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February 25, 2011

Dear Vadim

World Trade Center Transportation Hub
Structural Fire Engineering Analysis

This letter summarizes our review and recommendations regarding the possible omission of applied structural fireproofing on portions of the steel portal frame assemblies in the WTC Transit Hall. This letter and our analysis are applicable only to portions of these members 33' to 47' above the Transit Hall elevation of 274', as shown in Attachment A.

Analysis of Temperatures near Structural Members

We have reviewed our previous computational fluid dynamics (CFD) analysis results for a fire occurring at the Lower Concourse (el. 274') beneath the balcony overhang of the Upper concourse (el. 296') in order to determine expected maximum temperature exposures for the portal frame structural members. We have extended that analysis to review the thermal impacts of additional credible fire locations – namely fires originating at the upper concourse level and at the lower concourse level directly beneath the lowest point of the unprotected portion of the portal frame assemblies.

The 2,000 kW design fires used in the analysis are in accordance with the white paper *Design Fire Size for Transportation Hub other than Platform Areas*. No attenuation of the fire heat release rate due to sprinkler spray has been accounted for in this analysis. Also, all model surfaces were assumed adiabatic in order to maximize the resulting air temperatures.

Attachment B to this letter provides the results of our review of temperatures near the steel portal frame assemblies. The maximum temperature observed near the surface of these members in the range of elevations where omission of intumescent fireproofing is proposed is 105°C (221°F) resulting from an axi-symmetric fire located directly below the members on the Lower Concourse (see Figure B.4).

The average steel temperature can be expected to approach the ambient air temperature adjacent to the member over a long duration fire, though there will be a time delay or lag. Assuming that the steel temperature is equal to the adjacent air temperature introduces a factor of safety during the early portions of a fire. Given the assumption that the fire does not undergo a decay phase such that the heat release rate remains constant over the duration of the potential exposure, the average steel temperature will approach but not exceed the maximum predicted air/smoke temperatures.

Effects of Elevated Temperatures on Steel Members

The AISC *Specification for Structural Steel Buildings* (2010), Appendix 4, Table A-4.2.1, states that the strength of structural steel does not change over a temperature range from ambient to approximately 400°C (750°F). Above this temperature, the strength begins to decrease. The figure below shows the strength of steel as a function of temperature.

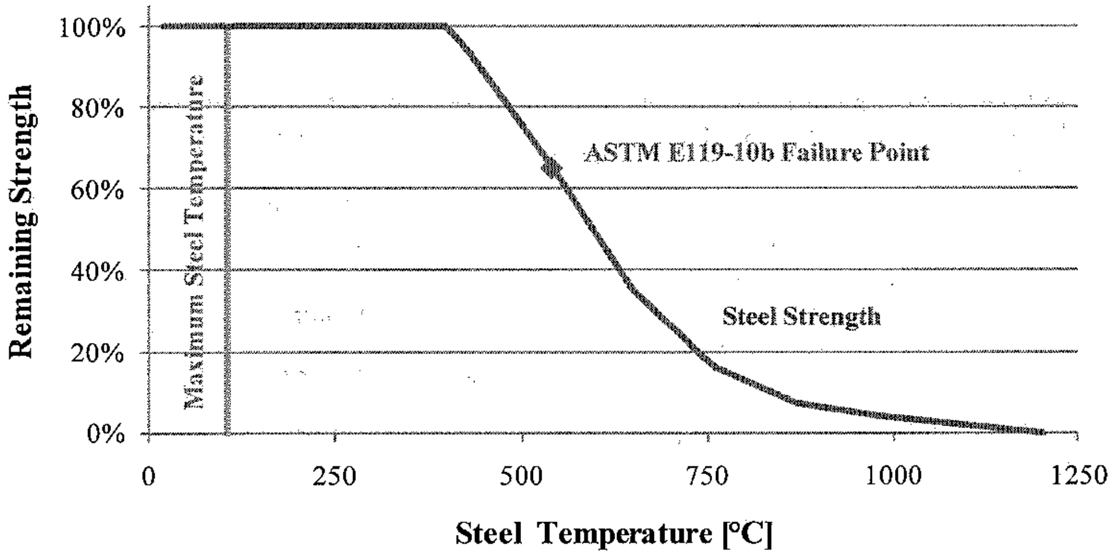


Figure 1 – Steel Strength as a Function of Steel Temperature

The vertical green line in this figure denotes the maximum predicted air/smoke temperature adjacent to the steel: 105°C. As discussed previously, the average temperature of the steel will not exceed this predicted maximum. At 105 °C, the steel is expected to retain up to 100% of its strength. For reference, the average temperature failure criteria for steel columns established by ASTM E119-10b (2010), 538°C (1000°F), is also shown.

