UNIVERSITY PROGRAMS

Structural Fire Engineering: Learning Module 2b

Column Capacity Throughout a Fire

Supplemental Material

Overview

Evaluate the effects of a fire on the nominal compression capacity of a column using the equations and information in Appendix 4, Chapter E of the AISC *Specification,* and a given steel time-temperature (*T-t*) response to a fire.

Learning Objectives

- Calculate the compression capacity of a column (P_n) throughout a fire, given the steel time-temperature (*T-t*) response to a fire using the equations and information in Appendix 4 and Chapter E of the AISC *Specification*.
- calculate the compression capacity of a column (P_n) throughout a fire, given the steel time-temperature (*T-t*) response to a fire, and considering the restraint of cooler floors above and below using the equations and information in the Commentary of Appendix 4 and in Chapter E of the AISC *Specification*.
- Explain how considering temperature-dependent slenderness of the column may change the boundary conditions of the column throughout a fire.

Students should already understand

- How to calculate the nominal compression capacity of a column (P_n) at ambient temperatures and a steady elevated temperature using Chapter E and Appendix 4 of the AISC *Specification* (Module 2a).
- Students have been exposed to the concept of slenderness ratios and effective length factors for columns with varying boundary conditions (Table C-A-7.1) for ambient and a steady elevated temperature.

Additional Notes

It is under the assumption Module 2a has already been taught. This teaching module, Module 2b, expands on Module 2a.

Column Capacity Throughout a Fire

Transient vs Steady Temperatures

As discussed in the previous modules (Module 1 and 2a), steel fire tests can occur under transient or steady temperatures. Transient tests reflect real fire scenarios where load is applied to the specimen first and then with the loading held constant, the specimen is exposed to a fire scenario. The fire scenario is characterized through a time-temperature (*T-t*) curve. On the contrary, steady tests heat the object first to a set temperature and then with the temperature held constant, apply the desired load.

Structural Fire Engineering

Appendix 4 of the AISC *Specification* has clear performance objectives in Section 4.1.1:

> *"Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.*
>
> *Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires evaluation of the deformation of the load-carrying structure.*
>
> *Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation."*

These performance objectives state that to implement structural fire engineering within Appendix 4, the building must maintain load-carrying capacity throughout a design basis fire. This design-basis fire can be either the ASTM E119 fire (material covered within Prescriptive Design) or a parametric fire that is calculated based on the characteristics of a compartment, namely:

- Thermal properties of the compartment boundaries (walls, floor, ceiling)
- Size of the compartment
- Occupancy/content
- Geometry of the openings

This learning module is not going to cover the calculation of a parametric fire. This fire is characterized through a time-temperature curve (*T-t*). Therefore, to perform structural fire engineering, an engineer would need to calculate the design-basis fire, perform a heat transfer analysis (not covered in this module) to calculate the steel time-temperature response to the fire, calculate the material and member properties at every time step of the fire, and then design the member to resist the imposed loads throughout the fire. The design of the member includes choosing a steel section size with the capacity to resist the demands and/or the thickness of fire protection to reduce the temperature of the steel.

Steel Time-temperature (*T-t*) Curves

Structural Fire Engineering and Performance-based Fire Design consists of designing structural members to maintain structural integrity throughout a complete burnout fire.

Columns are critical members of the building and column failure can lead to progressive collapse of a building, therefore, to implement Structural Fire Engineering we will have to design our columns (most of the time) to maintain load carrying capacity throughout a complete design fire. Steel time-temperatures (*T-t*) curves are determined through a heat transfer analysis. The heat transfer analysis is out of the scope for this learning module, so the steel temperatures will be given in this module. The *T-t* plots display the time (Figure 1), commonly in minutes, on the x-axis and temperature in °C on the y-axis. The steel *T-t* curve is commonly graphed on the same plot as the fire *T-t* curve to display the similarities and differences.

An example of a *T-t* graph can be seen in Figure 1. The red line in Figure 1 is the gas temperature of a design fire with heating and cooling (complete burnout), the black line is the steel *T-t* response when the steel is unprotected, and the blue line is the steel *T-t* response when the steel has a 2 hour fire resistance rating (FRR). For engineers to design columns to maintain load carrying capacity throughout a complete burnout fire (red curve), the engineer must know the *T-t* response of the steel column throughout the complete burnout fire. Therefore, the response of the steel material to a fire is important.

