

CISC Commentary
on CSA S16-14 Annex K
Structural Design for Fire Conditions



CANADIAN INSTITUTE OF STEEL CONSTRUCTION
INSTITUT CANADIEN DE LA CONSTRUCTION EN ACIER

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CISC Commentary on CSA-S16-14, Annex K, Structural design for fire conditions (June 2016)

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Preface

This Commentary has been adapted from an existing Commentary to AISC's Specification for Structural Steel Buildings, Appendix 4, Structural Design for Fire Conditions (AISC, 2010, 2nd edition), that has been prepared by AISC's Committee on Specifications, Task Committee, TC8, on Structural Design for Fire Conditions. AISC in developing the 3rd edition of Appendix 4 for the 2016 year has added some new commentary developed by the TC8 Task Committee, e.g., expanded the section on mechanical properties at elevated temperatures in regards to high-strength bolts, and this CISC Commentary includes those updates made by the TC8 Task Committee. The Commentary from the AISC document has been aligned to relevant CSA S16-14 Annex K clauses. CISC in providing this Annex K Commentary to users of Annex K acknowledge credit to AISC for their permission to adapt the aforementioned Commentary for CSA S16-14, Annex K.

K.1 General

Annex K 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 Clause K.1.3 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.

K.1.3 Performance objectives

The performance objective underlying the provisions in CSA S16-14 is that of life safety. Fire safety levels should depend on the building occupancy, height of 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.

K.1.4 Design by engineering analysis

The resistance design criteria for steel beams and columns at elevated temperatures reflect recent research (Tagaki and Deierlein, 2007 and 2009). These resistance equations do not transition smoothly to the resistance 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 Clause K.2 of Annex K is limited to temperatures above 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. Structural design for fire conditions by analysis should be performed using Limit States Design methods, in which the nonlinear structural actions arising during severe fire exposures and the temperature-dependent design strengths can be properly taken into account.

K.1.5 Load combinations and required resistance

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

$$P(F) = P(F|D,I) P(D|I) P(I) \quad (\text{K.1-1})$$

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 $P(F|D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation K.1-1. 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 in Clause K.1.6. This load combination is the same as Equation 4 that appears in Commentary A, Paragraph 25 of *User's Guide – NBC 2010: Structural Commentaries (Part 4 of Division B)*, (NRCC, 2011) and is based on Equation 3 given in Paragraph 12 of Commentary A, a load combination when there is a rare load or situation. The companion action load factors on L and S in Equation 3 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 probabilistic basis for load combinations for accidental or rare events is explained further in CSA S408, *Guidelines for the Development of Limit States Design Standards* (CSA, 2011).

Commentary on notional lateral loads that shall be applied with the gravity load combination is given in CISC Commentary on CSA-S16-14 (CISC, 2016) for Clause 8.4 dealing with stability effects.

K.2 Structural design for fire conditions by analysis

K.2.2 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. NFPA 557 (2012) and SFPE S.01 (2011), as well as other published standards, can be consulted in this regard. These heating effects 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, members, connections and edge details can be specified to provide a structure that is sufficiently robust.

K.2.2.2 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 centre 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.

K.2.2.3 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 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, 2016).

K.2.2.4 Exterior fires

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

K.2.2.5 Fire duration

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 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.

K.2.2.6 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 percent (Eurocode 1, 2002). 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 (2014), 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, 2016). 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 (2015).

K.2.3 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, open-web steel 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 shall be determined by:

$$\Delta T_s = \frac{a}{c_s \left(\frac{M}{D} \right)} (T_F - T_s) \Delta t \quad (\text{K.2-1})$$

where

$$a = \text{heat transfer coefficient, W/(m}^2\text{-}^\circ\text{C)} \\ = a_c + a_r \quad (\text{K.2-2})$$

a_c = convective heat transfer coefficient.

a_r = radiative heat transfer coefficient, given as:

$$a_r = \frac{S_B \epsilon_F}{T_F - T_s} (T_{FK}^4 - T_{SK}^4) \quad (\text{K.2-3})$$

c_s = specific heat of the steel, J/kg-°C
 D = heat perimeter, m
 S_B = Stefan-Boltzmann constant = 5.67×10^{-8} W/m²-°C⁴
 T_F = temperature of the fire, °C
 T_{FK} = temperature of the fire, °K
 = $(T_F + 273)$ for T_F in °C
 T_S = temperature of the steel, °C
 T_{SK} = temperature of the steel, °K
 = $(T_S + 273)$ for T_S in °C
 M = mass per unit length, kg/m
 ϵ_F = emissivity of the fire and view coefficient as suggested in Table 1
 Δt = time interval, s

For the standard exposure, the convective heat transfer coefficient, a_c , can be approximated as 25 W/(m²-°C).

