Fire Resistance of Protected Steel Columns

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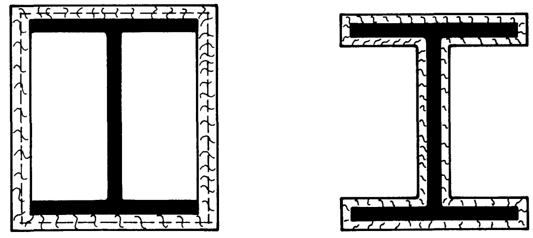
FULLY DEVELOPED building fires generally attain gas temperatures in the order of 2000°F. As the mechanical properties of steel deteriorate rapidly at temperatures of about one half this magnitude, it is necessary to provide some means of keeping steel columns relatively cool during exposure to fire, with the possible exception of extremely massive steel sections.[1] External insulation of a steel section to prevent excessive heat transfer during the expected period of fire exposure* is the most common method of providing fire resistance, although internal liquid-cooling has recently proved to be a viable protection method as well.

Typical forms and methods of fire protection in current use are illustrated in Figs. 1 to 4. Light protection (Fig. 1) using low density materials applied either to the profile of a section or in box form is most popular from an economic point of view. Massive protection, particularly concrete encasement, is used in special cases and forms the subject of a separate study.[2] External protections, which do not readily fall into either of these categories, have been labelled "complex protection" (Fig. 3) because their analysis may require special methods or engineering judgment. Box protected H-columns with core filling or very thick contour protection are examples. Liquid filling as fire protection (Fig. 4) can be accomplished by use of design methods described in Refs. 3 and 4. This paper will confine itself to the fire resistance of steel columns protected by relatively low-density materials, examining the problem by both fundamental and experimental engineering methods.

The ability to maintain its load carrying capacity is the only performance requirement of a building column during fire exposure. Consequently, the first applicable North American fire test standard[5] required a sample at least 9 ft in length to be tested under an applied load calculated to develop the theoretical working stresses of the design. In this standard the column is required to sustain the applied load for a period equal to the length of time for which classification is desired. Such classifications, measured in hours, form the basis of column protection required by building regulations.

Experience with the loaded column fire test indicated that failure of a protected steel column was reasonably predictable on the basis of the temperature attained by the steel cross section. This newer alternate test of protection for structural steel columns requires that a sample at least 8 ft in length be tested in a vertical position without applied load. The test is applicable when the protection is not required by design to carry any part of the column load. The applied protection must be restrained against longitudinal thermal expansion greater than that of the steel column. Temperatures are measured by at least three thermocouples located at each of four levels (cross sections). The upper and lower levels are 2 ft from the ends of the steel column and the two intermediate levels are equally spaced. The test is considered successful if the transmission of heat through the protection during the period of fire exposure for which classification is desired does not raise the average (arithmetical) temperature of the steel at any level above 1000°F, or above 1200°F at any one of the measured points.



(a) Box Protection (b) Contour Protection

Fig. 1. Light protection

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* *Fire regulations and "standards" concern themselves only with performance during a fire test, not with the degree of damage suffered by a structure or its possible reusability after a fire.*

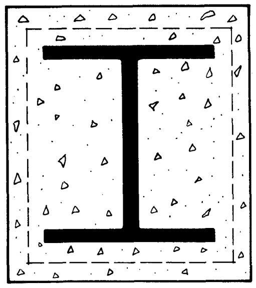

Fig. 2. Massive protection

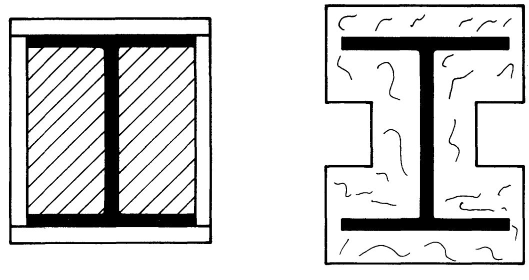

Fig. 3. Complex protection

These methods are stated in the current version[6] of the fire test standard, and standard test methods used in other countries are essentially similar.

The expense involved with large-scale fire testing (in the order of several thousand dollars per test) has encouraged a more fundamental approach to the evaluation of fire resistance. The present paper is the most recent effort, and is based on the findings of other research workers and the results of many calculations and experiments by the authors. Although the methods used to derive the information that follows were often complex and expensive, an effort was made to find those most suitable for practical use in the design of fire protection for steel building columns.

CRITICAL TEMPERATURES AND STRUCTURAL DESIGN

The critical temperature of a steel column is defined as the cross-sectional average temperature at which the member 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. With axially-loaded members the temperature at which the column buckles* is usually regarded as the critical temperature in fire resistance studies and depends on several factors. The most significant are: load intensity (stress); mechanical properties of the steel; shape, unit mass and length; end conditions; contribution of the protection to the strength of the structural unit.

In the present study it was assumed that the protection does not contribute to column strength and that the column is axially loaded to the allowable stress permitted by the American Institute of Steel Construction.[7] Accordingly, the following expressions were used in the calculations:

$$KL/r < C_c: \qquad F_a = \frac{\left[1 - \frac{(KL/r)^2}{2C_c^2}\right] F_{yo}}{\frac{5}{3} + \frac{3(KL/r)}{8C_c} - \frac{(KL/r)^3}{8C_c^3}} \tag{1}$$

$$KL/r > C_c: \qquad F_a = \frac{12\pi^2 E_o}{23(KL/r)^2} \tag{2}$$

where

$$C_c = \sqrt{\frac{2\pi^2 E_o}{F_{yo}}} \tag{3}$$

The allowable stresses calculated with the above equations are similar to those specified in several other countries.[8,9,10,11]

The mechanical properties that most significantly affect the critical temperature of a steel column are modulus of elasticity and yield strength of the steel. Both decrease as temperature increases. Data concerning the dependence of these properties on temperature have been reported by several authors.[12–19] Measured values of the modulus of elasticity and the yield strength of various structural carbon steels (ASTM A36, St-37,

