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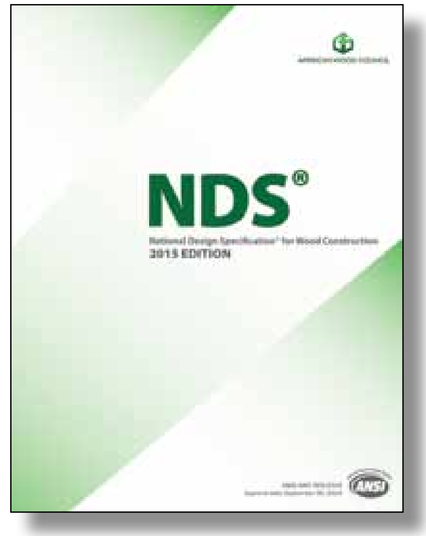

TECHNICAL REPORT NO. 10

CALCULATING THE FIRE RESISTANCE OF EXPOSED WOOD MEMBERS

The American Wood Council (AWC) is the voice of North American traditional and engineered wood products. From a renewable resource that absorbs and sequesters carbon, the wood products industry makes products that are essential to everyday life. AWC's engineers, technologists, scientists, and building code experts develop state-of-the-art engineering data, technology, and standards on structural wood products for use by design professionals, building officials, and wood products manufacturers to assure the safe and efficient design and use of wood structural components.

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Table of Contents

List of Figures

List of Tables

List of tables continued on next page

Appendix

Part 1: Development of Design Procedures for Exposed Wood Members

1.1 Introduction

Wood members have long been recognized for their ability to maintain structural integrity while exposed to fire. Early mill construction from the 19th century utilized timbers to carry large loads and to resist structural failure from fire. Exposed wood structural members are popular with architects and designers of modern buildings because they have a pleasing appearance, are economical and easy to use, and provide necessary fire resistance. Glued laminated (glulam) members are now commonly used where large sections and long spans are needed. Glulam members are composed of smaller laminates that are glued together. The small-section laminates are readily available. Glulam members offer the same fire performance advantages as large sawn members. Extensive research has demonstrated that adhesives used in the manufacture of glulam do not adversely affect fire performance [1].

The superior fire performance of timber can be attributed to the charring effect of wood. As wood members are exposed to fire, an insulating char layer is formed that protects the core of the section. Thus, beams and columns can be designed so that a sufficient cross section of wood remains to sustain the design loads for the required duration of fire exposure. A standard fire exposure is used for design purposes. In North America, this exposure is described in the standard fire resistance test ASTM E 119 [2]. Many other countries use a comparable test exposure found in ISO 834 [3]. In spite of the differences between standard fire resistance tests, experimental charring rates measured in various parts of the world appear to be consistent. This justifies the use of such data for design, regardless of origin.

1.2 Concepts of Fire Design of Wood

At fire exposure time, *t*, the initial breadth, *B*, and depth, *D*, of a member are reduced to *b* and *d*, respectively. This is illustrated in *Figure 1-1* for a section of a beam exposed on three sides. The original section is rectangular. However, since the corners are subject to heat transfer from two

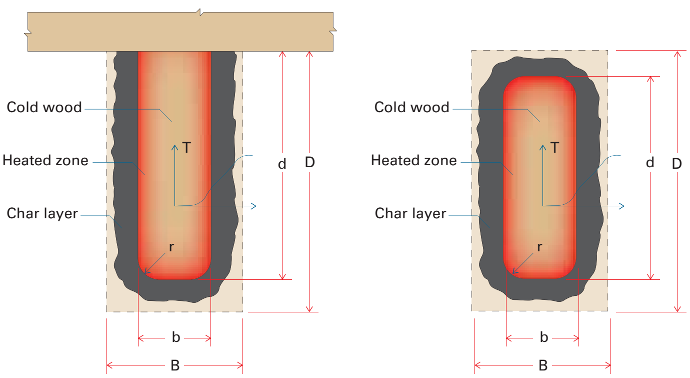

Figure 1-1 Reduction in member breadth and depth over time, t

directions, charring is faster at these corners. This has a rounding effect, and shortly after ignition the remaining cross section is no longer rectangular. The boundary between the char layer and the remaining wood section is quite distinct, and corresponds to a temperature of approximately 550°F. The remaining wood section is heated over a narrow region that extends approximately 1.5 inches from the char front. The inner core of the remaining wood section is at ambient (or initial) temperature. A section, smaller than the original section, is capable of supporting the design load because of the margin of safety provided in cold design. The original section is stressed only to a fraction of the maximum capacity. Failure occurs when the remaining cross section is stressed beyond the maximum capacity.

For members stressed in bending during fire exposure, failure occurs when the maximum bending capacity is exceeded due to the reduction in section modulus, *S*. For members stressed in tension parallel-to-grain during fire exposure, failure occurs when the maximum tension capacity is exceeded due to the reduction in cross-sectional area, *A*.

For members stressed in compression parallel-to-grain during fire exposure, the failure mode is a function of the column slenderness ratio, (L_e/D). The column slenderness ratio changes with exposure time. For short column members ($L_e/D \approx 0$) stressed in compression during fire exposure, failure occurs when maximum compressive capacity is exceeded due to the reduction in cross-sectional area, *A*. For long column members ($L_e/D \approx \infty$) stressed in compression during fire exposure, failure occurs when critical buckling capacity is exceeded due to the reduction in the moment of inertia, *I*. Current code-accepted design procedures in the *2012 National Design Specification® for Wood Construction* (*NDS®*) contains a single column equation which is used to calculate a stability factor, C_p, which approximates the column capacity for all slenderness ratios based on the calculated interaction of theoretical short and long column capacities [9].

1.3 Background

For over 20 years, the only building code-accepted design method for fire-resistive exposed wood members used in North America was based on analysis conducted by T.T. Lie at the National Research Council of Canada in the 1970s [4]. The method was first recognized by the U.S. model building codes in 1984 through a National Evaluation Report [5]. In subsequent years, the method was adopted by the three model code organizations, allowing engineers and architects to include fire-rated heavy timber members in their projects without conducting expensive standard fire resistance tests.

Lie assumed a charring rate of 1.42 in/hr, and accounted for a reduction in strength and stiffness due to heating of a small zone progressing over approximately 1.5 inches ahead of the char front. Lie reported that studies have shown that the ultimate strength and stiffness of various woods, at temperatures that the uncharred wood normally reaches in fires, reduces to about 0.85-0.90 of the original strength and stiffness. To account for this effect, reductions to strength and stiffness properties were implemented by uniformly reducing strength and stiffness values over the remaining cross section by a factor α. Furthermore, a factor *k* was introduced to account for the ratio of design strength to ultimate strength. To obtain conservative estimates, Lie recommended a *k* factor of 0.33 based on a safety factor of 3, and an α factor of 0.8 to account for a strength and stiffness reduction.

Lie ignored increased rate of charring at the corners, and assumed that the remaining section is rectangular. With

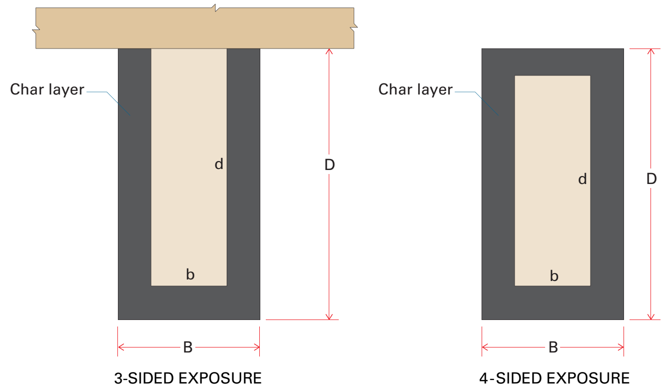

Figure 1-2 Reduction of member over time, t, per Lie's method

this assumption, initial breadth *B* and depth *D* of a member after *t* minutes of fire exposure are reduced to *b* and *d* respectively, as shown in *Figure 1-2*. Both *b* and *d* are a function of exposure time, *t*, and charring rate, *β*. Assuming the charring rate is identical in every direction, the exposure time *t* and the dimensions of the initial and remaining cross section are related via the charring rate, *β*:

$$t = \begin{cases} \dfrac{B-b}{2\beta} = \dfrac{D-d}{2\beta} & \textit{4-sided exposure} \\ \dfrac{B-b}{2\beta} = \dfrac{D-d}{\beta} & \textit{3-sided exposure} \end{cases} \qquad \text{Equation 1-1}$$

1.3.1 Beams

Lie's method assumed that a beam fails when the reduction in cross section results in a critical value for the section modulus *S* being reached. Assuming a safety factor reduction of *k*, a load factor of *Z*, and a uniform reduction in strength properties of α, the critical section is determined from:

$$kZ\,\frac{BD^2}{6} = \alpha\,\frac{bd^2}{6} \qquad \text{Equation 1-2}$$

Given the initial dimensions *B* (width) and *D* (depth), the fire resistance time can be calculated by combining equations (1-1) and (1-2), and solving the resulting equation for *t*. The roots to the resulting equations must be solved iteratively. To avoid these cumbersome iterative procedures, Lie approximated his solutions with a set of simple equations that allow for a straightforward calculation of fire resistance time as a function of member size for a realistic range of member dimensions. Lie approximated the solutions for $\alpha = 0.8$ and $k = 0.33$ to:

$$t_f = \begin{cases} 2.54\,Z\,B\,(4 - \frac{2B}{D}) & \text{4-sided exposure} \\ 2.54\,Z\,B\,(4 - \frac{B}{D}) & \text{3-sided exposure} \end{cases} \qquad \text{Equation 1-3}$$

with

$$Z = \begin{cases} 1.3 & R < 0.5 \\ 0.7 + \frac{0.3}{R} & R \geq 0.5 \end{cases} \qquad \text{Equation 1-4}$$

where R is the ratio of applied to allowable load, t_f is in minutes, and all dimensions are in inches.

1.3.2 Columns

As noted in Section 1.3.1, column failure mode depends on the slenderness ratio. Short columns fail when the reduction in cross section results in a critical value for the cross-sectional area A being reached. Assuming a safety factor reduction of k, a load factor of Z, and a uniform reduction in strength properties of α, the critical section is determined from:

$$k\,Z\,BD = \alpha\,bd \qquad \text{Equation 1-5}$$

Long columns fail when the reduction in cross section results in a critical value for the moment of inertia I being reached. Assuming a safety factor reduction of k, a load factor of Z, and a uniform reduction in strength properties of α, the critical section is determined from:

$$k\,Z\,\frac{BD^3}{12} = \alpha\frac{bd^3}{12} \qquad \text{Equation 1-6}$$

where D denotes the narrowest dimension of a column section and buckling is assumed to occur in the weakest direction.

Again, given the initial dimensions B (widest dimension) and D (narrowest dimension), the fire resistance time can be calculated for short columns by combining equations (1-1) and (1-5) or for long columns by combining equations (1-1) and (1-6). Again, to avoid the cumbersome iterative solution of these equations, Lie approximated his solutions with a set of simple equations using equation (1-2) as an average between equation (1-5) for short columns and equation (1-6) for long columns. Therefore, Lie approximated the solutions for $\alpha = 0.8$ and $k = 0.33$ to:

$$t_f = \begin{cases} 2.54\,Z\,D\,(3 - \frac{D}{B}) & \text{4-sided exposure} \\ 2.54\,Z\,D\,(3 - \frac{D}{2B}) & \text{3-sided exposure} \end{cases} \qquad \text{Equation 1-7}$$

where Z for short columns ($K_e l/D \leq 11$) follows from

$$Z = \begin{cases} 1.5 & R < 0.5 \\ 0.9 + \frac{0.3}{R} & R \geq 0.5 \end{cases} \qquad \text{Equation 1-8}$$

where Z for long columns ($K_e l/D > 11$) follows from

$$Z = \begin{cases} 1.3 & R < 0.5 \\ 0.7 + \frac{0.3}{R} & R \geq 0.5 \end{cases} \qquad \text{Equation 1-9}$$

where R is the ratio of applied to allowable load, t_f is in minutes, and all dimensions are in inches.

To determine the fire resistance of columns, Lie used the geometric mean of the equations for the extreme cases of short and long columns. Lie assumed that short columns fail due to crushing, and long columns fail due to buckling. In order to correct for underprediction of failure times for short columns, Lie recommended an increase to the load factor for such columns. In 1991, the NDS provisions for columns were changed from three equations for different ranges of slenderness to a single equation [9]. As a result, Lie's methodology for columns is not consistent with the current procedure for structural design of wood members.

Lie verified his method against experimental data from full-size column tests conducted in France [6], England [7], and Germany [8] in the 1960s and early 1970s. In his original paper [4], Lie noted that no beam data was available for comparison. Lie assumed that his calculation method would be valid for beams also, since it was based on the same assumptions and concepts as that for columns. Since Lie's initial work, standard fire test data has now been published for at least 7 heavy timber beams [16][17][18][23].

1.4 Mechanics-Based Design Method

Lie's design method for exposed wood members was based on actual fire test results and sound engineering; however, since the final equations were based on empirical solutions fit to limited beam and column test data, assumed loading and bracing conditions, and typical exposures, the application was limited. In 1999, a new mechanics-based design method was developed to permit the calculation of fire resistance for exposed wood members for other loading conditions and fire exposures not considered by Lie.

The new mechanics-based design method calculates the capacity of fire-resistive exposed wood members using the mechanics assumed by Lie. Failure of a member occurs when the load on the member exceeds the member capacity which has been reduced due to fire exposure. However, actual mechanical and physical properties are used and the capacity of the member is directly calculated for a given period of time. Section properties are computed assuming an effective char rate, β_{eff}, at a given time, t. Average member strength properties are approximated from test data or from procedures used to calculate design properties.

1.4.1 Char Rate

To estimate the reduced cross-sectional dimensions, b and d, the location of the char base must be determined as a function of time on the basis of empirical charring rate data. The char layer can be assumed to have zero strength and stiffness. The physical shape of the remaining section and its load carrying capacity should be adjusted to account for rounding at the corners, and for loss of strength and stiffness in the heated zone. In design there are various documented approaches to account for these affects:

- additional reduction of the remaining section [10][11];
- uniform reduction of the maximum strength and stiffness [4][10][12]; or
- more detailed analysis with subdivision of the remaining section into several zones at different temperatures [13][14].

Extensive char rate data is available for one-dimensional wood slabs. Data is also available for two-dimensional timbers, but most of this data is limited to larger cross-sections. Evaluation of linear char rate models using one-dimensional char rate data suggests that charring of wood is: slightly nonlinear, and estimates using linear models tend to underestimate char depth for short time periods (<60 minutes) and overestimate char depth for longer time periods (>60 minutes). One method for correcting for non-linear char is the use of empirical adjustments, such as the addition of an artificial "char time," t_c:

$$d_{char} = \beta\,(t+t_c) \qquad \text{Equation 1-10}$$

However, these types of corrections are awkward to handle in fire resistance models and tend to over-compensate when adjusting for shorter time periods.

To account for char rate nonlinearity, White developed a nonlinear, one-dimensional char rate model based on the results of 40 one-dimensional wood slab charring tests of various species [24]. White's non-linear model addressed accelerated charring which occurs early in the fire exposure by applying a power factor to the char depth, x_{char}, to adjust for char rate nonlinearity:

$$t = m\,(x_{char}{}^{1.23}) \qquad \text{Equation 1-11}$$

However, application of White's model is limited since the char slope (min/in$^{1.23}$), m, is species-specific and only limited data exists for different wood species fit to White's model. In addition, the model is limited to one-dimensional slabs.

To develop a two-dimensional, nonlinear char rate model, White's non-linear char rate model was modified to enable values for the slope factor m to be estimated using nominal char rate values (in/hr), β_n. The nominal char rate values, β_n, are calculated using measured char depth at approximately one hour. Substitution of this value allows the calculation of the slope factor:

$$1\ hour = m[(1\ hour)\ (\beta_n)]^{1.23}$$

$$m = \beta_n{}^{-1.23} \qquad \text{Equation 1-12}$$

Substituting and solving for the char depth, x_{char} in terms of time, t:

$$x_{char} = \beta_n\, t^{\,0.813} \qquad \text{Equation 1-13}$$

To account for rounding at the corners and reduction of strength and stiffness of the heated zone, the nominal char rate values, β_n, are increased 20%. The effective char rate can be estimated as:

$$\beta_{eff} = \frac{1.2\,\beta_n}{t^{0.187}} \qquad \text{Equation 1-14}$$

The section properties can be calculated using standard equations for area, section modulus and moment of inertia using reduced cross-sectional dimensions. The dimensions are reduced by $\beta_{eff}\,t$ for each surface exposed to fire. Cross-sectional properties for a member exposed on all four sides are shown in *Table 1.4.1*. Other exposures can be calculated using this method.

Table 1.4.1 Cross-Sectional Properties for Four-Sided Exposure

Cross-sectional Property	**Four-Sided Example**
Area of the cross-section, in^2	$A(t) = (B - 2\beta_{eff}\ t)(D - 2\beta_{eff}\ t)$
Section Modulus in the major-axis direction, in^3	$S(t) = (B - 2\beta_{eff}\ t)(D - 2\beta_{eff}\ t)^2/6$
Section Modulus in the minor-axis direction, in^3	$S(t) = (B - 2\beta_{eff}\ t)^2(D - 2\beta_{eff}\ t)/6$
Moment of Inertia in the major-axis direction, in^4	$I(t) = (D_{min} - 2\beta_{eff}\ t)(D_{max} - 2\beta_{eff}\ t)^3/12$
Moment of Inertia in the minor-axis direction, in^4	$I(t) = (D_{min} - 2\beta_{eff}\ t)^3(D_{max} - 2\beta_{eff}\ t)/12$

Table 1.4.2 Allowable Design Stress to Average Ultimate Strength Adjustment Factors

	F	1/*k*	*c*	*Assumed COV*	*K*
Bending Strength	F_b	2.1[1]	1-1.645 COV_b	0.16[2]	2.85
Tensile Strength	F_t	2.1[1]	1-1.645 COV_t	0.16[2]	2.85
Compression Strength	F_c	1.9[1]	1-1.645 COV_c	0.16[2]	2.58
Buckling Strength	E_{05}	1.66[3]	1-1.645 COV_E	0.11[4]	2.03

[1] Taken from Table 10 of ASTM D 245 Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.
[2] Taken from Table 5-6 of 2010 Wood Handbook for bending clear wood values.
[3] Taken from Appendices D and H of 2012 National Design Specification for Wood Construction.
[4] Taken from Appendix F of 2012 National Design Specification for Wood Construction.

Sides of individual timber decking members are shielded from full fire exposure by adjacent members collectively acting as a joint. Partial exposure can occur as members shrink and joints between members open. The degree of exposure is a function of the view angle of the radiant flame and the ability of hot volatile gases to pass through the joints. When the joint is completely open, such as can occur with butt-jointed timber decking, hot gases will carry into the joint and the sides of the decking members will char. This charring can be conservatively approximated assuming the sides of a member along the joint char at the effective char rate. When the joint is open but covered by sheathing, as with butt-jointed timber decking covered with wood structural panels, passage of hot gases is limited, and tests have shown that charring can be approximated, assuming a partial exposure char rate along the joint equal to one-third of the effective char rate [22]. For joints which are not open, as with tongue-and-groove timber decking, tests have shown that charring of the sides of members is negligible and can be ignored [21][22].

1.4.2 Approximation of Member Strength

Average unheated member strength can be approximated using allowable stress design (ASD) values. To estimate a lower bound of the average member strength, the ASD value can be multiplied by an adjustment factor, K, to adjust from an ASD value based on a 5% exclusion value to an average ultimate strength. The adjustment factor, K, has two components, the inverse of the applicable design value adjustment factor from ASTM D245 [15], denoted as $1/k$, and the inverse of the variability adjustment factor, denoted as c. To develop general design procedures for wood members, the D245 adjustment factors and estimates of COV listed in *Table 1.4.2* were used. The assumed COV values are estimates from clear wood properties.

1.4.3 Approximation of Member Capacity

As noted, average member capacity of a wood member exposed to fire for a given time, t, can be estimated using cross-sectional properties reduced for fire exposure time and average ultimate strength properties derived from allowable stress values.

Part 2: Comparison of Calculation Methods and Experiments

2.1 General

Given the theoretical derivation of the mechanics-based design method, existing test results from fire tests of exposed wood members was compared against the model predictions. International, as well as North American, test data were reviewed. The results indicate that the mechanics-based method will accurately estimate the fire resistance time of tested wood members. Overall, the mechanics-based model is more accurate than the T. T. Lie method, which also has limits as discussed in *Part 1*.

2.2 Beams

Lie was not able to compare his calculation method to experimental data for beams, because such data were not available [4]. Nonetheless, he assumed that it would be valid because the method for beams is conceptually identical to that for columns. At least 7 standard beam tests have been reported in the literature since Lie completed his work.