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

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 CAN/ULC-S101 (ULC, 2014) for building fires or ASTM E1529 (ASTM, 2014) 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:

$$c_s M/D > 2d_p \rho_p c_p \quad (\text{K.2-4})$$

Then, Equation K.2-5 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p} \frac{M}{D} (T_F - T_s) \Delta t \quad (\text{K.2-5})$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation K.2-4 is not satisfied), then Equation K.2-6 should be applied:

$$\Delta T_s = \frac{k_p}{d_p} \left[\frac{T_F - T_s}{c_s \frac{M}{D} + \frac{c_p \rho_p d_p}{2}} \right] \Delta t \quad (\text{K.2-6})$$

where

c_p = specific heat of the fire protection material, J/kg-°C

d_p = thickness of the fire protection material, m

k_p = thermal conductivity of the fire protection material, W/m-°C

ρ_p = density of the fire protection material, kg/m³

Note that the maximum limit for the time step, Δt , should be 5 sec.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a midrange temperature expected for that component or from calibrations to test data. For protected steel members, the material properties may be evaluated at 300 °C, and for protection materials, a temperature of 500 °C may be considered.

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

$$\Delta T_s = \frac{q''}{c_s \left(\frac{M}{D} \right)} \Delta t \quad (\text{K.2-7})$$

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:

- 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.

- Temperature-dependent material properties.
- Temperature variation within the steel member and any protection components, especially where the exposure varies from side to side.

K.2.4.3 Mechanical properties at elevated temperatures

The material properties used to assess the performance of steel and concrete structures at elevated temperatures should account for nonlinearities in stress versus strain response, thermal expansion, and time dependent creep effects. As these effects are highly variable, the uncertainties in the properties should be considered in measuring and using the derived properties to determine whether structural components and systems achieve the required reliability index target for deformation and strength limit states. While Annex K permits the determination of steel material properties from test data, in practice this is challenging given that there are no universally accepted test methods to consistently establish all of the required properties.

In lieu of test data on material properties, Annex K allows the use of the properties for steel and concrete at elevated temperatures that 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 (2005) and Eurocode 4 (2005), 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 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 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 ambient temperature yield strength F_y . Finally, at temperatures above 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 $F_u(T)$.

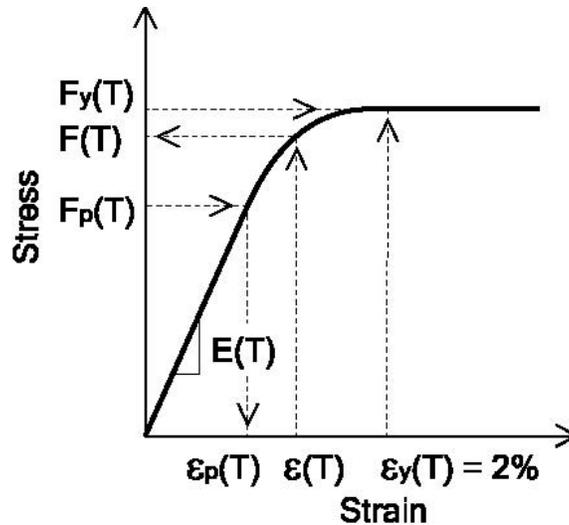


Figure 1
Parameters of idealized stress-strain curve at elevated temperatures
(Takagi and Deierlein, 2007 & 2009)

Table K.3 provides strength retention factors for high-strength bolts at elevated temperatures. These retention factors are based on a review of available experimental data (Gonzalez and Lange, 2009; Hanus et al., 2010, 2011; Kirby, 1995; Kodur et al., 2012; Li et al., 2001; Lou et al., 2010; Yu and Frank, 2009), and are consistent with values given in Eurocode 3 (2005). The available data indicates that retention factors are similar for both the shear and tensile strength of bolts, and are also similar for both ASTM A325 and A490 bolts. Consequently, Table K.3 specifies a single set of retention factors.