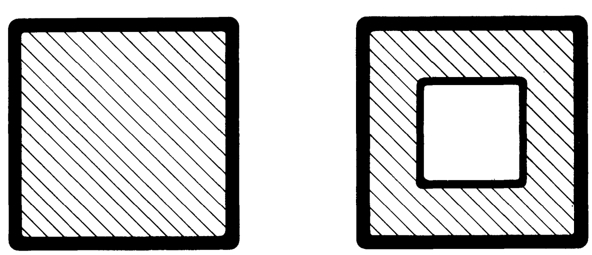

Fig. 4. Liquid-filling as fire protection

* *During fire tests it has been customary to permit a lateral deflection of 6 in. or more in the United States and 3 in. in England and most European countries. For the purposes of this work, however, failure will be taken as the point at which lateral deformation due to axial load begins, i.e., the point at which the column buckles. Buckling means the point at which any structure or part of a structure passes from one deflection pattern to another without a change in load, i.e., the point at which it first becomes unstable. In a fire test this would correspond approximately to the point of maximum expansion. Point of maximum expansion means the time at which the column is at its maximum length during the fire test. The column temperature at this time is usually less than the maximum temperature at the time of collapse. For tests of short duration (up to 2 hr), the points of maximum expansion and ultimate "failure" are separated by only a few minutes, but for longer fire test periods there may be a considerable time lapse between the two.*

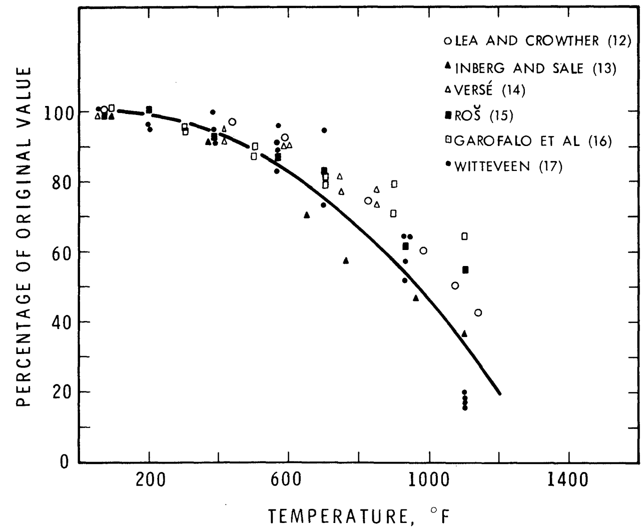

Fig. 5. Modulus of elasticity of carbon steels as a function of temperature

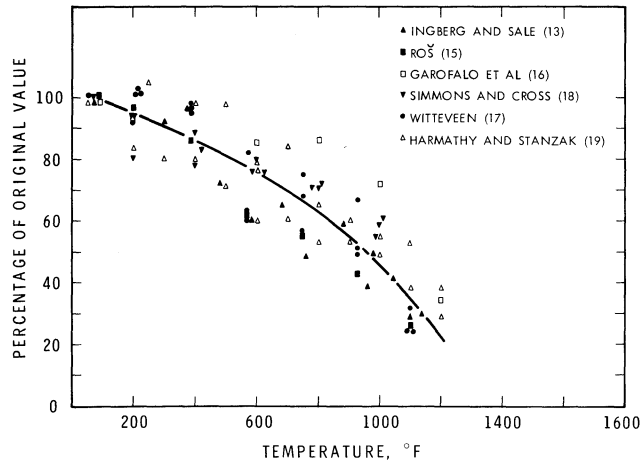

Fig. 6. Yield strength of carbon steels as a function of temperature

CSA G40.12) as a function of temperature are plotted in Figs. 5 and 6. The wide spread in the data can be attributed to many factors, the most important probably being the variability of steel quality and the influence of strain rate and creep properties on the test results.

Curves have already been drawn[20] to suit an analytical expression as well as the data reported in the literature. These will be used to evaluate the critical temperatures of steel columns and their dependence on column size, shape, and length. The authors point out that the yield strength curve represents the average decrease with temperature, but the modulus of elasticity curve, because it represents the more important variable, is somewhat conservative. The appropriate analytical expressions for yield strength and modulus of elasticity are:

$$F_y = F_{yo}(1 - 0.78\theta - 1.89\theta^4) \tag{4}$$

and

$$E = E_o(1 - 2.04\theta^2) \tag{5}$$

respectively. In these equations

$$\theta = \frac{T - 68}{1800}$$

The following assumptions were made in the calculations:

1) The effect of residual stresses can be ignored for rolled profiles. As these are due to stresses caused by faster cooling of the outer flanges than the flange-web junctions, it follows that on rapid heating the process is reversed. With longer fire resistance times, say 2 hr or more, sufficient time for stress relief due to metallurgical changes is available. With cold-rolled and welded sections the effect of residual stresses may be significant where fire exposure is brief.

2) The influence of creep is negligible at temperatures below approximately 950°F[19] and will not be separately taken into account. It should be realized, however, that the presence of creep deformation is already inherent in the expressions for mechanical properties used in the calculations.

3) The stress-strain curve at elevated temperatures can be obtained from an expression similar to that proposed by Galambos[21] for normal service temperatures:

$$\epsilon = \frac{F}{E} + \left[\frac{3F_y}{7E}\right]\left[\frac{F}{F_y}\right]^{10} \tag{6}$$

The following method was used to calculate buckling stresses and critical temperatures for steel columns with various slenderness ratios:

For stresses below the yield strength of the steel the buckling stress F_{cr} is given by

$$F_{cr} = \frac{\pi^2 E_t}{(KL/r)^2} \tag{7}$$

where E_t is the tangent modulus obtained by differentiating Eq. (6), so that

$$E_t = \frac{dF}{d\epsilon} = \frac{E}{\left[\dfrac{1 + 30(F/F_y)^9}{7}\right]} \tag{8}$$

For low slenderness ratios the calculated values of F_{cr} will exceed the yield strength of the steel. In this case buckling stress is considered to be the yield strength of the steel as determined by Eq. (4), at the temperature under consideration.