Conclusions

Based on the analysis discussed here, the strength of the steel portal frame members is not expected to be reduced as a result of the calculated fire exposures. For all fire scenarios considered, a factor of safety of at least 3.8 is maintained relative to the point at which steel strength begins to deteriorate (400 °C). When compared to ASTM failure criteria (538 °C), a factor of safety of at least 5.1 is provided. The analysis, therefore, justifies the omission of applied fire protective coatings on the portions of the steel portal frames more than 33' to 47' above the Transit Hall elevation of 274'. Portions of the portal frame assemblies above the entrance levels at grade level will require separate consideration.

Yours sincerely,

Jarrod Alston
Associate

David Jacoby
Associate Principal

Attachment A – Extent of Omission of Intumescent Fireproofing

The image below depicts areas of the steel portal frame assemblies from which omission of intumescent fireproofing is proposed. This image also indicates areas where sprinkler protection is provided.

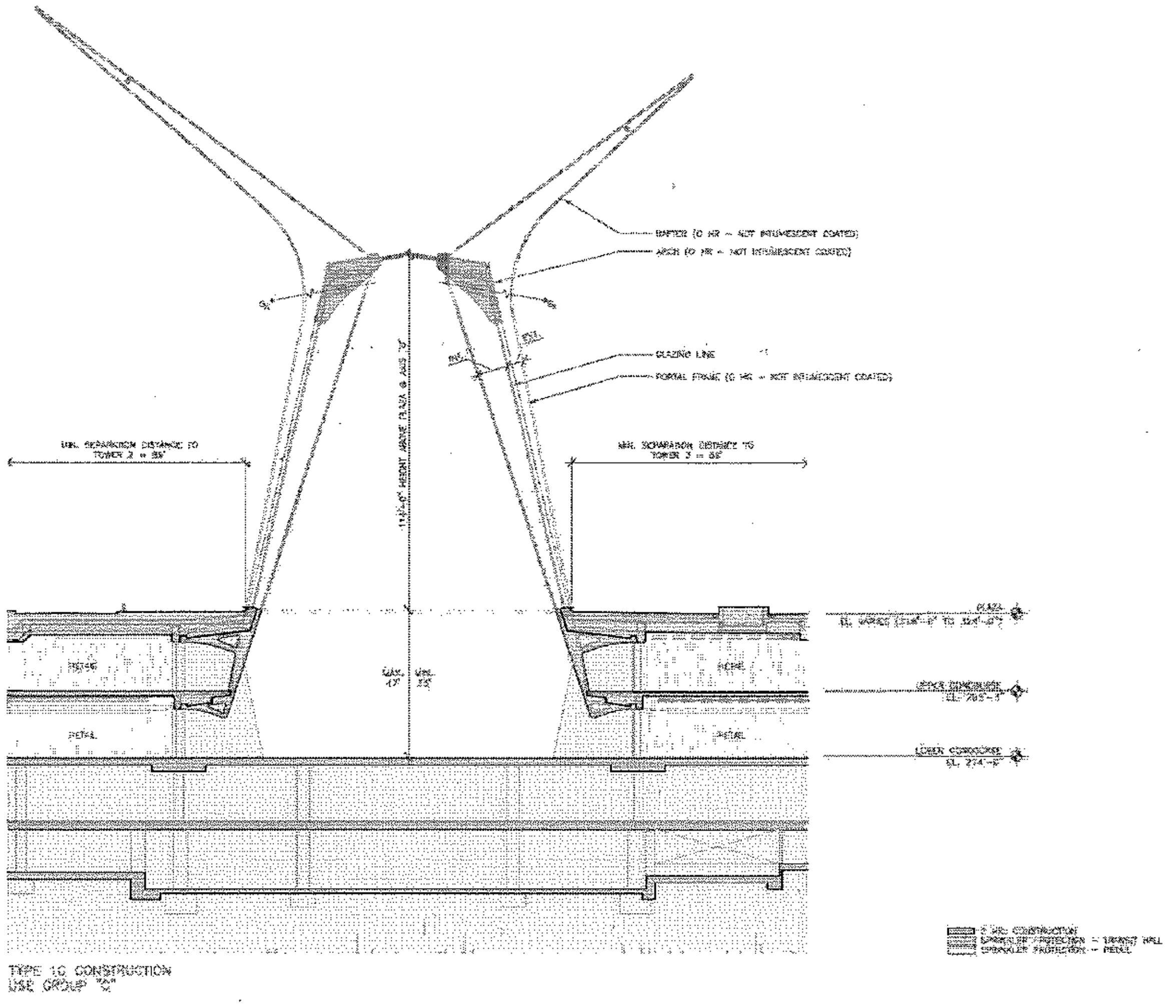


Figure A.1. Proposed Extent of Omission of Fireproofing

Attachment B – Analysis of Temperatures near Structural Members

The chart below depicts air/smoke temperatures at a range of elevations directly above a fire located on the Lower Concourse (274'-0" elevation). The structural members for which omission of applied fireproofing is proposed occur between 10.0 and 14.5 m above the Lower Concourse.