The strength of bolts depends both on temperature and temperature history. The strength retention factors given in Table K.3 assume the given temperature is the highest temperature to which the bolt has been exposed. For example, if a bolt is heated to 500 °C, and this is the highest temperature the bolt has seen, the strength of the bolt at 500 °C can be computed as 54% of its normal room temperature value, as indicated in Table K.3. However, if the bolt has been heated to, say 900 °C, and then cools to 500 °C, then the strength of the bolt at 500 °C may be less than 54% of the room temperature value. Limited data on the temperature history dependence of bolt strength is provided by Hanus et al. (2011). The temperature history dependence of bolt strength can be important when evaluating connection strength during the cooling stage of a fire. An additional important consequence of this behavior is that bolts can suffer a significant permanent loss of strength after being heated in a fire and then cooled to room temperature. This permanent loss of strength can be important when evaluating the condition of a steel structure after a fire. Information on the post-fire properties of high-strength bolts are reported by Yu and Frank (2009).

Annex K does not currently include provisions for computing the elevated temperature strength of welds because of the lack of experimental data on elevated temperature properties of welds made using typical North American welding processes, procedures and consumables. However, some guidance on the elevated temperature strength of welds is provided in Eurocode 3 (2005).

K.2.5 Structural design

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 resistance 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.

K.2.5.1 General structural integrity

The requirement for general structural integrity is consistent with that appearing in Commentary B, entitled *Structural Integrity of User's Guide – NBC 2010: Structural Commentaries (Part 4 of Division B)*, (NRCC, 2011). 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.

Commentary B of NRCC (2011) 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.

K.2.5.2 Strength requirements and deformation limits

As structural elements are heated, their expansion is restrained by adjacent element 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.

Membrane action of concrete floor slabs supported by steel beams has received growing international research attention over the last 15 years. Beginning with the landmark Cardington fire tests conducted during the mid-1990's in the United Kingdom (Newman, 1999), this high-temperature strength mechanism has been identified, better understood and developed as a fire resistant design alternative for steel beam and concrete floor slab systems. The novel advantage of this membrane action design is that it permits the secondary (infill) steel floor beams to be left unprotected, since they are designed for strength and stiffness primarily at ambient conditions. The tradeoffs are that the concrete slab, all the fire protected perimeter girders of the floor bays and their end connections must have adequate strength to bridge over an entire floor bay and the severely thermally weakened infill beams such that an adequate load path is maintained to transmit the gravity design loads of the floor bay. Agarwal and Varma (2014) and Agarwal et al. (2014b) have demonstrated that the presence of steel reinforcement (greater than the minimum shrinkage reinforcement) in the concrete slabs, and fire protection of the single-plate connections facilitates the redistribution of gravity loading through membrane action and reduces the risk of progressive

collapse of the structure. Bailey (2004) provides further background and the design criteria for how to effectively mobilize membrane action at large vertical deflections. There have been numerous other published papers on this research advancement, such as Zhao et al. (2008); Huang et al. (2004); and Bednar et al. (2013).

K.2.5.3 Methods of analysis

K.2.5.3.1 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. Advanced analysis should explicitly account for the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, large deformations, time-dependent effects such as creep, and uncertainties resulting from variability in material properties at elevated temperature. Boundary conditions and connection fixity must represent the proposed structural design. The models for advanced analysis models should account for all potential limit states, such as excessive deflections, connection ruptures, and overall or local buckling.

For example, Agarwal and Varma (2013) and Agarwal et al. 2014(b) conducted advanced analysis of 3D building structures while accounting for all potential limit states, namely, inelastic column buckling, composite slab cracking, yielding of the steel floor beams and reinforcement in the slabs, and deformation and fracture of the various shear connections. They used the Eurocode stress-strain-temperature relationships to account for the deterioration in strength and stiffness with increasing temperature. Sample results from one of their advanced analysis are shown in Figure 2.

