Structural Fire Engineering: Overview and Application Examples of Appendix 4 of the AISC Specification

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ABSTRACT

This paper presents an overview of current conventional practices for providing passive fire protection of building structures and describes alternative engineering approaches covered in Appendix 4 of the 2010 AISC *Specification*, ANSI/AISC 360-10. The concept of structural fire engineering is discussed, along with guidance and design references that are available to support performance-based structural fire engineering analyses. The roles and responsibilities typically assumed by design team members and other stakeholders in a structural fire engineering project are presented, as are considerations associated with peer reviews and approval by authorities having jurisdiction. The paper concludes with a series of four design examples that demonstrate a range of structural fire engineering applications for steel buildings.

Keywords: fire, structural fire engineering, performance-based fire design, fire engineering, AISC *Specification* Appendix 4.

INTRODUCTION

Beginning with the 2005 edition, Appendix 4 of the AISC *Specification for Structural Steel Buildings* has addressed structural design for fire conditions by analysis. By providing performance objectives and design requirements, guidance for the characterization of fires and their effects on steel members, and permitted methods of analysis, Appendix 4 supports the pursuit of structural fire engineering strategies that fall outside of the more traditional, prescriptive, code-based fire resistance design approach.

This article provides a general overview of prescriptive and performance-based structural fire-resistance design approaches and discusses how Appendix 4 of the 2010 AISC *Specification* can be used to support the latter. Four examples are included to demonstrate a range of possible structural fire engineering applications.

CURRENT PRACTICES

Building fire protection is achieved through either active or passive measures, or by a combination of both. Active measures, such as sprinkler systems, are intended to control the development and growth of fires. Passive measures are intended to protect structural elements from damage or collapse and to prevent the spread of fires. Examples of passive measures include sprayed fire-resistant materials (SFRMs) and construction of separating elements that prevent the transmission of heat and hot gases. By choosing and designing appropriate materials, assemblies and architectural arrangements, building designers can meet building code requirements for providing a prescribed level of fire resistance for the selected type of construction, occupancy and layout (height and area).

The term *fire resistance* refers to the ability of a given structure (or portion thereof) to maintain physical and thermal stability for some duration during a fire and to meet the acceptance criteria of the fire test standard(s) referenced by the applicable building code. This time period may be used for occupant evacuation, property protection and fire department response, depending on the type of the building, stakeholders' requirements and/or nature of the emergency event.

Model building codes, upon which the majority of jurisdictions in the United States and many international authorities base their local building codes, require minimum levels of structural fire resistance based on a building's size and use, among other factors. Prescriptive fire-resistance ratings for building construction in the United States have long been based on the test methods and acceptance criteria of ASTM E119 (referenced in UL 263 and NFPA 251) (ASTM, 2012; UL, 2011; NFPA, 2006). This fire resistance is most commonly achieved through specification of structural assemblies and systems, which are comprised of structural members as well as coatings, encasements, systems and other protective measures. A given rated assembly or system is prequalified to achieve a fire-resistance time through fire testing (per ASTM E119 or its UL or NFPA counterparts) or the derivative analytical methods contained in ASCE/SEI/SFPE 29 (ASCE, 2005).

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Prescriptive approaches such as these are usually conservative, and they can be easily implemented by a design team and enforced by building officials. Thus, they have had a generally successful and long history of providing for public life safety. However, prescriptive fire-resistance approaches are based on physical fire tests or calculation methodologies with limitations. Also, size-constrained assemblies for laboratory tests are considered in isolation rather than as part of a larger structural system. Furthermore, because these standardized fire test methods evaluate only the relative performance of particular assemblies subjected to standard fire exposures, they do not provide information regarding how the tested construction assembly, or a slightly different variant of it, might respond to a real fire as part of a structural system within a building. For these reasons, alternative methods based on the available scientific and engineering knowledge, modern computational tools and past experimental or event outcomes provide the only other recourse for some design conditions.

OVERVIEW OF APPENDIX 4

Appendix 4 of the 2010 AISC *Specification* is designed to support flexible approaches to structural fire resistance by providing methodologies and criteria to support evaluation of structural response to real fire exposures. Appendix 4 is organized into three main sections:

- Section 4.1: General Provisions
- Section 4.2: Structural Design for Fire Conditions by Analysis
- Section 4.3: Design by Qualification Testing

Section 4.1 (General Provisions) provides information regarding the performance objectives that should be used to determine if an assembly's performance is acceptable. It also defines the load combinations that should be used when evaluating structural performance under fire conditions.

Section 4.3 (Design by Qualification Testing) provides the engineer with the traditional option of using established fire testing protocols, such as ASTM E119 (ASTM, 2012), to determine the fire-resistance rating of a structural member or assembly.

The heart of Appendix 4 lies in Section 4.2 (Structural Design for Fire Conditions by Analysis). It is in this section that alternative methods, parameters and criteria are presented to guide performance-based structural fire engineering analysis. Key portions of Section 4.2 are described here.

Design-Basis Fire

An important aspect of an engineering evaluation of structural fire resistance is the definition of the design-basis fire(s). The selection of the design-basis fire(s) is usually performed by the fire protection engineer. If the bounding (worst-case) fire conditions against which the performance of a structure is evaluated are not accurately and fully described, the resulting conclusions will likely not be correct. Considerations and approaches are provided to help the engineer effectively describe the design-basis fire exposure, such as the fire compartment size and thermal characteristics of its boundaries, combustible fuel load density and ventilation conditions.

Material Strength and Properties at Elevated Temperatures

As construction materials are heated in a fire, their strength and mechanical properties degrade. Appendix 4 provides methodologies and material property data for structural steel and concrete for use in evaluating strength, modulus of elasticity and thermal expansion at elevated temperatures.

Structural Design Requirements

Criteria for providing structural integrity are given in terms of strength requirements and deformation limits. These are evaluated in the context of changing material properties at elevated temperatures and load combinations as defined earlier in the section. A structural system is required to be able to withstand local damage without experiencing loss of global stability. Connections must be designed to support the forces developed during the design-basis fire.

Methods of Analysis

Methods of analysis supported by Appendix 4 fall into two categories: simple and advanced. The simple methods are intended to predict the fire-induced response of individual members in tension, compression, flexural and composite floor action.

Advanced methods include approaches such as computational fluid dynamics modeling to describe temperature exposures, finite element modeling to evaluate heat transfer with structural members, and local and global structural frame response to predicted temperatures.

STRUCTURAL FIRE ENGINEERING

Performance-based structural fire engineering provides opportunities for engineers to seek innovative ways to meet code-required fire-resistance requirements. Prescriptive provisions do not typically support new or "outside-of-the-box" design solutions, and standard furnace testing of unique assemblies, even if feasible due to laboratory and furnace size constraints, can be a costly and time-consuming addition to a project. The structural fire engineering approaches supported by Appendix 4 can alleviate these challenges but may require more complex analyses of fire resistance with which some structural engineers may not be accustomed.

Performance-based approaches are common in the fire protection engineering community, especially for the design of smoke management systems. The Society of Fire Protection Engineers (SFPE) defines performance-based design as "an engineering approach to fire protection design based on: (1) agreed upon fire safety goals and objectives; (2) deterministic and/or probabilistic analysis of fire scenarios; and (3) quantitative assessment of design alternatives against the fire safety goals and objectives using accepted engineering tools, methodologies and performance criteria" (SFPE, 2007).

Available Guidance

Performance-based design, by definition, is flexible. The required methodology for one project may or may not be appropriate for another. Many factors influence the choice of engineering tools, performance measures and solutions. Because of this, specific analysis methodologies are difficult to document in codes and standards. Instead, the fire protection community has developed guidance for the overall approach to performance-based design. Four documents are available:

- ASCE/SEI 7-10, Section 1.3.1.3, Performance Based Procedures (ASCE, 2010)
- *SFPE Engineering Guide to Performance-Based Fire Protection* (SFPE, 2007)
- *SFPE Code Official's Guide to Performance-Based Design Review* (SFPE, 2004)
- *Guidelines for Peer Review in the Fire Protection Design Process* (SFPE, 2009)

These documents can be used to define an appropriate process for addressing a given structural fire engineering challenge, including definition and agreement of goals and objectives, documentation of the analysis and approval of the proposed solutions and associated justifications.