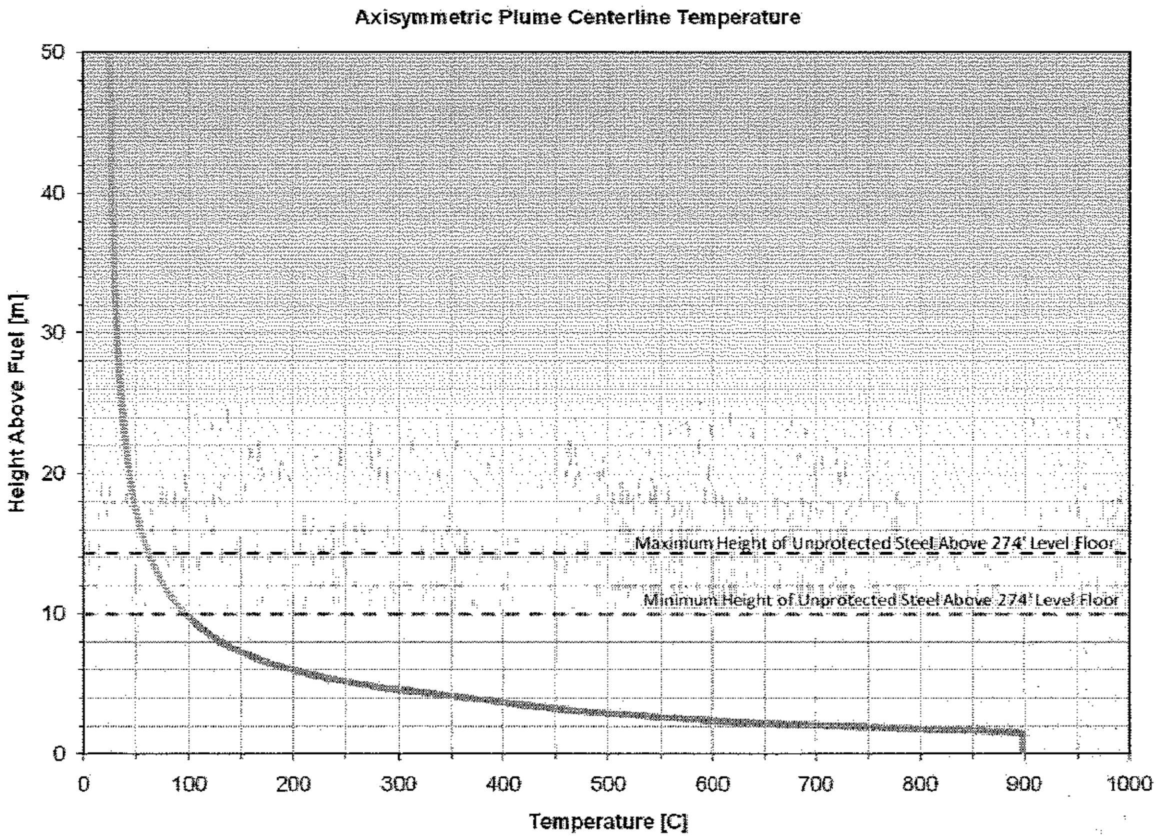


Figure B.1. Predicted Centerline Plume Temperatures above a 2,000 kW Axi-Symmetric Fire Located at the Lower Concourse