The Timber Research and Development Association (TRADA) in the United Kingdom conducted a series of tests on glulam beams in 1968 [16]. Only one of the tests was not terminated prior to structural failure, which occurred after 53 minutes of exposure to standard BS 476 fire conditions (similar to ISO 834). The ratio of induced load to design load was 80% for this test [13]. The reported allowable stresses were F_b = 2100 psi and E = 2.0E6 psi. The report also contained information which permitted the average ultimate bending strength to be estimated as $F_{b\text{-ult}}$ = 7530 psi. Each beam was braced against lateral translation and rotation at the supports and was loaded through 11 evenly spaced bearing blocks; therefore, an effective length, l_e = 1.84 l_u (l_u = full span), was assumed. Using the 2012 NDS behavioral equations, the resisting moment was estimated to be 45,340 ft-lbs compared to an induced moment of 9,830 ft-lbs.

The National Forest Products Association (NFoPA) (now the American Wood Council), sponsored a test on a Douglas fir glulam beam in 1986 [17]. The beam collapsed after 86 minutes of standard ASTM E 119 fire exposure. The ratio of induced load to design load was 72% for this test [13]. The reported allowable stresses were F_b = 2400 psi and E = 1.6E6 psi. Using the 2.85 allowable design stress to average ultimate strength adjustment factor derived in *Part 1* (see *Table 1.4.2*), the average ultimate bending strength was estimated as $F_{b\text{-ult}}$ = 6840 psi. The beam was braced against lateral translation and rotation at the supports and was loaded through 3 evenly spaced hydraulic cylinders. The center cylinder was braced to maintain a vertical orientation; however, the beam was not braced. Therefore, an effective length, l_e = 1.84 l_u (l_u = full span), was assumed. Using the 2012 NDS behavioral equations, the resisting moment was estimated to be 222,360 ft-lbs compared to an induced moment of 55,860 ft-lbs.

More recently, Dayeh and Syme reported results for Brush box and Radiata pine glulam beams tested by the Forestry Commission of New South Wales (FCNSW) according to AS 1720 Part 1 [18][26]. The ratios of induced load to design load were 46% and 18% and failure times were 59 minutes and 67 minutes for the Brush box and Radiata pine beam, respectively. Dayeh and Syme estimated the average ultimate strength for the Brush box beam as F_b = 7250 psi and E = 2.2E6 psi. The beam was braced against lateral translation and rotation at the supports and was loaded at 2 evenly spaced load points. The beam was apparently braced at the load points; therefore, an effective length, l_e = 1.68 l_u (l_u = full span/3), was assumed. Using the 2012 NDS behavioral equations, the resisting moment was estimated to be 161,570 ft-lbs compared to an induced moment of 74,790 ft-lbs.

Dayeh and Syme estimated the average ultimate strength for the Radiata pine beam as F_b = 5200 psi and E = 1.8E6 psi. The beam was braced against lateral translation and rotation at the supports and was loaded at 2 evenly spaced load points. The beam was apparently braced at the load points; therefore, an effective length, l_e = 1.68 l_u (l_u = full span/3), was assumed. Using the 2012 NDS behavioral equations, the

Table 2.2a Beams Tested

Designation	Breadth (in)	Depth (in)	Specific Gravity	$F_{b\text{-}ult}$ (psi)	E x 10^6 (psi)	Resisting Moment (ft-lbs)	Induced Moment (ft-lbs)
TRADA	5.5	9	0.49	7530	2.0	45,530	9,830
NFoPA	8.75	16.5	0.47	6840	1.6	222,360	55,860
AF&PA-27	8.75	16.5	0.47	6840	1.6	222,760	18,940
AF&PA-44	8.75	16.5	0.47	6840	1.6	222,760	30,710
AF&PA-91	8.75	16.5	0.47	6840	1.6	222,760	65,080
FCNSW-BB	5.9	16.5	0.82	7250	2.2	161,570	74,790
FCNSW-RP	5.9	16.5	0.52	5200	1.8	116,060	20,500

Table 2.2b Measured and Calculated Beam Fire Resistance Times

Designation	Measured t_f (min)	Calculated t_f (min) Lie Method [1, 2]	Calculated t_f (min) Mechanics-Based Method [3]
TRADA	53	51	52
NFoPA	86	86	84
AF&PA-27	147	100	134
AF&PA-44	114	100	125
AF&PA-91	85	79	92
FCNSW-BB	59	71	41
FCNSW-RP	67	71	72

[1] Assumed a char rate of 1.42 in/hr.
[2] Used stated design load ratio from report.
[3] Assumed a char rate of 1.5 in/hr.

resisting moment was estimated to be 116,060 ft-lbs compared to an induced moment of 20,500 ft-lbs.

In 1997, the American Forest & Paper Association (AF&PA) (now the American Wood Council) conducted a series of four experimental beam tests at Southwest Research Institute (SwRI) [23]. The primary objectives of the tests were to evaluate the effect of load on the fire resistance of glulam beams, and to determine whether the load factor equation in Lie's calculation method is valid for load ratios lower than 50%. The same type of beam was used as for the test conducted by NFoPA, so that the results from that test would provide an additional data point for the load ratio curve. The first of the four tests was conducted without external load, but with an extensive number of thermocouples distributed across the section to determine char rates in different directions as a function of time. In the remaining three tests, the beams were loaded at 27, 44, and 91% of the design load. The reported allowable stresses were F_b = 2400 psi and E = 1.6E6 psi. Using the 2.85 allowable design stress to average ultimate strength adjustment factor derived in *Part 1* (see *Table 1.4.2*), the average ultimate bending strength was estimated as $F_{b\text{-ult}}$ = 6840 psi. Each beam was braced against lateral translation and rotation at the supports and was loaded at 2 evenly spaced load points. The beam was braced at the load points; therefore, an effective length, $l_e = 1.68\ l_u$ (l_u = full span/3), was assumed. Using the 2012 NDS behavioral equations, the resisting moment was estimated to be 222,760 ft-lbs compared to induced moments of 18,940 ft-lbs, 30,710 ft-lbs and 65,080 ft-lbs for the 27, 44, and 91% design load cases, respectively. The corresponding failure times were 147, 114, and 85 minutes, respectively.

The section dimensions, average densities, resisting moment and induced moment for the seven beam tests are

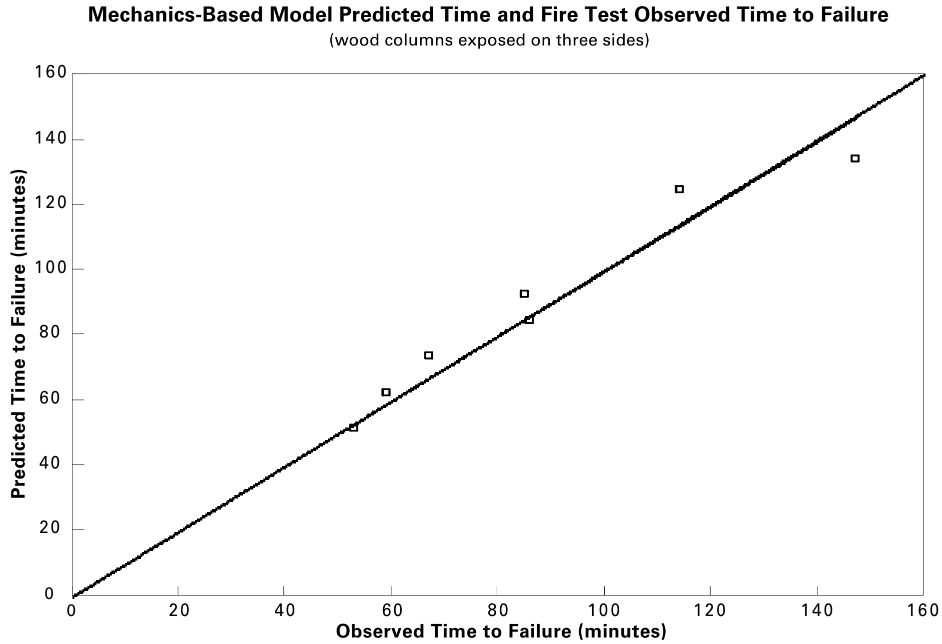

Figure 2-1 Comparison of predicted to observed time to failure (wood beams exposed on three sides)

summarized in *Table 2.2a*. The measured times to structural failure are compared to calculated results in *Table 2.2b* and in *Figure 2.1*.

2.3 Columns

Lie verified his method against experimental data for columns obtained in France [6], England [7], and Germany [8] in the 1960s and early 1970s. In this report, the same data sets are used to evaluate the mechanics-based calculation method.

Fackler reported results for 5 columns that were tested in the early 1960s at the laboratories of CSTB in France [6]. Two columns were glued-laminated, and the remaining three were bolted or nailed together. The two glulam columns were identical except for the type of adhesive. For one column, the laminates were glued together with a melamine adhesive. For the other column, a urea-formaldehyde adhesive was used. It was concluded that the type of adhesive did not have an effect on fire performance, because time to failure was identical for the two tests. Lie performed his calculations assuming the columns were tested under full design load, as mentioned in Fackler's report. Based on estimates of average ultimate bending strength for French Maritime Pine reported in the literature [19], the average ultimate compression strength was estimated as $F_{\text{c-ult}}$ = 2565 psi. The literature also reported E = 1.6E6 psi. Using the 2012 NDS behavioral equations and an effective length l_e = 90 inches, the resisting capacity was estimated to be 132,370 pounds compared to an induced load of 39,790 pounds. The section dimensions, specific gravities, mechanical properties, resisting capacities and induced loads for the 2 French column tests are summarized in *Table 2.3a*.

Stanke et al. reported results for numerous glulam columns that were tested in Germany in the 1970s [7]. Two types of adhesives were used; resorcinol (R designation), and urea

Table 2.3a Columns Tested in France

Designation	Depth (in)	Breadth (in)	Specific Gravity (lb/ft³)	$F_{\text{c-ult}}$ (psi)	$E \times 10^6$ (psi)	Resisting Capacity (lbs)	Induced Load (lbs)
CSTB44	7	7.875	0.56	2565	1.6	132,370	39,790
CSTB45	7	7.875	0.56	2565	1.6	132,370	39,790

based (H designation). As in the French tests, it was found that type of adhesive did not have a systematic effect on fire resistance. The load ratios were reported by Stanke et al. as 1.00, 0.75, and 0.50. Average ultimate compression strengths and *E* values were reported for some column tests. While the actual species tested were not identified, the average specific gravity for the laminations was recorded. Using the reported specific gravity and mechanical properties, average ultimate compression strengths and *E* values were estimated for the other columns tested. Using the 2012 NDS behavioral equations and an effective length l_e = 144 inches, the resisting capacities for each of the columns were estimated. The section dimensions, specific gravities, mechanical properties, resisting capacities and induced loads for each of the German column tests are summarized in *Table 2.3b*.

Malhotra and Rogowski reported results for 16 glulam column tests that were conducted at the Fire Research Station in the UK [8]. The tests were statistically designed to determine the effect of 4 variables. The variables were:

- species (first letter in designation): Douglas fir (F), Western hemlock (H), European redwood (R), and Western red cedar (C);
- adhesive (second letter in designation): urea (U), casein (C), resorcinol (R), and phenolic (P);
- shape: 9 x 9 inches, 12 x 6.9 inches, and 15 x 5.6 inches; and
- test load: 100% of design, 50% of design, and 25% of design.

Statistical analysis indicated that some columns with casein adhesive performed systematically below average. Since

Table 2.3b Columns Tested in Germany by Stanke et al.

Designation	Depth (in)	Breadth (in)	Specific Gravity	$F_{c\text{-}ult}$ (psi)	E x 10^6 (psi)	Resisting Capacity (lbs)	Induced Load (lbs)
R14A	5.5	5.5	0.44	7368	2.5	84,640	19,030
R14B	5.5	5.5	0.45	7929[a]	2.3[b]	80,310	19,030
R14C	5.5	5.5	0.45	8131[a]	2.4[b]	82,220	9,520
R14D	5.5	5.5	0.43	7447[a]	2.2[b]	75,740	14,260
H14A	5.5	5.5	0.44	8050	2.0	70,830	19,030
H14B	5.5	5.5	0.48	7652	2.4	82,600	19,030
H14C	5.5	5.5	0.45	8131[a]	2.4[b]	82,220	9,520
H14D	5.5	5.5	0.43	7447[a]	2.2[b]	75,740	14,260
H14/24A	5.5	9.5	0.41	6243[a]	1.7[b]	99,130	32,630
H14/24B	5.5	9.5	0.41	6169[a]	1.6[b]	98,030	32,630
H14/30A	5.5	11.75	0.45	6914[a]	1.7[b]	130,410	40,790
H14/30B	5.5	11.75	0.47	8690	2.7	198,240	20,390
H14/30C	5.5	11.75	0.46	7165[a]	1.8[b]	134,830	20,390
H14/40	5.5	15.75	0.45	6675[a]	1.6[b]	158,900	54,230
R15A	5.875	5.875	0.38	5995[a]	1.8[b]	78,940	24,030
R15B	5.875	5.875	0.38	5970[a]	1.8[b]	78,630	24,030
H15A	5.875	5.875	0.40	6515[a]	1.9[b]	85,340	24,030
H15B	5.875	5.875	0.37	5868[a]	1.7[b]	77,370	24,030
R16	5.875	5.875	0.31	4302[a]	1.3[b]	72,420	29,430
H16A	6.25	6.25	0.37	5723[a]	1.7[b]	94,690	29,430
H16B	6.25	6.25	0.40	6595[a]	1.9[b]	108,170	29,430

[a] Compression strength estimated from other specimens in test series.
[b] Modulus of elasticity estimated from other specimens in test series.

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Table 2.3b (cont'd) Columns Tested in Germany by Stanke et al.

Designation	Depth (in)	Breadth (in)	Specific Gravity	$F_{c\text{-}ult}$ (psi)	E x 10^6 (psi)	Resisting Capacity (lbs)	Induced Load (lbs)
R16/30	6.25	11.75	0.41	5944[a]	1.5[b]	163,780	27,560
H16/30A	6.25	11.75	0.42	6229[a]	1.6[b]	171,130	55,120
H16/30B	6.25	11.75	0.44	6666[a]	1.7[b]	182,350	55,120
H16/30C	6.25	11.75	0.43	6470[a]	1.6[b]	177,340	55,120
H16/30D	6.25	11.75	0.40	5710[a]	1.5[b]	157,740	27,560
R20A	7.875	7.875	0.40	5931	1.6	199,680	56,440
R20B	7.875	7.875	0.39	6657	1.7	219,630	56,440
R20C	7.875	7.875	0.46	9003	2.2	288,660	28,220
R20D	7.875	7.875	0.43	5685[a]	1.6[b]	198,700	28,220
H20A	7.875	7.875	0.38	5903	1.7	209,240	56,440
H20B	7.875	7.875	0.39	6031	1.8	218,960	56,440
H20C	7.875	7.875	0.45	8676	2.1	270,840	28,220
H20D	7.875	7.875	0.45	7370[a]	2.0[b]	254,220	28,220
H20/40A	7.875	15.75	0.44	6651[a]	1.6[b]	413,480	112,880
H20/40B	7.875	15.75	0.45	5415[a]	1.3[b]	340,470	112,880
H24A	9.5	9.5	0.40	5639[a]	1.5[b]	344,680	89,950
H24B	9.5	9.5	0.38	6616[a]	1.8[b]	401,960	89,950
H26A	10.25	10.25	0.42	6346[a]	1.7[b]	485,010	110,670
H26B	10.25	10.25	0.42	5579[a]	1.5[b]	428,170	110,670
R27A	10.625	10.625	0.38	5220	1.3	428,660	121,030
R27B	10.625	10.625	0.40	5504	1.6	483,440	121,030
R27C	10.625	10.625	0.41	6229[a]	1.6[b]	528,290	121,030
H27A	10.625	10.625	0.42	6216	1.9	555,830	121,030
H27B	10.625	10.625	0.40	5448	1.4	463,540	121,030
H27C	10.625	10.625	0.41	6181[a]	1.6[b]	524,300	121,030
H28A	11	11	0.40	5806[a]	1.5[b]	543,890	132,940
H28B	11	11	0.42	6260[a]	1.6[b]	585,190	132,940
H40	15.75	15.75	0.41	5659[a]	1.4[b]	1,257,230	308,650

[a] Compression strength estimated from other specimens in test series.

[b] Modulus of elasticity estimated from other specimens in test series.

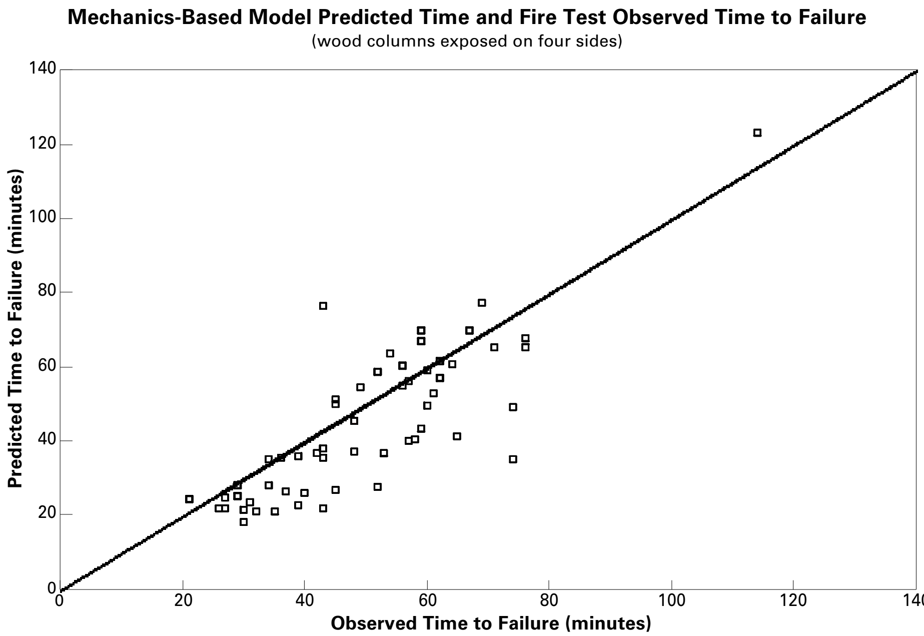

Figure 2-2 Comparison of predicted to observed time to failure (wood columns exposed on four sides)

Table 2.3c Columns Tested in England by Malhotra et al.

Designation	Depth (in)	Breadth (in)	Specific Gravity	$F_{c\text{-ult}}$ (psi)	$E \times 10^6$ (psi)	Resisting Capacity (lbs)	Induced Load (lbs)
FU1	9	9	0.59	5197	1.7	396,190	71,980
FR3	5.6	15	0.59	5197	1.7	327,260	35,990
FP4	9	9	0.59	5197	1.7	396,190	143,960
HU5	9	9	0.54	4454	1.5	339,410	31,030
HR7	6.9	12	0.54	4454	1.5	318,140	62,060
HP8	9	9	0.54	4454	1.5	339,410	62,060
RU9	5.6	15	0.54	3961	1.2	241,980	55,230
RR11	9	9	0.54	3961	1.2	299,830	110,450
RP12	6.9	12	0.54	3961	1.2	278,740	27,610
CU13	6.9	12	0.38	3218	1.0	227,700	89,510
CR15	9	9	0.38	3218	1.0	244,190	44,750
CP16	5.6	15	0.38	3218	1.0	198,650	44,750

Table 2.3d Measured and Calculated Column Fire Resistance Times

		Calculated t_f (min)	
Designation	**Measured t_f (min)**	**Lie Method [1, 2]**	**Mechanics-Based Method [3]**
CSTB44	48	38	45
CSTB45	48	38	45
R14A	29	28	25
R14B	21	28	24
R14C	36	36	36
R14D	29	31	28
H14A	26	28	22
H14B	27	28	25
H14C	43	36	36
H14D	34	31	28
H14/24A	35	34	21
H14/24B	32	34	21
H14/30A	39	35	23
H14/30B	59	46	43
H14/30C	53	46	36
H14/40	43	37	22
R15A	26	30	22
R15B	27	30	22
H15A	31	30	23
H15B	30	30	22
R16	30	32	18
H16A	31	32	23
H16B	37	32	26
R16/30	58	51	41
H16/30A	40	39	26
H16/30B	52	39	28
H16/30C	45	39	27
H16/30D	57	51	40
R20A	34	40	35
R20B	48	40	37
R20C	64	52	61
R20D	61	52	53

continued on next page

[1] Assumed a char rate of 1.42 in/hr.
[2] Used stated design load ratio from report.
[3] Assumed a char rate of 1.5 in/hr.