Additional methodology and data references specific to structural fire engineering are discussed later in this article.

Professional Roles and Responsibilities

The SFPE notes that "the team approach is essential to the success of a performance-based design" (SFPE, 2007). This team comprises building owners, architects, engineers, building and fire officials and others who may have a role in the project. Depending on the complexity of a proposed approach, performance-based design may require greater collaboration than is typical of a more conventional design project.

Structural fire engineering requires collaboration among five stakeholders.

Fire Protection Engineers

Because structural fire engineering will usually not rely on the fire exposure specified for standardized furnace testing, specialized knowledge in the calculation of real building fire exposures is required. Fire protection engineers are responsible for defining and interpreting the level of fire safety required by the code and for translating that information to appropriate performance criteria. They will also define the thermal environment to which the structure is exposed, including the combustible content, ventilation or wind effects, heat energy, flame shape and height, fire duration and affected area(s). This task may involve computer-based fire modeling or more simple hand calculations and may consider the effects of suppression systems, fire department activities and passive fire protection systems.

In many cases, the fire protection engineer will also characterize the transfer of heat into the structural member, the corresponding material temperature rise and the resulting thermal effects to the member. Fire protection engineers must be able to effectively convey the effects of material temperatures in a form that structural engineers can use to evaluate the response of the structure.

Fire protection engineers are also generally responsible for documenting these aspects of the performance-based design and supporting the approvals process.

If the structural fire engineering approach involves comparing unique or untested members or assemblies to conventional members that have been tested, fire protection engineers may be responsible for documenting this comparison and substantiating compliance.

Structural Engineers

The structural engineer's initial role in the structural fire engineering process is to assist the fire protection engineer in defining critical members for analysis. Analysis of every member in a structure would be inefficient and is generally unnecessary. The engineers should evaluate possible fire exposures, structural load paths, and any redundancies in order to determine the members or subassemblies that represent a limited number of critical cases.

After the results of the thermal exposure analysis are available, the structural engineer may consider them in a number of ways. If various members will reach temperatures that will substantially reduce their strength or stiffness, the structural engineer may evaluate the impact of these reductions on the response of the local and global structural systems. The structural engineer may also need to determine the ways in which the restraint of thermal expansion may affect a structure. If individual members are shown, through the fire analysis, to exceed failure criteria, then the structural engineer's role may be to consider load redistributions and structural redundancies in order to verify if these "local" failures can be tolerated to avoid progressive (disproportionate) collapse.

Through collaboration with the fire protection engineer, the structural engineer may also propose and evaluate changes to the structure to help withstand the predicted thermal exposures. Examples of this include increasing the size of given members, revising the framing layout and/or member design or using alternative framing connections, each of which could improve fire resistance.

Architects

Solutions developed through the performance-based design may impact the architecture of a building. The architect must be involved in this process in order to provide feedback regarding the acceptability of any proposed design alternatives. For example, while increasing the size of a concrete column will improve structural fire resistance, it may narrow an adjacent corridor to a width that may not be acceptable.

The architect (and building owner) must also provide details regarding interior finish materials, furnishings and the proposed uses of individual spaces. This information is required by the fire protection engineer for development of fire scenarios for evaluation of thermal exposures.

Owners

The building owner is the most directly affected stakeholder in the performance-based fire design approach and also has most to gain or lose. Thus, owners must be fully briefed beforehand on the critical reasons and benefits for utilizing this alternative approach, as well as its uncertainties, challenges and risks. The latter may include project schedule delays, budget extras and design revisions. The initial performance-based design plan may not be found totally acceptable, and certain changes may be required due to various considerations raised by other members of the project team, consultants, peer reviewers or the building officials. Good communication and coordination among the owner, the entire project team and the building authorities throughout this endeavor are paramount.

The building owner should remain fully committed to supporting the performance-based design approach during its execution and, accordingly, must manage the responsible professional members of the project team. In the same "buy-in" perspective, the owner could help influence the building official to be receptive to the new concepts and innovation resulting from this effort.

Building Officials

The building official's primary responsibility is to ensure that the goals and objectives of the laws, codes, standards and ordinances adopted by the jurisdiction are appropriately implemented in the design and construction of a building or structure. When a design uses an alternative approach, this responsibility can become more challenging and ambiguous. Therefore, the building official must determine if the appropriate skills are available within his or her office to properly contribute to the review and approval process when considering the proposed alternative design. If not, an external reviewer or peer review may be needed, or the design team may be required to petition a higher code authority, such as a state appeals board.

During the design process, the building official should be given the opportunity to actively contribute to discussions regarding overall strategy for performance-based or alternative approaches. The building official should promptly voice any concerns regarding the alignment of the proposed strategies and design approaches with the goals and objectives of the building and fire codes and impose any special requirements that must be implemented in order to meet the intent of the codes. This participatory approach represents a departure from the traditional role of the building official which has historically been more focused on post-design review.

Coordination with Authorities

A typical building or fire official may never have been presented with a performance-based approach to structural fire resistance, even though current building codes in the vast majority of jurisdictions contain provisions than will allow for this type of alternative design approach. Performance-based fire design remains uncommon because guidance for such an approach has been limited until relatively recently and also because it is not often applied on more common projects, such as residential and smaller commercial buildings. To date, the main applications of performance-based fire design have been on larger, more monumental projects with unique architectural or structural features or unusual fire exposures or risks.

This fact should not discourage building owners and engineers from pursuing a performance-based approach to structural fire resistance. However, the design team must address the needs and concerns of the officials at an early stage and throughout the design process. Authority buy-in is critical because if a building official is faced with evaluating a performance-based design at the final review stage, and if he or she disagrees with any of the underlying assumptions, methodologies or conclusions, the outcome can be disastrous. This scenario may result in costly redesigns (with associated delays) or complete abandonment of the performance-based approach. Ignoring concerns until late in the design can also damage the working relationship between the owner and the building official.

The building and fire officials are stakeholders in every building project within their jurisdiction. Their opinions, interpretations and goals need to inform the performance-based design from start to finish. They should be briefed on the intent to pursue an alternative approach once the feasibility of that approach is well understood by the design team, and they should be involved in stakeholder meetings early on. Authorities may influence decisions regarding fire scenarios, performance criteria, choice of structural members to be analyzed, analysis methodologies and documentation requirements. They may also require a peer review, which must be anticipated.

The SFPE Code Official's Guide to Performance-Based Design Review (SFPE, 2004) is intended to assist building and fire officials with the process of reviewing a performance-based design. This extensive guide includes many frequently asked questions, the answers to which can greatly inform both the approving authorities and the other project stakeholders. The engineer must understand the content of this document to be fully prepared to address the needs of the officials.

Peer Reviews

Often, when advanced analysis techniques are employed, one or more of the project stakeholders (frequently the building owner or the authority having jurisdiction) may not be suitably trained or experienced to evaluate the work and recommendations of the engineer, or they may not have resources available to review such a design. This is particularly true for structural fire engineering, which is historically relatively uncommon in the United States. In these cases, the stakeholder may require a peer review of the performance-based design. A peer review can provide an independent professional opinion regarding the appropriateness of the assumptions, methodologies and conclusions of a performance-based design.

The SFPE and ASCE publish guidelines for peer review of fire protection and performance-based designs (SFPE, 2009; ASCE, 2010), and this general approach can be a good fit for performance-based structural fire engineering. The document describes what the scope of a peer review should include, how the review should be conducted and what documentation should be produced. It also addresses such concerns as confidentiality and intellectual property.

