APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of structural steel components, systems, and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion, and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and stiffness of structural components and systems at elevated temperatures.

User Note: Throughout this chapter, the term "elevated temperatures" refers to temperatures due to unintended fire exposure only.

The appendix is organized as follows:

4.1. General Provisions
4.2. Structural Design for Fire Conditions by Analysis
4.3. Design by Qualification Testing

4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

1. Performance Objective

Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires evaluation of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the applicable building code (ABC).

Structural design for fire conditions using Appendix 4.2 shall be performed using the load and resistance factor design method in accordance with the provisions of Section B3.1 (LRFD).

3. Design by Qualification Testing

The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by the ABC.

4. Load Combinations and Required Strength

In the absence of ABC provisions for design under fire exposures, the required strength of the structure and its elements shall be determined from the gravity load combination as follows:

$$(0.9 \text{ or } 1.2)\, D + A_T + 0.5L + 0.2S \qquad \text{(A-4-1)}$$

where

A_T = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

D = nominal dead load

L = nominal occupancy live load

S = nominal snow load

User Note: ASCE/SEI 7 Section 2.5 contains this load combination for extraordinary events, which includes fire. Live load reduction is permitted.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

1. Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

The analysis methods in Section 4.2 shall be used in accordance with the provisions for alternative materials, designs, and methods as permitted by the ABC. When the analysis methods in Section 4.2 are used to demonstrate equivalency to hourly ratings based on qualification testing in Section 4.3, the design-basis fire shall be permitted to be determined in accordance with ASTM E119.

1a. Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array, and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

1b. Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics of the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

The fire duration in a particular area shall be determined from the total combustible mass, or fuel load in the space. In the case of either a localized fire or a post-flashover compartment fire, the fire duration shall be determined as the total combustible mass divided by the mass loss rate.

1c. Exterior Fires

The exposure effects of the exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be addressed along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1b shall be used for describing the characteristics of the interior compartment fire.

1d. Active Fire-Protection Systems

The effects of active fire-protection systems shall be addressed when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

3. Material Properties at Elevated Temperatures

The effects of elevated temperatures on the physical and mechanical properties of materials shall be considered in the analysis and design of structural members, components and systems. Any rational method that establishes material properties at elevated temperatures that is based on test data is permitted, including the methods defined in Sections 4.2.3a, and 4.2.3b.

3a. Thermal Elongation

The coefficients of thermal expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 7.8×10^{-6}/°F (1.4×10^{-5}/°C).

(b) For normal weight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 10×10^{-6}/°F (1.8×10^{-5}/°C).

(c) For lightweight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 4.4×10^{-6}/°F (7.9×10^{-6}/°C).

3b. Mechanical Properties of Structural Steel, Hot-Rolled Reinforcing Steel, and Concrete at Elevated Temperatures

The uniaxial engineering stress-strain-temperature relationship for structural steel, hot rolled reinforcing steel, and concrete shall be determined using this section. This applies only to structural and reinforcing steels with a specified minimum yield strength, F_y, equal to 65 ksi (450 MPa) or less, and to concrete with a specified compressive strength, f'_c, equal to 8 ksi (55 MPa) or less.

(a) Structural and Hot Rolled Reinforcing Steel

Table A-4.2.1 provides retention factors (k_E, k_y, and k_p) for steel which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, assumed to be 68°F (20°C). It is permitted to interpolate between these values. The properties at elevated temperature, T, and are defined as follows:

$E(T)$ is the modulus of elasticity of steel at elevated temperature, ksi (MPa), which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.

$G(T)$ is the shear modulus of elasticity of steel at elevated temperature, ksi (MPa) , which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.
$F_y(T)$ is the specified minimum yield stress of steel at elevated temperature, ksi (MPa) , which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.
$F_p(T)$ is the proportional limit at elevated temperature, which is calculated as a ratio to yield strength as specified in Table A-4.2.1.

$F_u(T)$ is the specified minimum tensile strength at elevated temperature, which is equal to $F_y(T)$ for temperatures greater than 750°F (400°C). For temperatures less than or equal to 750°F (400°C), F_u may be used in place of $F_u(T)$.

The engineering stress at elevated temperature, $F(T)$, at each strain range shall be determined as follows:

(a) When in the elastic range [$\varepsilon(T) < \varepsilon_p(T)$]

$$F(T) = E(T)\,\varepsilon(T) \qquad \text{(A-4-2)}^{[a]}$$

(b) When in the nonlinear range [$\varepsilon_p(T) \le \varepsilon(T) \le \varepsilon_y(T)$]

$$F(T) = F_p(T) - c + \frac{b}{a}\sqrt{a^2 - \left[\varepsilon_y(T) - \varepsilon(T)\right]^2} \qquad \text{(A-4-3)}^{[a]}$$

(c) When in the plastic range [$\varepsilon_y(T) \le \varepsilon(T) \le \varepsilon_u(T)$]

$$F(T) = F_y(T) \qquad \text{(A-4-4)}^{[a]}$$

where

$\varepsilon(T)$ = the engineering strain at elevated temperature, in./in. (m/m)

$\varepsilon_p(T)$ = the engineering strain at the proportional limit at elevated temperature, in./in. (m/m) = $F_p(T) / E(T)$

$\varepsilon_y(T)$ = the engineering yield strain at elevated temperature, in./in. (m/m) = 0.02 in./in. (m/m)

$$a^2 = \left[\varepsilon_y(T) - \varepsilon_p(T)\right]\left[\varepsilon_y(T) - \varepsilon_p(T) + \frac{c}{E(T)}\right] \qquad \text{(A-4-5)}^{[a]}$$

$$b^2 = E(T)\left[\varepsilon_y(T) - \varepsilon_p(T)\right]c + c^2 \qquad \text{(A-4-6)}^{[a]}$$

$$c = \frac{\left[F_y(T) - F_p(T)\right]^2}{E(T)\left[\varepsilon_y(T) - \varepsilon_p(T)\right] - 2\left[F_y(T) - F_p(T)\right]} \qquad \text{(A-4-7)}^{[a]}$$

User Note: The equation for the plastic range conservatively neglects the strain-hardening portion, but strain-hardening is permitted to be includedThe plateau of the plastic range does not exceed the ultimate strain, $\varepsilon_u(T)$, where $\varepsilon_u(T)$ = 15%.

User Note: This section applies to structural steel materials in Section A3.1 and to hot-rolled reinforcing steel with a specified minimum yield strength, F_y, equal to 65 ksi or less, which includes ASTM A615/A615M Gr. 60 (420) and ASTM A706/A706M Gr. 60 (420) steel reinforcement.

(b) Concrete

Table A-4.2.2 provides retention factors for concrete which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, assumed to be 68°F (20 °C). It is permitted to interpolate between these values. For lightweight concrete, values of $\varepsilon_{cu}(T)$ shall be obtained from tests. The properties at elevated temperature, T, are defined as follows:

[a] EC4, European Committee for Standardization (CEN), Eurocode 4 Design of Composite Steel and Concrete Structures: Part 1.2: General Rules, Structural Fire Design, EN 1994-1-2, CEN, Brussels, 2005.

$f_c'(T)$ = the specified compressive strength of concrete at elevated temperature, ksi (MPa), which is calculated as a ratio to the ambient property as specified in Table A-4.2.2.
$E_c(T)$ = the modulus of elasticity of concrete at elevated temperature, ksi (MPa)
$\varepsilon_{cu}(T)$ = the concrete strain corresponding to $f_c'(T)$ at elevated temperature, in./in. (m/m)

The uniaxial stress-strain-temperature relationship for concrete in compression is permitted to be calculated as follows:

$$F_c(T) = f_c'(T)\left\{\frac{3\left[\dfrac{\varepsilon_c(T)}{\varepsilon_{cu}(T)}\right]}{2+\left[\dfrac{\varepsilon_c(T)}{\varepsilon_{cu}(T)}\right]^3}\right\} \qquad \text{(A-4-8)[a]}$$

where $F_c(T)$ and $\varepsilon_c(T)$ are the concrete compressive stress and strain, respectively, at elevated temperature.

User Note: The tensile strength of concrete at elevated temperature can be taken as zero, or not more than 10% of the compressive strength at the corresponding temperature.

(c) Strengths of Bolts at Elevated Temperatures

Table A-4.2.3 provides retention factors for high-strength bolts which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, which is assumed to be 68°F (20°C). The properties at elevated temperature, T, are defined as follows:

$F_{nt}(T)$ = nominal tensile strength of the bolt, ksi (MPa)
$F_{nv}(T)$ = nominal shear strength of the bolt, ksi (MPa)

TABLE A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E = G(T)/G$	$k_p = F_p(T)/F_y$	$k_y = F_y(T)/F_y$
68 (20)	1.00	1.00	1.00
200 (93)	1.00	1.00	1.00
400 (200)	0.90	0.80	1.00
600 (320)	0.78	0.58	1.00
750 (400)	0.70	0.42	1.00
800 (430)	0.67	0.40	0.94
1000 (540)	0.49	0.29	0.66
1200 (650)	0.22	0.13	0.35

[a] EC4, European Committee for Standardization (CEN), Eurocode 4 Design of Composite Steel and Concrete Structures: Part 1.2: General Rules, Structural Fire Design, EN 1994-1-2, CEN, Brussels, 2005.

