscripts:

a = allowable
c = of concrete
cr = critical
d = of deck
o = at 68°F
s = pertaining to steel
T = at temperature T