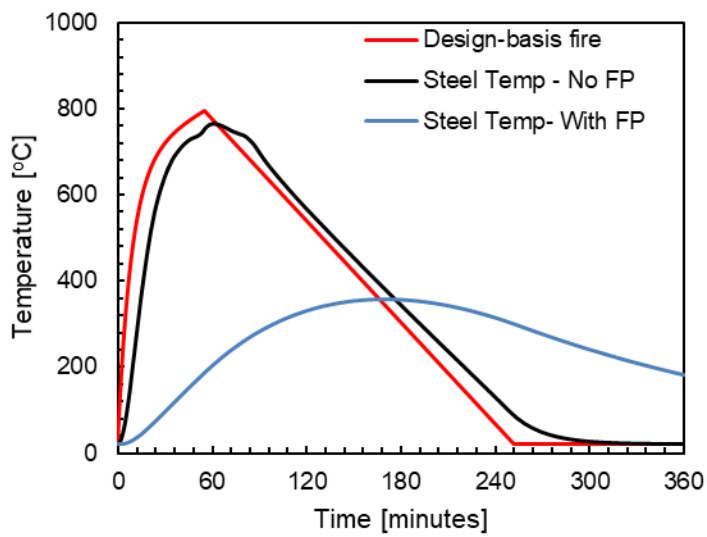


Figure 1: Fire and Steel Time-temperature Curve

Discussion Questions

1. What causes the offset between the fire temperature and the temperature of the unprotected steel (the red curve and the black curve)?
2. Why is there a "bump" around 750°C for the steel *T-t* response to the fire (circled in yellow)?

Answer:

As shown in Figure 1, the steel temperature is not the same as the fire temperature. This delay is due to the process of convection and radiative heat transfer. The steel T-t response reflects phase changes within the steel material itself. For instance, the "bump", as circled in Figure 1, at the peak temperature of the fire represents the phase change of steel at around 750°C.

The T-t curves give the engineer important information on how the fire affects the steel temperature. An unprotected steel member will have a similar response to the fire T-t curve as seen in Figure 1, however a protected steel member (e.g. columns) would have a different T-t than the fire due to heat transfer through the fire protection layers. A protected steel member is represented as the solid blue line in Figure 1. In later modules, there will be a more in-depth discussion of how fire protective materials influence the steel T-t curves.

Mechanical Properties of Steel throughout a Fire

For steel temperatures less than or equal to 400°F, the member and connection design strengths shall be determined without consideration of temperature effects.

Steel material properties at elevated temperatures

As discussed in Module 2a, steel strength and stiffness properties decrease as the temperature increases. If we, as engineers, know the temperature of the steel at each time increment throughout the fire, we can correlate this with the material property retention factors in AISC *Specification* Table A-4.2 shown in Table 1.

Table 1. Retention factors for structural steel per AISC *Specification* Appendix 4

TABLE A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E = G(T)/G$	$k_p = F_p(T)/F_y$	$k_y = F_y(T)/F_y$	$k_u = F_u(T)/F_y$
68 (20)	1.00	1.00	*	*
200 (93)	1.00	1.00	*	*
400 (200)	0.90	0.80	*	*
600 (320)	0.78	0.58	*	*
750 (400)	0.70	0.42	1.00	1.00
800 (430)	0.67	0.40	0.94	0.94
1000 (540)	0.49	0.29	0.66	0.66
1200 (650)	0.22	0.13	0.35	0.35
1400 (760)	0.11	0.06	0.16	0.16
1600 (870)	0.07	0.04	0.07	0.07
1800 (980)	0.05	0.03	0.04	0.04
2000 (1100)	0.02	0.01	0.02	0.02
2200 (1200)	0.00	0.00	0.00	0.00

*Use ambient properties

As with the steady state temperature, the retention factors for the steel's response to a fire are used to calculate the elevated temperature material properties by multiplying the retention factors in Table 1 by the ambient material property value (E, F_y, F_u). The values in Table 1 can be linearly interpolated between temperatures, and are consistent with Eurocode 3, Part 1-2.

Example

Calculate the *Young's Modulus* retention factor, k_E, when the steel has a temperature of 497°C.

Referencing Table 1: At 430°C, k_E = 0.67 and at 540°C, k_E = 0.49.

To find the k_E value at 497°C, the *Young's Modulus* retention factors at 430 °C and at 540°C are interpolated.

$$\frac{0.67 - 0.49}{430°\text{C} - 540°\text{C}} = \frac{0.97 - k_E(497°\text{C})}{430°\text{C} - 497°\text{C}}$$

$$k_E(497°\text{C}) = 0.786$$

For practice, pick a different steel temperature and find its corresponding retention factors using Table 1 and interpolation if necessary.