1400 (760)	0.11	0.06	0.16
1600 (870)	0.07	0.04	0.07
1800 (980)	0.05	0.03	0.04
2000 (1100)	0.02	0.01	0.02
2200 (1200)	0.00	0.00	0.00

TABLE A-4.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature, °F (°C)	$k_c = f'_c(T)/f'_c$		$E_c(T)/E_c$	$\varepsilon_{cu}(T)$, %
	Normal Weight Concrete	Lightweight Concrete		Normal Weight Concrete
68 (20)	1.00	1.00	1.00	0.25
200 (93)	0.95	1.00	0.93	0.34
400 (200)	0.90	1.00	0.75	0.46
550 (290)	0.86	1.00	0.61	0.58
600 (320)	0.83	0.98	0.57	0.62
800 (430)	0.71	0.85	0.38	0.80
1000 (540)	0.54	0.71	0.20	1.06
1200 (650)	0.38	0.58	0.092	1.32
1400 (760)	0.21	0.45	0.073	1.43
1600 (870)	0.10	0.31	0.055	1.49
1800 (980)	0.05	0.18	0.036	1.50
2000 (1100)	0.01	0.05	0.018	1.50
2200 (1200)	0.00	0.00	0.000	0.00

TABLE A-4.2.3
Properties of Group 120 and Group 150 High-Strength Bolts at Elevated Temperatures

Bolt Temperature, °F (°C)	$F_{nt}(T)/F_{nt}$ or $F_{nv}(T)/F_{nv}$
68 (20)	1.00
200 (93)	0.97
300 (150)	0.95
400 (200)	0.93
600 (320)	0.88
800 (430)	0.71
900 (480)	0.59
1000 (540)	0.42
1200 (650)	0.16
1400 (760)	0.08
1600 (870)	0.04
1800 (980)	0.01
2000 (1100)	0.00

4. Structural Design Requirements

4a. General Requirements

The structural frame and foundation shall be capable of providing the strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage with the structural system as a whole remaining stable. Frame stability and required strength shall be determined in accordance with the requirements of Section C1.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance.

The requirement for steam vent holes in concrete-filled composite members shall be evaluated. Any rational method that considers heat transfer through the cross-section, water content in concrete, fire protection, and the allowable pressure build up in the member is permitted for calculating the size and spacing of vent holes.

User Note: Section 4.3.2.2.1 provides a possible vent hole configuration for concrete-filled columns.

4b. Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall have the design strength necessary to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall be designed and detailed to resist the imposed loading and deformation demands during a design-basis fire as required to meet the performance objectives stated in Section 4.1.1. Where the means of providing fire resistance requires the evaluation of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

User Note: Typical simple shear connections may need additional design enhancements for ductility and resistance to large compression and tensile forces that may develop during the design-basis fire exposure. A fire exposure will not only affect the magnitude of member end reactions, but may also change the nature of the reaction to a limit state different from the controlling mode at ambient temperature.

It shall be permitted to include membrane action of composite floor slabs for fire resistance if the design provides for the effects of increased connection tensile forces and redistributed gravity load demands on the adjacent framing supports.

4c. Design by Advanced Methods of Analysis

Design by advanced methods of analysis is permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials, as per Section 4.2.2.

The mechanical response results in forces and deformations in the structural system due to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, inelastic behavior and load redistribution, large deformations, time-dependent effects such as creep, and uncertainties resulting from variability in material properties at elevated temperature. Support and restraint conditions (forces, moments, and boundary conditions) shall represent the behavior of the structure during a design-basis fire. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall address all relevant limit states, such as excessive deflections, connection ruptures, and global or local buckling.

4d. Design by Simple Methods of Analysis

The methods of analysis in this section are permitted to be used for the evaluation of the performance of individual members at elevated temperatures during exposure to a design-basis fire. When evaluating individual members, the support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures are permitted to be assumed to remain unchanged throughout the fire exposure.

For evaluating the performance of structural frames during exposure to a design-basis fire, member demands (forces and moments) are also permitted to be determined through consideration of reduced stiffness at elevated temperatures, appropriate boundary conditions, and thermal deformations.

It is permitted to model the thermal response of steel and composite members using a lumped heat capacity analysis with heat input as determined by the design-basis fire defined in Section 4.2.1, using the temperature equal to the maximum steel temperature. For composite beams, the maximum steel temperature shall be assigned to the bottom flange and a temperature gradient shall be applied to incorporate thermally induced moments, as stipulated in Section 4.2.4d(f) .

For steel temperatures less than or equal to 400°F (200°C), the member and connection design strengths is permitted to be determined without consideration of temperature effects on the nominal strengths.

The design strength shall be determined as in Section B3.1. The nominal strength, R_n, shall be calculated using material properties, as provided in Section 4.2.3b, at the temperature developed by the design-basis fire and as stipulated in Sections 4.2.4d(a) through (f).

User Note: Lumped heat capacity analysis assumes uniform temperature over the section and length of the member, which is generally a reasonable assumption for many structural members exposed to post-flashover fires. Consideration should be given to the use of the uniform temperature assumption as it may not always be applicable or conservative.

At temperatures below 400°F (200°C), the reduction in steel properties need not be considered in calculating member strengths despite small reductions in material properties in Table A-4.2.1. This is a simplifying assumption used only in the simple method of analysis.

(a) Design for Tension

The nominal strength for tension shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3b(a) and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(b) Design for Compression

For nonslender-element columns, the nominal strength for flexural buckling of compression members shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3b(a).Equation A-4-9 shall be used in lieu of Equations E3-2 and E3-3 to calculate the nominal compressive strength for flexural buckling:

$$F_{cr}(T) = \left[0.42^{\sqrt{\frac{F_y(T)}{F_e(T)}}} \right] F_y(T) \qquad \text{(A-4-9)}$$

where $F_y(T)$ is the yield stress at elevated temperature and $F_e(T)$ is the critical elastic buckling stress calculated from Equation E3-4 with the elastic modulus, $E(T)$, at elevated temperature. $F_y(T)$ and $E(T)$ are obtained using coefficients from Table A-4.2.1.

The strength of leaning (gravity) columns may be increased by rotational restraints from cooler columns in the stories above and below the story exposed to the fire. This increased strength applies to fires on only one floor and should not be used for multiple story fires. The increase in design strength can be accounted for by reducing the column slenderness (L_c/r) used to calculate $F_e(T)$ in Equation A-4-9 to $(L_c/r)_T$ as follows:

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T-32}{n(3{,}600)}\right)\left(\frac{L_c}{r}\right) - \frac{35}{n(3{,}600)}(T-32) \geq 0 \quad (^{\circ}\text{F}) \qquad \text{(A-4-10)}$$

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T}{n(2{,}000)}\right)\left(\frac{L_c}{r}\right) - \frac{35T}{n(2{,}000)} \geq 0 \quad (^{\circ}\text{C}) \qquad \text{(A-4-10M)}$$

where

L_c = KL = effective length of member, in. (mm)
L = laterally unbraced length of the member, in. (mm)
K = effective length factor
r = radius of gyration, in. (mm)
T = steel temperature, °F , °C
n = 1 for columns with cooler columns both above and below
n = 2 for columns with cooler columns either above or below only

User Note: The design equations for compression predict flexural buckling capacities of wide flange rolled shapes, but do not consider local buckling and torsional buckling. If applicable, these additional limit states must be considered with an alternative method. For most fire conditions, uniform heating and temperatures govern the design for compression. When uniform heating is not a reasonable assumption, alternative methods must be used to account for the effects of nonuniform heating and resulting thermal gradients on the design strength of compression members, as the simple method assumes a uniform temperature distribution.

(c) Design for Compression in Concrete-Filled Composite Columns

For concrete-filled composite columns, the nominal strength for compression shall be determined using the provisions of Section I2.2 with steel and concrete properties as stipulated in Section A-4.2.3b. Equation A-4-11 shall be used in lieu of Equations I2-2 and I2-3 to calculate the nominal compressive strength for flexural buckling:

$$P_n(T) = \left[0.45^{\left(\frac{P_{no}(T)}{P_e(T)}\right)^{0.3}}\right] P_{no}(T) \qquad \text{(A-4-11)}$$

where $P_{no}(T)$ is calculated at elevated temperature using Equations I2-

10, I2-11, and I2-12. $P_e(T)$ is calculated at elevated temperature using Equations I2-5, I2-13, and I2-14. . $F_y(T)$, $f'_c(T)$, $E_s(T)$, and $E_c(T)$ are obtained using coefficients from Tables A-4.2.1 and A-4.2.2.