Using Eqs. (4) to (8), and assuming values of F_{yo} and E_o of 36 and 2900 ksi, respectively, column curves have been calculated for various steel temperatures. These curves are plotted in Fig. 7, along with the AISC design curves given by Eqs. (1) to (3). The curves show that the

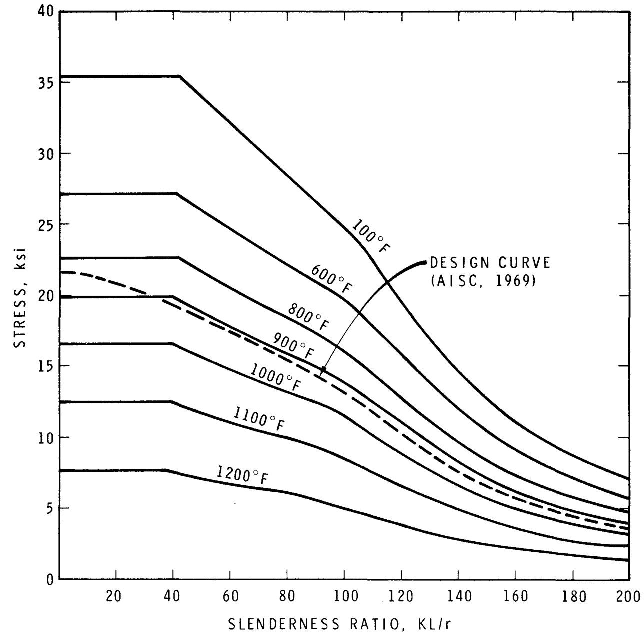

Fig. 7. Buckling stress as a function of the slenderness ratio for various steel temperatures

critical temperature of the shorter columns is approximately 880°F and that of intermediate and long columns about 950°F. Many fire tests on loaded columns[22–24] indicate that, for columns whose protection does not add significantly to the strength of the structural unit, complete failure occurs at temperatures 50° to 100°F higher than the temperature at which the column buckles. Consequently, the 1000°F critical cross-sectional temperature permitted in the current ASTM and CSA standards[6,25] appears appropriate because fire tests are not considered terminated until complete structural failure has occurred.

TEMPERATURE RISE OF PROTECTED STEEL COLUMNS

The temperature rise in a protected steel column is most reliably obtained by conducting a fire test, but it can also be calculated by engineering methods with a reasonable degree of accuracy. The problem, unlike fire resistance problems concerning more complex structural elements, is one of heat conduction and has, over the years, been the subject of several theoretical studies. The following methods already exist to explain the mechanism of heat transfer from a fire through insulation to the steel core (see Fig. 1a.)

Method 1, originally proposed by Geilinger and Bryl,[26] assumes one-dimensional heat transfer through the insulation. Accordingly, the model representing a protected steel column exposed to fire is a steel plate having the same weight to heated surface area ratio as the four sides of a unit length of the heated column, protected on the fire side and perfectly insulated on the other side.* The heat capacity of the protective material is neglected and the temperature gradient therein is assumed to be linear. Thermal resistance to heat transfer between the fire and an exposed surface is taken into account, but is usually negligible in comparison with the resistance of the protection, for which constant values of the thermal properties are assumed. If desired, the temperature dependence of the heat capacity of steel can be accounted for, but this is usually superfluous, considering the inaccuracies normally inherent in the assumptions previously made for protection.

Method 2 is the same as Method 1, with the following exceptions: the heat capacity of the protection is accounted for by adding one half its value to the heat capacity of the steel core, as proposed by McGuire et al.[27] Others have used one third of the heat capacity.[28,29] It is also assumed that the surface temperature of the protection is the same as the fire temperature.

Method 3, proposed by Lie,[30] again assumes one-dimensional heat transfer through the protection, but takes into account the actual heat capacity of the insulation. The temperature gradient in the protection is calculated but, as with the previous models, constant thermal properties for insulation and steel are assumed. The surface temperature of the protection is taken as the fire temperature, as in Method 2.

Method 4, originally investigated by Lie and Harmathy,[31] assumes two-dimensional heat transfer through the insulation. Radiative heat transfer is assumed from the fire to the surface of the insulation, and heat transfer through the insulation proceeds by a conduction mechanism. The thermal properties of the protection and steel core at any temperature can be taken into account. Of the four methods this one most realistically represents the actual physical situation and its accuracy has been borne out by experiment.[31]

The authors have examined these methods in detail with a view to determining their relative merits and limitations. A large number of fire resistance calculations, based on a 1000°F critical temperature, were performed for each model. The influence on fire resistance of section size and shape as well as thickness and material properties was examined; the results are listed in Table 1. Both light and heavy protective materials were considered. The equations used for the calculations are given in Appendix A.

In these calculations, a furnace boundary condition whose course satisfies the following equation was assumed:

$$T - T_0 = 2160 - 990e^{-0.6t} + 360e^{-3t} - 1530e^{-12t} \tag{9}$$

* *This model has already been employed with good results in calculating the temperature rise of unprotected steel columns.*[1]

This temperature course is practically the same as that of ASTM[6] and ISO[32] for the first 2 hr. After that, it follows approximately the ISO course, becoming slightly higher than that of ASTM. It was necessary to use such an expression of e-powers as a boundary condition in order to make the heat transfer equations integrable. The maximum difference in temperature, as compared with ASTM, is in the order of 6 percent.

To illustrate the effect of slight differences in the exposing atmosphere, calculations were repeated using an expression that accurately approximates the ASTM curve:[33]

$$T - T_0 = 1844.2 - 699.5e^{-0.02022t} - 1638e^{-0.3827t} + 493.3e^{-206.7t} \quad (10)$$

where $t \leq 120$.

Where $t \geq 120$,

$$T - T_0 = 1632 + 1.25t \quad (11)$$

The results of these calculations are given in the last column of Table 1 and show that the differences arising out of the use of the two curves are very small indeed.

Table 1 also shows that Method 3 gives practically the same results as Method 4, provided the assumption in Method 3 are closely satisfied, i.e.,

(a) The insulation is thin in comparison with the heated perimeter, so that heat transfer is approximately one-dimensional; generally, this condition is reasonably achieved when $l/D \leq 0.05$;

(b) The surface temperature of the insulation is approximately equal to the fire temperature, as is the case with a material that is light and a good insulator.[34] Figure 8 illustrates the close agreement obtained between Methods 3 and 4 for a few typical cases; the steel temperatures in Method 3 are obtained by exact solutions of the heat transfer equations and are precise provided the above assumptions are met; thus, the good agreement of the results obtained by Methods 3 and 4 also proves the accuracy of the numerical method.*