This same interpolation process is applied to the remaining temperatures and temperature-dependent values. Figure 2 depicts the retention factors at elevated temperatures.

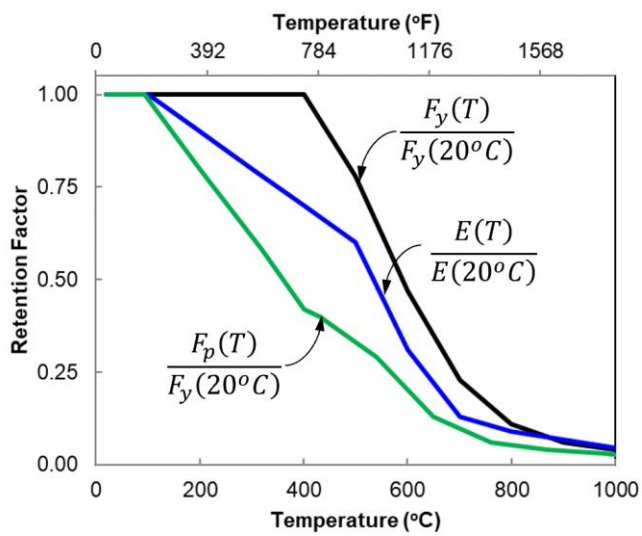


Figure 2: Retention Factors at Elevated Temperatures

Note, from Figure 2 and Table 1, that the retention factors do not degrade with temperature linearly. The retention factors for *Yield Stress*, k_y, and the *Ultimate Stress*, k_u, do follow the same degradation, but the other retention factors do not degrade the same way. Due to the retention factors having varying degradations, each must be calculated separately. In Figure 2, the retention factor for the yield stress, k_y, degrades at the faster rate the retention factor for the *Young's Modulus*, k_E. Both the proportional limit and *Young's Modulus* retention factors being reducing at 200°C, whereas both the *Yield Stress* and *Ultimate Stress* do not begin reducing until 430°C. At 1200°C, all the retention factors merge to 0, indicating the material does not have any remaining strength or stiffness.

Each retention factor needs to be interpolated separately when a steel temperature falls within a given value in Table 1. Once the retention factors have been calculated for each temperature, they are multiplied by the ambient material property (E, F_y, F_u) to find the elevated temperature material property, as shown in the Example.

Nominal capacity of the column at elevated temperatures per Appendix 4

These retention factors will be used to calculate the nominal column buckling capacity (P_n) using Chapter E and Appendix 4 of the AISC *Specification*. The critical buckling stress of the column is calculated per Equation A-4-2 in the AISC *Specification*.

$$F_{cr}(T) = \left[0.42^{\sqrt{F_y(T)/F_e(T)}}\right] * F_y(T)$$ AISC *Specification* Equation A-4-2

Where, $F_{cr}(T)$ is the temperature-dependent critical buckling stress and $F_e(T)$ is the temperature-dependent critical elastic buckling stress (Euler buckling stress) calculated using AISC *Specification* Equation E3-4, shown below:

$$F_e(T) = \frac{\pi^2 * E(T)}{\left(\frac{L_c}{r}\right)^2}$$ AISC *Specification* Equation E3-4

Where, $\frac{L_c}{r}$ is the slenderness ratio of the heated column, and L_c is the effective length of the column and *r* is the radius of gyration of the column section.

Like the retention factors, using an Excel spreadsheet or other common computer software (e.g. Python, MATLAB) will aid in substituting the elevated temperature material properties ($E(T)$, $F_y(T)$, $F_u(T)$) into Equation 1 and Equation 2 above.

Appendix 4 and Chapter E assume the slenderness ratio does not change with temperature. However, the restraint from thermal expansion that cooler floors above and below the heated column, has been shown to change the slenderness ratio with temperature.

Restraint of heated column by cooler floors above and below

The cooler columns above and/or below will act as rotational restraints. These restraints decrease the temperature-dependent slenderness ratio, increasing the temperature-dependent Euler buckling stress, $F_e(T)$, and temperature-dependent critical buckling stress, $F_{cr}(T)$. Typical gravity columns have pinned boundary conditions as shown in Figure 3a. These pinned boundary conditions allow for rotation at the column end as shown by the deformed column shape in Figure 3a. However, when considering a heated column with a cooler floor above and below, the rotation of the column ends is restrained as shown in Figure 3b. This restraint will change the buckling shape of the column.