(d) Design for Compression in Concrete-Filled Composite Plate Shear Walls

For concrete-filled composite plate shear walls, the nominal strength for compression shall be determined using the provisions of Section I2.3 with steel and concrete properties as stipulated in Section A-4.2.3b and Equation A-4-12 used in lieu of Equations I2-2 and I2-3 to calculate the nominal compressive strength for flexural buckling:

$$P_n(T) = \left[0.32^{\left(\frac{P_{no}(T)}{P_e(T)}\right)^{0.3}}\right] P_{no}(T) \qquad \text{(A-4-12)}$$

where $P_{no}(T)$ is calculated at elevated temperature using Equation I2-16. $P_e(T)$ is calculated at elevated temperature using Equations I2-5 and I1-1. $F_y(T)$, $f'_c(T_c)$, $E_s(T)$, and $E_c(T_c)$ are obtained using coefficients from Tables A-4.2.1 and A-4.2.2.

User Note: For composite members, the steel temperature is determined using heat transfer equations with heat input corresponding to the design-basis fire. The temperature distribution in concrete infill can be calculated using one- or two-dimensional heat transfer equations. The regions of concrete infill will have varying temperatures and mechanical properties. Concrete contribution to axial strength and effective stiffness can therefore be calculated by discretizing the cross-section into smaller elements (with each concrete element considered to have a uniform temperature) and summing up the contribution of individual elements.

(e) Design for Flexure

For steel beams, the calculated bottom flange temperature shall be constant over the depth of the member.

(1) The nominal strength for flexure shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3b(b).Equations A-4-13 through A-4-19 shall be used in lieu of Equations F2-2 through F2-6 to calculate the nominal flexural strength for lateral-torsional buckling of doubly symmetric compact rolled wide-flange shapes bent about their major axis: When $L_b \le L_r(T)$

$$M_n(T) = C_b\left\{F_L(T)S_x + \left[M_p(T) - F_L(T)S_x\right]\left[1 - \frac{L_b}{L_r(T)}\right]^{c_x}\right\} \le M_p(T) \qquad \text{(A-4-13)}$$

(2) When $L_b > L_r(T)$

$$M_n(T) = F_{cr}(T)S_x \le M_p(T) \qquad \text{(A-4-14)}$$

where

$$F_{cr}(T) = \frac{C_b \pi^2 E(T)}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(A-4-15)}$$

$$L_r(T) = 1.95 r_{ts} \frac{E(T)}{F_L(T)} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76 \left[\frac{F_L(T)}{E(T)}\right]^2}}$$

(A-4-16)

$$F_L(T) = F_y\left(k_p - 0.3k_y\right) \qquad \text{(A-4-17)}$$

$$M_p(T) = F_y(T) Z_x \qquad \text{(A-4-18)}$$

$$c_x = 0.53 + \frac{T}{450} \le 3.0 \text{ where } T \text{ is in °F} \qquad \text{(A-4-19)}$$

$$c_x = 0.6 + \frac{T}{250} \le 3.0 \text{ where } T \text{ is in °C} \qquad \text{(A-4-19M)}$$

and

T = elevated temperature of steel due to unintended fire, °F (°C)

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, and the k_p and k_y coefficients are calculated in accordance with Table A-4.2.1, and other terms are as defined in Chapter F.

User Note: $F_L(T)$ represents the initial yield stress, which assumes a residual stress of $0.3F_y$. Alternatively, 10 ksi (69 MPa) may be used in place of $0.3F_y$ for calculation of $F_L(T)$.

User Note: The equations for lateral-torsional buckling do not consider local buckling. If applicable, the effects of local buckling must be considered with an alternative method.

(f) Design for Flexure in Composite Beams

For composite beams, the calculated bottom flange temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25% from the mid-depth of the web to the top flange of the beam.

The nominal strength of a composite flexural member shall be determined using the provisions of Chapter I, with reduced yield stresses in the steel as determined from Table A-4.2.1. Steel properties will vary as the temperature along the depth of section changes.

Alternatively, the nominal flexural strength of a composite beam, $M_n(T)$, is permitted to be calculated using the bottom flange temperature, T, as follows:

$$M_n(T) = r(T) M_n \qquad \text{(A-4-20)}$$

where,

M_n = nominal flexural strength at ambient temperature calculated in accordance with provisions of Chapter I, kip-in. (N-mm)

$r(T)$ = retention factor depending on bottom flange temperature, T, as given in Table A-4.2.4

TABLE A-4.2.4
Retention Factor for Composite Flexural Members

Bottom Flange Temperature, °F (°C)	$r(T)$
68 (20)	1.00
300 (150)	0.98
600 (320)	0.95
800 (430)	0.89
1000 (540)	0.71
1200 (650)	0.49
1400 (760)	0.26
1600 (870)	0.12
1800 (980)	0.05
2000 (1100)	0.00

(g) Design for Shear

The nominal strength for shear yielding shall be determined in accordance with the provisions of Chapter G, with steel properties as stipulated in Section 4.2.3b(a) and assuming a uniform temperature over the cross section.

User Note: Shear yielding equations do not consider shear buckling or tension field action. If applicable, these limit states must be considered with an alternative method.

(h) Design for Combined Forces and Torsion

The nominal strength for combinations of axial force and flexure about one or both axes, with or without torsion, shall be in accordance with the provisions of Chapter H with the design axial and flexural strengths as stipulated in Sections 4.2.4d(a) to (d). Nominal strength for torsion shall be determined in accordance with the provisions of Chapter H, with the steel properties as stipulated in Section 4.2.3b(a), assuming uniform temperature over the cross section.

4e. Design by Critical Temperature Method

The critical temperature of a structural member is the temperature at which the demand on the member exceeds its capacity under fire conditions. The evaluation methods in this section are permitted to be used in lieu of Section 4.2.4d for tension members, continuously braced beams not supporting concrete slabs, or compression members that are assumed to be simply supported and develop a uniform temperature over the cross section throughout the fire exposure.

The use of the critical temperature methods shall be limited to steel members with wide-flange rolled shapes which have nonslender elements per Section B4.

(a) Design for Tensile Yielding

The critical temperature of a tension member is permitted to be calculated as follows:

$$T_{cr} = 816 - 306 \ln\left(\frac{R_u}{R_n}\right) \text{ in °F} \quad \text{(A-4-21)}$$

$$T_{cr} = 435 - 170 \ln\left(\frac{R_u}{R_n}\right) \text{ in °C} \quad \text{(A-4-21M)}$$

where

T_{cr} = critical temperature in °F (°C)
R_n = nominal yielding strength at ambient temperature determined in accordance with the provisions in Section D2, kips (N)
R_u = required tensile strength at elevated temperature, determined using the load combination in Equation A-4-1 and greater than $0.01R_n$, kips (N)

User Note: Tensile rupture in the net section is not considered in this critical temperature calculation and ought to be considered using alternative methods.

(b) Design for Compression

The critical temperature of a compression member is permitted to be calculated as follows:

$$T_{cr} = 1580 - 0.814\left(\frac{L_c}{r}\right) - 1300\left(\frac{P_u}{P_n}\right) \text{ in °F} \quad \text{(A-4-22)}$$

$$T_{cr} = 858 - 0.455\left(\frac{L_c}{r}\right) - 722\left(\frac{P_u}{P_n}\right) \text{ in °C} \quad \text{(A-4-22M)}$$

where

L_c = effective length of member, in. (mm)
r = radius of gyration, in. (mm)
P_n = nominal compressive strength at ambient temperature determined in accordance with the provisions in Section E3, kips (N)
P_u = required compressive strength at elevated temperature, determined using the load combination in Equation A-4-1, kips (N)

(c) Design for Flexural Yielding

The critical temperature of a continuously braced beam not supporting a concrete slab is permitted to be calculated as follows:

$$T_{cr} = 816 - 306 \ln\left(\frac{M_u}{M_n}\right) \text{ in °F} \quad \text{(A-4-23)}$$

$$T_{cr} = 435 - 170 \ln\left(\frac{M_u}{M_n}\right) \text{ in °C} \quad \text{(A-4-23M)}$$

where

T_{cr} = critical temperature in °F (°C)

M_n = nominal flexural strength due to yielding at ambient temperature determined in accordance with the provisions in Section F2.1, kip-in. (N-mm)

M_u = required flexural strength at elevated temperature, determined using the load combination in Equation A-4-1, kip-in. and greater than $0.01M_n$ (N-mm)

User Note: Lateral-torsional buckling of beams is not considered in this critical temperature calculation and ought to be considered using alternative methods.