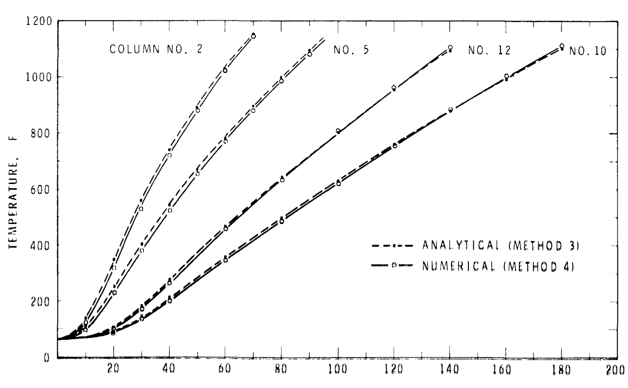

Fig. 8. Temperature rise of protected steel columns calculated by Methods 3 and 4 for a few typical cases (column and material characteristics are given in Table 1)

In summary, the calculated results of Table 1 show the following:

1. Neglect of the heat capacity of the protective material (Method 1) gives low values of fire resistance when compared with experimental data[35] and values calculated by other methods. An estimate[36] indicates that results obtained by Method 1 have practical value when the ratio

$$\frac{\text{heat capacity of insulation}}{\text{heat capacity of steel}} \leq 0.5$$

2. Adding one-third of the insulation's heat capacity to that of the steel gives better values than Method 1, but they are still low compared with the values produced by other methods and experimental data.

3. Adding one half of the insulations's heat capacity or using Method 3 (analytical) provides very good agreement with values obtained by Method 4 (numerical) for light insulating materials. For more dense materials, the results obtained by adding one half of the insulation's capacity or by Method 3 are somewhat lower than those of Method 4.

In this case, the assumption that the surface temperature of the insulation is equal to the fire temperature is probably not sufficiently satisfied. It can be shown[34] that with a heavy material, i.e., the product $k\rho c_i$ is large, the temperature of the surface will be appreciably lower than the fire temperature.

4. Values obtained using Method 4 are the most realistic, provided the thermal properties of the protective material are accurately known.

It should be pointed out that where models 1 to 3 are used for the solution of fire resistance problems a value can usually be obtained with the aid of a slide rule or desk calculator using materials properties reasonably representative of those at elevated temperatures and often found in the literature. Application of Method 4 requires a high-speed digital computer and materials properties obtainable from only a limited number of laboratories, including the authors'. Method 4 is, however, the one most suitable for calculations involving protective materials containing components that undergo significant chemical reactions at elevated temperatures, for example, cement paste and gypsum. The change in thermal properties of such materials, particularly increase in specific heat, will result in an increased fire resistance. Unfortunately, the variation of specific heat with temperature for these materials follows an irregular

* *In these calculations, hypothetical columns with box protection having the characteristics stated in Table 1 were used.*

Table 1. Comparison of Fire Resistances Calculated by Various Methods

Column No.	Size in.	Thickness Insulation in.	Thermal Properties of Insulation* (ρc) Btu/ft³° F	(k) Btu/ft h° F	W lb /ft	W/D lb/ft /in.	Fire Resistance (Time in minutes to reach 1000° F steel temperature) According to Method: No. 1 Capacity = 0	No. 2: 1/3 Capacity Added	No. 2: 1/2 Capacity Added	No. 3 Exact Solution	No. 4 (Numerical Method) ISO Curve	No. 4 (Numerical Method) ASTM Curve
			(Light Insulation)									
1	4x4	1	11.12	0.165	15	0.9375	42	50	54	54	54	55
2	6x6	1	"	"	25	1.042	45	53	57	58	59	60
3	10x10	1	"	"	15	0.375	21	30	34	34	37	37
4	10x10	1	"	"	40	1.0	44	52	54	56	58	59
5	10x10	1	"	"	70	1.75	69	76	79	80	82	83
6	16x16	1	"	"	70	1.094	47	55	59	59	62	63
7	4x4	2	"	"	15	0.9375	72	100	113	118	108	109
8	6x6	2	"	"	25	1.042	78	105	118	124	122	124
9	10x10	2	"	"	15	0.375	35	67	81	84	85	85
10	10x10	2	"	"	70	1.75	119	143	155	161	161	166
11	12x12	2	"	"	70	1.458	103	127	140	144	144	147
12	16x16	2	"	"	70	1.094	82	108	121	126	126	128
			(Heavy Insulation)									
13	4x4	1	29	0.7	15	0.9375	17	20	23	23	30	31
14	6x6	1	"	"	25	1.042	19	21	24	24	33	33
15	10x10	1	"	"	15	0.375	9	16	18	19	26	26
16	16x16	1	"	"	70	1.094	16	22	28	25	35	35
17	4x4	2	"	"	15	0.9375	23	45	54	56	59	61
18	6x6	2	"	"	15	0.625	18	39	48	50	57	59
19	6x6	2	"	"	25	1.042	25	46	56	57	64	65
20	10x10	2	"	"	15	0.375	13	35	45	47	54	56
21	12x12	2	"	"	70	1.458	33	52	62	64	72	72
22	16x16	2	"	"	70	1.004	27	47	57	58	67	67

* In the calculations, a value of 0.11 Btu/lb° F has been used for the specific heat of steel.

pattern and representative values at elevated temperatures are often difficult to provide, rendering models 1 to 3 unsuitable for such protective materials.

For lighter materials whose heat capacity is relatively small, on the other hand, the influence on fire resistance of changes in heat capacity with temperature is also relatively small. By examining the results obtained by means of Methods 1 to 4, it was possible to derive simple formulas for the fire resistance of steel columns thus protected. These are accurate enough for most practical purposes, as will be shown.

DESIGN FORMULAS

The derivation of design formulas was based on an examination of the parameters governing the rise of steel temperature during fire exposure. Empirical formulas based on the most significant parameters were derived as will now be described.

One of the assumptions common to all methods used in the analysis described in the previous section is that heat transmitted through the insulation to the steel core is equal to the increase in the heat content of the steel (the heat capacity of air spaces enclosed by the insulation is always so small that it is neglected). Thus, where one-dimensional heat transfer is assumed (models 1 to 3), the temperature rise of the steel is given by:

$$c_s W \frac{\partial T_s}{\partial t} = Ak \frac{\partial T_i}{\partial x} \qquad (12)$$

where

c_s = specific heat of steel
W = mass of steel per unit length
A = area of protection at the interface between protection and steel through which heat is transferred to the steel, per unit length
k = thermal conductivity of insulation
T_s = steel temperature
T_i = insulation temperature
t = time
x = coordinate perpendicular to insulation surface.