Table 2.3d (cont'd) Measured and Calculated Column Fire Resistance Times

		Calculated t_f (min)	
Designation	**Measured t_f (min)**	**Lie Method [1, 2]**	**Mechanics-Based Method [3]**
H20A	42	40	37
H20B	43	40	38
H20C	60	52	59
H20D	52	52	58
H20/40A	65	50	41
H20/40B	74	50	35
H24A	60	48	50
H24B	56	48	55
H26A	62	52	62
H26B	62	52	57
R27A	57	54	56
R27B	54	54	64
R27C	76	54	65
H27A	59	54	70
H27B	56	54	60
H27C	71	54	65
H28A	59	56	67
H28B	67	56	70
H40	114	96	123
FU1	55	60	77
FR3	74	48	49
FP4	45	55	51
HU5	73	60	96
HR7	49	55	54
HP8	69	69	77
RU9	47	55	35
RR11	45	55	50
RP12	76	69	68
CU13	35	51	35
CR15	43	69	76
CP16	39	48	36

[1] Assumed a char rate of 1.42 in/hr.
[2] Used stated design load ratio from report.
[3] Assumed a char rate of 1.5 in/hr.

these adhesives are not commonly used today for glulam, the test data for the casein (C) adhesives were discarded for the purpose of this report. The load ratios were reported by Malhotra and Rogowski as 1.00, 0.50, and 0.25. Allowable compression stresses and *E* values were also reported by Malhotra and Rogowski. Using the allowable/ultimate adjustments reported in the TRADA beam tests [16], average ultimate compression strengths and *E* values were estimated. Using the 2012 NDS behavioral equations and an effective length l_e = 82 inches (reported by Malhotra and Rogowski), the resisting capacities for each of the columns were estimated. The section dimensions, specific gravities, mechanical properties, resisting capacities and induced loads for each of the British column tests are summarized in *Table 2.3c*. The measured times to structural failure for the three separate series of column tests are compared to calculated results in *Table 2.3d* and *Figure 2-2*.

2.4 Tension Members

In 2000, the American Forest & Paper Association (now the American Wood Council) sponsored a series of four tension member tests at the U.S. Forest Products Laboratory (FPL) [27]. The primary objective of these tests was to validate this mechanics-based model against full-size tests of exposed wood members. The Douglas fir members were 117 inches long and loaded with a tension apparatus specially designed to induce intended tension loads. The center 72 inches of each member spanned through an intermediate-scale furnace and was subjected to an E119 exposure.

Using the 2012 NDS behavioral equations, the resisting capacities were estimated for each of the tension members. Due to a limitation in the furnace opening width, members were limited to less than 9 inches in width. In order to accommodate this limitation and to test members for up to two hours, load ratios in the range of 0.15-0.48 were used.

In the first two tests, it was determined that there was an unintended eccentricity caused by the bolted connection of the member to the test apparatus that resulted in a moment being induced in the member. This eccentricity resulted in a small moment in the first test of the 4x6 member, but induced a particularly large moment in the second test; therefore, the second test was not included in the analysis. A fourth test was conducted to repeat the configuration of the second test with the unintended eccentricity removed. Correcting the unintended eccentricity resulted in good agreement between the observed and predicted failure times.

The section dimensions, mechanical properties, resisting capacities and induced loads for the first, third and fourth tension members are provided in *Table 2.4a*. The measured times to structural failure are compared to calculated results in *Table 2.4b* and in *Figure 2-3*.

Table 2.4a Tension Members Tested

Designation	Breadth (in)	Depth (in)	$F_{t\text{-ult}}$ (psi)	Resisting Capacity (lbs)	Induced Load (lbs)
Test 1 – Lumber 4x6	3.375	5.313	2130	38,220	3,010
Test 3 – Glulam 5-1/8 x 9	5.063	8.813	4560	203,440	34,390
Test 4 – Glulam 8-3/4 x 9	8.75	8.563	4560	341,640	19,580 [1]

[1] For this test, a constant load of 6,000 lbs was applied for the first 120 minutes of the test. After 120 minutes, the load was gradually increased until failure occurred.

Table 2.4b Measured and Calculated Tension Member Fire Resistance Times

Designation	Measured t_f (min)	Calculated t_f (min)
Test 1 – Lumber 4x6	42	44
Test 3 – Glulam 5-1/8 x 9	58	60
Test 4 – Glulam 8-3/4 x 9	124	126

[1] Assumed a char rate of 1.5 in/hr.

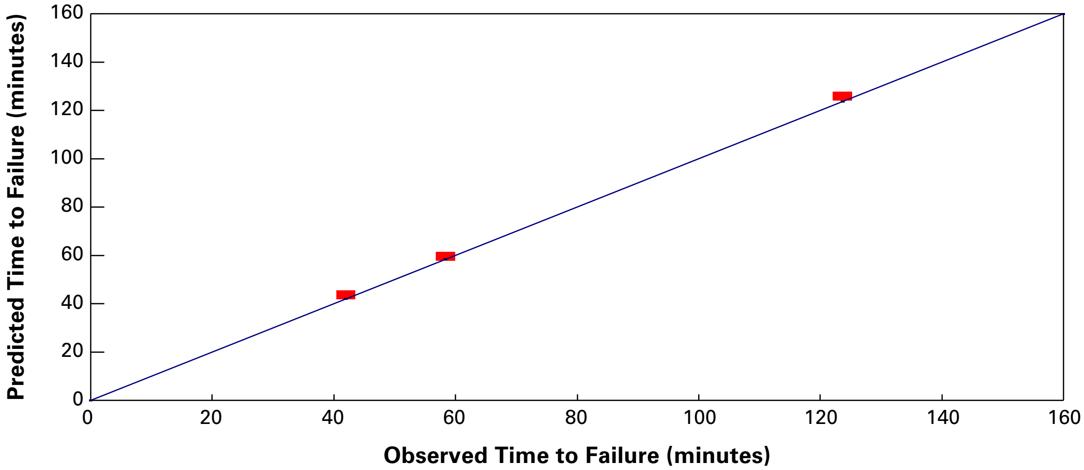

Figure 2-3 Comparison of predicted to observed time to failure (wood tension members exposed on four sides)

2.5 Decking

In 1964, Underwriters Laboratories (UL) conducted a series of four tests on roof constructions for the Douglas Fir Plywood Association (now APA–The Engineered Wood Association) [21]. Two of the tests, referred to as UL#2 and UL#4, were conducted on exposed timber decks consisting of 5.5- by 1.5-inch single tongue-and-groove Douglas fir planks. The decks were loaded to 46% and 59% of the design load for tests UL#2 and UL#4, respectively. The reported thermal penetration time (either excessive temperature rise or flame-through) was identical for the two tests at 20 minutes. First structural failure of a plank is not specifically mentioned in the report. However, for test UL#2, it is mentioned that deflection was noticeable (1.25 inches at the center of the deck) 13 minutes after the start of the test, and that the unsupported ends of some planks started to warp at 24 minutes. For test UL#4, noticeable deflection was observed at 11 minutes and warping was observed at 18 minutes.

In 1969, the American Iron and Steel Institute (AISI) conducted a comprehensive experimental program at Ohio State University (OSU) [22]. The program included six tests on exposed timber floor decks. The first two decks, referred to as HT1 and HT2, consisted of 1.625- by 3.625-inch members on edge and covered with ¾-inch wood flooring. Flame-through for the two tests was reported at 61 and 69 minutes, respectively. The first two decks were loaded at 21% of design load, and structural failure of the decking (not total structural failure) was reported at 62 minutes and 56 minutes for HT1 and HT2, respectively. Heavy charring occurred on the bottom of the decking, while lighter charring occurred on the sides. To use the mechanics-based model, charring on the sides due to the partial exposure at the butt-joints was addressed by assuming a charring rate of 30% of the effective charring rate for wood which is fully exposed.

The remaining four decks, referred to as HT3 through HT6, consisted of 5.625- by2.625-inch tongue-and-groove planks, covered with ¾-inch wood flooring. Flame-through for the four tests was reported at 54, 31, 35, and 49 minutes, respectively. The HT3 and HT4 decks were loaded at 42% of design load, and structural failure was reported at 54 minutes for HT3 (and not reported for HT4). The HT5 and HT6 decks were loaded at 50% of design load, and structural failure was reported at 45 minutes for HT6 (and not reported for HT5). Note that the fuel supply to the burners, instead of the temperature-time curve in the furnace, was controlled during the even-numbered tests. This resulted in slightly more severe exposure conditions than in the odd-numbered tests, which were conducted strictly according to ASTM E 119.

Using the 2.85 allowable design stress to average ultimate strength adjustment factor derived in *Part 1* (see *Table 1.4.2*), the ratio of induced moment to average ultimate bending moment can be estimated for each deck configuration. The section dimensions, induced moment to resisting moment ratio, measured structural failure time and calculated failure time are summarized in *Table 2.5* and *Figure 2-4*.

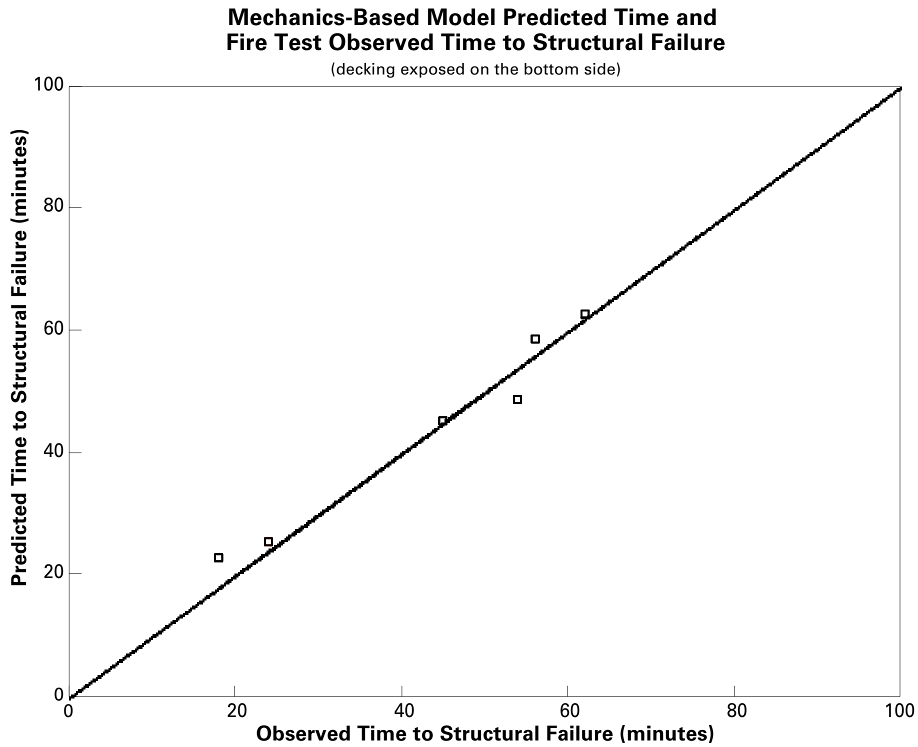

Figure 2-4 Comparison of predicted to observed time to failure (decking exposed on the bottom side)

Table 2.5 Measured and Calculated Decking Structural Fire Resistance Times

Designation	Species	Breadth (in)	Depth (in)	$\frac{M_{induced}}{M_{ult}}$	Measured (Structural) t_f (min)	Calculated (Structural) t_f [1] (min)
UL#2	Douglas fir	5.5	1.5	0.16	24+	25
UL#4	Douglas fir	5.5	1.5	0.21	18+	23
HT1	Subalpine fir	1.625	3.625	0.07	62	58
HT2	Subalpine fir	1.625	3.625	0.07	56	58
HT3	Southern pine	5.625	2.625	0.15	54	49
HT4	Southern pine	5.625	2.625	0.15	NR	49
HT5	Southern pine	5.625	2.625	0.18	NR	45
HT6	Southern pine	5.625	2.625	0.18	45	45

NR = Not Reported
[1] Assumed a char rate of 1.5 in/hr.

2.6 Unprotected Floor Joists

Several fire resistance tests of exposed wood joist floor assemblies have been conducted over the last 40 years. In review of these tests, the procedure derived in *Part 1* is applicable to light-frame wood members. A summary of each of the 21 tests reviewed and the assumptions used to calculate the fire resistance times for these assemblies are provided.

2.6.1 National Bureau of Standards Tests

In 1971, B. C. Son with the National Bureau of Standards (NBS) conducted a series of full-scale and small-scale fire resistance tests on unprotected floor assemblies [28]. All tests followed the E119 time-temperature curve. One full-scale floor assembly utilized nominal 2x10 sawn lumber joists spaced at 16 inches on center. A second full-scale floor assembly utilized nominal 2x8 sawn lumber joists spaced at 16 inches on center. Design values and dimensions for 2x8 and 2x10 joists were taken from FHA No. 300: *Minimum Property Standards for One and Two Living Units*. However, upon reading the report, it appears the writers did not know what was actually tested since they wrote, "To avoid overloading the joists, the lumber was assumed to be Rocky Mountain Region Douglas Fir. This has an allowable stress level of 1050 psi in bending according to Table III page 250 of the FHA Minimum Property Standards (4)."

In 1971, lumber sizes and grades had been standardized under U.S. Department of Commerce's Voluntary Product Standard 20: *American Softwood Lumber Standard* (PS 20-70) [29]. Sizes for dry 2x8 lumber were 1.5 inches x 7.25 inches and 2x10 lumber were 1.5 inches x 9.25 inches. Lumber grades had also been standardized and Construction grade was limited to 2x4 lumber. The lumber sizes and grades in the report appear to be for lumber available in the early to mid-1960s, not likely tested in 1971.

As a result of the confusion about the actual lumber sizes and grades tested, dimensions for both the 2x8 and 2x10 joists were based on PS 20-70. In addition, since the design values for Construction grade Douglas Fir lumber were not recognized in 1971, design values for a common grade of lumber used for floor joists, #2 Douglas Fir-Larch with a repetitive-member bending stress of 1450 psi, was used. Using the K factor of 2.85, to adjust the allowable bending design stress to a lower bound estimate of the average ultimate bending strength, results in a bending strength, $F_{b\text{-}ult}$, of 4133 psi.

The 2x10 floor was sheathed with two layers of ½-inch plywood (Test #2). Half of the 2x10 floor assembly was also covered with carpet (Test #4). The 2x8 floor was sheathed differently on each half. One half of the 2x8 floor assembly was sheathed with a single layer of ⅝-inch tongue-and-grove plywood (Test #9). The other half of the floor assembly was sheathed with a single layer of ½-inch plywood with all edges blocked using 2x3 lumber (Test #10).

The dead load of the 2x10 floor assembly was estimated to be 6.8 psf. A superimposed load of 63.7 psf was added, resulting in a total load of 70.5 psf. Using the 2012 NDS behavioral equations and standard dry dimensions of 1.5 inches x 9.25 inches, the allowable resisting moment was estimated to be 31,020 in-lb and the ultimate resisting moment was estimated to be 88,400 in-lb. Given a span of 163 inches, the induced moment was 25,910 in-lb (84% of full design load). Failure was recorded at 11:38 minutes.

The dead load of the 2x8 floor assembly was estimated to be 6.2 psf. A superimposed load of 21 psf was added, resulting in a total load of 27.2 psf. Using the 2012 NDS behavioral equations and standard dry dimensions of 1.5 inches x 7.25 inches, the allowable resisting moment was estimated to be 19,050 in-lb and the ultimate resisting moment was estimated to be 54,300 in-lb. Given a span of 163 inches, the induced moment was 9,940 in-lb (52% of full design load). Failure was recorded at 13:00 minutes.

2.6.2 Factory Mutual Tests

In 1974, Factory Mutual witnessed a series of full-scale fire resistance tests conducted at the NGC Research Center. The tests were conducted on unprotected floor assemblies constructed with lumber joists, and all tests followed the E119 time-temperature curve.

Two of the Factory Mutual full-scale floor assemblies consisted of #2 MG (medium-grain) grade 2x10 Southern Pine joists spaced at 24 inches on center sheathed with a single layer of 23/32-inch plywood [30][31]. The joists had an allowable bending stress for repetitive member assemblies of 1450 psi. Using K = 2.85, the ultimate bending strength was estimated as $F_{b\text{-}ult}$ = 4133 psi. The actual dimensions were reported as 1.5 inches x 9.125 inches. Test FC 209 was topped with vinyl tile flooring. Test FC 212 was topped with nylon carpet.

Dead loads of the 2x10 floor assemblies were estimated to be 4.7 psf and 4.8 psf for FC 209 and FC 212, respectively. Superimposed loads of 57.4 psf and 57.3 psf, respectively, were added to each assembly, resulting in a total load of 72.1 psf for both assemblies. Using the 2012 NDS behavioral equations and measured dry dimensions of 1.5 inches x 9.125 inches, the allowable resisting moment of joists in both assemblies was estimated to be 30,180 in-lb and the ultimate resisting moment was estimated to be 86,020 in-lb. Given a span of 157 inches, the moment induced in joists of both assemblies was 31,890 in-lb (106% of full design load). Failure was recorded at 13:34 and 12:06 minutes for FC 209 and FC 212, respectively.

Two additional Factory Mutual full-scale floor assemblies consisted of #2 grade 2x8 Douglas Fir sawn lumber joists spaced at 16 inches on center sheathed with a single layer of $^{19}/_{32}$-inch plywood [32][33]. The joists had an allowable bending stress for repetitive member assemblies of 1450 psi. The average bending strength was estimated as $F_{b\text{-}ult}$ = 4133 psi. The actual dimensions were reported as 1.5 inches x 7.25 inches. Test FC 213 was topped with vinyl tile flooring. Test FC 216 was topped with nylon carpet.

Dead loads of the 2x8 floor assemblies were estimated to be 4.4 psf and 5 psf for FC 213 and FC 216, respectively. Superimposed loads of 53.3 psf and 52.7 psf, respectively, were added to each assembly, resulting in a total load of 57.7 psf for both assemblies. Using the 2012 NDS behavioral equations and measured dry dimensions of 1.5 inches x 7.25 inches, the allowable resisting moment of joists in both assemblies was estimated to be 19,050 in-lb and the ultimate resisting moment was estimated to be 54,300 in-lb. Given a span of 157 inches, the moment induced in joists of both assemblies was 19,640 in-lb (102% of full design load). Failure was recorded at 10.2 and 12.9 minutes for FC 213 and FC 216, respectively.

2.6.3 NBS/HUD Tests

In 1982, NBS conducted a series of full-scale tests on selected residential floor assemblies for the U. S. Department of Housing and Urban Development (HUD) [34]. Two of the tests were conducted on unprotected floor assemblies constructed with lumber joists and followed the E119 time-temperature curve. The assemblies utilized nominal 2x8 sawn lumber joists spaced at 24 inches on center. The 2x8 joists were #2 grade Southern Pine lumber with an allowable bending stress for repetitive member assemblies of 1400 psi. Using the K factor of 2.85 to adjust the allowable bending design stress to a lower bound estimate of the average ultimate bending strength, results in a bending strength, $F_{b\text{-}ult}$, of 3990 psi. The floors were sheathed with a single layer of $^{23}/_{32}$-inch plywood (Test #6 & #7).

The dead load of the floor assembly was estimated to be 4.6 psf. A superimposed load of 54 psf was added, resulting in a total load of 58.6 psf. Using the 2012 NDS behavioral equations and standard dry dimensions of 1.5 inches x 7.25 inches, the allowable resisting moment was estimated to be 18,400 in-lb and the ultimate resisting moment was estimated to be 52,430 in-lb. Given a span of 110 inches, the induced moment was 14,770 in-lb (80% of full design load). Failure was recorded at 14:42 minutes in Test #6 and 13:10 minutes in Test #7.

2.6.4 USDA Forest Products Laboratory Tests

In 1983, the U.S. Forest Products Laboratory (FPL) funded a series of full-scale fire resistance tests on unprotected floor assemblies at Construction Technology Laboratories [35]. The floor assemblies were constructed with 2x10 sawn lumber joists spaced at 16 inches on center. The tests all followed the E119 time-temperature curve.

Materials for the floor assembly tests were obtained from a local lumber yard. Lumber joists were nominal 2x10 Douglas Fir joists, but the grade of the material was not reported. Results from limited destructive bending tests (20 pieces) and non-destructive testing (161 pieces) of the materials were reported. The average bending strength of the 20 piece sample was 5,280 psi with a COV=0.47 and the average edgewise Modulus of Elasticity (E) was 1.5E6 psi with a COV=0.25. While this limited sample cannot be used to determine bending design values for this sample, it does indicate that the material was at or below current #2 grade Douglas Fir-Larch bending design values. In order to use the procedures developed in *Part 1*, calculations were conducted assuming current design values for #2 grade Douglas Fir-Larch with an allowable bending stress for repetitive member assemblies of 1140 psi. Using K = 2.85, the average ultimate bending strength was estimated as $F_{b\text{-}ult}$ = 3246 psi. The actual dimensions were reported as 1.47 inches x 9.11 inches. The 2x10 floors were sheathed with a single layer of 23/32-inch plywood.

It should be noted that the $F_{b\text{-}ult}$ value calculated using the provisions of *Part 1* is lower than the average bending strength of the 20 piece sample. This difference is expected since the 2.85 factor adjusts the allowable design stress to a lower bound estimate of the average ultimate strength based on the assumed COV of clear wood (16%) rather than the COV of 47% measured in the 20 piece full-size, as-graded lumber sample. This added conservatism in the design procedure ensures that the calculated fire resistance time is a reasonable lower bound even for wood materials with highly variable properties.

Using the 2012 NDS behavioral equations and measured dry dimensions of 1.47 inches x 9.11 inches, the allowable resisting moment of the joists was estimated to be 23,150 in-lb and the average ultimate resisting moment was estimated to be 65,980 in-lb.