A peer review can affect a project's schedule in several ways. The review itself takes time, especially if the analysis and resulting design are complex and involve advanced calculation tools. Also, the review may call for changes to the methodology or final outcome of the analysis. Because of these concerns, it is most efficient if the reviewer becomes involved in the process well before the final design review.

DESIGN REFERENCES

While Appendix 4 includes valuable information needed for structural fire engineering assessments, additional references may be required, depending on the type of analysis being pursued. One study by AISC (AISC, 2005) provided in-depth information regarding available references to support structural fire engineering. The following six additional references identify more sources for a wide range of information needed when carrying out structural fire engineering analyses.

SFPE S.01: *Engineering Standard on Calculating Fire Exposures to Structures*

SFPE recently published its first standard, SFPE S.01: *Engineering Standard on Calculating Fire Exposures to Structures* (SFPE, 2011). It provides methodologies for describing thermal boundary conditions (heating effects) for structural elements exposed to both local and fully developed compartment fires. These types of natural (nonstandard) fire analyses will typically be performed by a fire protection engineer.

ASCE/SEI/SFPE 29-05: *Standard Calculation Methods for Structural Fire Protection*

ASCE/SEI/SFPE 29-05 (ASCE, 2005) provides simple empirical calculation methods for evaluating the structural fire resistance of individual members of multiple common construction materials. These methods are based on well-established equivalencies to results of standard fire resistance testing, but these methods cannot address effects of nonstandard fires, structural framing continuity, connections or member sizes/layouts that are outside the tested data base range.

SFPE Handbook of Fire Protection Engineering

The *SFPE Handbook of Fire Protection Engineering* (SFPE, 2008) includes chapters on "Methods for Predicting Temperatures in Fire Exposed Structures," "Structural Fire Engineering of Building Assemblies and Frames" and "Analytical Methods for Determining Fire Resistance of Steel Members." Heat transfer calculation approaches are discussed in depth. Advanced methodologies and performance-based approaches are discussed in concept, though technical content focuses on simple methods of predicting structural response to fire.

Eurocodes

The structural Eurocodes devote significant attention to fire-related issues. Each code includes a substantial amount of information on design for particular fire design case. For steel structures, Eurocode 1 (Basis of Design and Actions on Structures), Eurocode 3 (Design of Steel Structures) and

Eurocode 4 (Design of Composite Steel and Concrete Structures) apply (CEN, 2009a; CEN, 2009b; CEN, 2008b).

The Eurocodes support both prescriptive and performance-based design approaches, as well as consideration of individual members and whole frames. They discuss methods for characterizing fire exposures, predicting temperature-dependent thermal and mechanical properties using comprehensive mathematical expressions, choice of methodology and verification. Extensive tabulated data are included.

AISC *Design Guide 19*

While most of AISC *Design Guide 19* (AISC, 2003) explains and illustrates the conventional prescriptive approach to fire-resistive design of structural steel, one chapter introduces some basic computations for structural fire engineering. This guide contains many example problems and design aids, including tabulation of W/D properties for the standard steel shapes, and is an excellent beginning resource for practitioners less familiar with the subject.

NIST *Best Practice Guidelines for Structural Fire Resistance of Concrete and Steel Buildings*

The *NIST Best Practice Guidelines* (NIST, 2010) offer insights and recommendations for critical fire exposure variables, analysis-design of steel and concrete structures at high temperatures, risk and reliability of engineered structures when subjected to fire events, and general practical application considerations. The *Guidelines* provide a compact synthesis and guide on the overall existing state of the art in 2010 from a U.S. perspective.

EXAMPLES

The following four design examples are intended to demonstrate the application of various structural fire engineering techniques. They range from the comparatively elementary Example 1, which illustrates steel shape substitutions based on their weight to heated perimeter (W/D) property, to more complex problems. The focus of these examples is the effect of a fire on the structural performance of various types of members and on development of thermal restraint. In-depth discussion of the methodologies to calculate fire exposures to the structural elements and heat transfer are outside the scope of this paper, and the given information is only provided as direct input data for the examples. For actual project work of this type, a fire protection engineer would usually be tasked with performing the requisite fire/heating analyses and providing the final material temperature results to the structural engineer. The reader may reference SFPE S.01: Engineering Standard on Calculating Fire Exposures to Structures (SFPE, 2011) and the other noted references for additional information in this regard.

Since the 1970s, ASTM E119 and UL 263 have differentiated between restrained and unrestrained fire resistance ratings for beams in prescriptive design. In many cases, the required fire protection material thickness for thermally unrestrained beams is greater than for their thermally restrained counterparts with the same rating time. This thermal restraint classification, as defined in ASTM E119, can be quite different than the typical member end restraint connotation in structural engineering. Consequently, it has been a frequent source of confusion and interpretation questions over the decades. Section 4.3.2 of Appendix 4 of the 2010 AISC *Specification* provides specific guidance for structural steel beams and girders that support concrete slabs and are integrally connected by bolts or welds to adjacent steel framing: These can be considered as restrained (thermally) for purposes of such prescriptive fire resistance applications. Examples 1 and 3 illustrate some of the implications and effects of these fire-resistance rating distinctions.

For the purposes of these examples, various elevated material temperatures are provided as given information assumed to be properly determined either from tests or suitable analyses. Also, for similar practical reasons, computerized structural solutions are not fully described but are only presented as final results. These examples are intended to convey the capabilities of performance-based fire design approaches, their typical assumptions and computational steps, and the resulting sensitivity of the structural design to the fire and thermal exposures that have been postulated.

In many cases, agreement on the design basis fire scenario(s) may present the most critical project issue, followed by resolution of uncertainties in thermal properties of fire protection materials and in the fire response of member connections. For such instances, parameter variation and iterative sensitivity studies may be necessary to envelope the realistically expected performance range of the structure. As previously described, the entire project team and building official should review all analysis and design details prior to implementation.

Example 1: Shape Substitutions for Beams and Columns

Access to the *UL Fire Resistance Directory* (UL, 2013) in its published or online form is encouraged to enable a better understanding of this example, in particular the nature and details of the referenced fire resistive assemblies.

Problem Statement—Beams

A standard 2-hr fire resistive rating is required for a building floor system, which has been designated a "restrained" assembly. UL D902 (UL, 2013) is the specified rated floor assembly for this construction. The steel floor deck is to consist of all fluted, 2-in.-deep units, topped with 3¼ in. of lightweight concrete.

For the W24×84 steel beams in this floor system, compute the minimum contour thickness of spray-applied fire-resistive material (SFRM) required for a 2-hr unrestrained beam rating consistent with UL D 902, assuming Type 300 is the selected SFRM protection product.

Note: In accordance with ASTM E119 and the cited UL assembly listing, selection of a 1-hr unrestrained beam protection would also have been acceptable for the specified 2-hr restrained assembly rating, and would have accordingly resulted in a lower fire protection material thickness requirement.

Approach and Solution

The W/D steel shape property represents its ratio of weight to heated perimeter as the effective thermal inertia of the member. Shapes with larger W/D values are more resistant to heating effects than those with lower W/D values for identical exposure and fire protection cases. This shape parameter frequently recurs in the theoretical and design equations for steel fire resistance. AISC *Design Guide 19* (AISC, 2003) includes a tabulation of W/D properties for all the standard steel shapes.

The W24×84 beams (W/D = 1.14 lb/ft/in.) are substantially larger and heavier than the minimum W8×28 size (W/D = 0.80 lb/ft/in.) in the UL listing; hence, the proposed beam size complies with this requirement of UL D902. The easiest, but most conservative, thickness of the SFRM (of the type prescribed in the listing) can be simply taken as $^{11}\!/_{16}$ in. as provided within the UL D902 assembly listing for the 2-hr protection of the minimum W8×28 beam size.