4.3. DESIGN BY QUALIFICATION TESTING

1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. Demonstration of compliance with these requirements using the procedures specified for steel construction in Section 5 of *Standard Calculation Methods for Structural Fire Protection* (ASCE/SEI/SFPE 29) is permitted. It is also permitted to demonstrate equivalency to such standard fire resistance ratings using the advanced analysis methods in Section 4.2 in combination with the fire exposure specified in ASTM E119 as the design-basis fire.

User Note: There are other standard fire exposures which are more severe than that prescribed in ASTM E119, for example the hydrocarbon pool fire scenario defined in ASTM E1529 (UL 1709). Fire resistance ratings developed on the basis of ASTM E119 are not directly substitutable for such more demanding conditions.

The generic steel assemblies described in Table A-4.3.1 shall be deemed to have the fire resistance ratings prescribed therein.

Table A-4.3.1[a] Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e]

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-1.1	Carbonate, lightweight and sand-lightweight aggregate concrete, members 6 in. × 6 in. or greater (not including sandstone, granite and siliceous gravel).[a]	2-1/2	2	1-1/2	1
	1-1.2	Carbonate, lightweight and sand-lightweight aggregate concrete, members 8 in. × 8 in. or greater (not including sandstone, granite and siliceous gravel).[a]	2	1-1/2	1	1
	1-1.3	Carbonate, lightweight and sand-lightweight aggregate concrete, members 12 in. × 12 in. or greater (not including sandstone, granite and siliceous gravel).[a]	1-1/2	1	1	1
	1-1.4	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 6 in. × 6 in. or greater.[a]	3	2	1-1/2	1
	1-1.5	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 8 in.× 8 in.or greater.[a]	2-1/2	2	1	1
	1-1.6	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 12 in.× 12 in or greater.[a]	2	1	1	1
	1-2.1	Clay or shale brick with brick and mortar fill.[a]	3-3/4	–	–	2-1/4

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses						
	1-4.1	Cement plaster over metal lath wire tied to 3/4 in. cold-rolled vertical channels with 0.049 in. (No. 18 B.W. gage) wire ties spaced 3 to 6 in. on center. Plaster mixed 1:2.5 by volume, cement to sand.	–	–	2-1/2[b]	7/8

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-5.1	Vermiculite concrete, 1:4 mix by volume over paperbacked wire fabric lath wrapped directly around column with additional 2 × 2 in. 0.065 / 0.065 in. (No. 16/16 B.W. gage) wire fabric placed 3/4 in. from outer concrete surface. Wire fabric tied with 0.049 in. (No. 18 B.W. gage) wire spaced 6 in. on center for inner layer and 2 in. on center for outer layer.	2	–	–	–
	1-6.1	Perlite or vermiculite gypsum plaster over metal lath wrapped around column and furred 1-1/4 in. from column flanges. Sheets lapped at ends and tied at 6 in. intervals with 0.049 in. (No. 18 B.W. gage) tie wire. Plaster pushed through to flanges.	1-1/2	1	–	–
	1-6.2	Perlite or vermiculite gypsum plaster over self-furring metal lath wrapped directly around column, lapped 1 in. and tied at 6 in. intervals with 0.049 in. (No. 18 B.W. gage) wire.	1-3/4	1-3/8	1	–
	1-6.3	Perlite or vermiculite gypsum plaster on metal lath applied to 3/4 in. cold-rolled channels spaced 24 in. apart vertically and wrapped flatwise around column.	1-1/2	–	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-6.4	Perlite or vermiculite gypsum plaster over two layers of 1/2 in. plain full-length gypsum lath applied tight to column flanges. Lath wrapped with 1 in. hexagonal mesh of No. 20 gage wire and tied with doubled 0.035 in. diameter (No. 18 B.W. gage) wire ties spaced 23 in. on center. For three-coat work, the plaster mix for the second coat shall not exceed 100 pounds of gypsum to 2.5 cubic feet of aggregate for the 3-hour system.	2-1/2	2	–	–
	1-6.5	Perlite or vermiculite gypsum plaster over one layer of 1/2 in. plain full-length gypsum lath applied tight to column flanges. Lath tied with doubled 0.049 in. (No. 18 B.W. gage) wire ties spaced 23 in. on center and scratch coat wrapped with 1 in. hexagonal mesh 0.035 in. (No. 20 B.W. gage) wire fabric. For three-coat work, the plaster mix for the second coat shall not exceed 100 pounds of gypsum to 2.5 cubic feet of aggregate.	–	2	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-7.1	Multiple layers of 1/2 in. gypsum wallboard[c] adhesively[d] secured to column flanges and successive layers. Wallboard applied without horizontal joints. Corner edges of each layer staggered. Wallboard layer below outer layer secured to column with doubled 0.049 in. (No. 18 B.W. gage) steel wire ties spaced 15 in. on center. Exposed corners taped and treated.	–	–	2	1
	1-7.2	Three layers of 5/8 in. Type X gypsum wallboard.[c] First and second layer held in place by 1/8 in. dia. by 1-3/8 in. long ring shank nails with 5/16 in. dia. heads spaced 24 in. on center at corners. Middle layer also secured with metal straps at mid-height and 18 in. from each end, and by metal corner bead at each corner held by the metal straps. Third layer attached to corner bead with 1 in. long gypsum wallboard screws spaced 12 in. on center.	–	–	1-7/8	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-7.3	Three layers of 5/8 in. Type X gypsum wallboard,[c] each layer screw attached to 1-5/8 in. steel studs, 0.018 in. thick (No. 25 carbon sheet steel gage) at each corner of column. Middle layer also secured with 0.049 in. (No. 18 B.W. gage) double-strand steel wire ties, 24 in. on center. Screws are No. 6 by 1 in. spaced 24 in. on center for inner layer, No. 6 by 1-5/8 in. spaced 12 in. on center for middle layer and No. 8 by 2-1/4 in. spaced 12 in. on center for outer layer.	–	1-7/8	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-9.1	Minimum W8×35 wide flange steel column (w/d ≥ 0.75) with each web cavity filled even with the flange tip with normal weight carbonate or siliceous aggregate concrete (3,000 psi minimum compressive strength with 145 pcf ± 3 pcf unit weight). Reinforce the concrete in each web cavity with minimum No. 4 deformed reinforcing bar installed vertically and centered in the cavity, and secured to the column web with minimum No. 2 horizontal deformed reinforcing bar welded to the web every 18 in. on center vertically. As an alternative to the No. 4 rebar, 3/4 in. diameter by 3 in. long headed studs, spaced at 12 in. on center vertically, shall be welded on each side of the web midway between the column flanges.	–	–	–	See Note f
2. Webs or flanges of steel beams and girders	2.1-1	Carbonate, lightweight and sand-lightweight aggregate concrete (not including sandstone, granite and siliceous gravel) with 3 in. or finer metal mesh placed 1 in. from the finished surface anchored to the top flange and providing not less than 0.025 in.2 of steel area per foot in each direction.	2	1-1/2	1	1

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-1.2	Siliceous aggregate concrete and concrete excluded in Item 2-1.1 with 3 in. or finer metal mesh placed 1 in. from the finished surface anchored to the top flange and providing not less than 0.025 in.^{2} of steel area per foot in each direction.	2-1/2	2	1-1/2	1
	2-2.1	Cement plaster on metal lath attached to 3/4 in. cold-rolled channels with 0.04 in. (No. 18 B.W. gage) wire ties spaced 3 in. to 6 in. on center. Plaster mixed 1:2.5 by volume, cement to sand.	–	–	2-1/2[b]	7/8
	2-3.1	Vermiculite gypsum plaster on a metal lath cage, wire tied to 0.165 in. diameter (No. 8 B.W. gage) steel wire hangers wrapped around beam and spaced 16 in. on center. Metal lath ties spaced approximately 5 in. on center at cage sides and bottom.	–	7/8	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-4.1	Two layers of 5/8 in. Type X gypsum wallboard[c] are attached to U-shaped brackets spaced 24 in. on center. 0.018 in. thick (No. 25 carbon sheet steel gage) 1-5/8 in. deep by 1 in. galvanized steel runner channels are first installed parallel to and on each side of the top beam flange to provide a 1/2 in. clearance to the flange. The channel runners are attached to steel deck or concrete floor construction with approved fasteners spaced 12 in. on center. U-shaped brackets are formed from members identical to the channel runners. At the bent portion of the U-shaped bracket, the flanges of the channel are cut out so that 1-5/8 in. deep corner channels can be inserted without attachment parallel to each side of the lower flange. As an alternative, 0.021 in. thick (No. 24 carbon sheet steel gage) 1 in. × 2 in. runner and corner angles shall be used in lieu of channels, and the web cutouts in the U-shaped brackets shall not be required. Each angle is attached to the bracket with 1/2-in.-long No. 8 self-drilling screws. The vertical legs of the U-shaped bracket are attached to the runners with one 1/2 in. long No. 8 self-drilling screw. The completed steel framing provides a 2-1/8 in. and 1-1/2 in. space between the inner layer of wallboard and the sides and bottom of the steel beam, respectively. The inner layer of wallboard is attached to the top runners and bottom corner channels or corner angles with 1-1/4 in.-long No. 6 self-drilling screws spaced 16 in. on center. The outer layer of wallboard is applied with 1-3/4 in.-long No. 6 self-drilling screws spaced 8 in. on center. The bottom corners are reinforced with metal corner beads.	–	–	1-1/4	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