If thermal resistance between the insulation surface and fire is neglected and a linear temperature gradient through the insulation assumed, it becomes:

$$\frac{\partial T_i}{\partial x} = \frac{T_f - T_s}{l} \tag{13}$$

where

T_f = fire temperature
l = thickness of insulation

Substitution in Eq. (12) then yields:

$$\frac{c_s W l}{AK} \frac{\partial T_s}{\partial t} = T_f - T_s \tag{14}$$

which shows that the steel temperature is a function of the parameter Wl/AK if the specific heat of steel is taken as a constant. Because the heated area A is proportional to the heated perimeter D, the steel temperature is also a function of Wl/Dk.

A plot of the fire resistances obtained by means of Method 4 and the ASTM fire curve against the parameter Wl/Dk (Fig. 9) shows that this parameter alone cannot sufficiently describe the fire resistance of a protected steel column. This is not unexpected, because the parameter does not include the influence of the heat capacity of the insulation on the steel temperature. Adding another parameter, however, that is a function of l only, and can thus take into account to a certain degree the insulation's heat capacity, makes it possible to express the computed fire resistance by a single formula:

$$\tau = 0.09 \frac{Wl}{Dk} + Cl \tag{15}$$

where C is a constant. As indicated, the term Cl takes into account the heat capacity of the insulation and a value of 0.42 for C gives a good fit with the computed fire resistance (Fig. 9).

All parameters in the formula can be determined readily, except for thermal conductivity of the insulation, k, which almost always varies with temperature. If a constant value of k is used, therefore, it should be chosen so as to characterize, approximately, the actual thermal conductivity at elevated temperatures. Such approximate values are given in Refs. 34 and 37.

Normally, thermal conductivity increases with density. As density is a quantity that can be readily determined, an attempt was made to find a relation between density and thermal conductivity for use in Eq. (15). Figure 10 is a plot of k vs. ρ and indicates that the two can be approximately related by the expression:

$$k = 0.0046\rho \tag{16}$$

It should be noted that gypsum boards, whose thermal properties vary irregularly with temperature because of dehydration,[24] are not included in the graphs. Neither

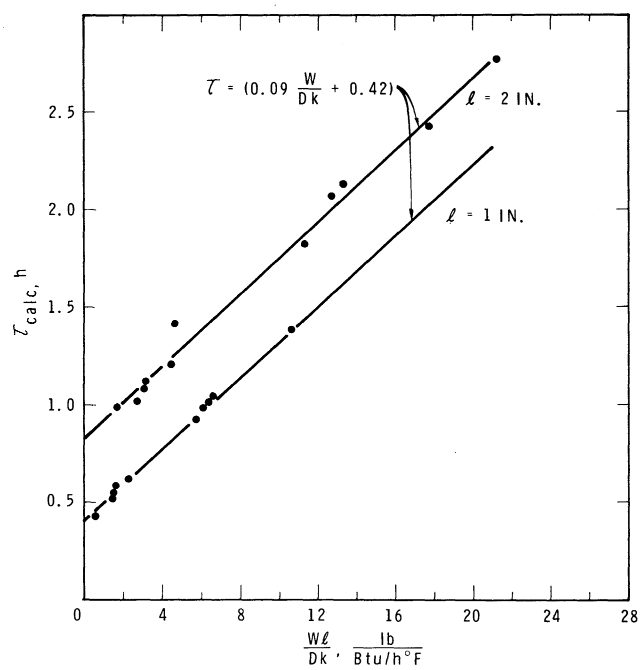

Fig. 9. Calculated fire resistance as a function of Wl/Dk for two protection thicknesses

is Eq. (16) applicable to porous mineral wool products with a density of less than about 20 lbs/ft^3 because their thermal conductivity increases very rapidly with temperature owing to radiation from fibre to fibre.[34] The reference states, however, that a value of approximately

$$k = 0.15 \text{ Btu/ft h °F}$$

can be used in Eq. (15) for the conductivity of mineral

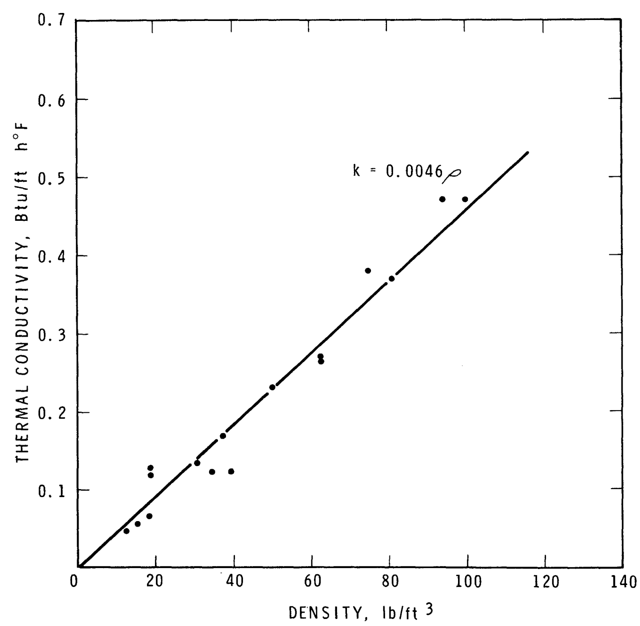

Fig. 10. Approximate thermal conductivity (k) at elevated temperatures of various materials as a function of their density (ρ)

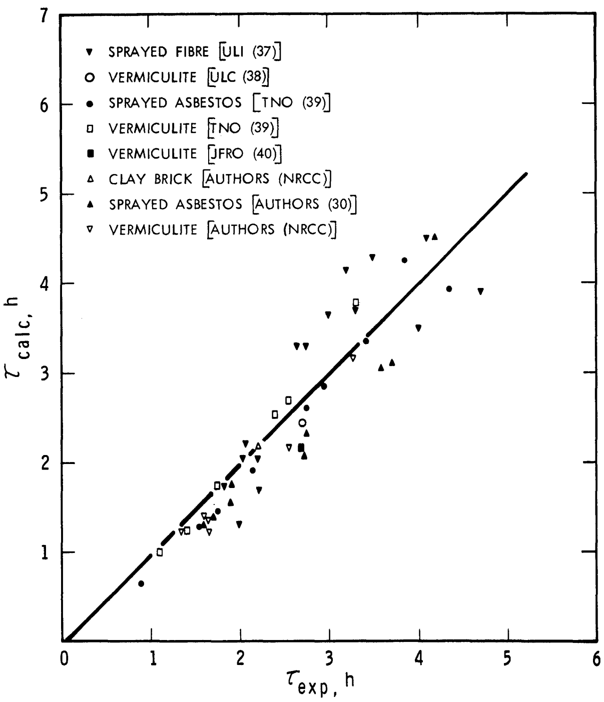

Fig. 11. Comparison between calculated and experimental fire resistances (calculated from Eq. (18) for light and chemically stable protections)