However, some efficiency and cost savings can be achieved by using the substitution equations given in the references (UL, 2013; ASCE, 2005; SFPE, 2008) and the 2012 *International Building Code* (ICC, 2012). This simple calculation adjusts the minimum required SFRM thickness on the basis of W/D for the actual beam shape to be protected, rather than the minimum size prescribed in the rated assembly. The required protective material thickness for the actual beam, t_2, is calculated based upon the thickness listed for the minimum beam size in the UL listing, $t_1 = {}^{11}\!/_{16}$ (or 0.688 in.), and the W/D ratios of the two beam sizes, as follows:

$$\begin{aligned} t_2 &= \frac{W_1/D_1 + 0.6}{W_2/D_2 + 0.6}(t_1) \\ &= \frac{0.8 + 0.6}{1.14 + 0.6}(0.688) \\ &= 0.553 \text{ in. or approximately } {}^{9}\!/_{16} \text{ in.} \end{aligned}$$

Thus, a minimum $^{9}\!/_{16}$-in. SFRM contour thickness could be used for the W24×84 beams in the 2-hr floor construction, resulting in a material thickness reduction of ⅛ in. relative to the baseline UL D902 assembly listing. While this material and cost savings may be marginal for the spraying of relatively few beams, it can quickly compound when multiplied over the many floors in a multi-story building.

This beam substitution equation must only be used within its stated limits of application, as given in the cited references.

Problem Statement—Columns

A 2-hr fire resistive rating is required for a built-up steel column (doubly symmetric I-shape), with MK-5 SFRM protection along its contour. UL X772 (UL, 2013) is the referenced rated assembly to be used.

For this given steel shape, compute W/D and the minimum required SFRM thickness.

Consider a doubly symmetric, built-up (nonstandard) I-shape column with the following dimensions:

- Total depth of I shape (d): 18 in.
- Flange width (b_f): 8 in.
- Flange thickness (t_f): 0.75 in.
- Web thickness (t_w): 0.5 in.

Approach and Solution

The weight per unit length, W, is calculated as follows:

$$W = \left[2b_f t_f + \left(d - 2t_f\right)t_w\right]\left[\frac{490 \text{ lb/ft}^3}{144 \text{ in.}^2/\text{ft}^2}\right] = 68.9 \text{ lb/ft}$$

The heated perimeter of the column, D, is calculated as follows, assuming that it in fully surrounded by fire, which induces the greatest heating effects.

$$D = 4b_f + 2d - 2t_w = 67 \text{ in.}$$

W/D then equals (68.9 lb/ft)/(67 in.) = 1.028 lb/ft/in.

Other partial-heating exposures can be represented by suitably modifying D for the conditions to be considered; for example, for a perimeter column that will have one flange face not subjected to the fire, the heated perimeter would decrease and slightly increase the W/D value relative to the all-around exposed case.

The UL X772 assembly includes the following formula for computation of the minimum required MK-5 SFRM thickness, h, as a function of W/D given a required fire resistance period, R:

$$h = \frac{R}{1.05\left(W/D\right) + 0.61} = \frac{2}{1.05\left(1.028\right) + 0.61} = 1.184 \text{ in.}$$

Practical round-up of this answer provides the required 1³⁄₁₆-in. thickness for this shape and the given conditions. One could also approximately check the accuracy of this solution by observing that the UL X772 listing itself required a minimum 1⅛-in. SFRM thickness for 2-hr protection of a W10×49 with $W/D = 0.83$.

This column equation must only be used within its stated limits of application, as given in the *UL Directory* (UL, 2013). Other column assemblies and SFRM products will have different curve-fitted formulas for this design purpose.

Example 2: Bending Strength of a Simply Supported Composite Beam

Problem Statement

A floor system has 2-in.-deep steel deck units, topped with 3¼ in. of 3,000-psi lightweight concrete. Simply supported and fully composite W16×26 beams—ASTM A992 steel, spaced 8 ft on center (o.c.), spanning 35 ft (see Figure 1) and running perpendicular to the deck flutes—have been designed for a uniformly distributed dead load of 60 psf and a live load of 100 psf (nominal, unfactored loads). Check only the adequacy of this beam's positive bending design strength for both ambient and fire conditions, assuming that ambient serviceability (deflections or floor vibrations) is to be separately assessed. Use the ultimate strength (fully yielded) model for both conditions. The shear connector design for full composite beam action is done conventionally and is assumed to be similarly effective at the elevated fire temperatures, consistent with the simple member analysis provision of Section 4.2.4.3.b of Appendix 4 of the 2010 AISC *Specification*.

The worst-case fire exposure for the strength limit state results in an average steel temperature of 1300 °F at the bottom flange and 600 °F at the top flange (much cooler due to its proximity to and heat shielding by the floor slab), as determined from past tests or heat transfer analysis (provided information).

Approach and Solution

First check factored loads and full composite beam design strength at ambient.

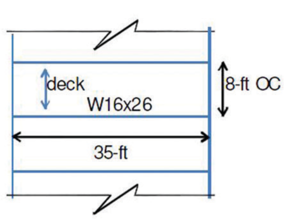

Fig. 1. Beam layout.

- 60 psf nominal dead load × 8 ft o.c. = $w_D = 0.480$ kips per lineal foot (klf)
- 100 psf nominal live load × 8 ft o.c. = $w_L = 0.800$ klf
- Beam span (L) = 35 ft
- Steel yield stress (ambient) (F_Y) = 50 ksi
- Concrete compressive stress (ambient) (f_c') = 3 ksi
- $w_u = 1.2w_D + 1.6w_L = 1.86$ klf

The required ambient strength for maximum positive bending at mid-span, M_u is calculated as follows:

$$M_u = \frac{w_u}{8}L^2 = 284.2 \text{ k-ft}$$

The design strength, ϕM_n, from conventional stress block calculations or from AISC *Manual* tables for Y2 = 4½ in., is 356 k-ft. The entire W16×26 member is fully yielded in tension at this limit state, $F_y = 50$ ksi. Because ϕM_n exceeds M_u, the composite beam has adequate strength for ambient design.

Next, check factored loads and full composite beam design strength for the design basis fire. The ASCE 7-10 load combination for an extreme event (fire) is:

$$w_{uf} = 1.2w_D + 0.5w_L = 1.0 \text{ klf}$$

Note that this required load combination for fire case is quite different from that used for ambient design conditions in terms of its live load component.

The required beam strength at the fire limit state is calculated as:

$$M_{uT} = \frac{w_{uf}}{8}L^2 = 149.4 \text{ k-ft}$$

For the composite beam design strength at elevated temperatures, use the given maximum average steel temperatures for this fire exposure to accordingly reduce the steel yield stress for thermal degradation (see 2010 AISC *Specification* Appendix 4, Table A-4.2.1). Because the beam web temperature is not explicitly given, assume a linear thermal gradient between the bottom and top flanges, which results in an average web temperature of:

$$(1300\ °\text{F} + 600\ °\text{F})/2 = 950\ °\text{F}$$

Consider the entire steel beam section to again be yielded in flexural tension.

Subdivide the steel beam into three distinct thermal regions (bottom flange, web and top flange) and assign the given average steel temperatures from the fire uniformly to each area (1300 °F, 950 °F and 600 °F, respectively) to correspondingly reduce the yield stress from ambient. Application of the web average 950 °F across the full web depth is a crude initial idealization, which will subsequently be refined.

Because the compressive stress block in the concrete slab is at the top of the floor and the heat transfer analyses have shown it to be much cooler than the steel beam temperatures (much less than 600 °F, the top flange temperature), the concrete strength is assumed to remain at its unreduced ambient value.

Use k_y retention factors from Table A-4.2.1 of the 2010 AISC *Specification* and interpolate as necessary to determine reduced yield strengths for each portion of the beam.