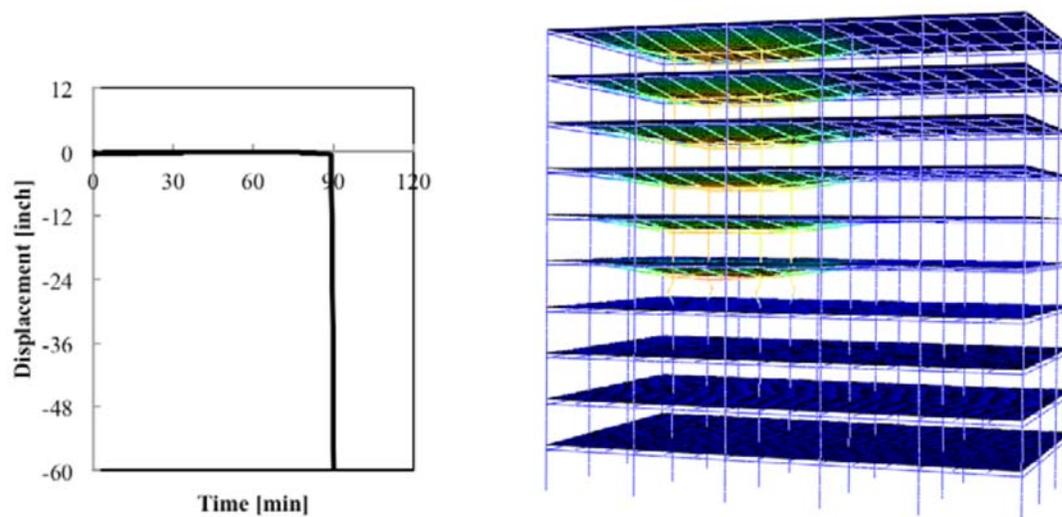


Figure 2

Advanced analysis of 3D building for design fire: (a) interior gravity column failure displacement history, and (b) failure mode

K.2.5.3.2 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 surrounded by fire.

As a practical measure, the nominal member resistances at elevated temperatures are generally calculated using the standard resistance equations of the CSA S16-14 Standard with steel properties (E , F_y and F_u) reduced for elevated temperatures by the factors in Table K.1. Recent research (Takagi and Deierlein, 2009) has shown this procedure to over-estimate considerably the resistances of members that are sensitive to stability effects. To reduce these unconservative errors, new equations, developed by Takagi and Deierlein (2009) are incorporated into Annex K to more accurately calculate the resistance of compression members subjected to flexural buckling and flexural members subjected to torsional-flexural buckling. As shown in Figure 3, the equations are much more accurate in comparison to detailed finite element method (FEM) analyses, which have been validated against test data, and to equations from the Eurocode 3 (2005).

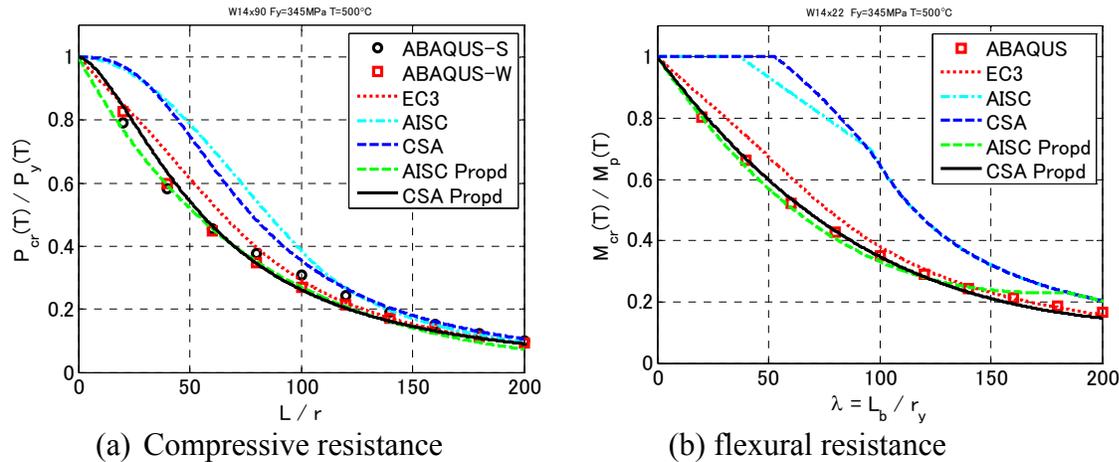


Figure 3

Comparison of compression and flexural resistances at 500°C (Takagi and Deierlein, 2009)

The stability of steel structures under fire loading is governed by the fire resistance of gravity columns because they are most likely to reach critical temperatures and structural failure due to high utilization ratios (Agarwal and Varma, 2011, 2014). The fire resistance of gravity columns may be improved due to the rotational restraints offered by cooler columns in the stories above and below. The increase in design strength can be accounted by reducing the column slenderness (KL/r) used to calculate $F_e(T)$ in Clause K.2.5.3.2 (b) to $(KL/r)_T$ as follows:

$$\left(\frac{KL}{r}\right)_T = \left(1 - \frac{T}{n(3,600)}\right) \left(\frac{KL}{r}\right) - \frac{35T}{n(3,600)} \geq 0 \quad (\text{K.2-8})$$

where

T = steel temperature, °C

- $n = 1$ for columns with cooler columns both above and below
- $n = 2$ for columns with cooler columns either above or below only

Figure 4 shows this reduction in the $(KL/r)_r$ with increasing temperature for columns with rotational restraints at both ends and one end only.