Dead loads of the floor assemblies were estimated to be 4.5 psf for all tests. Superimposed loads of 11.4 psf on six low load tests and 79.2 psf on five high load tests were added to the assemblies, resulting in total loads of 15.9 psf and 83.7 psf, respectively. Given the reported span of 156 inches, the induced moment that was intended to be applied to the floor joists was 5,330 in-lb (23% of full design load) for the lightly-loaded floor assemblies and 28,400 in-lb (123%) for the heavily-loaded floor assemblies. However, the loading was much higher.

The loading apparatus used at Construction Technologies Laboratories consisted of 16 interconnected hydraulic rams. At the ends of the hydraulic rams, a three-legged tripod structure was used to apply the load to the floor assembly. This tripod system had been used on tests of two-way concrete slabs, but had not been tested on repetitive member "ribbed" assemblies, such as a wood joist floor assembly. For decades, these tests have been reviewed and the validity of the results questioned because of the difficulties associated with estimating the loading on the assemblies over time [36, 37]. Different attempts to model the load distribution have met with unanswered questions about stiffness of the sheathing, charring of the sheathing, and the ability of the sheathing to distribute loads. Clearly the loads were non-uniform since "rippling," characterized as deflections of the sheathing along the line of the application of the loading was observed in the high-load tests between 3 and 6 minutes. Analysis, assuming that the sheathing was initially stiff enough to distribute the loads by bending to the joists, indicates that the initial load on the joist directly under the ram would have been approximately 200% of the assumed load and that joists on either side would have received approximately 50% of the assumed load. When this same analysis is conducted assuming sheathing had charred to the point that it could not distribute the load by bending, the load on the joist under the ram drops to 150% of the assumed load, and the joists on either side would increase to approximately 75% of the assumed load. Assuming that the latter case represents a lower bound estimate of the load on joists under the rams, the induced moments in these joists was assumed to be 150% of the induced moments assuming a uniform load. As a result, the apparent induced moment on the critical joists under the rams was estimated to be 8,000 in-lb (35% of full design load) in the lightly-loaded tests and 42,590 in-lb (184% of full design load) in the highly-loaded tests. First-joist failures ranged from 16.7 to 18.5 minutes in the lightly-loaded tests and 5.5 to 7.5 minutes in the highly-loaded tests. Floor assembly failure times were typically more than first-joist failure times, but due to non-uniform loading conditions on the joists, those failure times were not included in this analysis.

2.6.5 Underwriters Laboratory Tests

In 2008, Underwriters Laboratory conducted a series of full-scale fire resistance tests of unprotected floor assemblies [38]. For this series of tests, the loads placed on the floor assemblies were intended to represent typical loading conditions during a fire. A load of 40 psf was placed along two of the four perimeter sides of the floor assembly and two 300 pound concentrated loads were placed near the center of the floor assembly to represent two fire service personnel on the floor. One of the tests was a full-scale sawn lumber floor assembly with a non-uniform loading pattern on portions of the floor. The test followed the E119 time-temperature curve.

The floor assembly consisted of #2 grade 2x10 Spruce-Pine-Fir sawn lumber joists spaced at 16 inches on center and sheathed with a single layer of 1x6 subflooring and topped with 1x4 wood flooring. The joists had an allowable bending stress for repetitive member assemblies of 1110 psi. Using K = 2.85, the ultimate bending strength was estimated as $F_{b\text{-}ult}$ = 3160 psi. The actual dimensions were reported as 1.5 inches x 9.125 inches. Dead load of the 2x10 floor assembly was estimated to be 6 psf. As mentioned previously, a target uniform load of 40 psf was applied at the perimeter edge of two intersecting sides of the floor; however, due to proximity of the load to the joist bearing reaction on one edge of the floor and the use of three joists at the end of the floor assembly, the effective loads on the interior joists were likely much less than reported. In fact, deflection measurements taken during the test suggest that joists at the end of the assembly were only loaded to about the same level as joists in the middle of the floor assembly; therefore, all calculations were done assuming that the maximum load ratios were those reported at the middle joists. Using the 2012 NDS behavioral equations and measured dry dimensions of 1.5 inches x 9.125 inches, the allowable resisting moment was estimated to be 23,040 in-lb and the ultimate resisting moment was estimated to be 65,670 in-lb. Given a span of 155 inches, the induced moment was estimated to be 7,760 in-lb (34% of full design load). Failure was recorded at 18.75 minutes.

In 2011, Underwriters Laboratory conducted another series of full-scale fire resistance tests of unprotected floor assemblies [39]. For this series of tests, a uniform load was placed on the floor assembly. Two of the tested floor assemblies utilized sawn lumber. Floor assembly #6 utilized #2 grade Douglas Fir-Larch 2x10 joists and appeared to be loaded at approximately 91% of design load. In Test #6, the furnace temperature was initially allowed to run at temperatures nearly 50% higher than the E119 time-temperature curve and resulted in failure at about 7 minutes. Since the furnace temperature was not controlled at standard E119 conditions assumed in this model, this test was not included in the comparison of test results to the analysis procedure.

Floor assembly #7 utilized 2x8 Douglas Fir joists taken from deconstruction of a circa 1940 home in Ohio. The grade of the material was not known, but based on the time period and using the mid-quality grade of Douglas Fir joists reported in the 1944 NDS [40], Structural grade Douglas Fir 2x10 joists were assumed with an allowable bending design value of 1900 psi. Using K = 2.85, the ultimate bending strength of the joists was estimated as $F_{b\text{-}ult}$ = 5415 psi. The actual dimensions were reported as 1.75 inches x 7.56 inches. The floor was sheathed with a single layer of 23/32-inch OSB sheathing.

Dead load of the 2x8 floor assembly was estimated to be 4.5 psf. A superimposed load of 42.3 psf was added, resulting in a total load of 46.8 psf. Using the 2012 NDS behavioral equations and reported dimensions of 1.75 inches x 7.56 inches, the allowable resisting moment of the joists was estimated to be 31,670 in-lb and the ultimate resisting moment was estimated to be 90,270 in-lb. Given a span of 155 inches, the induced moment was 15,690 in-lb (50% of full design load). Failure was recorded at 18.1 minutes.

2.6.6 Results of Analysis

The fire resistance of joists from each of the unprotected floor joist assemblies is provided in *Table 2.6* and *Figure 2-5*. On average, the calculated fire resistance times underpredicted the actual observed fire resistance times by approximately 1 minute, ranging from the maximum underprediction of 2.8 minutes to the maximum overprediction of 2.2 minutes (see *Table 2.6*).

These results are consistent with expected results, since the loading issues and resistance estimates were expected to increase the variability of the analysis. In addition, the model is expected to underpredict the fire resistance times since the model underestimates the average ultimate strength for wood members that have higher property variability, like sawn lumber joists.

2.7 Structural Composite Lumber

Over the last decade, a number of public and proprietary tests have been conducted to demonstrate that the procedures in Chapter 1 of this report can be used to design Structural Composite Lumber (SCL) manufactured in accordance with requirements of ASTM D5456 [41] and designed per the NDS. This section contains a summary and analysis of available test results analyzed for this report.

2.7.1 FPL Tension Tests

In 2006, White reported on fire resistance testing of SCL at the U.S. Forest Products Laboratory (FPL) [42]. Fourteen SCL products were exposed to a standard E119 time-temperature curve in a small vertical furnace to determine the

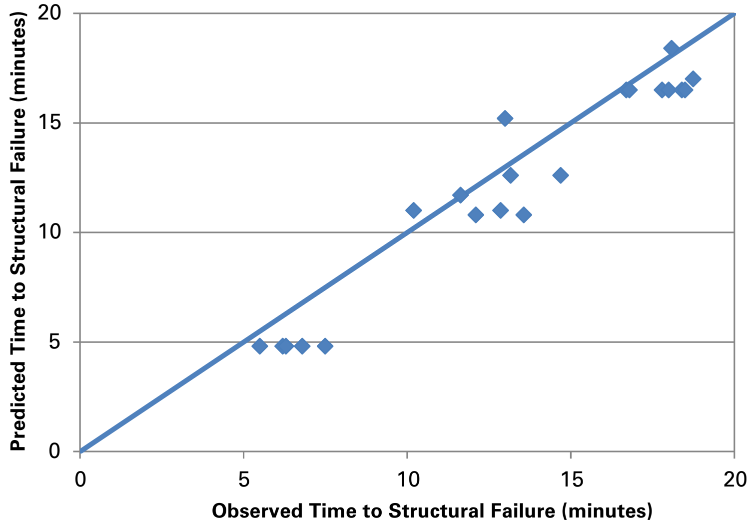

Figure 2-5 Comparison of predicted to observed time to failure (floor joists exposed on three sides)

Table 2.6 Measured and Calculated Floor Joist Structural Fire Resistance Times

Designation	Species	Breadth (in)	Depth (in)	$\frac{M_{induced}}{M_{design}}$	Measured (Structural) t_f (min)	Calculated (Structural) t_f [1] (min)
NBS#2 & #4	Douglas fir	1.5	9.25	0.84	11.6	12.5
NBS#9 & #10	Douglas fir	1.5	7.25	0.52	13.0	15.2
FC 209	Southern pine	1.5	9.13	1.06	13.6	10.8
FC 212	Southern pine	1.5	9.13	1.06	12.1	10.8
FC 213	Douglas fir	1.5	7.25	1.03	10.2	11.0
FC 216	Douglas fir	1.5	7.25	1.02	12.9	11.0
NBSIR #6	Southern pine	1.5	7.25	0.80	14.7	12.6
NBSIR #7	Southern pine	1.5	7.25	0.80	13.2	12.6
FPL Trial	Douglas fir	1.5	9.11	0.35	16.7	16.5
FPL #1	Douglas fir	1.47	9.11	0.35	17.8	16.5
FPL #2	Douglas fir	1.47	9.11	0.35	16.8	16.5
FPL #3	Douglas fir	1.47	9.11	0.35	18.0	16.5
FPL #4	Douglas fir	1.47	9.11	0.35	18.4	16.5
FPL #5	Douglas fir	1.47	9.11	0.35	18.5	16.5
FPL #6	Douglas fir	1.47	9.11	1.84	6.2	4.8
FPL #7	Douglas fir	1.47	9.11	1.84	6.8	4.8
FPL #8	Douglas fir	1.47	9.11	1.84	7.5	4.8
FPL #9	Douglas fir	1.47	9.11	1.84	5.5	4.8
FPL #10	Douglas fir	1.47	9.11	1.84	6.3	4.8
UL NC9140#1	Spruce-Pine-Fir	1.5	9.13	0.34	18.8	17.0
UL 2011#7	Douglas fir	1.75	7.56	0.50	18.1	18.4

[1] Assumed a char rate of 1.5 in/hr.

one-dimensional char rate for each product. Char rates were determined to be in the range expected for other wood products. Ten of these SCL products were then tension tested in an intermediate-scale furnace while being exposed to the E119 time-temperature curve. These fire resistance tests utilized the same configuration as the sawn lumber and glued-laminated timber tension tests reported in *Section 2.4* of this report.

In reviewing the intermediate-scale tension test data, it was noted that some of the larger laminated-veneer lumber (LVL) cross-sections appeared to fail early. In consultation with the SCL manufacturers and FPL staff, it was determined that some of the thicker LVL cross-sections were made from thinner LVL that were field-glued by an unknown third party. The type of adhesives and the quality of the bond was also unknown. In at least some cases, it was suspected that the secondary bond lines failed prematurely, causing char to fall off and increasing the effective char rate; therefore, for purposes of this analysis, all of the test results for specimens with secondary bond lines were excluded. In addition, one test of an experimental product in the study was also excluded. All other specimens were analyzed as solid cross-sections using the provisions of this report. The measured times to structural failure are compared to calculated results in *Table 2.7a* and in *Figure 2-6*.

2.7.2 AWC Beam Tests

In 2014, AWC funded a series of SCL bending tests at the Western Fire Center (WFC) [43]. Six fire resistance tests of SCL beams were conducted in accordance with ASTM E119. The beams were exposed on three surfaces, the bottom and sides of the beams, and loaded in flexure to various percentages of their design load. The test plan was developed to encompass a range of SCL types, beam sizes and load ratios. Several of these products had larger cross-sections that required gluing of thinner laminations to form larger cross-sections. This gluing was done by the manufacturers using adhesives that meet the elevated temperature performance requirements for glulam and SCL, and were bonded under in-plant controlled conditions.

All beams were loaded to a predetermined load using two hydraulic cylinders. Lateral bracing was provided at the ends and at the load points. After the full-scale tests, WFC found that the actual load was slightly greater than the target load for each beam. The ASD stress ratios and calculated structural fire resistance times, based on the actual loads reported by WFC, are reported in *Table 2.7b* and compared against measured times in *Figure 2-6*.

2.7.3 Proprietary Beam Tests

In 1994, the Technical University Braunschweig (TUB) conducted two fire resistance tests of loaded parallel-strand lumber (PSL) beams [44]. The beam tests were conducted in accordance with DIN 4102-2 [45], a European fire test standard similar to ASTM E119. In each test, two PSL beams spanned 187 inches across the horizontal furnace and were spaced 47 inches apart. The PSL beams were covered with foam concrete slabs which were positioned and able to deform freely with the beams. Lateral bracing was provided by friction of the slabs. The measured times to structural failure are compared to calculated results in *Table 2.7c* and in *Figure 2-6*.

Table 2.7a Measured and Calculated SCL Tension Member Structural Fire Resistance Times

Test No.	Designation	Species	SCL Width (in)	SCL Depth (in)	ASD Stress Ratio	Measured (Structural) t_f (min)	Calculated (Structural) t_f[1] (min)
2	LVL #3	Aspen	1.65	9.53	0.64	13	16
5	LVL #5	Douglas Fir	1.69	9.45	0.08	21	23
6	LVL #5	Douglas Fir	3.54	9.45	0.33	46	47
9	LVL #7	Eucalyptus	1.61	9.41	0.28	18	19
10	LVL #8	Southern Pine	1.77	9.41	0.26	18	22
12	LVL #11	Yellow Poplar	1.73	9.06	0.46	14	19
13	PSL #12	Douglas Fir	6.93	9.88	0.26	101	101

[1] Assumed a char rate of 1.5 in/hr.

In 1997, the Southwest Research Institute (SWRI) conducted two fire resistance tests of loaded parallel-strand lumber (PSL) beams [46]. The beam tests were conducted in accordance with ASTM E119. The beams were exposed on three surfaces, the bottom and sides of the beams, and loaded to full design load using three hydraulic cylinders. Lateral bracing was provided at the ends and at the load points. While the fire resistance model in this report significantly underpredicted the actual structural fire resistance times, analysis of deflection data recorded during the tests indicate an issue with the loading that resulted in less than full design load at the end of the tests. The initial ASD stress and calculated structural fire resistance times are reported in *Table 2.7c* and compared against measured times in *Figure 2-6*.

2.7.4 Proprietary Column Tests

In 1994, TUB conducted two fire resistance tests of loaded parallel-strand lumber (PSL) columns [47]. The column tests were conducted in accordance with DIN 4102-2. The first column was 7.87 inches x 7.87 inches and had an unbraced length of 148 inches. The second column was 7.09 inches x 7.09 inches and had an unbraced length of 118 inches. The columns were loaded concentrically through steel plates at each end. For design purposes, the columns were assumed to be pinned at each end; however, analysis of the results suggest that the bearing moment created by concentrically-loaded wood columns with square-cut bearing areas bearing on rigid steel or concrete plates, such as found in a fire test laboratory, would result in an effective length, L_e, of approximately $0.7L_u$. For analysis purposes in this report, the effective length was estimated to be $L_e = 0.7L_u$. This shorter effective length was used to estimate ASD stress ratios, and calculate structural fire resistance times reported in *Table 2.7d* and compared against measured times in *Figure 2-6*.

In 1997, the National Research Council of Canada (NRC) conducted two fire resistance tests of loaded parallel-strand lumber (PSL) columns [48]. The column tests were conducted in accordance with CAN/ULC S101 [49]. The first

Table 2.7b Measured and Calculated SCL Beam Structural Fire Resistance Times

Test No.	Designation	SCL Width (in)	SCL Depth (in)	ASD Stress Ratio	Measured (Structural) t_f (min)	Calculated (Structural) t_f[1] (min)
1	LSL	3.5	9.5	0.56	33	30
3	PSL	5.25	9.5	1.13	50	49
5	PSL	3.5	9.5	0.84	35	26
6	PSL	7.0	9.5	1.12	26	23
7	LVL	3.5	9.5	0.56	66	58
9	LVL	7.0	9.5	0.28	119	99

[1] Assumed a char rate of 1.5 in/hr.

Table 2.7c Measured and Calculated SCL Beam Structural Fire Resistance Times

Designation	SCL Width (in)	SCL Depth (in)	ASD Stress Ratio	Measured (Structural) t_f (min)	Calculated (Structural) t_f[1] (min)
TUB PSL 1	3.94	3.94	1.0	24	24
TUB PSL 2	4.53	19.21	1.0	44	42
SWRi PSL 1	7.87	16.0	1.0	99	73
SWRi PSL 2	8.86	11.8	1.0	112	73

[1] Assumed a char rate of 1.5 in/hr.

column was 9.84 inches x 9.84 inches. The second column was 10.50 inches x 10.50 inches. Both columns had unbraced lengths of 150 inches and the exposed length of the column was 120 inches. The columns were loaded concentrically through steel plates at each end. For design purposes, the columns were initially assumed to be fixed at each end by lightweight steel braces; however, the braces were not stiff enough to prevent the column from rotating at the ends. A separate analysis suggested that the effective length factor for design of these columns would be approximately 0.9 which would result in an effective length of $L_e = 0.9L_u$. The first column test had furnace temperatures well above the ASTM E119 curve throughout the entire duration of the test, so the results were not reported. The second column test was run with loads calculated for an effective length, L_e, of $0.9L_u$ which resulted in underprediction of the fire resistance time. Based on a review of these and other column tests, it appears that the effective length adjustment previously used with the TUB tests, $L_e = 0.7L_u$, results in the best estimate of fire resistance of the second column test, and was used to estimate the ASD stress ratio for the second column test and calculated structural fire resistance time reported in *Table 2.7d* and compared to measured times in *Figure 2-6*.

2.8 Cross-Laminated Timber

In 2011, FPInnovations (FPI), in collaboration with the National Research Council of Canada, conducted a series of 8 full-scale fire resistance tests of CLT floors and walls [50]. All tests followed the ULC S101 time-temperature curve, a fire exposure comparable to the ASTM E119 time-temperature curve. Two of the CLT floors and two of the CLT walls were exposed directly to the flames (unprotected).

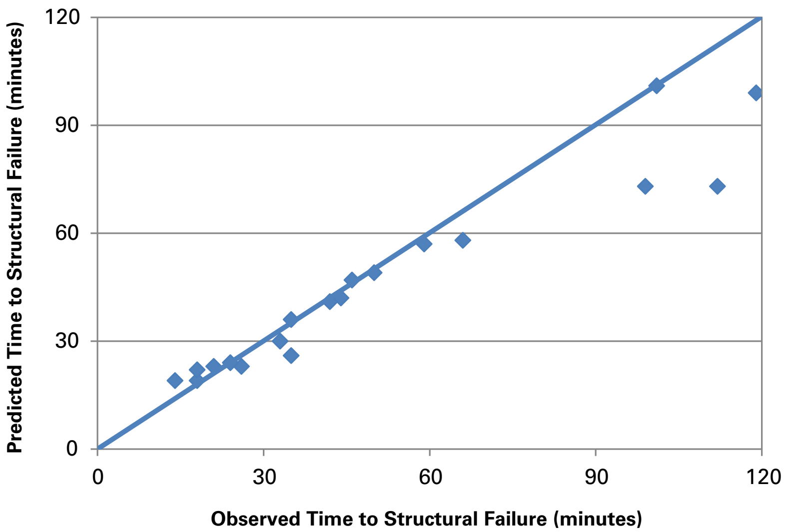

Figure 2-6 Comparison of Predicted to Observed Time to Failure (SCL tests)

Table 2.7d Measured and Calculated SCL Column Structural Fire Resistance Times

Designation	Species	SCL Width (in)	SCL Depth (in)	ASD Stress Ratio	Measured (Structural) t_f (min)	Calculated (Structural) t_f [1] (min)
TUB PSL 1	Southern Pine	7.87	7.87	0.66	42	41
TUB PSL 2	Southern Pine	7.09	7.09	0.75	35	37
NRC PSL 2	Southern Pine	10.50	10.50	0.92	59	57

[1] Assumed a char rate of 1.5 in/hr.

Loading of the floors and walls was determined using Canadian standards. For purposes of this analysis, allowable stress design (ASD) values were determined by using the relevant grades from the CLT product standard, PRG-320 [51]. Structural fire resistance was then calculated using the provisions in *Chapter 1* of this report substituting the appropriate ASD design values from PRG-320.

2.8.1 Test #3–Unprotected Floor

The first unprotected floor test (FPI #3) was a 5-ply CLT slab. The plies were each 1-3/8 inches thick for a total thickness of 6-7/8 inches thick. The CLT was constructed using SPF 1950f-1.7E lumber for the face and center laminations, and SPF No. 3 grade lumber for the two crossing laminations. This construction matched the CLT Grade E1 from PRG-320.