- At the top flange (600 °F), F_{tf} = 1.0, F_y = 50 ksi (no reduction due to temperature)
- At the web (950 °F), $F_w = 0.73$, $F_y = 36.5$ ksi
- At the bottom flange (1300 °F), $F_{bf} = 0.255$, $F_y = 12.8$ ksi

Cross-sectional areas are as follows for the W16×26 beam:

- Top flange area (A_{tf}) = 1.9 in.^{2}
- Web area (A_w) = 3.8 in.^{2}
- Bottom flange area (A_{bf}) = 1.9 in.^{2}

Assuming the entire steel beam is in tension due to composite action, summation of steel beam tensile yield forces, with high-temperature reductions, gives:

$$F_T = F_{tf}A_{tf} + F_wA_w + F_{bf}A_{bf} = 257.9 \text{ kips}$$

Impose force equilibrium of steel tension with a concrete compression block of $0.85f'b$ and solve for the depth of the concrete stress block at the top of the slab, a. The effective concrete width, b, is equal to the beam spacing, which is 8 ft or 96 in.

$$a = \frac{F_T}{0.85 f'_c b} = 1.05 \text{ in.}$$

Because a = 1.05 in. is less than the concrete slab topping height of 3¼ in. and the plastic neutral axis is above the steel beam, the original assumption of the entire steel beam acting only in flexural tension has been confirmed. The composite beam flexural resistance is computed from the summation of moments (by parts) generated by the steel flange and web area tension relative to the center of the concrete compression block ($a/2$), as shown in Figure 2.

- Vertical distance between concrete and top flange centroids (L_{tf}) = 4.89 in.
- Vertical distance between concrete and beam web centroids (L_w) = 12.57 in.
- Vertical distance between concrete and bottom flange centroids (L_{bf}) = 20.25 in.

$$M_{nT} = F_{tf}A_{tf}L_{tf} + F_wA_wL_w + F_{bf}A_{bf}L_{bf}$$

$$\phi M_{nT} = 0.9\frac{M_{nT}}{(12 \text{ in./ft})} = 202.4 \text{ k-ft}$$

The design strength during fire is therefore 202.4 k-ft, which represents approximately a 43% reduction from the ambient case.

Because $\phi M_{nT} > M_{uT}$ = 149 k-ft, the composite beam has adequate strength for the given fire exposure based on this simple idealization of the fire-induced temperature effects in the web.

A slightly more refined bending model and analysis follow, which subdivides the steel web into two parts—upper and lower halves—with corresponding average temperatures for each. This improved web discretization will more accurately reflect the beam's effective web bending due to the vertical steel temperature variations along its height. The average temperature in the bottom half of the web is 1125 °F and that for the top half is 775 °F (see Figure 3). Consideration of the beam flanges as single individual areas at one temperature is generally sufficient because the thermal gradient through the relatively thin flange thickness has inconsequential effects.

The axial force balance remains unchanged from before, with a = 1.05 in., as does the resistance of both flanges. The only difference appears in the bending moment summation of the two web half-areas, as follows, wherein the additional subscripts for the variables F and L refer to the top and bottom halves of the web area.

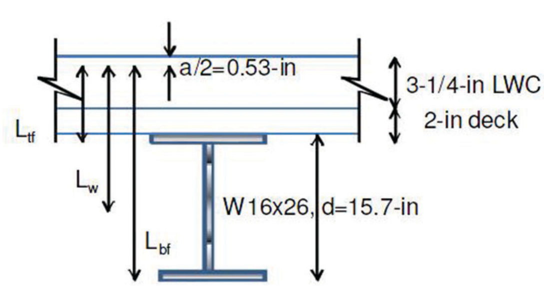

Fig. 2. Assembly cross section.

Again use k_y retention factors from Table A-4.2.1, with interpolation as necessary.

- At the top half of the web (775 °F), $F_{tw} = 0.97F_y = 48.5$ ksi
- At the bottom half of the web (1150 °F), $F_{bw} = 0.43F_y = 21.5$ ksi
- Vertical distance between concrete and beam top web centroids (L_{tw}) = 8.82 in.
- Vertical distance between concrete and beam bottom web centroids (L_{bw}) = 16.32 in.

The following two-part M_{web} expression now replaces the previous single $F_wA_wL_w$ term, with the flange model remaining the same.

$$M_{web} = F_{tw}A_w\frac{L_{tw}}{2} + F_{bw}A_w\frac{L_{bw}}{2} = 1.5\times10^3 \text{ k-in.}$$

$$M_{nT} = F_{tf}A_{tf}L_{tf} + M_{web} + F_{bf}A_{bf}L_{bf}$$

$$\phi M_{nT} = 0.9\frac{M_{nT}}{(12 \text{ in./ft})} = 182.6 \text{ k-ft}$$

The revised ϕM_{nT} value of 182.6 k-ft is approximately 10% less than the 202.4 k-ft value computed previously and about a 49% reduction from ambient.

Because $\phi M_{nT} > M_{uT}$, the composite beam again demonstrates adequate strength for the given fire exposure, with approximately 23% reserve bending strength (183/149). One additional computational iteration could be attempted with additional web subdivisions to confirm the satisfactory convergence of this bending moment solution at a value exceeding the required strength.

As a side note, if the steel beam temperature had resulted from a lumped mass heat transfer analysis, Section 4.2.4.3b.4 of Appendix 4 of the 2010 AISC *Specification* would have required a prescribed (conservative) temperature distribution through the cross-section to be used in the determination of its moment resistance, with the lumped mass temperature assumed over the bottom half of the steel beam shape (flange and web), then linearly decaying at no more than 25% through the upper web half to the top flange. Because this problem identified specific steel temperature inputs for both beam flanges, this more general provision may be considered to be superseded by the given thermal profile input.

If the more severe maximum uniform temperature profile had been imposed for the bottom half of the W16×26 (1300 °F through lower beam $d/2$, then linearly varying to 600 °F in the steel top flange), the concrete compressive stress block depth is reduced to $a = 0.84$ in. For these modified thermal conditions, the composite beam design strength ϕM_{nT} additionally decreases to 137 k-ft, which is now about 8% less than the required 149 k-ft moment. A slightly larger beam size or an incremental increase in the initial steel beam fire protection thickness would decrease the fire heating effects and enhance the member's design strength to compensate for this strength differential.

The most conservative assumption of a uniform maximum 1300 °F temperature over the entire steel beam results in the lower bound composite beam design strength of approximately 85 k-ft, which would likewise have required a redesign.

This problem illustrates the basic structural limit state model for this type of design problem and the effects of variations in the temperature distribution through the steel beam depth on the composite member design strength. As demonstrated, the critical heating parameter is not only the maximum steel temperature in the bottom flange, but also the thermal gradient along the beam web. The fidelity of the prior heat transfer analysis and/or empirical data used as the

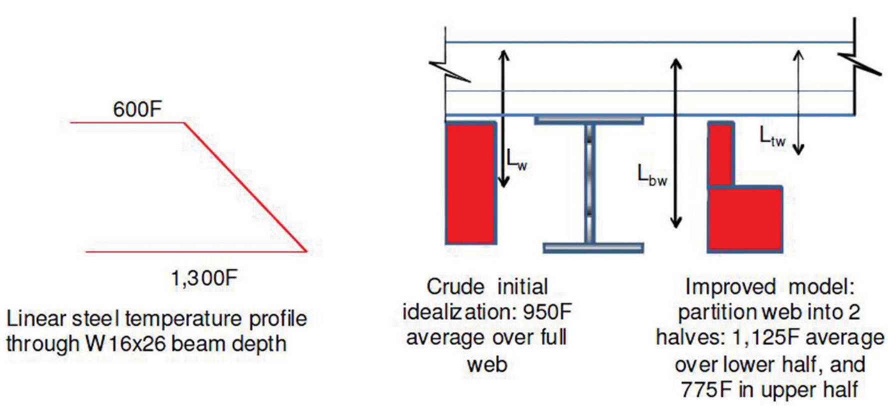

Fig. 3. Influence of refined consideration of web temperature distribution.

thermal input for these structural calculations should help guide selection of the most appropriate and bounding steel beam temperature distribution for the design fire exposure.

Example 3: Restrained Beam

Problem Statement

This example illustrates an application of advanced analysis to better understand, assess and design for thermally restrained or unrestrained conditions, as defined in ASTM E119 (ASTM, 2012) and UL 263 (UL, 2011).