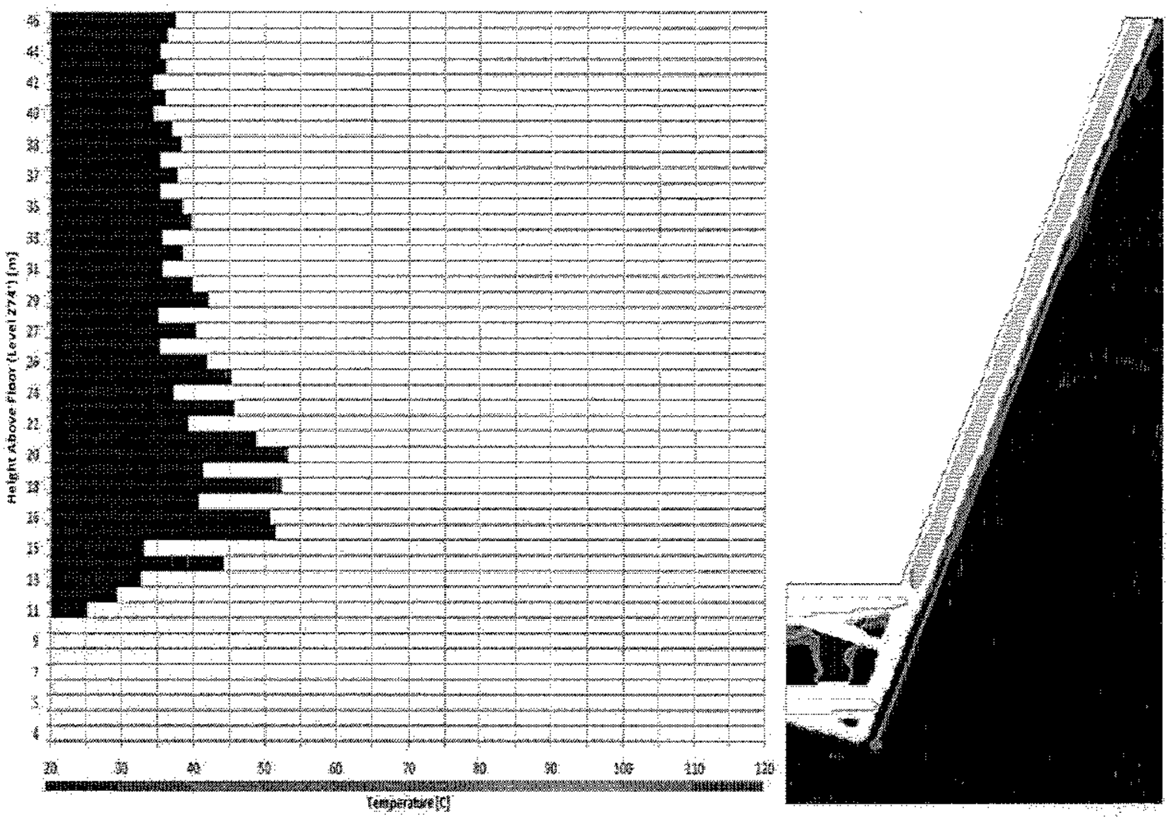


Figure B.2. Air/Smoke Temperatures Derived from CFD Modeling Near Portal Frame for 2,000 kW Balcony Spill Fire at the Upper Concourse
(the orange ray indicates the approximate line of temperature sampling points)

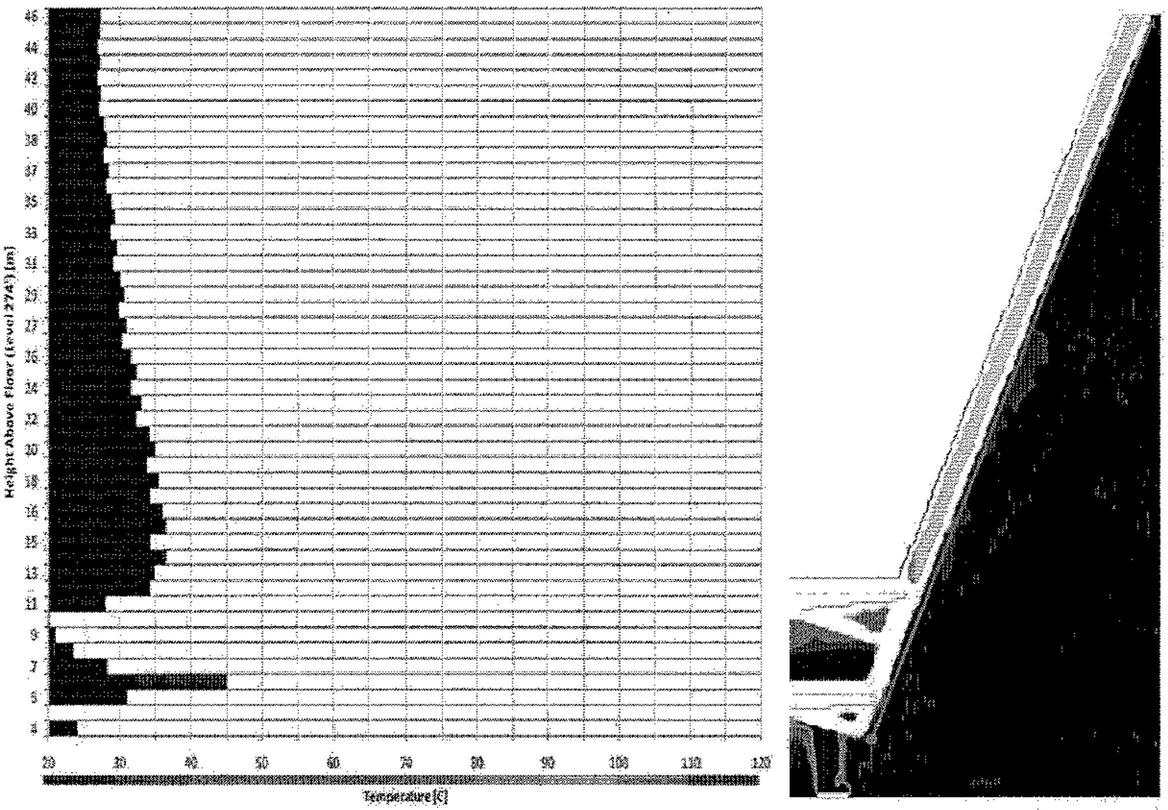


Figure B.3. Air/Smoke Temperatures Derived from CFD Modeling Near Portal Frame for 2,000 kW Balcony Spill Fire at Lower Concourse
(the orange ray indicates the approximate line of temperature sampling points)