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Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-4.2	Three layers of 5/8 in. Type X gypsum wallboard[c] attached to a steel suspension system as described immediately above utilizing the 0.018 in. thick (No. 25 carbon sheet steel gage) 1 in. × 2 in. lower corner angles. The framing is located so that a 2-1/8 in. and 2 in. space is provided between the inner layer of wallboard and the sides and bottom of the beam, respectively. The first two layers of wallboard are attached as described immediately above. A layer of 0.035 in. thick (No. 20 B.W. gage) 1 in. hexagonal galvanized wire mesh is applied under the soffit of the middle layer and up the sides approximately 2 in. The mesh is held in position with the No. 6 1-5/8-in.-long screws installed in the vertical leg of the bottom corner angles. The outer layer of wallboard is attached with No. 6 2-1/4 in.-long screws spaced 8 in. on center. One screw is also installed at the mid-depth of the bracket in each layer. Bottom corners are finished as described above.	–	1-7/8	–	–

[a] Reentrant parts of protected members to be filled solidly.
[b] Two layers of equal thickness with a 3/4-in. airspace between.
[c] For all of the construction with gypsum wallboard, gypsum base for veneer plaster of the same size, thickness and core type is permitted to be substituted for gypsum wallboard, provided attachment is identical to that specified for the wallboard, the joints on the face layer are reinforced, and the entire surface is covered with not less than 1/16-inch gypsum veneer plaster.
[d] An approved adhesive qualified under ASTM E119.
[e] Generic fire-resistance ratings (those not designated as PROPRIETARY* in the listing) in GA 600 shall be accepted as if herein listed.

[a] ICC IBC-2018 *International Building Code,* International Code Council.

2. Structural Steel Assemblies

The provisions of this section contain procedures by which the standard fire-resistance ratings of structural steel assemblies are established by calculations. Use of these provisions is permitted in place of and/or as a supplement to published fire resistive assemblies based on ASTM E119. The installation of the fire protection material shall comply with the applicable requirements of the building code, the referenced approved assemblies, and manufacturer instructions.

The weight-to-heated-perimeter ratios (*W/D*) and area-to-heated-perimeter ratios (*A/P*) shall be determined in accordance with the definitions given in this section. As used in these sections, *W* is the average weight of a shape in pounds per linear foot and *A* is the area in square inches. The heated perimeter, *D* or *P*, is the inside perimeter of the fire-resistant material or exterior contour of the steel shape in inches, as defined for each type of member.

User Note: These procedures establish a basis for determining the fire resistance rating of steel construction assemblies as a function of the thickness of fire-resistant material, the weight, *W*, or area, *A*, and the applicable heated perimeter, *D* or *P*, of the fire protection material or structural steel member. The *W/D* and *A/P* ratios are equivalent and mutually convertible section properties that represent their thermal inertia. *W/D* has conventionally been used for open wide-flange shapes, while *A/P* has been used for closed hollow structural sections.

The heated perimeter, *D* or *P*, is a function of the configuration of the steel fire protection material installation, which can be in either a contour or box profile, together with the nature of the heat exposure on the steel member. The latter is typically characterized as either an all-around exposure of the steel shape, as for an interior column, or as a 3-sided exposure of a floor beam supporting a concrete floor. Tabulations of *W/D* and *A/P* values for these cases and for the standard rolled steel shapes are available from multiple sources, including AISC Design Guide 19 (a free download for members from www.aisc/org/dg) and other publications.

2.1 Steel Columns

The fire-resistance ratings of columns shall be based on the size of the member and the type of protection provided in accordance with this section.

The application of these procedures for noncomposite steel column assemblies shall be limited to designs in which the fire-resistant material is not designed to carry any of the load acting on the column.

Mechanical, electrical, and plumbing elements shall not be embedded in required fire-resistant materials, unless fire-endurance test results are available to establish the adequacy of the resulting condition.

User Note: The International Building Code requires fire resistance rated columns to be protected on all sides for the full column height, including connections with other structural members and protection continuity through any ceilings to the top of the column.

2.1.1 Gypsum Wallboard Protection

The fire resistance of columns with weight-to-heated perimeter ratios (*W/D*) less than or equal to 3.65 lb/ft/in. and protected with Type X gypsum wallboard is permitted to be determined from the following expression for a maximum column rating of 4-hours:

$$R = 130\left[\frac{h\left(\frac{W'}{D}\right)}{2}\right]^{0.75} \qquad \text{(A-4-24)}^{[a],[b]}$$

$$R = 96\left[\frac{h\left(\frac{W'}{D}\right)}{2}\right]^{0.75} \qquad \text{(A-4-24M)}^{[a]}$$

where

D = inside heated perimeter of the gypsum board, in. (mm)
R = fire resistance, minutes
W = nominal weight of steel shape, lb/ft (kg/m)
h = total nominal thickness of Type X gypsum wallboard, in. (mm)

and

$$\frac{W'}{D} = \frac{W}{D} + \frac{50h}{144} \qquad \text{(A-4-25)}^{[a],[b]}$$

$$\frac{W'}{D} = \frac{W}{D} + 0.0008h \qquad \text{(A-4-25M)}^{[a]}$$

For columns with weight-to-heated-perimeter ratios (*W/D*) greater than 3.65 lb/ft/in., the thickness of Type X gypsum wallboard required for specified fire-resistance ratings shall be the same as the thickness determined for *W/D* = 3.65 lb/ft/in.

User Note: This equation has been developed and long used for steel column fire protection with any Type X gypsum board. Since Type C gypsum board has demonstrated improved fire performance relative to Type X board, these provisions may also be conservatively applied to column protection with any Type C gypsum board. The supporting test data and accompanying gypsum board installation methods limit the computed fire resistance rating of the steel column to a maximum of 3-hours or 4-hours, as specified in the next section.

The gypsum board or gypsum panel products shall be installed and supported as required either in UL X526 for fire-resistance ratings of four hours or less, or in UL X528 for fire-resistance ratings of three hours or less.

User Note: The attachment of the Type X gypsum board protection for the steel columns must be done in accordance with the referenced UL assem-

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[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.

blies. UL X526 is applicable only when exterior steel covers are installed over the gypsum board. Otherwise, UL X528 describes the more general gypsum board installation.

2.1.2 Sprayed and Intumescent/Mastic Fire-Resistant Materials

The fire resistance of columns protected with sprayed or intumescent/mastic fire-resistant coatings shall be determined on the basis of standard fire-resistance rated assemblies, any associated computations and limits as provided in the applicable rated assemblies.

The fire resistance of wide-flange columns protected with sprayed fire-resistant materials is permitted to be determined as:

$$R = \left[C_1 \left(\frac{W}{D} \right) + C_2 \right] h \qquad \text{(A-4-26)}^{[a],[b]}$$

$$R = \left[C_3 \left(\frac{W}{D} \right) + C_4 \right] h \qquad \text{(A-4-26M)}^{[a]}$$

where

R = fire resistance, minutes
h = thickness of sprayed fire-resistant material, in.
D = heated perimeter of the column, in.
C_1, C_2 , C_3, and C_4= material-dependent constants prescribed in specified rated assembly.
W = weight of columns, pounds per linear foot

The material dependent constants, C_1, C_2, C_3, and C_4 shall be determined for specific fire-resistant materials on the basis of standard fire endurance tests. The computational usage for each correlation, protection product and its material-dependent constants shall be limited to the range of their underlying fire test basis reflected in the selected rated assembly.

User Note: The fire resistance rated steel column assemblies, published by UL and by other test laboratories, will often include such interpolation equations and specific constants that depend on the particular fire protection product. The applicability limits of each given design correlation relative to the column assembly, sprayed fire-resistant protection product, *W/D*, rating duration, minimum required thickness, and the like must be followed to remain within the range of the existing fire test data range.

The fire resistance of HSS columns protected with sprayed fire-resistant materials is permitted to be determined from empirical correlations similar to Equation A-4-25 expressed in terms of *A/P* values, wherein *A* is the area in in.^{2} (mm^2) and *P* is the heated perimeter. The applicability limits specified in the rated column assembly for each correlation and its material-dependent constants shall be followed.