A W16×40 ASTM A992 member has been chosen through ambient temperature design for a 30-ft span. The simply supported beam is noncomposite to the floor deck above and can be assumed to be continuously braced for lateral torsional buckling. The building is subdivided by full-height (slab-to-structure) fire barriers that align with the column grid such that a fire in one compartment will not directly heat beams in an adjacent compartment, assuming the fire barriers do not fail. The flexural resistance of this beam design when exposed to elevated temperatures during a fire is to be reviewed for an interior bay (restrained condition) and for an exterior bay (assumed to be unrestrained). The uniformly distributed dead load is 0.48 kips/ft and the uniformly distributed live load is 0.80 kips/ft.

Approach

Based on a load combination for fire of 1.2D + 0.5L per ASCE 7-10 and Appendix 4 of the 2010 AISC *Specification*, the maximum required moment at center span (M_u) is 110 kip-ft.

Per Section 4.2.4.3b.(3) of Appendix 4, the hottest bottom flange temperature is conservatively taken as being representative of the temperature of the rest of the cross-section.

W16×40 section properties:

A = 11.8 in.^{2} $\quad$ t_f = 0.505 in.

d = 16 in. $\quad$ I_x = 518 in.^{4}

t_w = 0.305 in. $\quad$ Z_x = 73 in.^{3}

b_f = 7 in.

Per the user note to Specification Section F2, W16×40 is a compact section.

Ambient temperature material properties for ASTM A992 steel:

F_y = 50 ksi

E = 29,000 ksi

F_y and E are temperature dependent per Table A-4.2.1 of Appendix 4. The coefficient of thermal expansion is 7.8×10^{-6}/°F at temperatures greater than 150° F (AISC, 2010). The 30-ft-long beam expands as its temperature increases as shown in Figure 4.

Unrestrained Case—Exterior Bay

In the proposed structural design, the exterior wall provides minimal lateral restraint against the axial expansion of the beam, which is ignored such that the beam is conservatively assumed to behave as simply supported without development of significant second-order moments due to P-Δ effects as a result of the applied loading and heating. The moment capacity of the beam is:

$$R_f = \phi_b M_{n,fire} = \phi_b F_y k_y Z_x$$

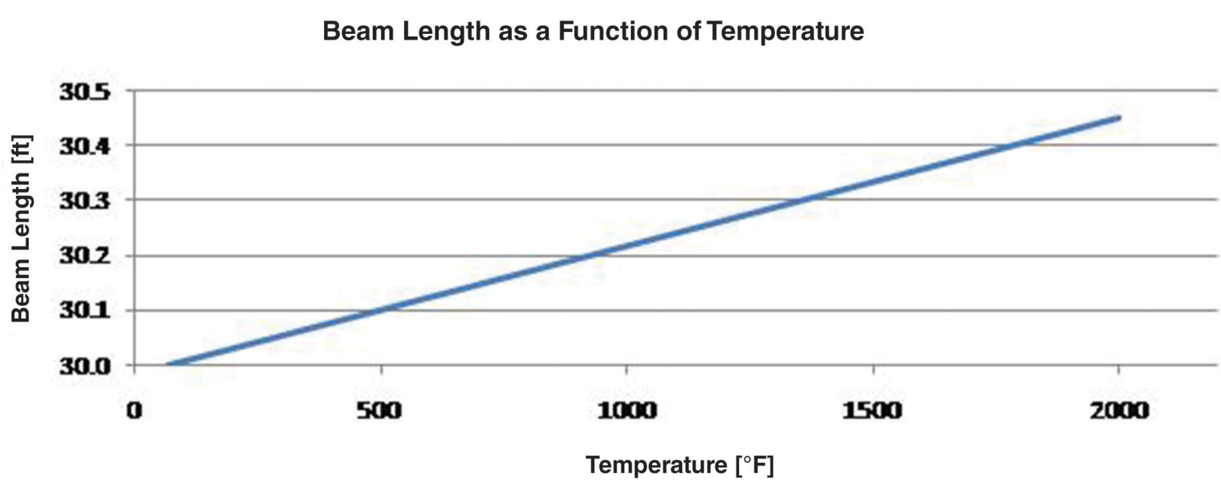

Fig. 4. Thermal expansion.

where R_f is the flexural resistance during fire, $\phi_b = 0.9$ and k_y is the temperature-dependent strength reduction factor obtained from Table A.4.2.1 from Appendix 4.

Restrained Case—Interior Bay

If a fire occurs in an interior compartment and the compartment's fire barriers do not fail, only the structural members in the interior bay will experience high temperatures, and the structure of the surrounding bays will provide restraint against fire-induced axial forces in the heated beams. The level of restraint will vary based upon the design of the structure. For this example, 75% restraint is used given that the frame is bolted (nonsliding connection) but the floor construction is not composite with the beams.

When thermal expansion (as discussed earlier) is induced by elevated temperatures but the ends of the beam are restrained against this expansion, high axial thrust forces can develop at the supports. Depending on the design of the connection, this thrust force can result in second-order moments that either increase or reduce the moment-carrying capacity of the member as long as the end connections do not fail. For this example, the connection is designed with consideration of this condition such that the thrust force occurs below the centroid of the beam and the connection has sufficient capacity to resist this force at elevated temperatures—a case that can result in improved moment capacity. The axial force, P, induced by thermal expansion is calculated as follows:

$$P = Ek_E A\alpha\Delta T$$

where

P = axial force, kips
k_E = temperature-dependent reduction factor for E
A = cross-sectional area, in.^{2}
α = coefficient of thermal expansion
L = beam length, in.
ΔT = temperature rise above ambient, °F

The critical buckling load for the W16×40 with a 30-ft unbraced length is calculated using the Euler formula and changes as the beam is heated given the temperature dependence of the modulus of elasticity. Figure 5 compares the calculated axial force due to thermal restraint with the critical buckling load. The critical buckling load will only be surpassed above 1800 °F (indicated in Figure 5 by the "x" denoting the intersection of the curves); otherwise, the restraining axial thrust reaction can be included in the beam's flexural strength.

Local member buckling at the connections should be reviewed because it might be an important factor given the high axial loads concentrated at the bottom flange. At elevated temperatures, this complex behavior is best reviewed through computer modeling, which is beyond the scope of this example.

The second-order moments induced by restraint of thermal expansion can be calculated as follows:

$$M_{axial\ thrust} = P\Delta$$

where P is the axial thrust force at the connection and Δ is the eccentricity associated with the location of the thrust

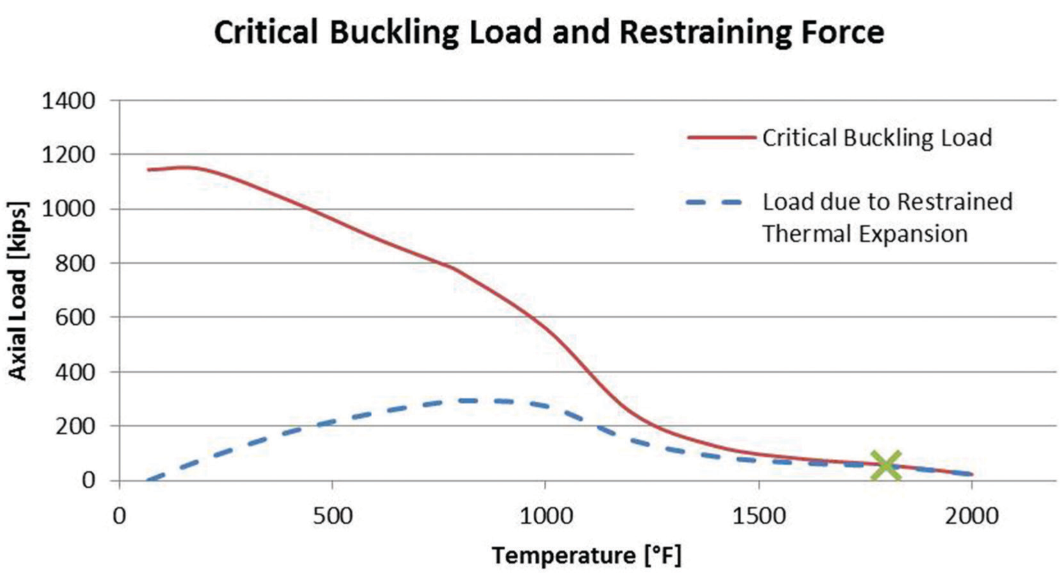

Fig. 5. Critical buckling load and restraining force for W16×40.