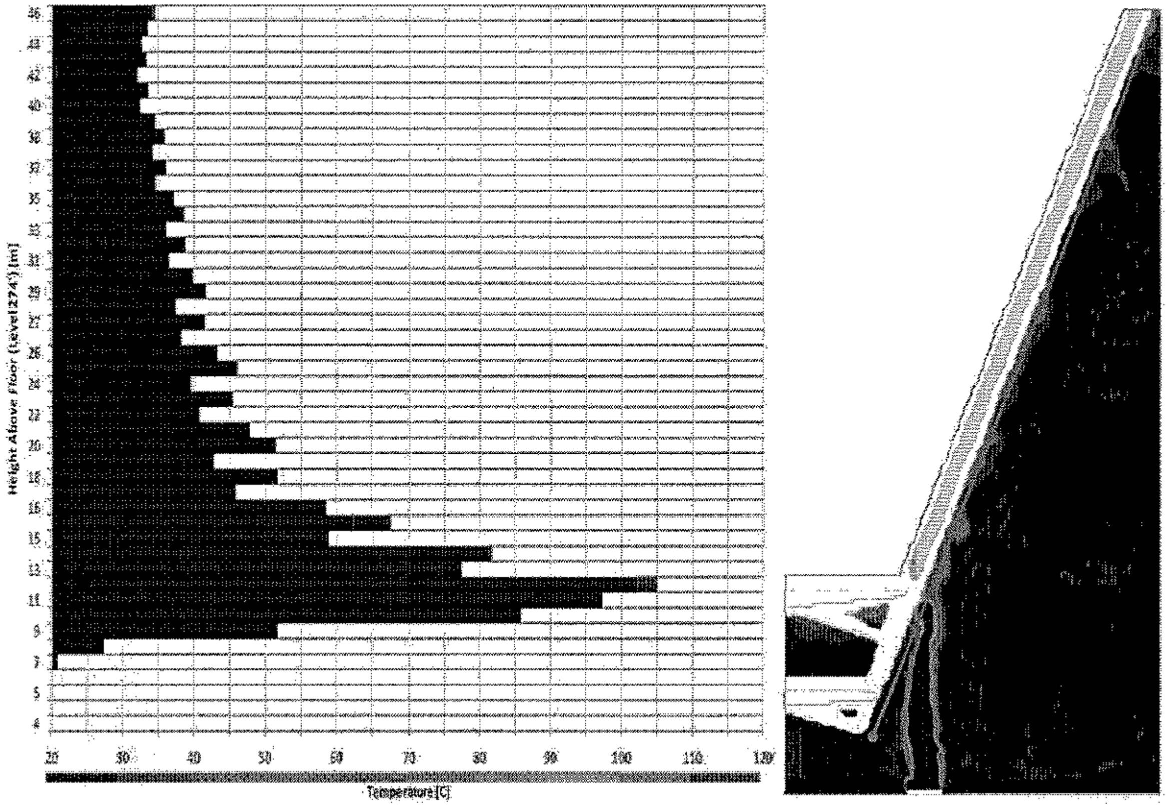


Figure B.4. Air/Smoke Temperatures Derived from CFD Modeling Near Portal Frame for a 2,000 kW Axi-Symmetric Fire at the Lower Concourse
(the orange ray indicates the approximate line of temperature sampling points)

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1. GENERAL PROVISIONS

Appendix 4 provides structural engineers with criteria for designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Additional guidance is provided in this Commentary. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

4.1.1. Performance Objective

The performance objective underlying the provisions in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of the building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements serving as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the above general performance objective and limit states. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

4.1.2. Design by Engineering Analysis

The strength design criteria for steel beams and columns at elevated temperatures have been revised from the 2005 *Specification for Structural Steel Buildings* (AISC, 2005a) to reflect recent research (Tagaki and Deierlein, 2007). These strength equations do not transition smoothly to the strength equations used to design steel members under ambient conditions. The practical implications of the discontinuity are minor, as the temperatures in the structural members during a fully developed fire are far in excess of the temperatures at which this discontinuity might otherwise be of concern in design. Nevertheless, to avoid the possibility of misinterpretation, the scope of applicability of the analysis methods in Section 4.2 of Appendix 4 is limited to temperatures above 400 °F (204 °C).

Structural behavior under severe fire conditions is highly nonlinear in nature as a result of the constitutive behavior of materials at elevated temperatures and the relatively large deformations that may develop in structural systems at sustained elevated temperatures. As a result of this behavior, it is difficult to develop design equations to ensure the necessary level of structural performance during severe fires using elastically based ASD methods. Accordingly, structural design for fire conditions by analysis should be performed using LRFD methods, in which the nonlinear structural actions arising during severe fire exposures and the temperature-dependent design strengths can be properly taken into account.