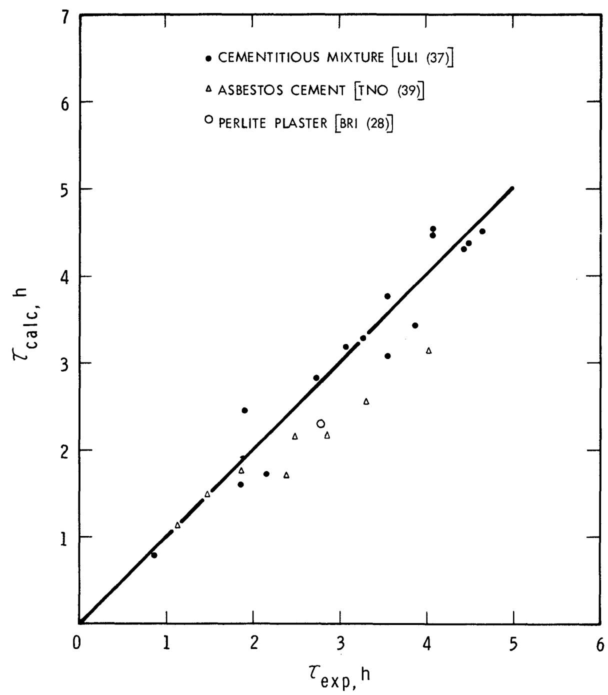

Fig. 12. Comparison between calculated and experimental fire resistances (calculated from Eq. (19) for light protections containing cementitious components)

wool in the density range 7 to 20 lbs/ft^3. This illustrates that caution should be applied in the correct use of these formulas.

Generally, light, fibrous, porous materials such as mineral wool products will provide lower fire resistances than those calculated. Others that undergo chemical changes (gypsum, cement paste, some concrete or plaster aggregates) will provide higher fire resistances. Chemically stable materials (vermiculite, perlite, dense mineral wool, asbestos, clay) are expected to yield fire resistances very close to those calculated by the design formula. On substitution of Eq. (16) into Eq. (15), this becomes:

$$\tau = 20 \frac{Wl}{D\rho} + Cl \qquad (17)$$

All parameters in this expression can be readily determined.

Using a value of $C = 0.42$ (which gave the best fit with fire resistances computed by Method 4), the accuracy of Eq. (17) was examined by comparing calculated fire resistances with experimental data from laboratories in Britain (JFRO), Canada (NRCC and ULC), Holland (TNO), Japan (BRI) and the United States (ULI). The materials in the tests were for the most part common, protective materials such as vermiculite, perlite and sprayed fibres with various binders, and mineral wool. One test involved a clay brick. Both box and contour type protections were represented. The comparison shows that calculated values of fire resistance are in fair agreement with experimental results, although generally slightly lower for chemically stable materials. As a result, the following expression was chosen as yielding the most representative answers (Fig. 11):

$$\tau = \left(20 \frac{W}{D\rho} + 0.5\right) l \qquad (18)$$

for relatively lightweight protective materials ($\rho \leq 50$ lbs/ft^3).

For materials that contain cement paste or gypsum, Eq. (18) provides conservative answers. Using $C = 1.2$ gives good results (Fig. 12) and for these materials the expression

$$\tau = \left(20 \frac{W}{D\rho} + 1.2\right) l \qquad (19)$$

should be used ($\rho \leq 50$ lbs/ft^3).

Design formulas (18) and (19) were developed by a semiempirical approach and offer a far simpler solution to column fire resistance problems than has previously been available. Users should appreciate, however, that because of their generality certain pitfalls can be encountered if they are applied to a problem indiscriminately, as has already been illustrated for mineral wool products. It is evident from the section "Temperature Rise of Protected Steel Columns" that the accuracy of calculated results can be improved for any material by returning to Eq. (15) whenever sufficient test data are available.

Further examination of low-density mineral wool protections serves as an example. It has already been stated that Eq. (16) and hence Eq. (17) are not valid for these protections. If the actual thermal conductivity of $k = 0.15$ Btu/ft h °F is used in Eq. (15), as has been recommended, reasonable agreement with experimental data may be obtained for the density range 7 to 20 lbs/ft³. With even lighter products, such as are normally used for sound absorption, a value of $k = 0.25$ Btu/ft h °F or higher was found to be appropriate.

DISCUSSION

The present study was designed to provide methods for:

1. Assessing the effect of construction (geometry) dependent parameters on the fire resistance of protected steel columns,
2. Extension of fire test data on protective materials to enable wider and more realistic application of known information in building construction,
3. Facilitating development of new products by reducing the amount of fire test data required to gain acceptance.

Each of these items will be discussed in turn and, where possible, illustrated by examples.

1. Construction Dependent Parameters—These are the "size and shape" factor W/D, the protection thickness l, type of protection (box or contour), and whether or not a continuous air gap is present between the protective cover and steel. Quality of workmanship is naturally of vital importance, but variations therein are usually not amenable to quantitative treatment. Quality of workmanship in commercial construction generally falls considerably short of that in specimens submitted for fire test.

In the examples that follow, a W10×49 section is used as a basis of comparison because it is the shape most commonly fire tested in North America. A sprayed fibre having a density of 15 lbs/ft³ serves as a typical protective material

(*a*) *Size and shape factor W/D:* If a box type protection for a W10×49 with $l = 0.75$ is assumed, Eq. (18) yields a fire endurance time of 96 min and a fire resistance classification of 1½ hr. The same protection on an 8 x 8 x ¼ HSS (tubing, 25.4 lbs/ft) provides a fire resistance of only 70 min, making the difference between a 1½ hr and a 1 hr rating. A similar rating would be obtained for a W6×15.5 section, one of the lightest rolled sections used for building columns.