force relative to the centroid of the top flange of the beam. The value of Δ will change as the beam deforms and deflects due to the reduction of the modulus of elasticity at elevated temperatures. For steel temperatures not more than 1800 °F, the total flexural resistance, R_f, of this restrained beam then becomes:

$$R_f = M_{gravity} + M_{axial\ thrust}$$

Summary of Results

Figure 6 summarizes the total flexural capacity of the W16×40 beam as a function of temperature for the unrestrained and restrained cases. Based on flexural capacity and ignoring local buckling at the connections, the moments induced by the axial restraint condition allow the beam to sustain the applied gravity load at higher temperatures than for the unrestrained case.

As the beam continues to deflect, Δ may approach zero, reducing or eliminating the benefits of the second-order moment. At some point, the orientation of the second-order moment is reversed and the thrust force will reduce the flexural capacity of the beam, as seen in Figure 6 for temperatures above about 1600 °F.

This example has only considered an overall general temperature regime without any particular maximum exposure value. Credible design fire(s) must be used to evaluate the imposed heating demands and expected structural performance. The effects of cooling, and the resulting reduction in the length of the beam, may need to be reviewed to determine if the cooling phase might lead to failure of the connections.

Example 4: Exterior Tension Rods

Problem Statement

A new building includes a large atrium with a cable-stayed glass façade. The architectural design includes exterior steel cables or rods that span from the top of the wall (50 ft above grade) down to concrete foundations at the ground level. The original structural system utilized steel cables, and the ambient temperature design called for these members to be 3.5 in. in diameter. They are approximately 62 ft in total length and are spaced approximately 13.1 ft apart. The members span above a road surface adjacent to the building's main entrance.

The design team identified the following objectives:

- The structural members require a 1-hr fire resistance rating per the building code.
- The members should appear to be steel and should not be coated in protective material (i.e., omit applied fireproofing).
- Either cables or rods can be used.
- Large passenger vehicles (buses) should be allowed to utilize the access road.

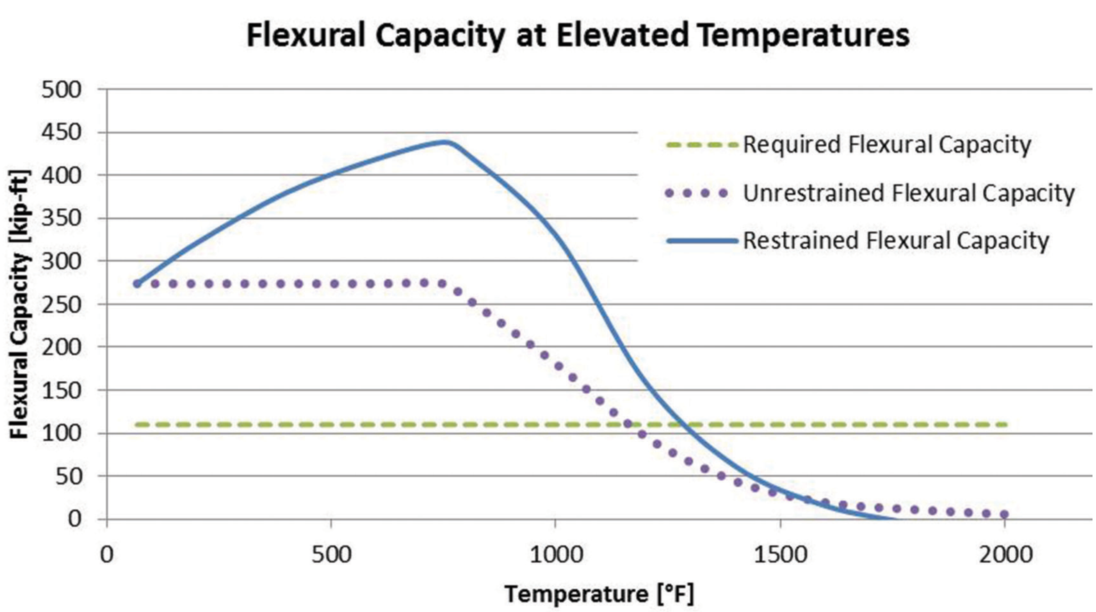

Fig. 6. W16×40 beam flexural capacity at elevated temperatures.

The fire protection engineer identified the most severe credible design fire for this case. The relevant details of this design fire are as follows:

- The fire source is a passenger bus.
- Up to four structural members (tension rods or cables) could be directly exposed to a fire engulfing the bus.
- Barriers prevent the bus from being closer than 11.8 ft to the base of the cables/rods.
- The burning rate of the bus has been determined based on the fuel load and ventilation.

Steel Temperatures

Based upon the preceding design fire description, the fire protection engineer has calculated the heat transfer from the bus fire to the adjacent members. Available research (SFPE, 2008) indicates that school bus fires may achieve peak heat release rates near 35 MW, as shown in Figure. 7.

The heat transfer analysis, which considered flame extension from the windows of the burning bus, resulted in estimates of steel temperatures along the length of the members as shown in Figure. 8, with a maximum expected steel cable/rod temperature of 1200 °F. Similar temperature profiles have been used for the four cables/rods directly adjacent to the bus (fire source) in order to represent the most severe exposure expected. The members immediately adjacent to the fire, but not directly above it, attained a maximum temperature of only 570 °F.

Reduction in Steel Strength

The loss in strength and stiffness of steel at high temperatures depends on how the steel was processed. Steel cables are typically cold worked and lose strength and stiffness at high temperatures more quickly than hot rolled steel. At 1200 °F, cold worked steel retains only 8% of its ambient strength (CEN, 2008a). Hot rolled steel retains 35% of its ambient strength at this temperature (CEN, 2008a). Figure 9 compares the loss of strength of these materials at elevated temperatures.

Thermal Expansion

Steel expands as it is heated. The coefficient of thermal expansion for the analysis was taken as a constant 7.8×10^{-6}/°F when the steel temperature is greater than 150 °F (AISC, 2010). More refined temperature-dependent representations of this coefficient exist. Taking the temperatures shown in Figure 8 as the average temperature of each 1-m-long portion of the member, the total thermal expansion of each of the four members directly above the bus fire is 4.3 in. The next adjacent members expand by approximately 1.9 in. given a maximum temperature of 570 °F and a similar profile to that shown in Figure 8.

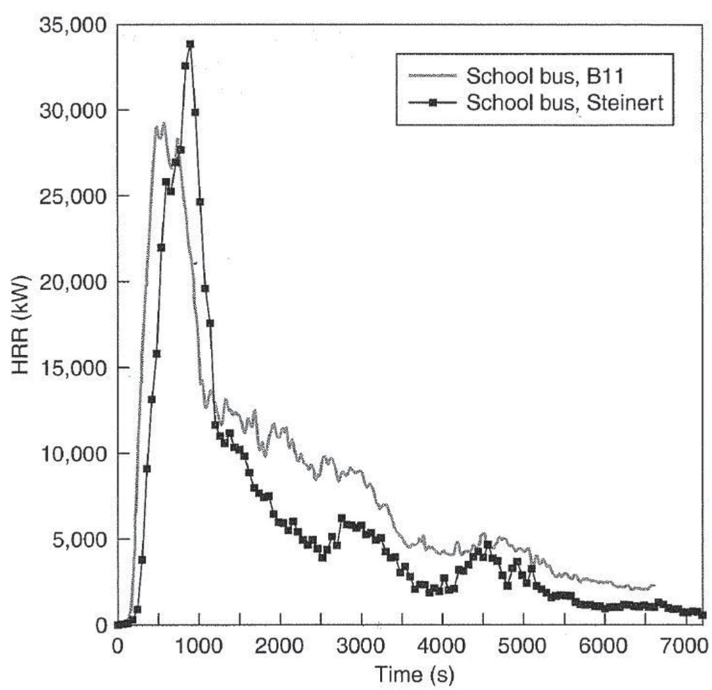

Fig. 7. School bus fire sizes (SFPE, 2008).