The dead load of the CLT floor was estimated to be 16 psf. A superimposed load of 245 psf was added, resulting in a total load of 261 psf. Using the 2015 NDS behavioral equations and standard properties from PRG 320, the allowable resisting moment was 124,800 in-lb/ft; however, for the purposes of modeling this specific test result, additional conservatisms built into the derivation of CLT design values were removed (these calculations assumed $C_F = 1.0$, $C_{fu} = 1.10$, $C_V = 1.0$, and removal of the 0.85 bending factor) resulting in an allowable resisting moment of 161,590 in-lb/ft and an ultimate resisting moment was estimated to be 460,520 in-lb/ft. Given a span of 186 inches, the induced moment was 94,340 in-lb/ft (58% of full design load). Failure was recorded at 96 minutes due to burn-through at the lap joint.

2.8.2 Test #4–Unprotected Wall

The first unprotected wall test (FPI #4) was a 5-ply CLT slab. The plies were each 1-3/8 inches thick for a total thickness of 6-7/8 inches thick. The CLT was constructed using SPF 1950f-1.7E lumber for the face and center laminations, and SPF No. 3 grade lumber for the two crossing laminations. This construction matched the CLT Grade E1 from PRG-320.

The dead load of the CLT wall was estimated to be 153 plf. A superimposed load of 22,820 plf was added, resulting in a total load of 22,970 plf. Using standard design properties from PRG 320 and the 2015 NDS behavioral equations, assuming an unbraced wall height of 120 inches and a buckling length coefficient, K_e, of 0.7 (see justification in *Section 2.7.4* of this report) for columns bearing on a rigid foundation, the initial allowable compression capacity was estimated to be 82,050 plf and the ultimate resisting compression capacity was estimated to be 203,470 plf. Failure was recorded at 113 minutes due to structural failure.

2.8.3 Test #7–Unprotected Floor

The second unprotected floor test (FPI #7) was a 7-ply CLT slab. The plies were each 1-3/8 inches thick for a total thickness of 9-5/8 inches thick. The CLT was constructed using SPF No.1/No.2 grade lumber for all laminations. For the relevant design properties needed for fire calculations, this construction matched CLT Grade V2 from PRG-320.

The dead load of the CLT floor was estimated to be 22 psf. A superimposed load of 304 psf was added, resulting in a total load of 326 psf. Using the 2015 NDS behavioral equations and standard properties from PRG 320, the allowable resisting moment is 99,300 in-lb/ft; however, for the purposes of modeling this specific test result, additional conservatisms built into the derivation of CLT design values were removed (these calculations assumed $C_f = 1.3$, $C_{fu} = 1.15$, $C_V = 1.0$, and removal of the 0.85 bending factor) resulting in an allowable resisting moment of 174,750 in-lb/ft and an ultimate resisting moment was estimated to be 498,030 in-lb/ft. Given a span of 186 inches, the induced moment was 117,810 in-lb/ft (67% of full design load). Failure was recorded at 179 minutes due to structural failure.

2.8.4 Test #8–Unprotected Wall

The second unprotected wall test (FPI #8) was a 5-ply CLT slab. The plies were each 13/16 inch thick for a total thickness of 4-1/16 inches thick. The CLT was constructed using SPF No.1/No.2 grade lumber for all laminations. For the relevant design properties needed for fire calculations, this construction matched CLT Grade V2 from PRG-320.

The dead load of the CLT wall was estimated to be 92 plf. A superimposed load of 4,933 plf was added, resulting in a total load of 5,025 plf. Using standard design properties from PRG 320 and the 2015 NDS behavioral equations, assuming an unbraced wall height of 120 inches and a buckling length coefficient, K_e, = 0.7, the initial allowable compression capacity was estimated to be 28,450 plf and the ultimate resisting compression capacity was estimated to be 66,900 plf. Failure was recorded at 57 minutes due to structural failure.

2.8.5 Proprietary CLT Wall Test

In May 2012, Intertek conducted a full-scale fire resistance test of a CLT wall [52]. All tests followed the ULC S101 time-temperature curve. The CLT wall was exposed directly to the flames (unprotected). Loading of wall was determined using Canadian standards. For purposes of this analysis, allowable stress design (ASD) values were determined by using the relevant grades from the CLT product standard, PRG-320. Structural fire resistance was then calculated using the provisions in *Chapter 1* of this report, substituting the appropriate ASD design values from PRG-320. The combined dead load and superimposed load resulted in a total load of 20,250 plf. Using standard design properties from PRG 320 and the 2015 NDS behavioral equations, assuming an unbraced wall height of 120 inches and a buckling length coefficient, K_e, = 0.7, the initial allowable compression capacity was estimated to be 42,720 plf and the ultimate resisting compression capacity was estimated to be 93,990 plf. Failure was recorded at 32 minutes due to structural failure.

2.8.6 Results of Analysis

Adjustments to the general design provisions derived in *Chapter 1* of this report were required to calculate the structural fire resistance of the CLT floor and wall assemblies tested by NRC. First, the nominal char rate of the CLT was found to be approximately 1.5 inches/hr in small-scale tests; however, during the full-scale tests, lamination falloff was observed. Working backwards from thermocouple data, lamination falloff was noted to occur at a time approximately related to the approach of the char front to the glueline interface. Calculation of the char depth, a_{char}, was adjusted to account for the lamination falloff as follows:

$$a_{char} = 1.2\left[n_{lam} \bullet h_{lam} + \beta_n \left(t - (n_{lam} \bullet t_{gi})\right)^{0.813}\right] \qquad \text{Equation 2-1}$$

where:

β_n = nominal char rate (in/hr), linear char rate based on 1-hour exposure

t = exposure time (hrs)

and

$$t_{gi} = \left(\frac{h_{lam}}{\beta_n}\right)^{1.23} \qquad \text{Equation 2-2}$$

t_{gi} = time to reach glued interface (hr)

h_{lam} = lamination thickness (in)

and

$$n_{lam} = \frac{t}{t_{gi}} \qquad \text{Equation 2-3}$$

n_{lam} = number of laminations charred (rounded down to lowest integer)

As previously mentioned, a second adjustment was related to conservative assumptions made when assigning CLT bending design values. When design values were assigned for the various grades of CLT, bending stresses were based on reference design values for lumber, not on adjusted design values. As a result, bending design values for CLT E-grades using E-rated laminations were not increased by the flat-use factor, C_{fu}, provided in the NDS. Similarly, design values for CLT V-grades using visually-graded laminations were not increased by the size factor, C_F, nor C_{fu}. In addition, a factor of 0.85 was taken on bending stresses. While significant in overall magnitude, the effect of these combined conservatisms typically has little impact on structural design of floors because spans tend to be limited by deflection and vibration concerns. However, for fire design, these conservatisms can result in significant underpredictions of structural fire resistance. For model verification purposes using these CLT fire test results, these conservatisms were removed so that actual fire resistance times could be compared with the fire resistance prediction times. Removal of these conservatisms for the purpose of verifying this model should not be construed as a recommendation to deviate from standard design values and assumptions.

The third adjustment was related to shear stiffness modeling. Initial calculations attempted to estimate the change in both the effective bending stiffness, EI_{eff}, and the effective shear stiffness, GA_{eff}. After reviewing the sensitivity of these calculations, it was found that tracking changes in GA_{eff} rather than using the relative EI_{eff} change for both EI_{eff} and GA_{eff} generally resulted in less than 1% difference in the final results. For that reason, it is recommended that changes to GA_{eff} due to charring be rolled into the changes in EI_{eff}, greatly simplifying the calculations and avoiding the need for development of additional adjustments to the procedures in *Chapter 1* of this report.

A fourth adjustment was related to estimating the effective length of the wall height when designing the wall as a column. In these tests, the strong axis laminations were loaded parallel to grain and the bearing was directly on a rigid base. As a result, the initial loading calculations were done assuming an effective length of $L_e = 0.7L_u$ as discussed previously in *Section 2.7.4* of this report regarding SCL column tests. As with the bending design value

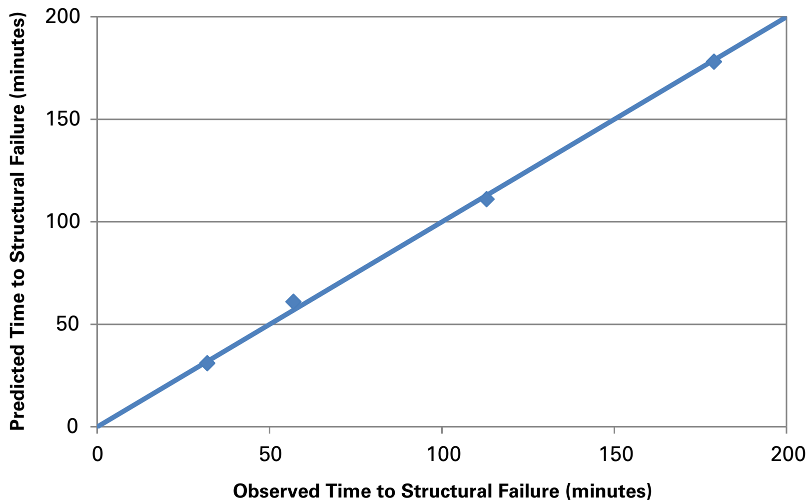

Figure 2-7 Comparison of Predicted to Observed Time to Failure (CLT tests)

Table 2.8 Measured and Calculated CLT Structural Fire Resistance Times

Designation	Species	CLT Application	CLT Thickness (in)	ASD Stress Ratio	Measured (Structural) t_f (min)	Calculated (Structural) t_f[1] (min)
FPI Test #3	Black Spruce	Floor	6.875	0.58	-[2]	114
FPI Test #4	Black Spruce	Wall	6.875	0.28	113	111
FPI Test #7	Black Spruce	Floor	9.625	0.67	179	178
FPI Test #8	Black Spruce	Wall	4.0625	0.18	57	61
Intertek Test	Black Spruce	Wall	4.125	0.48	32	31

[1] Assumed a char rate of 1.5 in/hr.
[2] Test halted at 96 minutes due to burn-through at unbacked lap joint.

adjustment, a less conservative effective length was used for model verification purposes in order to represent actual laboratory conditions but should not be construed as a recommendation to deviate from standard design assumptions.

The fire resistance of each unprotected CLT test is provided in *Table 2.8* and *Figure 2-7*. The first exposed floor test was terminated early, at about 96 minutes, due to burn-through of the CLT at one of the lap joints. While burn-through would technically be a failure in an E119 test, it did not result in a structural failure and could have easily been avoided by covering joints on the unexposed side with a floor covering; therefore, the observed failure time was not included in the final comparison of predicted structural failure. For the remaining tests, the calculated fire resistance times predicted the actual observed fire resistance times very well (see *Table 2.8*).

2.9 Summary

As can be seen in *Figures 2-1* through *2-7*, the mechanics-based method which uses a standard nominal char rate, B_n = 1.5 in/hr, for all species, a non-linear char rate adjustment, a constant char acceleration factor of 1.2, and a standard variability adjustment in the design to ultimate adjustment factor predicts average resistance times for beams, columns, decks, and light-frame wood members that closely track actual resistance times for tested members. While further refinements of this method are possible, these comparisons suggest that standardized adjustments to design stresses, a standardized accelerated char rate, and the use of the NDS behavioral equations adequately address fire design of exposed wood members.

Part 3: Design Procedures for Exposed Wood Members

3.1 Design Procedures for Wood Members

Failure of a member occurs when the load on the member exceeds the member capacity which has been reduced due to fire exposure. This new mechanics-based design procedure calculates the capacity of exposed wood members using basic wood engineering mechanics and was originally incorporated into the 2001 NDS [25] for fire resistance calculations of up to 2 hours. Actual mechanical and physical properties of the wood are used and the capacity of the member is directly calculated for a given period of time. Section properties are computed assuming an effective char rate, β_{eff}, at a given time, t. Reductions of strength and stiffness of wood directly adjacent to the char layer are addressed by accelerating the char rate 20%. Average member strength properties are approximated from existing accepted procedures used to calculate design properties. Finally, wood members are designed using accepted engineering procedures found in the NDS.

3.1.1 Char Rate

The effective char rate to be used in this procedure can be estimated from published nominal one-hour char rate data using the equation from *Section 1.4.1*:

$$\beta_{eff} = \frac{1.2\,\beta_n}{t^{0.187}} \qquad \text{Equation 1-14}$$

Where:

β_{eff} = Effective char rate (in/hr), adjusted for exposure time, t

β_n = Nominal char rate (in/hr), linear char rate based on 1-hour exposure

t = Exposure time (hrs)

A nominal char rate, β_n, of 1.5 inches/hour is commonly assumed for sawn lumber and timbers, glued-laminated timbers, laminated veneer lumber, parallel strand lumber, laminated strand lumber, and cross-laminated timber.

3.1.1.1 For sawn lumber and timbers, glued laminated timbers, laminated veneer lumber, parallel strand lumber, and laminated strand lumber with a nominal char rate, where β_n = 1.5 inches/hour, the effective char rates, β_{eff}, and effective char layer depths, a_{char}, for each exposed surface are:

Table 3.1.1.1 Effective Char Rates and Char Layer Depths (for β_n = 1.5 inches/hour)

Required Fire Resistance (hr)	Effective Char Rate, β_{eff} (in/hr)	Effective Char Layer Depth, a_{char} (in)
1-Hour	1.8	1.8
1½-Hour	1.67	2.5
2-Hour	1.58	3.2

Section properties can be calculated using standard equations for area, section modulus and moment of inertia using the reduced cross-sectional dimensions. The dimensions are reduced by the effective char depth, a_{char}, for each surface exposed to fire.

3.1.1.2 For cross-laminated timber, falloff of laminations has been noted during full-scale tests. The falloff appears to occur as the char front reaches the glueline. To model this effect, the time required to reach the glueline for each lamination can be calculated as:

$$t_{gl,i} = \left(\frac{h_{lam,i}}{\beta_n}\right)^{1.23} \qquad \text{Equation 3.1}$$

Where:

$t_{gi,i}$ = time to reach glued interface for each lamination (hr)

h_{lam} = lamination thickness (in)

The number of laminations that could potentially falloff is estimated by subtracting each t_{gl} from the total time until the last partial lamination is determined. The value of n_{lam} is the maximum value in which the following equation is true:

$$t - \sum_{i=1}^{n_{lam}} t_{gl,i} \geq 0 \qquad \text{Equation 3.2}$$

Where:

n_{lam} = number of laminations charred (rounded to lowest integer)

The values of $t_{gl,i}$ and n_{lam} determined in Equation 3.2 are used to calculate the effective char depth, a_{char}:

$$a_{char} = 1.2\left[\sum_{i=1}^{n_{lam}} h_{lam,i} + \beta_n \left(t - \sum_{i=1}^{n_{lam}} t_{gl,i}\right)^{0.813}\right] \quad \text{Equation 3.3}$$

For cross-laminated timber manufactured with laminations of equal thickness, calculation of the effective char depth, a_{char}, can be simplified as follows:

$$a_{char} = 1.2\left[n_{lam} \bullet h_{lam} + \beta_n \left(t - (n_{lam} \bullet t_{gi})\right)^{0.813}\right] \quad \text{Equation 3.4}$$

Where:

$$t_{gi} = \left(\frac{h_{lam}}{\beta_n}\right)^{1.23} \quad \text{Equation 3.4.1}$$

and

$$n_{lam} = \frac{t}{t_{gi}} \quad \text{Equation 3.4.2}$$

For cross-laminated timber manufactured with laminations of equal thickness and assuming a nominal char rate, β_n, of 1.5 in/hr, the effective char depths for each exposed surface are:

Table 3.1.1.2 Effective Char Depth (for CLT with β_n = 1.5 inches/hour)

Required Fire Resistance (hr)	Effective Char Depth, a_{char} (in) — lamination thicknesses, h_{lam} (in)						
	5/8	3/4	7/8	1	1¼	1 3/8	1½
1-Hour	2.2	2.2	2.1	2.0	2.0	1.9	1.8
1½-Hour	3.4	3.2	3.1	3.0	2.9	2.8	2.8
2-Hour	4.4	4.3	4.1	4.0	3.9	3.8	3.6

For cross-laminated timber, reduced section properties must account for the influence of char depth on the actual laminations. Unlike other laminated wood products with the strength axis oriented in one major axis, the influence of the char depth on cross-laminated timber has more influence on laminations oriented in the axis being stressed and less in the perpendicular axis. While the product standards have developed models for the effects of the lamination properties in the major and minor strength axis, the effect of char depth has not be included. Therefore, effects of the char depth on actual section properties should be calculated using equations provided by the cross-laminated timber manufacturer based on the actual layup used in the manufacturing process. For an approximate conservative estimate, the procedures in 3.1.1.2 can be used to determine which laminations have charred and thinner cross-laminated members of the same configuration and the same number of laminations as the remaining uncharred laminations can be used.

Table 3.1.2 Allowable Design Stress to Average Ultimate Strength Adjustment Factor

Member Capacity	K
Bending Moment Capacity, in-lbs.	2.85
Tensile Capacity, lbs.	2.85
Compression Capacity, lbs.	2.58
Beam Buckling Capacity, lbs.	2.03
Column Buckling Capacity, lbs.	2.03

3.1.2 Approximation of Member Strength and Capacity

For fire design, the estimated member capacity is evaluated against the loss of cross-section and mechanical properties as a result of fire exposure. While the loss of cross-section and mechanical properties are addressed by reducing the section properties using the effective char layer thickness, the average member strength properties must be determined from published allowable design stresses. The average member capacity of a wood member exposed to fire for a given time, *t*, can be estimated using the average member strength and reduced cross-sectional properties. For sawn lumber and timbers, glued-laminated timbers, and structural composite lumber and cross-laminated timber members, the average member capacity can be approximated by multiplying the allowable design values by the adjustment factors, *K* shown in *Table 3.1.2*.

Axial/bending interactions can be calculated using this procedure. All member strength and cross-sectional properties should be adjusted prior to the interaction calculations. The interaction calculations should then be conducted in accordance with appropriate NDS provisions.

3.1.3 Design of Members

Once the member capacity has been determined using the effective section properties from *Section 3.1.1* and the member strength approximations from *Section 3.1.2*, the wood member can be designed using accepted NDS design procedures for the following loading condition:

$$D + L \leq K\, R_{ASD} \qquad \text{Equation 3-5}$$

Where:
D = Design dead load
L = Design live load
R_{ASD} = Nominal allowable design capacity
K = Factor to adjust from nominal design capacity to average ultimate capacity

3.2 Design Procedures for Timber Decks

Timber decks consist of planks that are at least 2 inches (nominal) thick. The planks span the distance between supporting beams, and can be arranged in different ways depending on the available lengths [20]. Usually, a single or double tongue-and-groove joint is used to connect adjoining planks, but splines or butted joints are also common.

In order to meet requirements for a given fire resistance rating, a timber deck needs to maintain its thermal separation function and load carrying capacity for the specified duration of exposure to standard fire conditions. The thermal separation requirement limits the temperature rise on the unexposed side of the deck to 250°F above ambient temperature over the entire surface area, or 325°F above ambient temperature at a single location. When the limits cannot be met by the decking alone, additional floor coverings can be used to increase the thermal separation time. The calculation procedures in this report do not address the adequacy of thermal separation.

The load carrying capacity requires that the deck carry the specified load for the required resistance time. The structural design procedures described in *Section 3.1* also apply to timber decks. Single and double tongue-and-groove (T&G) decking should be designed as an assembly of wood beams fully-exposed on one face. Butt-jointed decking should be designed as an assembly of wood beams partially-exposed on the sides and fully-exposed on one face. To compute the effects of partial exposure of the decking on its sides, the char rate for this limited exposure should be reduced to 33% of the effective char rate.

3.3 Special Provisions for Glued Laminated Timber Beams

For glued laminated timber bending members rated for 1-hour fire resistance, an outer tension lamination shall be substituted for a core lamination on the tension side for unbalanced beams and on both sides for balanced beams as shown in *Figure 3-1(b)* and *Figure 3-2(b)*, respectively. For glued laminated timber bending members rated for 1½ or 2-hour fire resistance, two outer tension laminations shall be substituted for two core laminations on the tension side for unbalanced beams and on both sides for balanced beams as shown in *Figure 3-1(c)* and *Figure 3-2(c)*, respectively.

3.4 Wood Connections

Where one-hour fire resistance is required, connectors and fasteners must be protected from fire exposure by wood, fire-rated gypsum board, or any coating approved for the required resistance time. Typical details for commonly used fasteners and connectors in timber framing are shown in *Figure 3-3* (Beam to Column Connection Not Exposed to Fire), *Figure 3-4* (Beam to Column Connection Exposed to Fire Where Appearance is a Factor), *Figure 3-5* (Ceiling Construction), *Figure 3-6* (Beam to Column Connection Exposed to Fire Where Appearance is Not a Factor), *Figure 3-7* (Column Connections – Covered), *Figure 3-8* (Beam to Girder – Concealed Connection).