4.1.4. Load Combinations and Required Strength

Fire safety measures are aimed at three levels: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F|D,I)\ P(D|I)\ P(I) \tag{C-A-4-1}$$

where $P(I)$ = probability of ignition, $P(D|I)$ = probability of development of a structurally significant fire, and $P(F|D,I)$ = probability of failure, given the occurrence of the two preceding events. Measures taken to reduce $P(I)$ and $P(D|I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact the term $P(F|D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation C-A-4-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos et al., 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10^{-5} to 10^{-4} per year. For redundant steel frame systems, $P(F)$ is on the order of 10^{-6} to 10^{-5}. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is on the order of 10^{-7} to 10^{-6} per year (Pate-Cornell, 1994). If $P(I)$ is on the order of 10^{-4} per year for typical buildings and $P(D|I)$ is on the order of 10^{-2} for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F|D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is similar to Equation 2.5-1 that appears in ASCE/SEI 7-10 (ASCE, 2010), where the probabilistic bases for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise, the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

The overall stability of the structural system is checked by considering the effect of a small notional lateral load equal to 0.2% of the story gravity force, as defined in Section C2.2, acting in combination with the gravity loads. The required strength of the structural component or system designed using load combination A-4-1 is on the order of 60% to 70% of the required strength under full gravity or wind load at normal temperature.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

4.2.1. Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. These relations may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, connections and edge details can be specified to provide a structure that is sufficiently robust.

4.2.1.1. Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a steady-state fire may be assumed.

4.2.1.2. Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces; for example, those with an open (or exposed) floor area in excess of 5,000 ft^2 (465 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook of Fire Protection Engineering* (SFPE, 2002).

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with a floor area in excess of 5,000 ft^2 (465 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This clause should not limit the use of temperature-time relationships to those where duration is calculated, as stated above, as these tend to be localized fires and external fire.

4.2.1.3. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

4.2.1.4. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60% [Eurocode 1 (CEN, 1991)]. The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability; for example, reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002a), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002b).

4.2.2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a "lumped heat capacity analysis" where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated mid-point of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis shall consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire resistive materials in the form of insulation, heat screens, or other protective measures shall be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected Steel Members. The temperature rise in an unprotected steel section in a short time period is determined by:

$$\Delta T_s = \frac{a}{c_s\left(\dfrac{W}{D}\right)}\left(T_F - T_s\right)\Delta t \qquad \text{(C-A-4-2)}$$

The heat transfer coefficient, a, is determined from

$$a = a_c + a_r \qquad \text{(C-A-4-3)}$$

where

a_c = convective heat transfer coefficient

a_r = radiative heat transfer coefficient, given as:

$$a_r = \frac{5.67\times10^{-8}\varepsilon_F}{T_F - T_S}\left(T_F^4 - T_s^4\right) \qquad \text{(C-A-4-4)}$$

TABLE C-A-4.1
Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Embedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width-to-beam depth ratio ≥ 0.5	0.5
Flange width-to-beam depth ratio < 0.5	0.7
Box girder and lattice girder	0.7

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 25 W/m^2-°C [4.4 Btu/(ft^2-hr-°F)]. The parameter, ε_F, accounts for the emissivity of the fire and the view factor. Estimates for ε_F, are suggested in Table C-A-4.1.

For accuracy reasons, a maximum limit for the time step, Δt, is suggested as 5 s.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2009d) for building fires or ASTM E1529 (ASTM, 2006) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words, where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted which determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s W/D > 2 d_p \rho_p c_p \tag{C-A-4-5}$$

Then, Equation C-A-4-6 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \left(\dfrac{W}{D}\right)} \left(T_F - T_s\right) \Delta t \tag{C-A-4-6}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-5 is not satisfied), then Equation C-A-4-7 should be applied:

$$\Delta T_s = \frac{k_p}{d_p}\left[\frac{T_F - T_s}{c_s\left(\dfrac{W}{D}\right) + \dfrac{c_p \rho_p d_p}{2}}\right]\Delta t \qquad \text{(C-A-4-7)}$$

The maximum limit for the time step, Δt, should be 5 s.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a mid-range temperature expected for that component. For protected steel members, the material properties may be evaluated at 572 °F (300 °C), and for protection materials, a temperature of 932 °F (500 °C) may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s\left(\dfrac{W}{D}\right)}\Delta t \qquad \text{(C-A-4-8)}$$

where q'' is the net heat flux incident on the steel member.

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

(1) Exposure conditions established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is dependent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.
(2) Temperature-dependent material properties.
(3) Temperature variation within the steel member and any protection components, especially where the exposure varies from side-to-side.