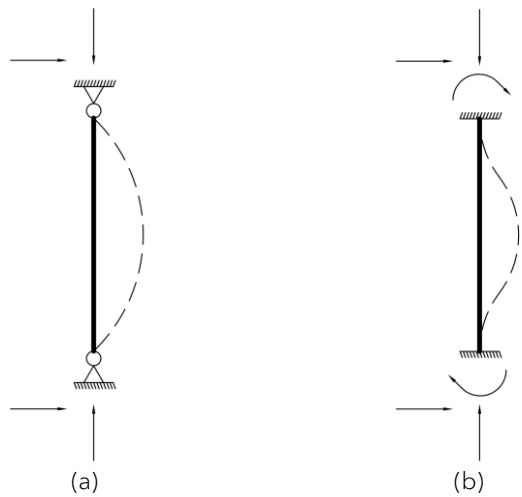


Figure 3: Deformed Shape of a Column; (a) with pin-pin restraints and (b) with pin-pin restraints and cooler floors above and below the heated column

The dotted line depicts the deformed shape of the column, whereas the solid line is the undeformed shape. When the column has pin-pin restraints, the deformed shape has rotation at the ends of the column. When the column has cooler floors above and below the heated column, the floors act as rotational restraints making the column act as if it has fixed- fixed boundary conditions. The cooler floors prevent rotation at the ends of the column, causing the deformation to start further away of the ends. As the rest of this learning module will explain, the cooler floors can increase the nominal compression capacity compared to when the changing boundary conditions are not considered.

These cooler floors above and below are intended to be stories that do not have smoke infiltration or fires thereby keeping these floors near ambient temperatures.

Consideration of temperature-dependent slenderness ratio due to column restraint

The commentary of Appendix 4 provides guidance for engineers to calculate temperature-dependent slenderness ratios that depend on the temperature of the column and if the column has cooler floors above and/or below.

When considering the slenderness is temperature-dependent, AISC *Specification* Equation C-A-4-9 considers the potential increased strength due to the restraint on thermal expansion from cooler floors above and below the heated column. Equation C-A-4-9 shows how the temperature-dependent slenderness ratio is calculated.

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T-32}{n(3600)}\right)\left(\frac{L_c}{r}\right) - \frac{35}{n(3600)}(T-32) \geq 0 \qquad \text{AISC } \textit{Specification} \text{ Equation C-A-4-9}$$

T is the temperature of the steel column in Fahrenheit and *n* considers if there is only a cooler floor (non-heated floor) above or below, or if there are cooler floors above and below the heated column. If both the columns above and below the affected column are cooler, then *n* is taken as 1. If only one of the columns either above or the below the affected column is cooler, then *n* is taken as 2.

As discussed above, the same process of finding the temperature dependent slenderness ratio is easily computed using Excel or a similar computer program.

Once $\left(\frac{L_c}{r}\right)_T$ is calculated, this value is then used in Equation E3-4 of the AISC *Specification* to calculate the Euler buckling stress of the column at elevated temperatures. The rest of the calculations for the nominal compression capacity of the column are the same as previously described.

Research performed by Agarwal and Varma (2011) found the slenderness ratio has a lower bound of 10.5. However, using the fundamentals of structural mechanics, the slenderness ratio cannot be less than that of a column with fix-fix boundary conditions (refer to Figures 4a and 4b in the later example problem). For the purposes of this teaching module, we will remain consistent with this approach.

Resistance Factor (ϕ)

The resistance factor for design, ϕ , is applied the same as with ambient temperatures. The resistance factor can be found in Chapter E of AISC *Specification* when designing the member for compression.