Figure 13 shows how fire resistance varies with W/D for a particular sprayed fibre applied to a thickness of ¾-in.; typical steel sections are noted on the graph. The

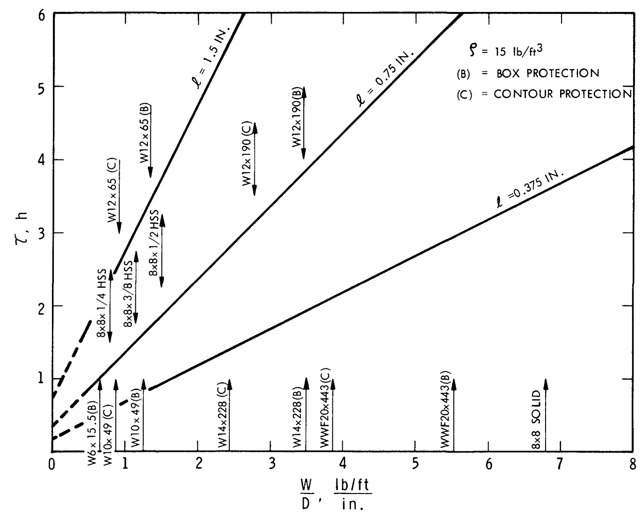

Fig. 13. Effect of "size and shape" factor W/D on fire resistance

fire resistance of the thickness chosen varies almost directly with W/D, and although the smallest columns obtain a fire rating of 1 hr, a rating of 2 or 3 hr can be obtained with the same protective thickness for many sections commonly used in high-rise steel frame construction.

(*b*) *Thickness of protection l:* Doubling the protection on a W10×49 section of the previous example to 1.5 in. yields a fire resistance of 192 min or 3.2 hr. Thus, where a small margin of safety is available in a fire test providing a given rating, any increase or decrease in desired rating can be provided by a proportionate increase or decrease in thickness of protection.* (This is true only if the protective material is not subject to large cracks and remains in place.)

(*c*) *Type of protection (box or contour):* The type of protection has a considerable influence on the fire resistance of flanged profiles. Continuing with a W10×49 section protected with ¾-in. sprayed fibre, the fire resistance was 96 min with box protection but only 72 min with contour protection, resulting in fire resistance classification of 1½ and 1 hr, respectively. Figure 13 shows examples of other common sections.

In the past it has been common to relate column fire resistance only to size by stating that classifications derived by test apply to a minimum size, for example, W10×49. This practice can be misleading in that a W12×65 column with contour protection will have practically the same fire resistance as a W10×49 column

* *McGuire et al*[27] *have proposed that* $l_1/l_2 = (\tau_1/\tau_2)^{1.25}$ *for identical columns with different protective thicknesses. Consequently, the authors of this study caution that a margin of safety of at least* $\tau_1/5$ *should be present in a test result for each doubling of fire resistance to be calculated by Eq. (18). Conversely, it is always safe to make calculations resulting in a lesser fire resistance than one that is known by test.*

similarly protected. If a test were performed on the W12 column, the rating might well be rejected for use with a W10 column because the latter is "smaller."

(*d*) *Air gaps:* To illustrate the influence of a continuous air gap on fire resistance, the W10×49 example is continued. With no air gap between the box protection and flanges, the fire resistance time was 96 min; with a 1-in. air gap it is 84 min; with a 2-in. gap, 75 min; with a 3-in. gap, 68 min, etc. Thus the presence of an air gap can sufficiently lower fire resistance to change the fire resistance classification, in this case from 1½ hr to 1 hr.

In general, the presence of a continuous air gap in a construction increases its fire resistance, as is the case with suspended fire-resistive ceilings versus those directly applied. With protected columns, this is only true where the air gap occurs between layers of protective material, not between the protective material and the column. The latter point has been proved by Lie and Harmathy[31] who found that assuming heat transfer to steel by radiation or conduction produced little difference in the calculated temperature rise (Fig. 14). This fact also dispels the common belief that the increase in fire endurance achieved with flanged profiles by the change from contour protection to box protection is due to the beneficial influence of the air space thus created. It is due to the decrease of the heated perimeter D.

2. Extension of Fire Test Data—Some possible extensions of fire test data have already been illustrated in the examples. These can be extremely important in facilitating the introduction of new products or methods, and in overcoming everyday problems encountered in building practice.

Protection for columns smaller than the standard W10×49, for example a W6×15.5, has been questioned. It is now possible to calculate the protection required on the basis of a known standard fire test result for a W10×49 section.

A plot such as that illustrated in Fig. 13 can also be drawn for most common protective materials, using information available from existing fire test data. This greatly facilitates the design of fire protection for new steel products (tubular columns are a fairly recent example) or custom-made sections. Similar design methods can be applied to protection of steel joists and beams if the heat sink effect of the floor slab is ignored.

As a final example, if an air gap is required to accommodate a certain non-combustible building service such as a water pipe, but test data is available only for contact protection, the increase in fire protection required by creation of the air gap can be calculated. The result will usually be quite conservative owing to the extra core heat capacity provided by the building service in question.

3. New Products—Several full-scale fire tests costing many thousands of dollars are usually carried out in developing and gaining acceptance for a new column fire protection material. It should be clear that only one non-loaded fire test need be conducted, and that it should be designed so as to yield the maximum fire resistance classification that may be desired (usually 3 or 4 hr). This is necessary to ensure that the material is capable of remaining in place and relatively intact during the maximum fire exposure contemplated for the product. All other fire resistance requirements can be conservatively calculated by means of the design formulae.