Nomenclature:

D = heat perimeter, in. (m)
T = temperature, °F (°C)
W = weight (mass) per unit length, lb/ft (kg/m)
a = heat transfer coefficient, Btu/ft^2-sec-°F (W/m^2-°C)

c = specific heat, Btu/lb·°F (J/kg-°C)
d = thickness, in. (m)
k = thermal conductivity, Btu/ft-sec-°F (W/m-°C)
Δt = time interval, s
ρ = density, lb/ft^3 (kg/m^3)

***Subscripts*:**
c = convection
p = fire protection material
r = radiation
s = steel

4.2.3. Material Strengths at Elevated Temperatures

The properties for steel and concrete at elevated temperatures are adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), Section III.2, "Material Properties." These generic properties are consistent with those in Eurocode 3 (CEN, 2005) and Eurocode 4 (CEN, 2003), and reflect the consensus of the international fire engineering and research community. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Kirby and Preston (1988).

The stress-strain response of steel at elevated temperatures is more nonlinear than at room temperature and experiences less *strain hardening*. As shown in Figure C-A-4.1, at elevated temperatures the deviation from linear behavior is represented by the proportional limit, $F_p(T)$, and the yield strength, $F_y(T)$, is defined at a 2% strain. At 1,000 °F (538 °C), the yield strength, $F_y(T)$, reduces to about 66% of its value at room temperature, and the proportional limit $F_p(T)$ occurs at 29% of the elevated temperature yield strength $F_y(T)$. Finally, at

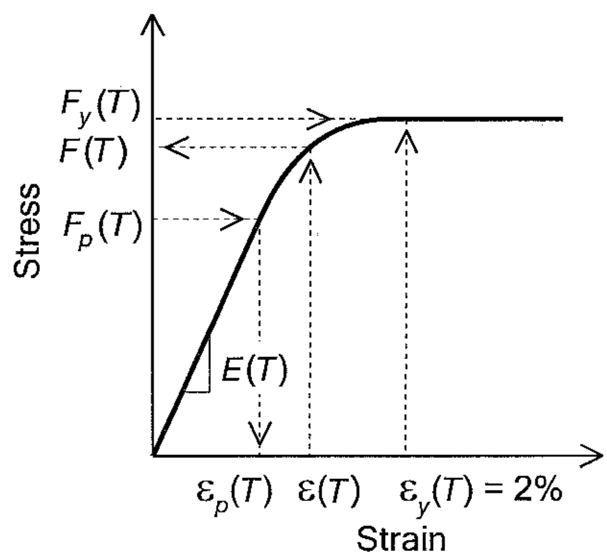


Fig. C-A-4.1. Parameters of idealized stress-strain curve at elevated temperatures (Takagi and Deierlein, 2007).

temperatures above 750 °F (399 °C), the elevated temperature ultimate strength is essentially the same as the elevated temperature yield strength; in other words, $F_y(T)$ is equal to $F_u(T)$.

4.2.4. Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(a) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated
(b) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities
(c) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation, and geometric nonlinearity are considered

4.2.4.1. General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2010). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 to Section 1.4 of ASCE (2010) contains guidelines for the provision of general structural integrity. Compartmentation (subdivision of buildings/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

4.2.4.2. Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent elements and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation as well as the overall load bearing capacity of the structural system is maintained.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered.

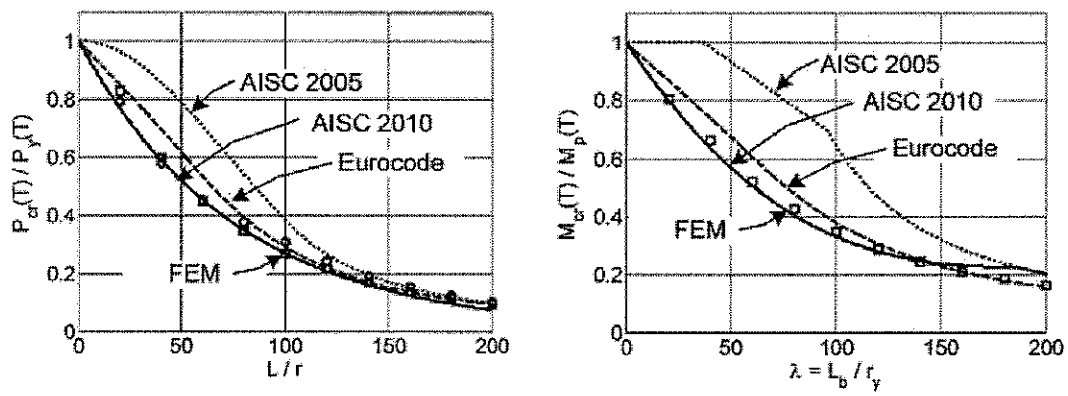
4.2.4.3b. Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column.

In the 2005 Specification, nominal member strengths at elevated temperatures were calculated using the standard strength equations of the Specification with steel properties (E, F_y and F_u) reduced for elevated temperatures by appropriate factors. Recent research (Takagi and Deierlein, 2007) has shown this procedure to over-estimate considerably the strengths of members that are sensitive to stability effects. To reduce these unconservative errors, new equations, developed by Takagi and Deierlein (2007) are introduced in the 2010 edition of the Specification to more accurately calculate the strength of compression members subjected to flexural buckling and flexural members subjected to lateral-torsional buckling. As shown in Figure C-A-4.2, the 2010 Specification equations are much more accurate in comparison to detailed finite element method analyses (represented by the square symbol in the figure), which have been validated against test data, and to equations from the Eurocode (ECCS, 2001).