User Note: *A/P* is a directly convertible and equivalent steel section property to *W/D* which has traditionally been used in fire resistive compu-

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.

tations for HSS sections. Similar to *W/D* for open wide flange shapes, tabulation of *A/P* values for standard closed shapes with contour and box protection applications are available from multiple sources, including AISC and the published literature. The applicability limits of each given design correlation relative to the column assembly, sprayed fire-resistant protection product, *A/P*, rating duration, minimum required thickness, and the like must be followed to remain within the range of the existing fire test result range .

2.1.3 Noncomposite Columns Encased in Concrete

The fire resistance of noncomposite columns fully encased within concrete protection is permitted to be determined from the following expression:

$$R = R_o\left(1 + 0.03m\right) \quad \text{(A-4-27)}^{[a],[b]}$$

where

$$R_o = 10\left(\frac{W}{D}\right)^{0.7} + 17\left(\frac{h^{1.6}}{k_c^{\,0.2}}\right) \times \left\{1 + 26\left[\frac{H}{p_c c_c h\left(L+h\right)}\right]^{0.8}\right\} \quad \text{(A-4-28)}^{[a],[b]}$$

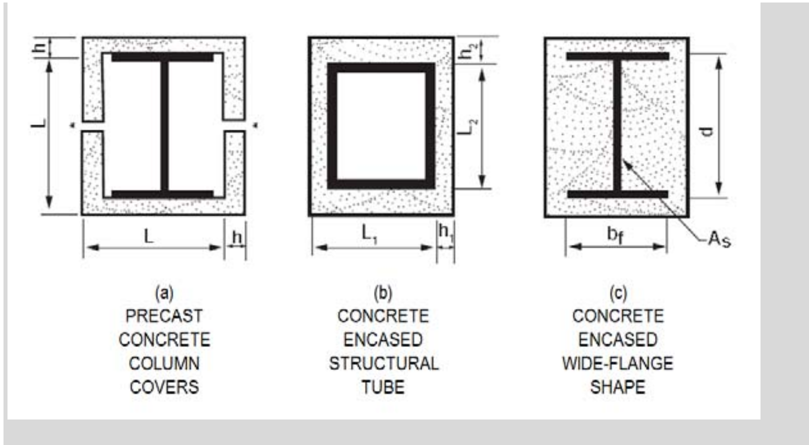
$$R_o = 73\left(\frac{W}{D}\right)^{0.7} + 0.162\frac{h^{1.6}}{k_c^{0.2}}\left\{1 + 31{,}000\left[\frac{H}{p_c c_c h(L+h)}\right]^{0.8}\right\} \quad \text{(A-4-28M)}^{[a]}$$

R = fire endurance at equilibrium moisture conditions, minutes
R_o = fire endurance at zero moisture content, minutes
m = equilibrium moisture content of the concrete by volume, percent
W = average weight of the column, lb/ft (kg/m)
D = heated perimeter of the column, in. (mm)
h = thickness of the concrete cover, measured between the exposed concrete and nearest outer surface of the encased steel column section, in. (mm)
k_c = ambient temperature thermal conductivity of the concrete, Btu/hr ft °F. (W/m K)
H = ambient temperature thermal capacity of the steel column, Btu/ ft °F (W/kJ m K)
= $0.11W$ ($0.46\,W$)
p_c = concrete density, lb/ft^3 (kg/m^3)
c_c = ambient temperature specific heat of concrete, Btu/lb °F (kJ/kg K)
L = interior dimension of one side of a square concrete box protection, in. (mm)

When the inside perimeter of the concrete protection is not square, L shall be taken as the average of its two rectangular side lengths (L_1 and L_2). If the thickness of the concrete cover is not constant, h shall be taken as the average of h_1 and h_2.

User Note: The variables in these equations are illustrated in the figure

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.



For wide-flange columns completely encased in concrete with all reentrant spaces filled, the thermal capacity of the concrete within the reentrant spaces is permitted to be added to the ambient thermal capacity of the steel column, as follows:

$$H = 0.11W + \left(\frac{p_c c_c}{144}\right)\left(b_f d - A_s\right) \qquad \text{(A-4-29)}^{[a],[b]}$$

$$H = 0.46W + \left(\frac{p_c c_c}{1{,}000{,}000}\right)\left(b_f d - A_s\right) \qquad \text{(A-4-29M)}^{[a]}$$

where:

b_f = flange width of the column, in. (mm)

d =depth of the column, in. (mm)

A_s = area of the steel column, in.^{2} (mm^2)

User Note: It is conservative to neglect this additional concrete term in the column fire resistance calculation.

In the absence of more specific data for the ambient properties of the concrete encasement, it is permitted to use the values provided in Table A-4.3.2.

Table A-4.3.2[c]

Ambient Properties of Concrete Encasement for Steel Column Fire Resistance

Property	Normal Weight Concrete	Light Weight Concrete
Thermal conductivity, k_c	0.95 Btu/hr ·ft ·°F (1.64 W/m K)	0.35 Btu/hr · ft ·°F (0.61 W/m K)
Specific heat, c_c	0.20 Btu/lb °F (840 J/kg K)	0.20 Btu/lb °F (840 J/kg K)

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*

[b] ICC IBC-2018 *International Building Code,* International Code Council.

[c] ICC IBC-2018 *International Building Code,* International Code Council.

Density, p_c	145 lb/ft^3 (2300 kg/m^3)	110 lb/ft^3 (1800 kg/m^3)
Equilibrium (free) moisture content (m) by volume	4%	5%

User Note: The estimated free moisture content of concrete given in Table A-4.3.2 may not be appropriate for all conditions, particularly for older concrete that has already been in service for a longer time. For these and similar situations of uncertainty, it is conservative to not rely on this beneficial effect of the free moisture and to assume the concrete is completely dry with m=0 for fire resistance of R_o.

2.1.4 Noncomposite Columns Encased in Masonry Units of Concrete or Clay

The fire resistance of noncomposite columns protected by encasement with concrete masonry units or with clay masonry units is permitted to be determined from the following expression:

$$R = 0.17\left(\frac{W}{D}\right)^{0.7} + \left[0.285\left(\frac{T_e^{1.6}}{K^{0.2}}\right)\right]\left\{1.0 + 42.7\left[\frac{\left(A_s/d_m T_e\right)}{\left(0.25p + T_e\right)}\right]^{0.8}\right\} \quad \text{(A-4-30)}^{[a]}$$

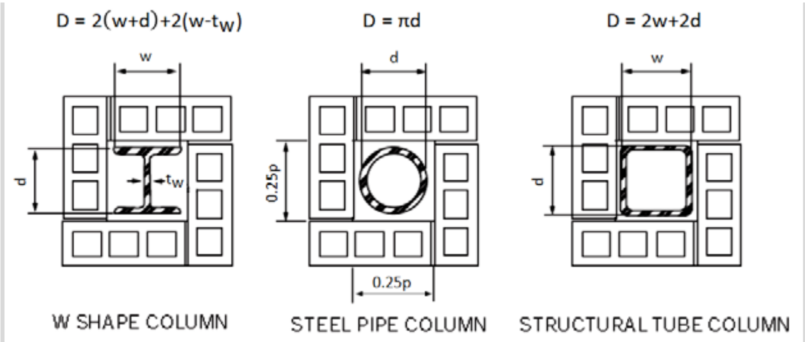
$$R = 1.22\left(\frac{W}{D}\right)^{0.7} + \left[0.0027\left(\frac{T_e^{1.6}}{K^{0.2}}\right)\right]\left\{1.0 + 1249\left[\frac{\left(\frac{A_s}{d_m T_e}\right)}{(0.25p + T_e)}\right]^{0.8}\right\} \quad \text{(A-4-30M)}$$

where

R = fire-resistance rating of column assembly, hours
W = average weight of column, lb/ft (kg/m)
D = heated perimeter of column, in. (mm)
T_e = equivalent thickness of concrete or clay masonry unit, in accordance with ACI 216.1, in. (mm)
K = thermal conductivity of concrete or clay masonry unit, Btu/hr · ft · °F (see Table A-4.3.3).
A_s = cross-sectional area of column, in.^{2} (mm^2)
d_m = density of the concrete or clay masonry unit, lb/ft^3 (kg/m^3) p = inner perimeter of concrete or clay masonry protection, in. (mm)

The thermal conductivity values given in Table A-4.3.3 as a function of the concrete or clay masonry unit density is permitted for use with this encasement protection formulation.