Load Combinations in a Fire

If the applicable building code (ABC) does not provide provisions for design under fire exposures, then the required load combination can be found in Equation A-4.1of Appendix 4 of the AISC *Specification*.

$$(0.9\ or\ 1.2)D + A_T + 0.5L + 0.2S$$ AISC *Specification* Equation A-4.1

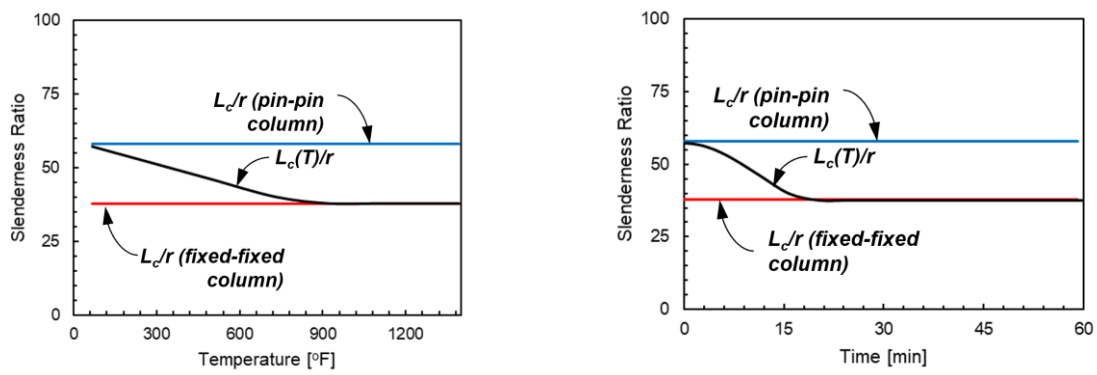
Where A_T is the nominal forces and deformation due to the design-basis fire defined in Section 4.2.1. of the AISC *Specification*. *D* is the nominal dead load, *L* is the nominal occupancy live load, and *S* is the nominal snow load. The 0.9 and 1.2 factor in front of the dead load is dependent if ASD or LRFD load cases are used.

At ambient conditions, ASCE 7 provides the load combinations required in Section 2.3. The load combinations during a fire and at ambient temperature are different. In a fire, the load combination requires the forces and deformations due to a fire and the factors in front of the nominal loads are less than they are at ambient temperature. Therefore, due to the load combinations, there will be less gravity loads on a column during a fire than there is at ambient temperature. For simplified design using the AISC *Specification,* the A_T term of Equation A-4.1 is ignored, thereby not considering deformations and forces induced in the member due to fire effects.

Example Problem

Provided in Figure 1 (and the provided spreadsheet) is a fire's *T-t* curve (red curve) and the steel's response to the fire (black curve) for an unprotected column (has no fire protective material). The steel member is a W14x82 column with an unbraced length (L_c) of 12ft at ambient temperature. The column is designed as a gravity column with pin-pin boundary conditions.

As discussed, the slenderness ratio of a column can change as a function of temperature due to the rotational restraint of cooler floors above and below. Figure 4 displays three different slenderness ratios as a function of temperature. The blue line is the slenderness ratio of the gravity column not considering the rotational restraint of cooler floors above and below the heated column. As shown, and supported by Appendix 4, this slenderness ratio remains constant throughout the fire (58.1). The red line is the slenderness ratio of the column if it had fixed-fixed boundary conditions and an effective length factor (k) of 0.65.



(a) (b)

Figure 4: Slenderness ratio vs. (a) temperature and (b) time

Discussion Questions

Q1. How does the temperature-dependent slenderness ratio (black line in Figure 4a) change as a function of temperature?

Q2. At what steel temperature is the temperature-dependent slenderness ratio equal to that of a fixed-fixed column for a column with the same height and section size?

Q3. How will the answer to Q2 relate to the column buckling capacity?

Answers:
The black line represents the temperature-dependent slenderness ratio that considers pin-pin boundary conditions and cooler floors above and below the heated column. As shown in Figure 4a, this temperature-dependent slenderness ratio has a constant declining slope. At ambient temperature, the slenderness ratio starts at a value close to the slenderness ratio of the pin-pin slenderness ratio.

At approximately 450°C (842°F), the temperature-dependent slenderness ratio is equal to the slenderness ratio of the fixed-fixed boundary conditions. After the 450°C (842°F), the temperature-dependent slenderness ratio continues to decrease at a constant rate.

The slenderness ratio is inversely related to the column buckling capacity, therefore the higher the value for the slenderness ratio then the lower the buckling capacity is. While the temperature-dependent starts with the greatest slenderness ratio at ambient temperature, it quickly decreases as the temperature increases. This is important to note when considering the buckling strength of the column, which can be seen in Figure 5.