4.2.4.4. Design Strength

The design strength for structural steel members and connections is calculated as ϕR_n, in which R_n = nominal strength, when the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is computed as in Chapters C through K and Appendix 4 of the Specification, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2. While ECCS (2001) and Eurocode 1 (CEN, 1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Accordingly, the resistance factors herein are the same as those at ordinary conditions.



(a) Compression strength *(b) Flexural strength*

Fig. C-A-4.2 Comparison of compression and flexural strengths at 500 °C (932 °F) (Takagi and Deierlein, 2007).

4.3. DESIGN BY QUALIFICATION TESTING

4.3.1. Qualification Standards

Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. Fire resistance ratings of building elements are generally determined in accordance with procedures set forth in ASTM E119, *Standard Test Methods for Fire Tests of Building Construction and Materials* (ASTM, 2009d). Tested building element designs, with their respective fire resistance ratings, may be found in special directories and reports published by testing agencies. Additionally, calculation procedures based on standard test results may be used as specified in *Standard Calculation Methods for Structural Fire Protection* (ASCE, 2005a).

For building elements that are required to prevent the spread of fire, such as walls, floors and roofs, the test standard provides for measurement of the transmission of heat. For loadbearing building elements, such as columns, beams, floors, roofs and loadbearing walls, the test standard also provides for measurement of the load-carrying ability under the standard fire exposure.

For beam, floor and roof specimens tested under ASTM E119, two fire resistance classifications—restrained and unrestrained—may be determined, depending on the conditions of restraint and the acceptance criteria applied to the specimen.

4.3.2. Restrained Construction

The ASTM E119 standard provides for tests of loaded beam specimens only in the restrained condition, where the two ends of the beam specimen (including slab ends for composite steel-concrete beam specimens) are placed tightly against the test frame that supports the beam specimen. Therefore, during fire exposure, the thermal expansion and rotation of the beam specimen ends are resisted by the test frame. Similar restrained condition is provided in the ASTM E119 tests on restrained loaded floor or roof assemblies, where the entire perimeter of the assembly is placed tightly against the test frame.

The practice of restrained specimens dates back to the early fire tests (over 100 years ago), and it is predominant today in the qualification of structural steel framed and reinforced concrete floors, roofs and beams in North America. While the current ASTM E119 standard does provide for an option to test loaded floor and roof assemblies in the unrestrained condition, this testing option is rarely used for structural steel and concrete. However, unrestrained loaded floor and roof specimens, with sufficient space around the perimeter to allow for free thermal expansion and rotation, are common in the tests of wood and cold-formed-steel framed assemblies.

Gewain and Troup (2001) provide a detailed review of the background research and practices in the qualification fire resistance testing and rating of structural steel (and composite steel/concrete) girders, beams, and steel framed floors and roofs. The restrained assembly fire resistance ratings (developed from tests on loaded restrained floor or roof specimens) and the restrained beam fire resistance ratings (developed from tests on loaded restrained beam specimens) are commonly applicable to all types (with minor exceptions) of steel framed floors, roofs, girders and beams, as recommended in Table X3.1 of ASTM E119, especially where they incorporate or support cast-in-place or prefabricated concrete slabs. Ruddy et al. (2003) provides several detailed examples of steel framed floor and roof designs by qualification testing.

4.3.3. Unrestrained Construction

An unrestrained condition is one in which thermal expansion at the support of load-carrying elements is not resisted by forces external to the element and the supported ends are free to expand and rotate.

However, in the common practice for structural steel (and composite steel-concrete) beams and girders, the unrestrained beam ratings are developed from ASTM E119 tests on loaded restrained beam specimens or from ASTM E119 tests on loaded restrained floor or roof specimens, based only on temperature measurements on the surface of structural steel members. For steel framed floors and roofs, the unrestrained assembly ratings are developed from ASTM E119 tests on loaded restrained floor and roof specimens, based only on temperature measurements on the surface of the steel deck (if any) and on the surface of structural steel members. As such, the unrestrained fire resistance ratings are temperature-based ratings indicative of the time when the steel reaches specified temperature limits. These unrestrained ratings do not bear much direct relevance to the unrestrained condition or the load-bearing functions of the specimens in fire tests.

Nevertheless, unrestrained ratings provide useful supplementary information, and they are used as a conservative estimate of fire resistance (in lieu of the restrained ratings) in cases where the surrounding or supporting construction cannot be expected to accommodate the thermal expansion of steel beams or girders. For instance, as recommended in Table X3.1 of ASTM E119, a steel member bearing on a wall in a single span or at the end span of multiple spans should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

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