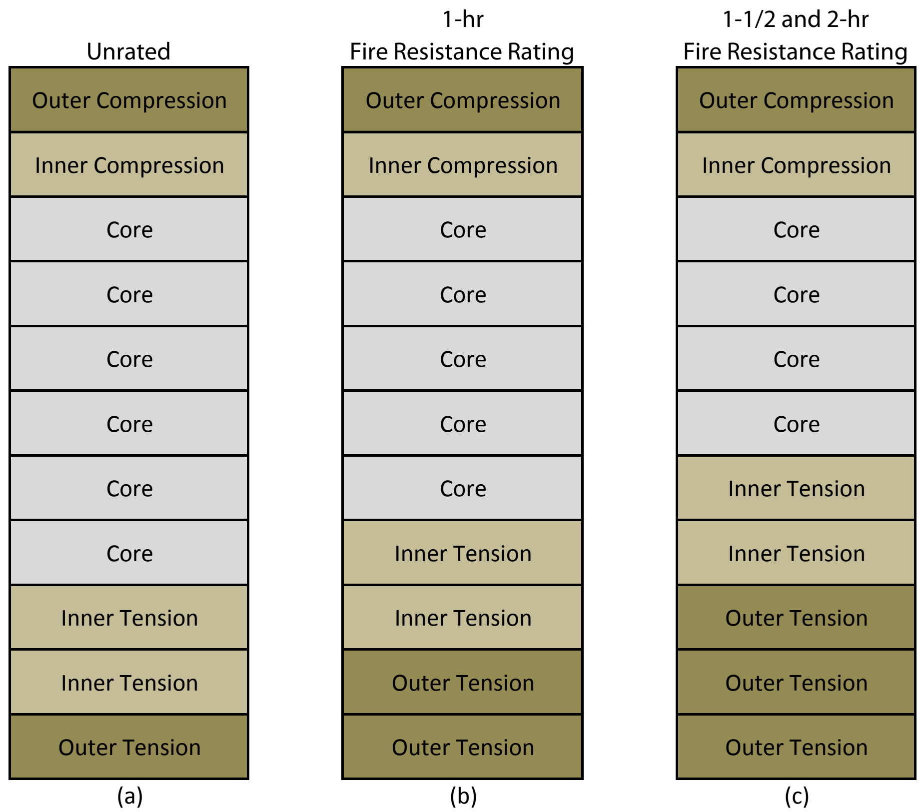

Figure 3-1 Typical glulam unbalanced beam layups

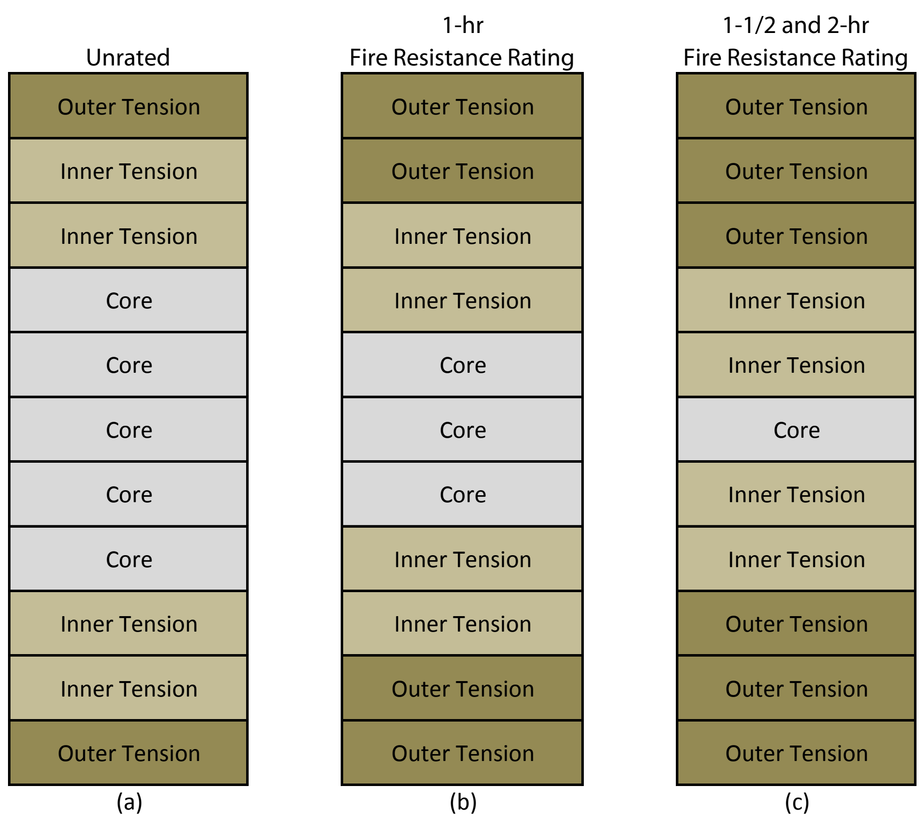

Figure 3-2 Typical glulam balanced beam layups

3.5 Application Guidelines for Wood Members

For given member sizes, different resistance times can be achieved by varying the percent of maximum design load applied to the member. Examples of the relationship between section size, load ratio, and fire resistance time are provided in *Part 4*. Tabulated design aids have been developed for some common design cases and are provided in the *Appendix A*.

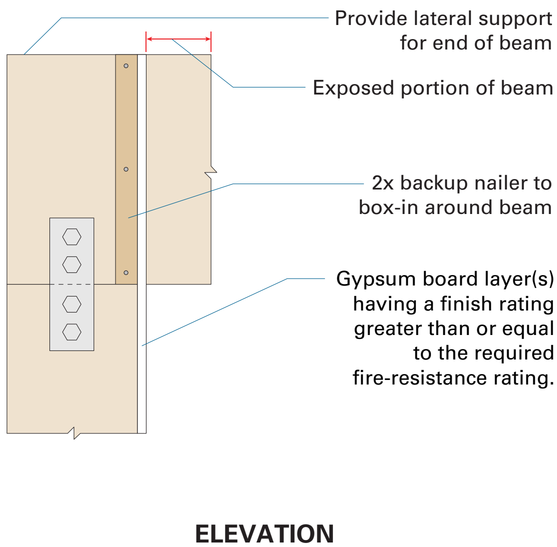

Figure 3-3 Beam to column connection, connection not exposed to fire

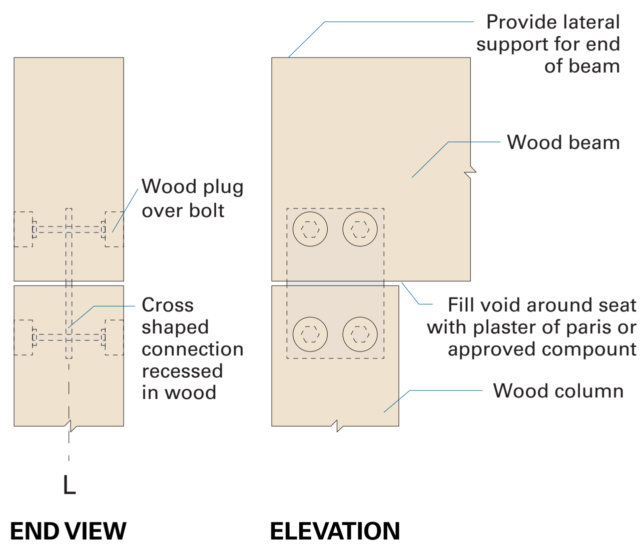

Figure 3-4 Beam to column connection, connection exposed to fire where appearance is a factor

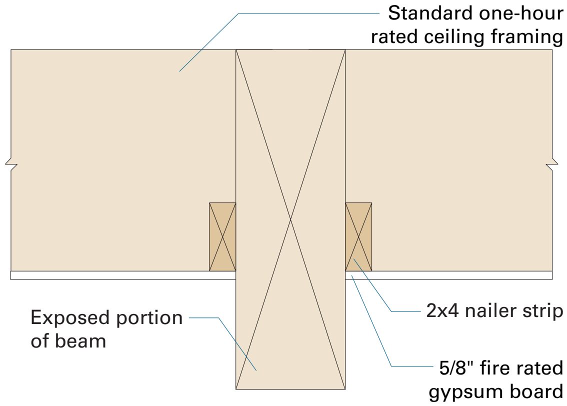

Figure 3-5 Ceiling construction

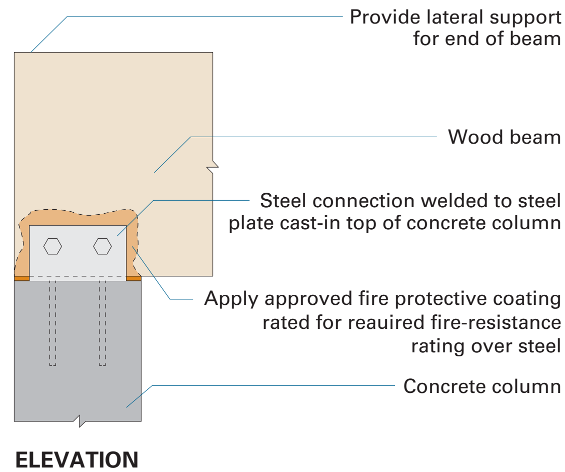

Figure 3-6 Beam to column connection, connection exposed to fire where appearance is not a factor

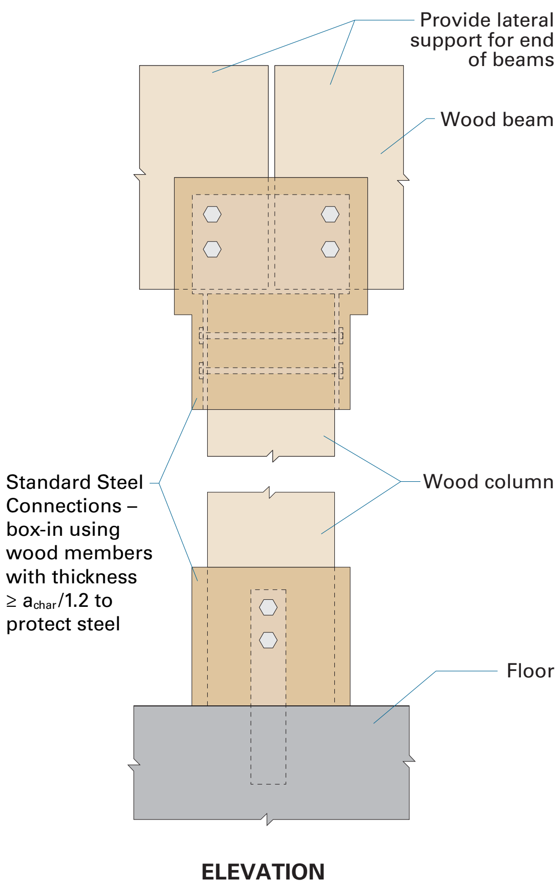

Figure 3-7 Column connections – covered

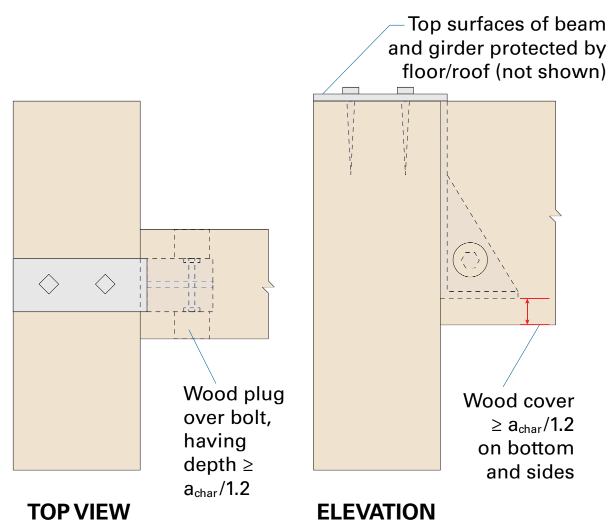

Figure 3-8 Beam to girder – concealed connection

Part 4: Design Procedure Examples

4.1 Exposed Beam Example (*Allowable Stress Design*)

Simply-supported Douglas fir glulam beams span L = 18 ft, and are spaced at s = 6 ft. The design loads are q_{live} = 100 psf and q_{dead} = 25 psf. Timber decking nailed to the compression edge of the beams provides lateral bracing for at least the same fire resistance as the beams (i.e. $C_L = 1.0$). Calculate the required section dimensions for a one-hour fire resistance time.

For the structural design of the wood beam, calculate the maximum induced moment. Calculate beam load:

$$w_{load} = s\,(q_{dead} + q_{live}) = (6)(25 + 100) = 750 \text{ plf}$$

Calculate maximum induced moment:

$$M_{max} = w_{load}\,L^2 / 8 = (750)(18^2)/8 = 30{,}375 \text{ ft-lb}$$

Select a 6¾" x 13½" 24F visually-graded Douglas-fir glulam beam with a tabulated bending stress, F_b, equal to 2400 psi.
Calculate beam section modulus:

$$S_s = bd^2/6 = (6.75)(13.5)^2/6 = 205 \text{ in}^3$$

Calculate the adjusted allowable bending stress (assuming $C_D = 1.0$: $C_M = 1.0$: $C_t = 1.0$: $C_L = 1.0$: $C_V = 0.98$)

$$F'_b = F_b\,(C_D)(C_M)(C_t)(\text{lesser of } C_L \text{ or } C_V) = 2400\,(1.0)(1.0)(1.0)(0.98) = 2343 \text{ psi (NDS 5.3.1)}$$

Calculate design resisting moment:

$$M'_s = F'_b\,S_s = (2343)(205.0)/12 = 40{,}032 \text{ ft-lb}$$

Structural Check: $\mathbf{M'_s \geq M_{max}}$ **40,032 ft-lb > 30,375 ft-lb** √

For the fire design of the wood beam, the loading is unchanged. Therefore, the maximum induced moment is unchanged. The fire resistance must be calculated.
Calculate beam section modulus exposed on three-sides:

$$S_f = (b\text{-}2a)(d\text{-}a)^2/6 = (6.75\text{-}3.6)(13.5\text{-}1.8)^2/6 = 71.9 \text{ in}^3 \text{ (NDS 16.2.1)}$$

Calculate the adjusted bending strength (assuming C_D = N/A: C_M = N/A: C_t = N/A: $C_L = 1.0$: $C_V = 0.98$)

$$F'_{b,f} = (2.85)\,F_{b,f}\,(\text{lesser of } C_L \text{ or } C_V) = 2.85(2400)(0.98) = 6703 \text{ psi (NDS 16.2.2)}$$

Calculate the resisting moment:

$$M'_f = F'_{b,f}\,S_f = (6703)(71.9)/12 = 40{,}145 \text{ ft-lb (NDS 16.2.2)}$$

Fire Check: $\mathbf{M'_f \geq M_{max}}$ **40,145 ft-lb > 30,375 ft-lb** √

4.1.1 Simplified Alternative Approach (*using design aid in Appendix A*):

Select the maximum design load ratio limit from *Appendix A, Table A-1 (1-hr)* or calculate using the following equation:

$$R_s = \frac{2.85\ S_f}{S_s\ C_D\ C_M\ C_t} = \frac{(2.85)(71.9)}{(205)(1.0)(1.0)(1.0)} = 1.00$$

Fire Check: $M'_s R_s \geq M_{max}$ (40,032 ft-lb)(1.00) > 30,375 ft-lb √

For this simple case where only bending moment in one axis is being checked, the design load ratio limit from Appendix Table A1 (1-hr), R_s = 1.00, need only be greater than the structural design load ratio, $r_s = M_{max}/M'_s$ = 30,375/40,032 = 0.76.

4.2 Exposed Column Example (*Allowable Stress Design*)

A Southern pine glulam column with an effective column length, L_e = 168 inches. The design loads are P_{snow} = 16,000 lb and P_{dead} = 6,000 lb. Calculate the required section dimensions for a one-hour fire resistance time.

For the structural design of the wood column, calculate the maximum induced compression stress, f_c. Calculate column load:

$$P_{load} = P_{dead} + P_{snow} = 6{,}000 + 16{,}000 = 22{,}000 \text{ lb}$$

Select a 8½" x 9⅝" Combination #48 Southern pine glulam column with a tabulated compression parallel-to-grain stress, F_c, equal to 2200 psi and a tabulated minimum modulus of elasticity, E_{min}, equal to 900,000 psi. Calculate column area:

$$A_s = bd = (9.625)(8.5) = 81.81 \text{ in}^2$$
$$I_s = bd^3/12 = (9.625)(8.5)^3/12 = 492.6 \text{ in}^4$$

Calculate the adjusted allowable compression stress (assuming C_D = 1.15: C_M = 1.0: C_t = 1.0):

$$E'_{min} = E_{min}\,(C_M)(C_t) = 900{,}000\ (1.0)(1.0) = 900{,}000 \text{ psi (NDS 5.3.1)}$$
$$L_e/d_1 = 168/9.625 = 17.5 : L_e/d_2 = 168/8.5 = 19.7 : \text{Maximum } L_e/d = 19.7 \text{ (NDS 3.7.1.3)}$$
$$F_{cE} = 0.822\ E'_{min} / (L_e/d)^2 = 0.822\ (900{,}000) / (19.7)^2 = 1894 \text{ psi (NDS 3.7.1.5)}$$
$$F_c^* = F_c\,(C_D)(C_M)(C_t) = 2200\ (1.15)(1.0)(1.0) = 2530 \text{ psi (NDS 3.7.1.5)}$$
$$c = 0.9 \text{ for glued laminated timbers (NDS 3.7.1.5)}$$
$$\alpha_c = F_{cE}/F_c^* = 1894/2530 = 0.7485$$
$$C_p = \frac{1+\alpha_c}{2c} - \sqrt{\left(\frac{1+\alpha_c}{2c}\right)^2 - \frac{\alpha_c}{c}} = \frac{1+0.7485}{2(0.9)} - \sqrt{\left(\frac{1+0.7485}{2(0.9)}\right)^2 - \frac{0.7485}{0.9}} = 0.6369 \text{ (NDS 3.7.1.5)}$$
$$F'_c = F_c^*\,C_p = 2530\ (0.6368) = 1611 \text{ psi (NDS 5.3.1)}$$

Calculate the resisting column compression capacity:

$$P'_s = F'_c\,A_s = (1611)(81.81) = 131{,}819 \text{ lb}$$

Structural Check: $P'_s \geq P_{load}$ 131,819 lb > 22,000 lb √

For the fire design of the wood column, the loading is unchanged. Therefore, the total load is unchanged. The fire resistance must be calculated.

Calculate column area, A, and moment of inertia, I, for column exposed on four-sides:

$A_f = (b - 2a)(d - 2a) = (9.625 - 3.6)(8.5 - 3.6) = 29.52\ in^2$
$I_f = (b - 2a)(d - 2a)^3/12 = (9.625 - 3.6)(8.5 - 3.6)^3/12 = 59.07\ in^4$

Calculate the adjusted compression strength (assuming C_D = N/A: C_M = N/A: C_t = N/A):

$L_e/d_1 = 168/(9.625 - 3.6) = 27.9 : L_e/d_2 = 168/(8.5 - 3.6) = 34.3$: Maximum $L_e/d = 34.3$ (NDS 3.7.1.3)
$F_{cE,f} = (2.03)\ 0.822\ E' / (L_e/d)^2 = (2.03)(0.822)(900{,}000) / (34.3)^2 = 1278$ psi (NDS 16.2.2)
$F^*_{c,f} = (2.58)\ F_c = (2.58)(2200) = 5676$ psi (NDS 16.2.2)
$\alpha_c = F_{cE,f}/F^*_{c,f} = 1278/5676 = 0.2251$

$$C_{p,f} = \frac{1 + 0.2251}{2(0.9)} - \sqrt{\left(\frac{1 + 0.2251}{2(0.9)}\right)^2 - \frac{0.2251}{0.9}} = 0.2189$$

$F'_{c,f} = 5676\ (0.2189) = 1243$ psi

Calculate the resisting column compression capacity:

$P'_f = F'_{c,f}\ A_f = (1243)(29.52) = 36{,}689$ lbs

Fire Check: $\mathbf{P'_f \geq P_{load}}$ **36,689 lb > 22,000 lb** √

4.2.1 Simplified Alternative Approach (*using design aid in Appendix A*)

Calculate the maximum design load ratio using R_{s1} and R_{s2} terms from *Appendix A, Table A2 (1-hr)*:

$R_{s1} = 0.25 : R_{s2} = 1.09$
$R_s = R_{s1}\ R_{s2} = (0.25)(1.09) = 0.27 \leq 1.0$ (use 0.27)

Fire Check: $\mathbf{P'_s R_s \geq P_{load}}$ **(131,819 lb)(0.27) = 35,600 lb > 22,000 lb** √

For this simple case where only concentrically-loaded compression is being checked, the design load ratio limit from Appendix Table A2 (1-hr), $R_s = 0.27$, need only be greater than the structural design load ratio, $r_s = P_{load} / P'_s = 22{,}000/131{,}819 = 0.17$.

Note: While it does not make a difference in the final result of this example, the simplified alternative method will usually yield a slightly conservative value for the fire design check as illustrated in this example by the difference between 36,689 lb and 35,600 lb. This is because the R_{s1} and R_{s2} values are derived based on the most conservative result using c = 0.8 and c = 0.9. This allows the simplified alternative method to be used for sawn lumber, structural glued laminated timber, and structural composite lumber.