User Note: Equation A-4-30 is derived from Equation A-4-27 assuming m = 0, c_c = 0.2 Btu/lb °F, h = T_e, and L = p/4. The following cross-sections illustrate three different configurations for concrete masonry units or clay masonry unit encasement of steel columns, along with the applicable fire protection design variables.



d = depth of a wide flange column, outside diameter of pipe column, or outside dimension of hollow structural section column, in. (mm)
t_w = thickness of web of wide flange column, in. (mm)
w = width of flange of wide flange or hollow structural section, in. (mm)

Table A-4.3.3[a]
Thermal Conductivity of Masonry Units for Steel Column Encasement

Unit Density, d_m, lb/ft^3 (kg/m^3)	Unit Thermal Conductivity K, Btu/hr ft °F (W/m K)
Concrete Masonry Units	
80 (1280)	0.207 (0.358)
85 (1360)	0.228 (0.395)
90 (1440)	0.252 (0.436)
95 (1520)	0.278 (0.481)
100 (1600)	0.308 (0.533)
105 (1680)	0.340 (0.589)
110 (1760)	0.376 (0.651)
115 (1840)	0.416 (0.720)
120 (1920)	0.459 (0.795)
125 (2000)	0.508 (0.879)
130 (2080)	0.561 (0.971)
135 (2160)	0.620 (1.07)
140 (2240)	0.685 (1.19)
145 (2320)	0.758 (1.31)
150 (2400)	0.837 (1.45)
Clay Masonry Units	
120 (1920)	1.25 (2.16)
130 (2080)	2.25 (3.89)

2.2 Composite Steel-Concrete Columns

The fire resistance rating of columns acting compositely with concrete (concrete-filled or encased) is permitted to be based on the size of the composite member and concrete protection in accordance with this section.

2.2.1 Concrete-Filled Columns

The fire resistance rating of hollow structural section (HSS) columns filled with unreinforced normal weight concrete, steel-fiber-reinforced normal

[a] ICC IBC-2018 *International Building Code,* International Code Council.

weight concrete or bar-reinforced normal weight concrete is permitted to be determined in accordance with the following expressions:

(A-4-31)[a]

$$R = \frac{0.58a(f_c' + 2.9)D^2\left(\frac{D}{C}\right)^{0.5}}{L_c - 3.28}$$

(A-4-31M)[a]

$$R = \frac{a(f_c' + 20)D^2\left(\frac{D}{C}\right)^{0.5}}{[60(L_c - 1000)]}$$

R = fire resistance rating in hours
a = constant determined from Table A-4.3.4
f'_c = 28-day compressive strength of concrete, ksi(MPa)
L_c = column effective length, ft (mm)
D = outside diameter for circular columns, in. (mm)
= outside dimension for square columns, in. (mm)
= least outside dimension for rectangular columns, in. (mm)
C = compressive force due to unfactored dead load and live load, kips (kN)

The application of these equations shall be limited by all of the following conditions:

1. The required fire resistance rating R shall be less than or equal to the limits specified in Tables A-4.3.5 or A-4.3.5M.
2. The specified compressive strength of concrete, f'_c, the column effective length, L_c, the dimension D, the concrete reinforcement ratio, and the thickness of the concrete cover shall be within the limits specified in Tables A-4.3.5 or A-4.3.5M.
3. C shall not exceed the design strength of the concrete or the reinforced concrete core determined in accordance with this Specification.
4. Two minimum 1/2 in.(12.7 mm) diameter holes shall be placed opposite each other at the top and bottom of the column and at maximum 12-ft on center spacing along the column height. Each set of vent holes should be rotated 90° relative to the adjacent set of holes to relieve steam pressure.

User Note: Concrete-filled hollow structural sections (HSS) can effectively sustain load during a fire exposure without benefit of any external protection for the steel HSS. The concrete infill mass provides both an increased capacity for absorbing the heat caused by the fire and loadbearing strength to thereby extend the column fire resistance duration. Research conducted at the National Research Council of Canada has provided a basis for establishing an empirical equation to predict the standard fire resistance of concrete-filled round and square HSS section for commonly used story heights and steel sections. This empirical equation was derived from and can only be used within the allowable range of design variables, as given, and is not applicable to lightweight concrete infill.

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*

The fire performance of a concrete-filled HSS column is improved when heat absorption occurs as the moisture in the concrete is converted to steam. The heat absorbed during this phase change is significant, however the resulting steam must be released to prevent the adverse effects of an internal pressure build-up within the HSS column. Thus, vent holes must be provided in the steel section, as indicated in the given limitation #4.

Table A-4.3.4 Values of Constant *a* for Normal Weight Concrete				
Aggregate Type	**Concrete Fill Type**	**Reinf. Ratio (%)**	***a***	
			Circular Columns	**Sq. or Rect. Columns**
siliceous	unreinforced	NA	0.070	0.060
siliceous	steel-fiber-reinforced	2 %	0.075	0.065
siliceous	steel-bar-reinforced	1.5 – 3	0.080	0.070
		3 – 5	0.085	0.070
carbonate	unreinforced	NA	0.080	0.070
carbonate	steel-fiber-reinforced	2	0.085	0.075
carbonate	steel-bar-reinforced	1.5 - 3	0.090	0.080
		3 – 5	0.095	0.085

Table A-4.3.5 Limits for the use of Equation A-4.25 Parameters			
Parameter	**Concrete Fill Type**		
	Unreinforced	**steel-fiber-reinforced**	**steel-bar reinforced**
R (hours)	≤ 2	≤ 3	≤ 3
fc’ (ksi)	2.9 – 5.8	2.9 – 8.0	2.9 – 8.0
L_c (ft)	6.5 – 13.0	6.5 – 15.0	6.5 – 15.0
D (round) (in)	5.5 – 16.0	5.5 – 16.0	6.5 – 16.0
D (sq. or rect.) (in)	5.5 – 12.0	4.0 – 12.0	7.0 – 12.0
Reinf. (%)	NA	2% of concrete mix by mass	1.5 – 5% of section area
Concrete cover (in)	NA	NA	≥ 1.0

Table A-4.3.5M. Limits for the use of Equation A-4.25MParameters			
Parameter	**Concrete Fill Type**		
	unreinforced	**steel-fiber-reinforced**	**steel-bar-reinforced**
R (hours)	≤ 2	≤ 3	≤ 3
fc’ (MPa)	20 – 40	20-55	20-55
L_c (mm)	2000 – 4000	2000 - 4500	2000 - 4500
D (round) (mm)	140 – 410	140 - 410	165 - 410
D (sq. or rect.) (mm)	140 – 305	102 - 305	175 - 305
Reinf. (%)	NA	2% of concrete mix by mass	1.5 – 5% of section area
Concrete cover (mm)	NA	NA	≥ 25

2.2.2 Composite Columns Encased in Concrete

The fire resistance of composite columns fully encased within normal weight or lightweight concrete and with no unfilled spaces is permitted to be determined as the lesser of Equation A-4-30 and the values in Table A-4.3.6.

Table A-4.3.6 Minimum size and concrete cover limits for fire resistance of composite steel columns encased in concrete with no unfilled spaces

Fire Resistance Rating, hrs	Minimum Concrete Cover, *h*, in. (mm)	Minimum Column Outside Dimension, in. (mm)
1	1 (25)	8 (200)
2	2 (50)	10 (250)
3	2 (50)	12 (300)
4	2 (50)	14 (350)

User Note: The fire resistance ratings and requirements in Table A-4.3.7 were directly adapted from the ACI 216.1 provisions for conventional bar-reinforced concrete columns. Substitution of an embedded structural steel shape for steel bar reinforcement should not reduce the fire resistance of the loadbearing concrete parts of column, and the R computed for the same but assumed non-composite steel column accordingly verifies the fire resistance of the loadbearing steel shape. The concrete cover, *h*, is defined identical to that used for non-composite steel columns encased in concrete.

2.3 I-Shaped Beams and Girders

The fire-resistance ratings of beams and girders shall be based upon the size of the element and the type of protection provided in accordance with this section.

These procedures establish a basis for determining resistance of structural steel beams and girders that differ in size from that specified in approved fire-resistance-rated assemblies as a function of the thickness of fire-resistant material and the weight (W) and heated perimeter (D) of the beam or girder.

The beams provided in approved fire-resistance-rated assemblies shall be considered to be the minimum permissible size. Other beam or girder shapes is permitted to be substituted provided that the weight-to-heated-perimeter ratio (W/D) of the substitute beam is equal to or greater than that of the minimum beam specified in the approved assembly.

User Note: In the past, the substitution of larger beams for the minimum required sizes has been permitted based upon the thickness of web and flange elements, W/D ratio, or the beam size designation. Extensive fire research has shown that the heat transfer to a protected steel beam or girder is actually a direct function of the W/D ratio. As a result, beam substitutions should be more directly based upon W/D ratios. The significance of the thickness of web and flange elements and beam size is inherently included in the determination of W/D ratios.
It is acceptable and conservative to protect a larger steel beam or girder, which has a greater W/D value than the W/D of the minimum member size

specified in an approved assembly, with the thickness of fire protection material required for the minimum member size.