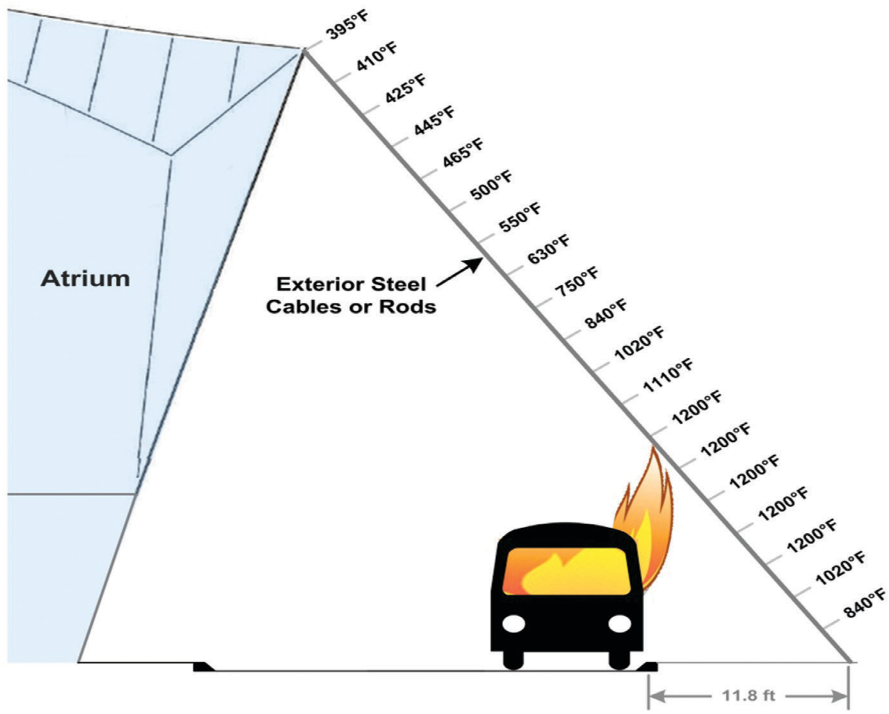

Fig. 8. Steel temperatures for tension rods/cables adjacent to bus fire source.

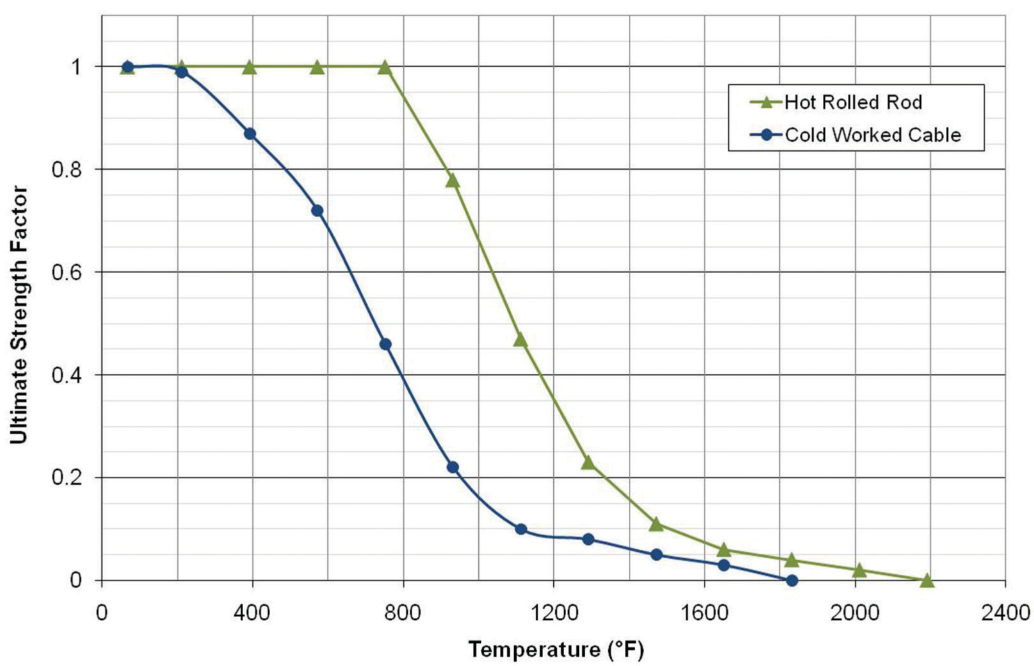

Fig. 9. Steel strength as a function of temperature.

Table 1. Structural Analysis Results				
Rod	Required Tensile Strength (kips)	Available Tensile Strength at 68 °F (kips)	Available Tensile Strength at 1200 °F (kips)	Remaining Material Safety Factor
3.5-in.-diamater Rods				
R1	112	454	159	1.4
R2	157	454	159	1.0
R3	168	454	159	0.9
R4	105	454	159	1.5
4.1-in.-diameter Rods				
R1	112	617	216	1.9
R2	157	617	216	1.4
R3	168	617	216	1.3
R4	105	617	216	2.1

Structural Analysis

Based on the steel temperature analysis discussed earlier, the design team chose to move forward using hot-rolled steel rods because they showed the most promise for meeting the goal of omitting applied fireproofing. The following load combinations were used to evaluate structural performance for the fire case (ASCE, 2010; AISC, 2010):

$$1.2D + 0.5L + 0.2S$$

$$1.2D + 0.5L + 0.2W$$

where D represents the nominal dead load, L is the nominal occupancy live load, S is the nominal snow load and W is the nominal wind load. ASTM A588 steel was chosen for this application ($F_y = 46$ ksi; $F_u = 67$ ksi).

The normal-temperature structural design of the atrium, including the exterior members discussed here, was accomplished using a finite element model given the highly complex geometry and interactions between different members. The same model was used to evaluate the effects of reduced member strength in the fire case. The model also accounted for the calculated 4.3-in. increase in the length of the four rods directly above the fire, as well as the lesser expansion of other rods in the vicinity. The complex response of the structural system to the weakening and expansion of individual members required this type of advanced analysis.

The structural engineer determined the required tensile strengths in the fire case with the four critical tension rods heated to the temperatures indicated in Table 1 summarizes the results of this analysis.

As can be seen in Table 1, the available strength in the fire case is not sufficient for rod R3 with a diameter of 3.5 in. However, a safety factor of at least 1.3 is maintained if the diameter of the rods is increased to 4.1 in. This level of performance is maintained for the 1-hr duration required by the applicable building code.

The completed structural fire engineering analysis demonstrated that increasing the diameter of the steel tension rods to 4.1 in. provides 1-hr fire resistance performance without the need for applied fire-resistive materials on the rods.

CONCLUSION

This article has presented an overview of Appendix 4 of the 2010 AISC *Specification*, with focus on its provisions for structural fire engineering. While movement to such advanced and performance-based approaches to structural fire resistance has been somewhat slow in the United States, it is further advanced in some other countries with a relative wealth of information available to support its undertaking. Building codes and referenced standards in the United States now provide means of gaining approval to use these types of approaches.

The four design examples demonstrate the types of approaches that are available with their potential outcomes and benefits. As more experience, confidence and successful project applications are developed with performance-based structural fire design, it is expected that its popularity will accordingly grow.

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