4.3 Exposed Tension Member Example (*Allowable Stress Design*)

Sawn Hem-Fir timbers used as heavy timber truss bottom chords with an unbraced length L_u = 20 ft. The total design tension load from a roof live and dead load are P_{load} = 2,000 lb. The bending load due to the dead load of the timber will be determined based on timber size. Calculate the required section dimensions for a one-hour fire resistance time.

Select a nominal 6x6 (5½" x 5½") Hem-Fir #2 grade timber with a tabulated bending stress, F_b, equal to 575 psi, a tabulated tension stress, F_t, equal to 375 psi, and a tabulated minimum modulus of elasticity, E_{min}, equal to 400,000 psi.

Calculate timber area:

$A_s = bd = (5.5)(5.5) = 30.25\ in^2$
$S_s = bd^2/6 = (5.5)(5.5)^2/6 = 27.73\ in^3$

Calculate the maximum induced tension stress, f_t:

$P_{load} = 2{,}000$ lb
f_t = Pload / As = 2000/30.25 = 66.1 psi

Calculate the adjusted allowable tension stress (assuming $C_D = 1.25$: $C_M = 1.0$: $C_t = 1.0$):

$F'_t = F_t\ (C_D)(C_M)(C_t) = 375\ (1.25)(1.0)(1.0) = 469$ psi (NDS 4.3.1)

The weight of the timber is estimated 30 pcf:

w_{load} = (30 pcf / 144)(30.25) = 6.3 plf

Calculate maximum induced bending stress, f_b:

$M_{max} = w_{load}\ L^2 / 8 = (6.3)(20^2)/8 = 315$ ft-lb = 3,780 in-lb
$f_b = M_{max} / S_s = 3780/27.73 = 136$ psi

Calculate the adjusted allowable bending stress (assuming $C_D = 1.25$: $C_M = 1.0$: $C_t = 1.0$):

$F^*_b = F_b\ (C_D)(C_M)(C_t) = 575\ (1.25)(1.0)(1.0) = 719$ psi (NDS 3.3.3.8)
Since b = d, $C_L = 1.0$ (NDS 3.3.3.1)
$F'_b = F^*_b\ C_L = 719\ (1.0) = 719$ psi (NDS 4.3.1)

Structural Check:	**$F'_t \geq f_t$**	**469 psi ≥ 83 psi**	√
Structural Check:	**$F'_b \geq f_b$**	**719 psi ≥ 136 psi**	√
Structural Check:	**$f_t / F'_t + f_b / F^*_b \leq 1.0$**	**66.1/469 + 136/719 = 0.33 ≤ 1.0**	√

For the fire design of the timber tension member, the loading is unchanged. Therefore, the total load is unchanged; however, the dead load of the timber has changed.
Calculate effective section properties for member exposed on four-sides:

$A_f = (b - 2a)(d - 2a) = (5.5 - 3.6)(5.5 - 3.6) = 3.61\ in^2$
$S_f = (b - 2a)(d - 2a)^2/6 = (5.5 - 3.6)(5.5\text{-}3.6)^2 /6 = 1.14\ in^3$

Calculate the maximum induced tension stress, f_t:

$P_{load} = 2{,}000$ lb
$f_t = P_{load} / A_s = 2000/3.61 = 554$ psi

Calculate the adjusted tension strength (assuming C_D = N/A: C_M = N/A: C_t = N/A):

$F'_{t,f} = (2.85)\ F_t = 2.85\ (375) = 1069$ psi (NDS 16.2.2)

The initial weight of the timber was estimated to be 30 pcf; however, the volume of the beam has changed due to charring. The weight of the char layer is assumed to be negligible, but the actual char depth is used rather than the effective char depth. The actual char depth can be estimated by dividing the effective char depth by 1.20:

w_{load} = (30 pcf /144) (b – 2(a/1.2))(d – 2(a/1.2)) = (30 pcf / 144) (5.5-3.0)(5.5-3.0) = 1.3 plf

Calculate maximum induced bending stress, f_b:

$M_{max} = w_{load} L^2 / 8 = (1.3)(20^2)/8$ = 65 ft-lb = 780 in-lb
$f_b = M_{max} / S_s = 780/1.14$ = 683 psi

Calculate the adjusted bending strength (assuming C_D = N/A: C_M = N/A: C_t = N/A):

$F^*_{b,f} = (2.85)\ F_b\ (C_D)(C_M)(C_t) = (2.85)(575)$ = 1639 psi (NDS 3.3.3.8)
Since (b – 2a) = (d – 2a), C_L = 1.0 (NDS 3.3.3.1)
$F'_{b,f} = F^*_b\ C_L = 1639\ (1.0)$ = 1639 psi (NDS 4.3.1)

Fire Check:	$F'_{t,f} \geq f_t$	**1069 psi ≥ 554 psi**	√
Fire Check:	$F'_{b,f} \geq f_b$	**1639 psi ≥ 683 psi**	√
Fire Check:	$f_t / F'_{t,f} + f_b / F^*_{b,f} \leq 1.0$	**554/1069 + 683/1639 = 0.93 ≤ 1.0**	√

4.4 Exposed Deck Example (*Allowable Stress Design*)

Decking spans L = 6 feet. A single layer of 3/4 inch sheathing is installed over the decking. The design loads are q_{live} = 40 psf and q_{dead} = 10 psf. Calculate the required decking depth for a one-hour fire resistance time using tongue-and-groove or butt-jointed timber decking.

4.4.1 Tongue-and-Groove Decking

Calculate deck load on a one-foot-wide strip:

$w_{load} = B(q_{dead} + q_{live})$ = (1 ft)(50 psf) = 50 plf

Calculate maximum induced moment:

$M_{max} = w_{load} L^2 / 8 = (50)(6^2)/8$ = 225 ft-lbs

Select nominal 3x6 (2½" x 5½") Hem-Fir tongue-and-groove Commercial decking with a tabulated repetitive member bending stress, $F_b(C_r)$, equal to 1350 psi.
Calculate the section modulus of a one-foot-wide strip:

$S_s = bd^2/6 = (12)(2.5)^2/6 = 12.5$ in^3

Calculate the adjusted allowable bending stress (assuming C_D = 1.0: C_M = 1.0: C_t = 1.0: C_F = 1.04):

$F'_{b,s} = F_b(C_r)\ (C_D)(C_M)(C_t)(C_F) = 1350\ (1.0)(1.0)(1.0)(1.04)$ = 1404 psi (NDS 4.3.1)

Calculate resisting moment:

$M'_s = F'_b\ S_s = (1404)(12.5)/12$ = 1463 ft-lbs

Structural Check:	$M'_s \geq M_{max}$	**1463 ft-lbs > 225 ft-lbs**	√

For the fire design of the timber deck, the loading is unchanged. Therefore, the maximum induced moment is unchanged. The fire resistance must be calculated.
Calculate the section modulus of a one-foot-wide strip exposed on the bottom surface:

$$S_f = (b)(d\text{-}a)^2/6 = (12)(2.5\text{-}1.8)^2/6 = 0.98 \text{ in}^3$$

Calculate the adjusted allowable bending stress (assuming C_D = N/A: C_M = N/A: C_t = N/A: C_F = 1.04):

$$F'_{b,f} = (2.85)\ F_b(C_r)(C_F) = 2.85(1350)(1.04) = 4001 \text{ psi}$$

Calculate resisting moment:

$$M'_f = F'_{b,f}\ S_f = (4001)(0.98)/12 = 327 \text{ ft-lbs (NDS 16.2.2)}$$

Fire Check: $M'_f \geq M_{max}$ 327 ft-lbs > 225 ft-lbs √

4.4.1.1 Simplified Alternative Approach (*using design aid in Appendix A*)

Select the maximum design load ratio limit from *Appendix A, Table A3.2* or calculate using the following equation:

$$R_s = \frac{2.85\ S_f}{S_s\ C_D\ C_M\ C_t} = \frac{(2.85)(0.98)}{(12.5)(1.0)(1.0)(1.0)} = 0.22$$

Fire Check: $M'_s R_s \geq M_{max}$ (1463 ft-lb)(0.22) = 322 > 225 ft-lb √

For this simple case where only bending moment in one axis is being checked, the design load ratio limit from Appendix Table A3.2, R_s = 0.22, need only be greater than the structural design load ratio, r_s = M_{max} /M' = 225/1463 = 0.15.

4.4.2 Butt-Jointed Decking

Assume a board width of 5.5 inches
Calculate deck load:

$$w_{load} = B(q_{dead} + q_{live}) = (5.5 \text{ in} / 12 \text{ in/ft})(50 \text{ psf}) = 22.9 \text{ plf}$$

Calculate maximum induced moment on each member:

$$M_{max} = w_{load}\ L^2 / 8 = (22.9)(6^2)/8 = 103 \text{ ft-lbs}$$

Select nominal 3x6 (2½" x 5½") Hem-Fir butt-jointed Commercial decking with a tabulated repetitive member bending stress, $F_b(C_r)$, equal to 1350 psi.
Calculate the section modulus of each member:

$$S_s = bd^2/6 = (5.5)(2.5)^2/6 = 5.73 \text{ in}^3$$

Calculate the adjusted allowable bending stress (assuming C_D = 1.0: C_M = 1.0: C_t = 1.0: C_F = 1.04):

$$F'_{b,s} = F_b(C_r)\ (C_D)(C_M)(C_t)(C_F) = 1350\ (1.0)(1.0)(1.0)(1.04) = 1404 \text{ psi (NDS 4.3.1)}$$

Calculate resisting moment:

$$M'_s = F'_b\ S_s = (1404)(5.73)/12 = 670 \text{ ft-lbs}$$

Structural Check: $M' \geq M_{max}$ 670 ft-lbs > 103 ft-lbs √

For the fire design of the timber deck, the loading is unchanged. Therefore, the maximum induced moment is unchanged. The fire resistance must be calculated.
Calculate the section modulus of a member, exposed fully on the bottom surface, with 33% of the effective char rate on the butt-jointed sides:

$S_f = (b - 2(a/3))(d - a)^2/6 = (5.5 - 2(1.8/3))(2.5 - 1.8)^2/6 = 0.351\ in^3$

Calculate the adjusted allowable bending stress (assuming C_D = N/A: C_M = N/A: C_t = N/A: C_F = 1.04):

$F'_{b,f} = (2.85)\ F_b(C_r)(C_F) = 2.85(1350)(1.04) = 4001$ psi

Calculate resisting moment:

$M'_f = F'_{b,f}\ S_f = (4001)(0.351)/12 = 117$ ft-lbs (NDS 16.2.2)

Fire Check:	$\mathbf{M'_f \geq M_{max}}$	**117 ft-lbs > 103 ft-lbs**	√

4.4.2.1 Simplified Alternative Approach (*using design aid in Appendix A*)

Select the maximum design load ratio limit from *Appendix A, Table A3.1* or calculate using the following equation:

$$R_s = \frac{2.85\ S_f}{S_s\ C_D\ C_M\ C_t} = \frac{(2.85)(0.351)}{(5.73)(1.0)(1.0)(1.0)} = 0.18$$

Fire Check:	$\mathbf{M'_s R_s \geq M_{max}}$	**(670 ft-lb)(0.18) = 120 > 103 ft-lb**	√

For this simple case where only bending moment in one axis is being checked, the design load ratio limit from Appendix Table A3.1, R_s = 0.17, need only be greater than the structural design load ratio, $r_s = M_{max}/M' = 225/1463 = 0.15$.

4.5 Exposed CLT Floor Example (*Allowable Stress Design*)

Simply-supported cross-laminated timber (CLT) floor spanning L=18 ft in the strong-axis direction. The design loads are q_{live} = 80 psf and q_{dead} = 30 psf including estimated self-weight of the CLT panel. Floor decking, nailed to the unexposed face of CLT panel, is spaced to restrict hot gases from venting through half-lap joints at edges of CLT panel sections. Calculate the required section dimensions for a one-hour fire resistance time.

For the structural design of the CLT panel, calculate the maximum induced moment. Calculate panel load (per foot of width):

$W_{load} = (q_{dead} + q_{live}) = (30\ psf + 80\ psf)(1ft\ width) = 110$ plf/ft of width

Calculate maximum induced moment (per foot of width):

$M_{max} = w_{load}\ L^2 / 8 = (110)(18^2)/8 = 4{,}455$ ft-lb/ft of width

From PRG 320, select a 5-ply CLT floor panel made from 1³/₈ in x 3¹/₂ inch lumber boards (CLT thickness of 6 ⁷/₈ inches). For CLT grade V2, tabulated properties are:

Bending moment, $F_bS_{eff,0}$ = 4,675 ft-lb/ft of width (PRG 320 Annex A, Table A2)

Calculate the allowable design moment (assuming C_D = 1.0: C_M = 1.0: C_t = 1.0: C_L = 1.0)

$M_s' = F_b(S_{eff})\,(C_D)(C_M)(C_t)(C_L) = 4{,}675\,(1.0)(1.0)(1.0) = 4{,}675$ ft-lb/ft of width (NDS 10.3.1)

Structural Check: $M_s' \geq M_{max}$ **4,675 ft-lb/ft > 4,455 ft-lb/ft** √

(note: serviceability check is not performed to simplify the design example, but should be done in typical structural design).

For the fire design of the CLT panel, the loading is unchanged. Therefore, the maximum induced moment is unchanged. The fire resistance must be calculated.
Determine the char depth, a_{char}.

a_{char} = 1.9 inches (NDS Table 16.2.1B)

In this example, the depth of the char has penetrated through the first face lamination and partially into the second lamination. The contribution of the partially charred cross-ply is neglected and the resisting moment is calculated using the tabulated bending moment for a 3-ply CLT where $F_bS_{eff,0}$, equals 2,030 ft-lb/ft of width.

Calculate the resisting moment (assuming C_D = N/A: C_M = N/A: C_t = N/A: C_L = 1.0)

$M_f' = (2.85)\,F_b(S_{eff})\,(C_L) = 2.85(2{,}030)(1.0) = 5{,}785$ ft-lb/ft of width (NDS 16.2.2)

Fire Check: $M_f' \geq M_{max}$ **5,785 ft-lb/ft > 4,455 ft-lb/ft** √

4.6 Exposed CLT Wall Example (*Allowable Stress Design*)

Cross-laminated timber (CLT) wall with an unbraced height of L=120 inches and loaded in compression in the strong-axis direction. The design loads are w_{live} = 14,000 plf and w_{dead} = 6,150 plf including estimated self-weight of the CLT panel. Walls above are supported on a CLT floor slab and aligned with a CLT wall below. Use of fire-rated caulking of wall joints restricts hot gases from venting through half-lap joints at edges of CLT panel sections. Calculate the required section dimensions for a 2-hr fire resistance time from the CLT.

Calculate column load:

$P_{load} = P_{dead} + P_{snow}$ = 6,150 plf + 14,000 plf = 20,150 lb/foot of width.

From PRG 320, select a 7-ply CLT panel made from $1^3/_8$ in x $3^1/_2$ inch lumber boards (CLT thickness of $9^5/_8$ inches). For CLT grade E1, tabulated properties are:

Compression stress, $F_{c,0}$ = 1800 psi	(PRG 320 Annex A, Table A1)
Bending moment, $F_bS_{eff,0}$ = 18,375 ft-lb/ft of width	(PRG 320 Annex A, Table A2)
Bending stiffness, $EI_{eff,0} = 1{,}089 \times 10^6$ lb-in^2/ft of width	(PRG 320 Annex A, Table A2)
Shear stiffness, $GA_{eff,0} = 1.4 \times 10^6$ lb/ft of width	(PRG 320 Annex A, Table A2)

Calculate the effective wall compression capacity:

$A_{parallel}$ = bd of strong axis plies = 4(12)(1.375) = 66 in^2 (NDS 10.3.1)
$P_c = F_c(A_{parallel})$ = (1800)(66) = 118,800 lb/ft of width (NDS 10.3.1)

Calculate the apparent wall buckling capacity:

Currently, $(EI)_{app\text{-}min}$ properties are not provided by most CLT manufacturers. However, the provisions from the NDS can be used to calculate these properties if certain information is provided. Using NDS Equation 10.4-1, the value for $(EI)_{app}$ can be estimated. Since PRG-320 assumes that E/G = 16 for CLT, NDS Equation 10.4-1 can be rewritten as:

$$(EI)_{app} = \frac{EI_{eff}}{1 + \dfrac{16K_sI_{eff}}{A_{eff}L^2}} = \frac{EI_{eff}}{1 + \dfrac{K_sEI_{eff}}{GA_{eff}L^2}}$$

For columns under constant moment due to buckling stresses, $K_s = 11.8$; therefore:

$$(EI)_{app} = \frac{1{,}089\text{x}10^6}{1 + \dfrac{(11.8)(1{,}089\text{x}10^6)}{(1.4\text{x}10^6)(120)^2}} = 665\text{x}10^6 \text{ psi}$$

To estimate $(EI)_{app\text{-}min}$ the value for $(EI)_{app}$ is adjusted per provisions of NDS Appendix H and the coefficient of variation of 0.10 from PRG-320:

$(EI)_{app\text{-}min}$ = (665x10^6)(1-1.645(0.10))(1.03)/1.66 = 345x10^6 psi (NDS 10.3.1)

Calculate the adjusted allowable column capacity (assuming $C_D = 1.0$: $C_M = 1.0$: $C_t = 1.0$):

$(EI)_{app\text{-}min}' = (EI)_{app\text{-}min}\,(C_M)(C_t)$ = 345x10^6(1.0)(1.0) = 345x10^6 psi (NDS 10.3.1)

Using the general form of the Euler buckling equation:

$$P_{cE} = \frac{\pi^2\,(EI)_{app\text{-}min}}{L^2} = \frac{\pi^2\,(345\text{x}10^6)}{(120)^2} = 236{,}500 \textit{ lb per ft of width}$$

$P_c^* = P_c\,(C_D)(C_M)(C_t)$ = 118,800 (1.0)(1.0)(1.0) = 118,800 lb/ft of width (NDS 3.7.1.5)
Assume c = 0.9 for glued laminated timbers (NDS 3.7.1.5)
$\alpha_c = P_{cE}/P_c^*$ = 236,500/118,800 = 1.991

$$C_p = \frac{1+\alpha_c}{2c} - \sqrt{\left(\frac{1+\alpha_c}{2c}\right)^2 - \frac{\alpha_c}{c}} = \frac{1+1.991}{2(0.9)} - \sqrt{\left(\frac{1+1.991}{2(0.9)}\right)^2 - \frac{1.991}{0.9}} = 0.9208 \qquad \text{(NDS 3.7.1.5)}$$

$P'_s = P_c^*\,C_p$ = 118,800 (0.9208) = 109,400 lb/ft of width (NDS 10.3.1)

Structural Check: $P'_s \geq P_{load}$ 109,400 lb/ft > 20,150 lb/ft √

For the fire design of the CLT wall, the loading is unchanged. Therefore, the total load is unchanged. The fire resistance must be calculated.
Determine the char depth, a_{char}.

a_{char} = 3.8 inches (NDS Table 16.2.1B)

Calculation of the actual $(EI)_{app}$ and $(EI)_{app\text{-}min}$ as a function of fire resistance time is a complex equation that would require several pages of calculations. In this particular example, it can be seen that a char depth of 3.8 inches involves the first 3 laminations which includes the first 2 strong-axis laminations. Rather than attempt to account for the relatively small contribution from the remaining portion of the 2nd strong axis lamination, the wall can be designed as an eccentrically-loaded 3-ply CLT column.