2.3.1 Sprayed and Intumescent/Mastic Fire-Resistant Materials

The provisions in this section apply to beams and girders protected with sprayed or intumescent/mastic fire-resistant materials.

Larger or smaller beam and girder shapes protected with sprayed fire-resistant materials are permitted to be substituted for beams specified in approved unrestrained or restrained fire-resistance-rated assemblies, provided that the thickness of the fire-resistant material is adjusted in accordance with the following expression:

$$h_2 = h_1\,[(W_1 / D_1) + 0.60] / [(W_2 / D_2) + 0.60] \qquad \text{(A-4-32)}^{[a],[b]}$$

$$h_2 = h_1[(W_1/D_1) + 0.036]/[(W_2/D_2) + 0.036] \qquad \text{(A-4-32M)}^{[a]}$$

where:
h = thickness of sprayed fire-resistant material, in. (mm)
W = weight of the beam or girder, lb/ft (kg/m)
D = heated perimeter of the beam, in. (mm)

Subscript 1 refers to the substitute beam or girder and the required thickness of fire-resistant material.
Subscript 2 refers to the beam and fire-resistant material thickness in the approved assembly.

The use of this Equation is limited to the following conditions:

1. The weight-to-heated-perimeter ratio for the substitute beam or girder (W_1/D_1) shall be not less than 0.37 (customary units) or 0.022 (SI units).
2. The thickness of fire protection materials calculated for the substitute beam or girder (T_1) shall be not less than 3/8 in. (10 mm).
3. The unrestrained or restrained beam rating shall be not less than 1 hour.
4. Where used to adjust the material thickness for a restrained beam, the use of this procedure is limited to sections classified as compact.

User Note: This substitution equation based on W/D for beams protected with spray-applied fire resistive materials was developed by UL with the given limitations. The minimum *W/D* ratio of 0.37 prevents the use of this equation for determining the fire resistance of very small shapes that have not been tested. The 3/8-in. (10 mm) minimum thickness of protection is a practical application limit based upon the most commonly used spray-applied fire protection materials.

The fire resistance of beams and girders protected with intumescent or mastic fire-resistant coatings shall be determined on the basis of standard

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*.
[b] ICC IBC-2018 *International Building Code,* International Code Council.

fire-resistance rated assemblies, and associated computations and limits as provided in the applicable rated assemblies.

2.4 Trusses

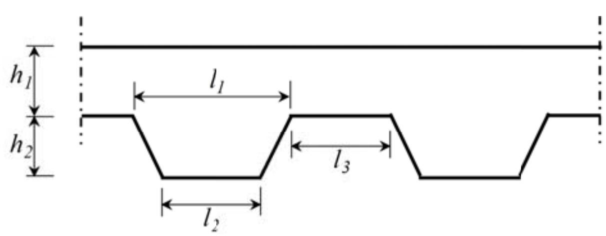
The fire resistance of trusses with members individually protected by fire-resistant materials applied onto each of the individual truss elements is permitted to be determined for each member in accordance with the Appendix 4, Section 4.3.1. The protection thickness of truss elements that can be simultaneously exposed to fire on all sides shall be determined for the same weight-to-heated perimeter ratio (*W/D*) as columns. The protection thickness of truss elements that directly support floor or roof assembly is permitted to be determined for the same weight-to-heated-perimeter ratio (*W/D*) as for beams and girders.

User Note: For trusses, application of the column fire resistance equation is more technically correct than the beam equation, since truss members are predominantly axially loaded and will require larger protection thicknesses than beams. Also, most truss elements can be exposed to fire on all four sides simultaneously. As a result, the heated perimeter and protection thickness of most truss members should be determined in the same manner as for columns. However, an exception is included for top chord elements that directly support floor or roof construction. The heated perimeter and protection thickness of such elements may be determined in the same manner as for beams and girders, or they may be conservatively determined in the same manner as for columns.

2.5 Concrete Floor Slabs on Steel Deck

For composite concrete floor slabs on trapezoidal steel decking wherein the upper width of the deck rib is equal to or greater than its bottom rib width, the fire resistance rating, based on the thermal insulation criterion for the unexposed surface temperature, shall be permitted to be calculated using the following equation:

$$R = a_0 + a_1 h_1 + a_2 h_2 + a_3 l_2 + a_4 l_3 + a_5 m + a_6 h_1^2 + a_7 h_1 h_2 + a_8 h_1 l_2 + a_9 h_1 l_3 + a_{10} h_1 m + a_{11} h_2 l_2 + a_{12} h_2 l_3 + a_{13} h_2 m + a_{14} l_2 l_3 + a_{15} l_2 m + a_{16} l_3 m \qquad \text{(A-4-33)}$$



where

R = fire resistance rating in minutes
h_1 = concrete slab thickness above steel deck, in (mm)
h_2 = depth of steel deck, in (mm)
l_1 = largest upper width of deck rib, in (mm)
l_2 = bottom width of deck rib, in (mm)
l_3 = width of deck upper flange, in (mm)

m = moisture content of the concrete slab. Range of applicability is between 0% (0.0) and 10% (0.1)

The coefficients a_0 to a_{16} are shown in Table A-4.3.7.

TABLE A-4.3.7 Coefficients a_0 to a_{16} for use with Equation A-4-33

Coefficient	Coefficient Value	
	Normal-weight concrete	Lightweight concrete
a_0	38.6 min	68.7 min
a_1	-5.08 min/in (-0.2 min/mm)	-36.58 min/in (-1.44 min/mm)
a_2	−1.45 min/in (−0.057 min/mm)	-2.79 min/in (-0.11 min/mm)
a_3	−3.30 min/in (−0.13 min/mm)	−12.70 min/in (−0.5 min/mm)
a_4	−2.08 min/in (−0.082 min/mm)	20.07 min/in (0.79 min/mm)
a_5	-118.1 min	-784.2 min
a_6	4.06 min/in^2 (0.0063 min/mm^2)	8.84 min/in^2 (0.0137 min/mm^2)
a_7	1.48 min/in^2 (0.0023 min/mm^2)	3.61 min/in^2 (0.0056 min/mm^2)
a_8	1.87 min/in^2 (0.0029 min/mm^2)	3.68 min/in^2 (0.0057 min/mm^2)
a_9	0	-2.39 min/in^2 (-0.0037 min/mm^2)
a_{10}	263.1 min/in (10.36 min/mm)	444.5 min/in (17.5 min/mm)
a_{11}	1.16 min/in^2 (0.0018 min/mm^2)	2.06 min/in^2 (0.0032 min/mm^2)
a_{12}	0	-3.42 min/in^2 (-0.0053 min/mm^2)
a_{13}	0	91.44 min/in (3.6 min/mm)
a_{14}	-0.65 min/in^2 (-0.001 min/mm^2)	-0.97 min/in^2 (-0.0015 min/mm^2)
a_{15}	0	42.42 min/in (1.67 min/mm)
a_{16}	0	-66.04 min/in (-2.6 min/mm)

User Note: If moisture content values are not available, m = 4% and 5% can be used for normal-weight concrete and lightweight concrete, respectively, consistent with Annex D of Eurocode 4. Dry conditions (m = 0%) will yield the most conservative fire resistance rating.

2.6 Composite Plate Shear Walls

For unprotected composite plate shear walls meeting the requirements of Chapter I and Section 4.3.2.6, the fire resistance rating is permitted to be determined in accordance with Equation A-4-34.

$$R = \left[-18.5\left(\frac{P_u}{P_n}\right)^{\left(0.24-\frac{L/t_{sc}}{230}\right)} + 15\right]\left(\frac{1.9t_{sc}}{8} - 1\right) \qquad \text{(A-4-34)}$$

$$R = \left[-18.5\left(\frac{P_u}{P_n}\right)^{\left(0.24-\frac{L/t_{sc}}{230}\right)} + 15\right]\left(\frac{1.9t_{sc}}{200} - 1\right) \qquad \text{(A-4-34M)}$$

where R is the fire rating in hours, P_u is the applied axial load in kips (kN), and L, t_{sc}, and P_n are as defined in Chapter I.

The use of Equation A-4-34 shall be limited to walls satisfying all the following conditions:

1. Wall slenderness ratio (L/t_{sc}) is less than or equal to 20
2. Axial load ratio (P_u/P_n) is less than or equal to 0.2
3. Wall thickness, t_{sc}, is between 8 in. and 24 in. (200 mm and 600 mm)

3. Restrained Construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting forces and accommodating deformations caused by thermal expansion throughout the range of anticipated elevated temperatures. Cast-in-place or prefabricated concrete floor or roof construction secured to steel framing members, and individual steel beams and girders that are welded or bolted to integral framing members shall be considered restrained construction.

4. Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist effects of elevated temperatures.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.