To summarize, the design formulas are very useful and their reliability increases with availability of the following information:

(a) density of the protective material

(b) knowledge of the chemical reactions that take place while the material is heated

(c) average thermal conductivity characteristic of the behaviour at elevated temperatures for use in Eq. (15), obtained by laboratory tests or calculated from fire test results

(d) a value of C, calculated from fire test data on a given material for use in Eqs. (15) or (17), that is more appropriate than the general values for C used in Eqs. (18) and (19)

(e) the amount of available fire test data, including a fire test for the maximum fire exposure contemplated

CONCLUSION

Means of solving fire resistance problems for protected steel columns have been presented. All rely on careful engineering judgement in the choice of solution and assessment of the confidence level of that solution. With known and widely used materials, for which considerable fire test data are available, it is suggested that Eqs. (18) and (19) be incorporated in building regulations in the form

$$\tau = \left(20 \frac{W}{D\rho} + C\right) l \qquad (17)$$

where

C = 0.5 for protections mainly consisting of chemically stable materials such as vermiculite, perlite, sprayed asbestos with various binders, and dense ($\rho \geq 20$ lbs/ft^3) mineral wool

C = 1.2 for protections containing cement paste or gypsum, such as asbestos-cement board, plasters, and cementitious mixtures

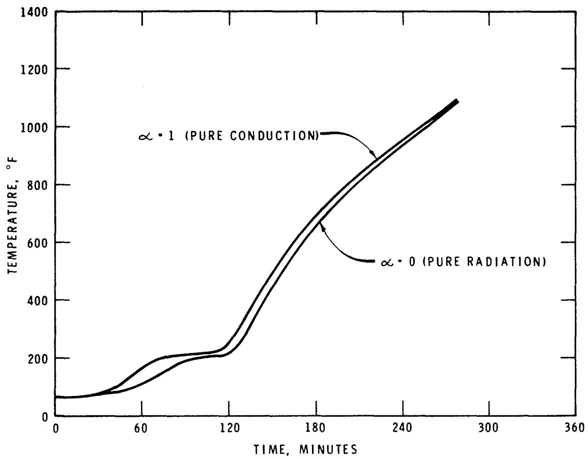

Fig. 14. Effect of heat transfer mechanism from protection to steel on the steel temperature rise

Equation (17) is valid for relatively light protective materials ($\rho \leq 50$ lbs/ft^3). When used for heavier materials, it is expected to give conservative estimates of fire resistance.

NOMENCLATURE

- a Thermal diffusivity, ft^2/h
- A Area of protection at the interface between protection and steel through which heat is transferred to steel, per unit length ft^2/ft
- c Specific heat, Btu/lb °F
- C Constant, taking into account the heat capacity of insulation
- D Perimeter of protection at the interface between protection and steel through which heat is transferred to steel, in.
- E Modulus of elasticity, ksi
- F Stress, ksi
- k Thermal conductivity of insulation, Btu/h ft °F
- K Effective length factor
- l Thickness of insulation, in.
- L Height of column, ft
- r Radius of gyration, ft
- t Time, hr
- T Temperature, °F
- W Mass of steel section per ft, lb/ft
- x Coordinate perpendicular to insulation surface

Subscripts

- a Allowable
- cr Critical
- f Of fire
- i Of insulation
- o At room temperature
- s Of steel
- t Tangent
- y Yield

Greek Letters

- α Fraction of insulation capacity added to that of steel
- ϵ Relative strain
- ρ Density of insulation, lb/ft^3
- τ Fire resistance, h

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APPENDIX A
HEAT TRANSFER EQUATIONS

Models 1 and 2—The general form of the equation that describes heat transfer through insulation from a fire to steel is

$$kA\,\frac{(T_f - T_s)}{l} = (c_s W + \alpha \rho c_i A l)\,\frac{dT_s}{d_t} \tag{1}$$

where α is the fraction of the thermal capacity of the insulation that is added to the steel capacity. In Model 1 the value of α is 0; in Model 2 it is $1/3$ or $1/2$.

If the temperature rise at the exposed surface follows the temperature time relation,

$$T_f - T_0 = 2160 - 990e^{-0.6t} + 360e^{-3.0t} - 1530e^{-12.0t} \tag{2}$$

The solution of Eq. (1) is:

$$\begin{aligned} T_s - T_0 = {} & 2160(1 - e^{-\gamma t}) \\ & - \frac{990}{0.6\gamma - 1)}\,(e^{-\gamma t} - e^{-0.6t}) \\ & + \frac{360}{3.0\gamma - 1}\,(e^{-\gamma t} - e^{-3.0t}) \\ & - \frac{1530}{1.20\gamma - 1}\,(e^{-\gamma t} - e^{-12.0t}) \end{aligned} \tag{3}$$

where

$$\gamma = \frac{kA}{l(c_s W + \alpha \rho c_i A l)}$$

Model 3—Heat transfer equations that determine temperatures in insulation and steel are as follows:

The temperature of the insulation satisfies the differential equation for thermal conduction

$$\rho c_i \frac{\partial T_i}{\partial t} = k \frac{\partial^2 T_i}{\partial x^2} \tag{4}$$

At the exposed surface of the insulation, temperature is assumed to follow the fire temperature course T_f. Thus, for $x = 0$

$$T_i = T_f \tag{5}$$

At the interface of the insulation, the heat flow to the steel is equal to the increase in the heat content of the steel per unit time. Hence, for $x = l$

$$-kA \frac{\partial T_i}{\partial x} = c_s W \frac{\partial T_i}{\partial t} \tag{6}$$

where

$$T_i = T_s \tag{7}$$

Initially, the temperature of a column is equal to the room temperature, so that for $t = 0$

$$T_i = T_0 \tag{8}$$

The method of solving Eqs. (4) to (8) is given in Ref. 30. For a temperature rise at the exposed surface that follows the temperature time relation given by Eq. (2), the solution is as follows:

$$\begin{aligned} T_s - T_0 = 2160 &- 2160 \sum_{n=1}^{\infty} f(\beta_n) e^{-a\beta_n^2 t} \\ &- 990 \sum_{n=1}^{\infty} f(\beta_n) \frac{a\beta_n^2}{a\beta_n^2 - 0.6} (e^{-0.6t} - e^{-a\beta_n^2 t}) \\ &+ 360 \sum_{n=1}^{\infty} f(\beta_n) \frac{a\beta_n^2}{a\beta_n^2 - 3.0} (e^{-3.0t} - e^{-a\beta_n^2 t}) \\ &- 1530 \sum_{n=1}^{\infty} f(\beta_n) \frac{a\beta_n^2}{a\beta_n^2 - 12.0} (e^{-12.0t} - e^{-a\beta_n^2 t}) \end{aligned} \tag{9}$$

where

$a = \dfrac{k}{\rho c_i}$ thermal diffusivity of insulation

β_n = roots of $\beta \tan \beta = hl$

$h = \dfrac{\rho c_i}{c_s W/A}$

l = thickness of insulation

$f(\beta_n) = \dfrac{2(\beta_n^2 + h^2) \sin \beta_n l}{\beta_n [l(\beta_n^2 + h^2) + h]}$

Model 4—The equations are described completely in Ref. 31.