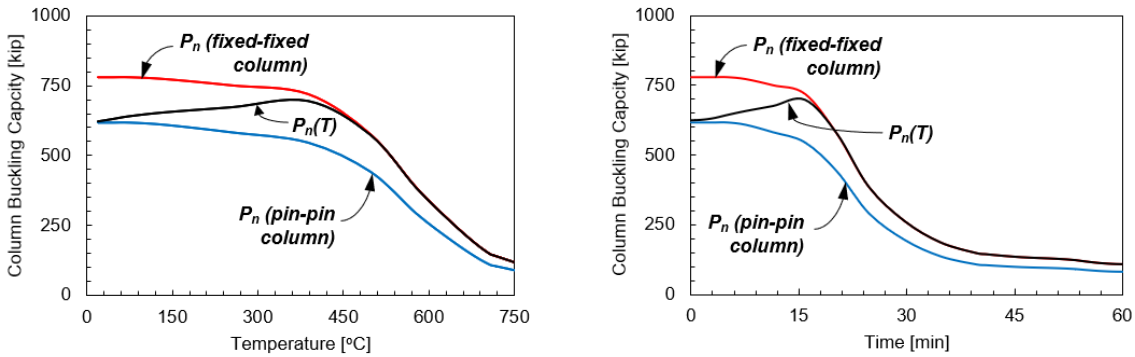


Figure 5: Column buckling capacity vs. (a) temperature and (b) time

Discussion Questions

Q4. How would we describe the relationship between the blue and red lines (column buckling capacity for a pin-pin column and fixed-fixed column) shown in Figure 5? Is the difference between the two constant as a function of temperature?

Q5. At ambient temperature what is the relationship between the blue, black, and red lines in Figure 5? Does this make sense?

Q6. At what temperature does the column buckling capacity change from aligning with a pin-pin column to more of a fixed-fixed column?

Q7. Do the boundary conditions actually change? Hypothesize what the implications are for the remainder of the framing and connections due to the rotational restraint of cooler floors above and below?

Answer:

The fixed-fixed column buckling capacity and the pin-pin column buckling capacity (red and blue lines, respectively) have the same shape and trend as a function of temperature. However, there is a greater difference between the two at lower temperature than at elevated temperatures.

The temperature-dependent buckling capacity at ambient temperature is equivalent to the pin-pin buckling capacity value for the column. However, as temperature increases between ambient and about 450°C, the buckling capacity of the column considering a temperature-

dependent slenderness ratio gradually increases. At approximately 450°C, the temperature-dependent buckling capacity equals the fixed-fixed buckling capacity value and then starts to follow a similar shape as the fixed-fixed curve, decreasing as a function of temperature.

450°C was the temperature in Figure 4 where the temperature-dependent slenderness ratio equaled the fixed-fixed slenderness ratio. At ambient temperature, cooler floors above and below the heated column will not impact the rotational restraint of the column itself, therefore, the buckling capacity of the column considering a temperature-dependent slenderness ratio should align with the Chapter E value at ambient temperature. However, as the column is heated and thermal elongation is restricted we would expect a greater rotational restraint by the cooler floors above and below the heated column.

Figures 4 and 5 illustrate the inverse relationship between the slenderness ratio and the buckling capacity. This is especially seen through the temperature-dependent slenderness ratio and temperature-dependent column buckling capacity where at ambient temperature the slenderness ratio and buckling capacity values are both of equal value to the pin-pin boundary conditions and then at 450°C the temperature-dependent values begin to more closely resemble the values of the fixed-fixed boundary condition values.

Regardless of the rotational restraint of the column by the cooler floors above and below, it is important to note that the column was originally designed for a pin-pin boundary condition at ambient temperature. Therefore, this additional rotational restraint may impose flexural demands on the column and connections that they were not designed for.

Additional Resources

Agarwal, A. and Varma, A.H. (2011). "Design of Steel Columns at Elevated Temperatures Due to Fire: Effects of Rotational Restraints," *Engineering Journal*, American Institute of Steel Construction, Vol. 48, pp. 297-314. (*Available for free download for AISC Educator and Student Members* aisc.org/Design-of-Steel-Columns-at-Elevated-Temperatures-Due-to-Fire-Effects-of-Rotational)

AISC (2016), *Specification for Structural Steel Buildings* (Appendix 4), Standard ANSI/AISC 360-16, American Institute of Steel Construction, Inc., Chicago. IL. *(Available for free download at* aisc.org/Specification-for-Structural-Steel-Buildings-ANSIAISC-360-16-1*)*

ASTM E119-20 (2020), *Standard Test Methods for Fire Tests of Building Construction and Materials*, American Society for Testing and Materials, West Conshohocken, PA. (https://www.astm.org/Standards/E119.htm)