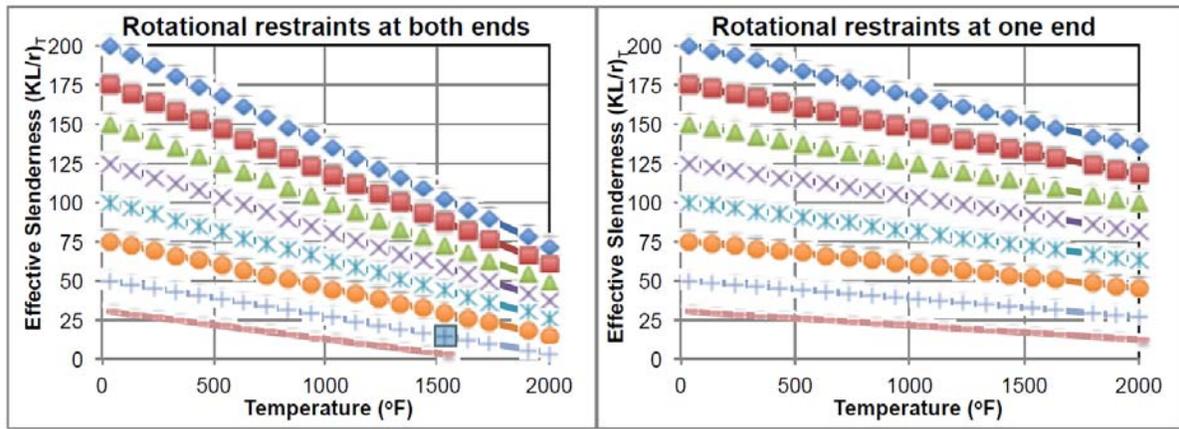


Figure 4

Effects of rotational restraints on column slenderness as a function of elevated temperature (from Equation K.2-8)

Compression members subject to uniform heating have greater heat flux from all sides than members subjected to nonuniform heating. As a result, compression members subjected to uniform heating reach their failure temperatures much earlier than members subjected to nonuniform heating. Uniform heating will be the governing case for most fire scenarios (Agarwal et al., 2014a) in terms of time to failure.

Thermal gradients due to non-uniform heating reduce the axial load capacity of compression members due to (1) elevated temperatures, (2) bowing deformations resulting from uneven thermal expansion, and (3) asymmetry in the column cross section resulting from uneven degradation of material properties (yield stress and elastic modulus). Several researchers have discussed these effects and proposed alternate design methods for columns with thermal gradients. Agarwal et al. (2014a) conducted experimental and numerical studies to develop and verify design equations for compression members with thermal gradients. The parameters included in the study were member length, cross section, and axial loading magnitude. Three different heating scenarios were considered: uniform heating, thermal gradient along the flanges, and thermal gradient along the web. The studies indicated that columns subjected to uniform heating have much greater heat influx, and therefore reach higher average temperatures faster than columns exposed to nonuniform heating. In most cases, uniformly heated columns reached their failure temperature earlier than nonuniform heated columns with thermal gradients. Exceptions were slender columns with very high axial compression (more than 50% of ambient capacity). The design strength of such columns can be calculated using equations presented in Agarwal et al. (2014a). These equations quantify the effects of (1) elevated temperature, (2) bowing, and (3) cross-section asymmetry mentioned earlier. They were verified using the results of large-scale tests and numerical parametric studies.

The factored resistances for structural steel members and connections is calculated as ϕR , in which R = nominal resistance, in which the deterioration in resistance at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal resistance is computed as in Clause 13 of the CSA S16-14, using material strength and stiffnesses at elevated temperatures defined in Tables K.1, K.2 and K.3. For limit states governed by steel yielding or fracture, the ambient equations for nominal resistance are used with elevated temperature material properties from Clause K.2.4 and the corresponding Tables. For limit states governed by buckling or instability, equations for nominal resistance are provided in this section. For example, nominal resistance equations are provided for design for compression and for flexure governed by lateral-torsional buckling.

While ECCS (2001) and Eurocode 1 (2002) 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. Research is continuing on this topic. In the interim, ambient resistance factors should be used when determining factored resistances.

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