From PRG 320, select a 3-ply CLT panel made from the same 1³/₈ in x 3¹/₂ inch lumber boards (CLT thickness of 4¹/₈ inches) and the same CLT grade E1. The tabulated properties are:

Compression stress, $F_{c,0}$ = 1800 psi	(PRG 320 Annex A, Table A1)
Bending moment, $F_bS_{eff,0}$ = 4,525 ft-lb/ft of width	(PRG 320 Annex A, Table A2)
Bending stiffness, $EI_{eff,0}$ = $115\text{x}10^6$ lb-in²/ft of width	(PRG 320 Annex A, Table A2)
Shear stiffness, $GA_{eff,0}$ = $0.46\text{x}10^6$ lb/ft of width	(PRG 320 Annex A, Table A2)

Calculate the effective wall compression capacity:

$A_{parallel}$ = bd of strong axis plies = 2(12)(1.375) = 33 in² (NDS 10.3.1)
$P_c = F_c(A_{parallel})$ = (1800)(33) = 59,400 lb/ft of width (NDS 10.3.1)

Calculate the apparent wall buckling capacity:

$$(EI)_{app} = \frac{115\text{x}10^6}{1 + \dfrac{(11.8)(115\text{x}10^6)}{(0.46\text{x}10^6)(120)^2}} = 95.4\text{x}10^6 \text{ psi}$$

$(EI)_{app\text{-}min} = (95.4\text{x}10^6)(1\text{-}1.645(0.10))(1.03)/1.66 = 49.5\text{x}10^6$ psi (NDS 10.3.1)

Calculate the adjusted allowable column capacity (assuming C_D = N/A: C_M = N/A: C_t = N/A):

$(EI)_{app\text{-}min}' = 49.5\text{x}10^6$ psi (NDS 10.3.1)

Using the general form of the Euler buckling equation:

$$P_{cE,f} = 2.03\,\frac{\pi^2\,(EI)_{app\text{-}min}}{L^2} = 2.03\,\frac{\pi^2(49.55\text{x}10^6)}{(120)^2} = 68{,}900 \textit{ lb per ft of width}$$

$P^*_{c,f} = 2.58P_c$ = 2.58(59,400) = 153,300 lb/ft of width (NDS 3.7.1.5)
Assume c = 0.9 for glued laminated timbers (NDS 3.7.1.5)

$\alpha_c = P_{cE,f}/P^*_{c,f}$ = 68,900/153,300 = 0.4494

$$C_p = \frac{1+\alpha_c}{2c} - \sqrt{\left(\frac{1+\alpha_c}{2c}\right)^2 - \frac{\alpha_c}{c}} = \frac{1+0.4494}{2(0.9)} - \sqrt{\left(\frac{1+0.4494}{2(0.9)}\right)^2 - \frac{0.4494}{0.9}} = 0.4192 \qquad \text{(NDS 3.7.1.5)}$$

$P'_f = P^*_c\ C_p = 153{,}300\ (0.4192) = 64{,}250$ lb/ft of width (NDS 10.3.1)

Fire Check: $P'_f \geq P_{load}$ 64,250 lb > 20,150 lb √

Initially, the CLT wall is assumed to be loaded concentrically; however, one-sided charring of the wall creates load eccentricities. While the eccentricity created by an effective char depth, $a_{char} = 3.8$ inches would be approximately 1.9 inches, a 3-ply CLT is utilized for this fire design example. The eccentricity in this wall would therefore be:

$e = (d_{7\text{-ply}} - d_{3\text{-ply}})/2 = (9.625 - 4.125)/2 = 2.75$ inches

Calculate the resisting moment (assuming C_D = N/A: C_M = N/A: C_t = N/A: C_L = 1.0)

$M_f' = (2.85)\ F_b(S_{eff})\ (C_L) = 2.85(4{,}525)(1.0) = 12{,}900$ ft-lb/ft of width (NDS 16.2.2)

Using the general form of the wood column equations based on NDS Equation 15.4-3:

$$\left(\frac{P_{Load}}{P'_f}\right)^2 + \frac{(P_{Load}\ e)[1 + 0.234(P_{Load}/P_{cE,f})]}{M'_f[1 - (P_{Load}/P_{cE,f})]} \leq 1.0$$

Fire Check: $$\left(\frac{20{,}150}{64{,}250}\right)^2 + \frac{(20{,}150)(2.75)[1 + 0.234(20{,}150/68{,}900)]}{(12{,}900)(12\text{ in/ft})[1 - (20{,}150/68{,}900)]} = 0.64 \leq 1.0 \quad √$$

There is conservatism in this example due to the simplifying assumption that the remaining cross-section after two hours is a 3-ply CLT wall. The conservatism can be estimated by back-calculating the time required for the first 3 laminations (includes 2 strong-axis and 1 weak-axis laminations) to char. The time required to char each lamination can be estimated using the equations in NDS 16.2-2 as:

$$t_{gl} = \left(\frac{h_{lam}}{\beta_n}\right)^{1.23} = \left(\frac{1.375}{1.5}\right)^{1.23} = 0.90 \text{ hrs}$$

$t = n_{lam}\ t_{gl}/1.2 = 3(0.90)/1.2 = 2.25$ hrs

Note that, while the structural contribution of the fourth lamination (a crossing ply) was ignored in these calculations, it does protect the last 3 laminations in the CLT so the fourth lamination could also be added:

$t = 4(0.90)/1.2 = 3$ hrs

In fact, this CLT wall would be expected to have similar structural fire resistance from 2.25 to 3 hours. A more rigorous analysis would demonstrate that the expected fire resistance of this CLT wall under these loading conditions is about 3 hours.

Part 5: References

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APPENDIX A: Derivation of Load Ratio Tables

For members stressed in one principle direction, simplifications can be made which allow the tabulation of load factor tables. These load factor tables can be used to determine the structural design load ratio, R_s, at which the member has sufficient capacity for a given fire endurance time. This appendix provides the rational used to develop the load ratio tables provided later in this Appendix. For more complex calculations where stress interactions must be considered, the user should consider using the provisions of this technical report with the appropriate NDS provisions.

A.1 Bending Members

Structural:

$$D+L \leq R_s\, F_b\, S_s\, C_{L\text{-}s}\, C_D\, C_M\, C_t \qquad \text{Equation A-1}$$

Fire:

$$D+L \leq 2.85\, F_b\, S_f\, C_{L\text{-}f} \qquad \text{Equation A-2}$$

Where;

D = Design dead load
L = Design live load
R_s = Design load ratio
F_b = Tabulated bending design value
S_s = Section modulus using full (initial) cross-section dimensions
S_f = Section modulus using cross-section dimensions reduced from fire exposure
$C_{L\text{-}s}$ = Beam Stability factor using full cross-section dimensions
$C_{L\text{-}f}$ = Beam Stability factor using cross-section dimensions reduced from fire exposure
C_D = Load Duration factor
C_M = Wet Service factor
C_t = Temperature factor

Solve for R_s:

$$R_s = \frac{2.85\, S_f\, C_{L\text{-}f}}{S_s\, C_{L\text{-}s}\, C_D\, C_M\, C_t} \qquad \text{Equation A-3}$$

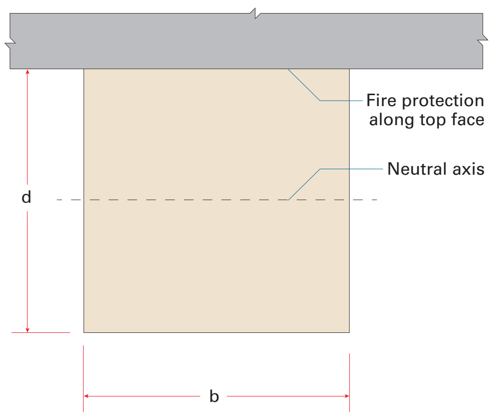

Figure A-1 Flexural member cross-section

For cases in which the compression edge does not have continuous lateral support, the beam stability factor must be calculated separately for both the full (initial) cross-sectional dimensions ($C_{L\text{-}s}$) and for the cross-sectional dimensions reduced from fire exposure ($C_{L\text{-}f}$). The calculation of $C_{L\text{-}s}$ and $C_{L\text{-}f}$ require the designer to consider both the change in bending section relative to bending strength and the change in buckling stiffness relative to buckling strength. While these relationships can be directly calculated using NDS provisions, they cannot be easily tabulated. However, for most beams exposed on three-sides, the beams are braced on the protected side.

Design load ratios, R_s, for fire design of flexural members are given in Table A1(1-hr), Table A1(1.5-hr) and Table A1(2-hr) for 1-hour, 1.5-hour and 2-hour fire-resistance ratings, respectively. These values were developed for standard reference conditions (C_D = 1.0; C_M = 1.0; C_t = 1.0; $C_{L\text{-}f}$ = 1.0), assuming three-sided exposure (protected from fire exposure along the top face), and continuous lateral support along the compression edge of the beam. The dimension "d" is the actual cross-sectional dimension measured in the direction normal to the axis about which bending occurs, and is not necessarily greater than "b" (see *Figure A-1*).

Table A1(1-hr) Design Load Ratios, R_s, for Flexural Members Exposed on Three Sides, 1 – Hour Rating (Structural Calculations at Standard Reference Conditions: C_D = 1.0, C_M = 1.0, C_t = 1.0, C_i = 1.0, C_L = 1.0) (Protected Surface Along Width, b, on Top Edge; With Continuous Lateral Support)

Width, b	5-½	6	6-¾	6-⅞	7-¼	7-½	8-¼	8-½	8-¾	9	9-¼	9-⅝	10-½	10-¾	11	11-¼	12	12-¼	12-⅜	13-¼	13-½	13-¾	15
Depth, d	Design Load Ratio, R_s																						
5-½	0.45	0.52	0.60	0.61	0.65	0.67	0.73	0.74	0.76	0.77	0.79	0.81	0.85	0.86	0.87	0.88	0.90	0.91	0.91	0.94	0.95	0.95	0.98
6	0.48	0.56	0.65	0.67	0.70	0.73	0.79	0.81	0.82	0.84	0.85	0.87	0.92	0.93	0.94	0.95	0.98	0.99	0.99	1.00	1.00	1.00	1.00
6-¾	0.53	0.61	0.72	0.73	0.77	0.80	0.86	0.88	0.90	0.92	0.94	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
6-⅞	0.54	0.62	0.72	0.74	0.78	0.81	0.88	0.90	0.91	0.93	0.95	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7-¼	0.56	0.64	0.75	0.77	0.81	0.84	0.91	0.93	0.95	0.97	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7-½	0.57	0.66	0.77	0.78	0.83	0.86	0.93	0.95	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8-¼	0.60	0.70	0.81	0.83	0.88	0.91	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8-½	0.61	0.71	0.83	0.84	0.89	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8-¾	0.62	0.72	0.84	0.86	0.91	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
9	0.63	0.73	0.85	0.87	0.92	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
9-¼	0.64	0.74	0.86	0.88	0.93	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
9-⅝	0.65	0.75	0.88	0.90	0.95	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
10-½	0.68	0.78	0.91	0.93	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
10-¾	0.68	0.79	0.92	0.94	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
11	0.69	0.80	0.93	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
11-¼	0.69	0.80	0.94	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12	0.71	0.82	0.96	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12-¼	0.72	0.83	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12-⅜	0.72	0.83	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-¼	0.74	0.85	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-½	0.74	0.86	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-¾	0.74	0.86	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15	0.76	0.88	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15-⅛	0.76	0.88	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
16-½	0.78	0.90	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17	0.79	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17-⅞	0.80	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
18	0.80	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19	0.81	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-¼	0.81	0.94	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-½	0.81	0.94	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20-⅝	0.82	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
21	0.82	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22	0.83	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22-½	0.83	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23	0.84	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23-⅜	0.84	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
24	0.84	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
30	0.87	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
36	0.89	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
60	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Design load ratios (R_s) assume bending about the X-X axis, and continuous lateral support along the compression edge.
Design load ratios (R_s) may be interpolated for depths (d) other than those shown.
For the purposes of this table, the dimension d is measured in the direction normal to the axis about which bending occurs. The dimensions d and b are dressed, dry dimensions.
The design moment for fire, $F'_{b,f}S_f$, is approximated by multiplying the adjusted ASD design moment used in structural design, F'_bS, by R_s. ($F'_{b,f}S_f = F'_bS\,R_s$)

Table A1(1.5-hr) Design Load Ratios, R_s, for Flexural Members Exposed on Three Sides, 1.5 – HOUR RATING (Structural Calculations at Standard Reference Conditions: C_D = 1.0, C_M = 1.0, C_t = 1.0, C_i = 1.0, C_L = 1.0) (Protected Surface Along Width, b, on Top Edge; With Continuous Lateral Support)

Width, b	**6-¾**	**6-⅞**	**7-¼**	**7-½**	**8-¼**	**8-½**	**8-¾**	**9**	**9-¼**	**9-⅞**	**10-½**	**10-¾**	**11**	**11-¼**	**12**	**12-⅜**	**13-¼**	**13-¾**	**15**	**16-½**	**18**	**24**	**36**
Depth, d											**Design Load Ratio, R_s**												
5-½	0.22	0.23	0.26	0.28	0.33	0.35	0.36	0.38	0.39	0.41	0.44	0.45	0.46	0.47	0.49	0.50	0.53	0.54	0.56	0.59	0.61	0.67	0.73
6	0.25	0.26	0.30	0.32	0.38	0.40	0.41	0.43	0.44	0.46	0.51	0.52	0.53	0.54	0.56	0.58	0.60	0.62	0.65	0.67	0.70	0.77	0.83
6-¾	0.29	0.31	0.35	0.38	0.44	0.46	0.48	0.50	0.52	0.54	0.59	0.60	0.61	0.63	0.66	0.67	0.70	0.72	0.75	0.79	0.81	0.89	0.97
6-⅞	0.30	0.31	0.36	0.38	0.45	0.47	0.49	0.51	0.53	0.55	0.60	0.62	0.63	0.64	0.67	0.69	0.72	0.73	0.77	0.80	0.83	0.91	0.99
7-¼	0.32	0.33	0.38	0.41	0.48	0.50	0.52	0.54	0.56	0.59	0.64	0.65	0.67	0.68	0.71	0.73	0.76	0.78	0.81	0.85	0.88	0.97	1.00
7-½	0.33	0.34	0.39	0.42	0.50	0.52	0.54	0.56	0.58	0.61	0.66	0.68	0.69	0.70	0.74	0.75	0.79	0.80	0.84	0.88	0.91	1.00	1.00
8-¼	0.36	0.38	0.43	0.46	0.54	0.57	0.59	0.61	0.63	0.66	0.72	0.74	0.75	0.77	0.81	0.82	0.86	0.88	0.92	0.96	1.00	1.00	1.00
8-½	0.37	0.39	0.44	0.47	0.56	0.58	0.61	0.63	0.65	0.68	0.74	0.76	0.77	0.79	0.83	0.84	0.88	0.90	0.95	0.99	1.00	1.00	1.00
8-¾	0.38	0.40	0.45	0.48	0.57	0.60	0.62	0.64	0.67	0.70	0.76	0.78	0.79	0.81	0.85	0.87	0.90	0.92	0.97	1.00	1.00	1.00	1.00
9	0.38	0.40	0.46	0.49	0.58	0.61	0.64	0.66	0.68	0.71	0.78	0.79	0.81	0.82	0.87	0.88	0.92	0.94	0.99	1.00	1.00	1.00	1.00
9-¼	0.39	0.41	0.47	0.50	0.60	0.62	0.65	0.67	0.70	0.73	0.79	0.81	0.83	0.84	0.88	0.90	0.94	0.96	1.00	1.00	1.00	1.00	1.00
9-⅝	0.40	0.42	0.48	0.52	0.61	0.64	0.67	0.69	0.72	0.75	0.82	0.83	0.85	0.87	0.91	0.93	0.97	0.99	1.00	1.00	1.00	1.00	1.00
10-½	0.43	0.45	0.51	0.55	0.65	0.68	0.71	0.73	0.76	0.79	0.87	0.88	0.90	0.92	0.96	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00
10-¾	0.43	0.46	0.52	0.56	0.66	0.69	0.72	0.74	0.77	0.81	0.88	0.90	0.91	0.93	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
11	0.44	0.46	0.53	0.57	0.67	0.70	0.73	0.75	0.78	0.82	0.89	0.91	0.93	0.94	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
11-¼	0.45	0.47	0.53	0.57	0.68	0.71	0.74	0.76	0.79	0.83	0.90	0.92	0.94	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12	0.46	0.49	0.55	0.59	0.70	0.73	0.76	0.79	0.82	0.86	0.93	0.95	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12-¼	0.47	0.49	0.56	0.60	0.71	0.74	0.77	0.80	0.83	0.87	0.94	0.96	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
12-⅜	0.47	0.49	0.56	0.60	0.71	0.75	0.78	0.80	0.83	0.87	0.95	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-¼	0.48	0.51	0.58	0.62	0.74	0.77	0.80	0.83	0.86	0.90	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-½	0.49	0.51	0.59	0.63	0.74	0.78	0.81	0.84	0.87	0.91	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-¾	0.49	0.52	0.59	0.63	0.75	0.78	0.82	0.85	0.87	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15	0.51	0.54	0.61	0.66	0.78	0.81	0.85	0.88	0.91	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15-⅛	0.51	0.54	0.61	0.66	0.78	0.82	0.85	0.88	0.91	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
16-½	0.53	0.56	0.63	0.68	0.81	0.84	0.88	0.91	0.94	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17	0.54	0.56	0.64	0.69	0.82	0.85	0.89	0.92	0.95	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17-⅞	0.54	0.57	0.65	0.70	0.83	0.87	0.90	0.94	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
18	0.55	0.57	0.65	0.70	0.83	0.87	0.90	0.94	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19	0.56	0.58	0.67	0.71	0.84	0.88	0.92	0.95	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-¼	0.56	0.59	0.67	0.72	0.85	0.89	0.92	0.96	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-½	0.56	0.59	0.67	0.72	0.85	0.89	0.93	0.96	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20-⅝	0.57	0.60	0.68	0.73	0.87	0.90	0.94	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
21	0.57	0.60	0.68	0.74	0.87	0.91	0.95	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22	0.58	0.61	0.69	0.74	0.88	0.92	0.96	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22-½	0.58	0.61	0.70	0.75	0.89	0.93	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23	0.58	0.62	0.70	0.75	0.89	0.93	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23-⅜	0.59	0.62	0.70	0.76	0.89	0.93	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
24	0.59	0.62	0.71	0.76	0.90	0.94	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
30	0.62	0.65	0.74	0.80	0.94	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
36	0.64	0.67	0.76	0.82	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
60	0.68	0.71	0.81	0.87	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Design load ratios (R_S) assume bending about the X-X axis, and continuous lateral support along the compression edge.
Design load ratios (R_S) may be interpolated for depths (d) other than those shown.
For the purposes of this table, the dimension d is measured in the direction normal to the axis about which bending occurs. The dimensions d and b are dressed, dry dimensions.
The design moment for fire, $F'_{b,f} S_f$, is approximated by multiplying the adjusted ASD design moment used in structural design, $F'_b S$, by R_S. ($F'_{b,f} S_f = F'_b S R_S$)

Table A1(2-hr) Design Load Ratios, R_s, for Flexural Members Exposed on Three Sides, 2 – HOUR RATING (Structural Calculations at Standard Reference Conditions: C_D = 1.0, C_M = 1.0, C_t = 1.0, C_i = 1.0, C_L = 1.0) (Protected Surface Along Width, b, on Top Edge; With Continuous Lateral Support)

Width, b	8-½	8-¾	9	9-¼	9-⅝	10-½	10-¾	11	11-¼	12	12-⅜	13-¼	13-½	13-¾	15	16-½	17	18	21	24	36	48	60
Depth, d	Design Load Ratio, R_s																						
5-½	0.13	0.14	0.15	0.16	0.18	0.20	0.21	0.22	0.23	0.24	0.25	0.27	0.27	0.28	0.30	0.32	0.32	0.33	0.36	0.38	0.42	0.45	0.46
6	0.16	0.18	0.19	0.20	0.22	0.25	0.26	0.27	0.28	0.30	0.31	0.33	0.34	0.34	0.37	0.39	0.40	0.41	0.45	0.47	0.53	0.55	0.57
6-¾	0.21	0.22	0.24	0.25	0.28	0.32	0.33	0.34	0.35	0.38	0.39	0.42	0.43	0.43	0.47	0.50	0.51	0.52	0.56	0.59	0.66	0.70	0.72
6-⅞	0.21	0.23	0.25	0.26	0.28	0.33	0.34	0.35	0.36	0.39	0.41	0.43	0.44	0.45	0.48	0.51	0.52	0.54	0.58	0.61	0.69	0.72	0.74
7-¼	0.23	0.25	0.27	0.29	0.31	0.36	0.37	0.39	0.40	0.43	0.44	0.47	0.48	0.49	0.52	0.56	0.57	0.59	0.63	0.67	0.75	0.79	0.81
7-½	0.24	0.26	0.28	0.30	0.33	0.38	0.39	0.41	0.42	0.45	0.47	0.50	0.51	0.51	0.55	0.59	0.60	0.62	0.67	0.70	0.79	0.83	0.85
8-¼	0.28	0.30	0.32	0.34	0.37	0.43	0.45	0.46	0.47	0.51	0.53	0.57	0.58	0.59	0.63	0.67	0.68	0.70	0.76	0.80	0.89	0.94	0.97
8-½	0.29	0.31	0.33	0.36	0.39	0.45	0.46	0.48	0.49	0.53	0.55	0.59	0.60	0.61	0.65	0.69	0.71	0.73	0.79	0.83	0.93	0.98	1.00
8-¾	0.30	0.32	0.35	0.37	0.40	0.46	0.48	0.49	0.51	0.55	0.57	0.61	0.62	0.63	0.67	0.72	0.73	0.75	0.81	0.86	0.96	1.00	1.00
9	0.31	0.33	0.36	0.38	0.41	0.48	0.49	0.51	0.52	0.57	0.59	0.63	0.64	0.65	0.69	0.74	0.75	0.78	0.84	0.88	0.99	1.00	1.00
9-¼	0.32	0.34	0.37	0.39	0.42	0.49	0.51	0.52	0.54	0.58	0.60	0.65	0.66	0.67	0.71	0.76	0.78	0.80	0.86	0.91	1.00	1.00	1.00
9-⅝	0.33	0.36	0.38	0.41	0.44	0.51	0.53	0.55	0.56	0.61	0.63	0.67	0.68	0.69	0.74	0.79	0.81	0.83	0.90	0.95	1.00	1.00	1.00
10-½	0.36	0.39	0.41	0.44	0.48	0.55	0.57	0.59	0.61	0.66	0.68	0.73	0.74	0.75	0.80	0.86	0.87	0.90	0.97	1.00	1.00	1.00	1.00
10-¾	0.36	0.39	0.42	0.45	0.49	0.56	0.58	0.60	0.62	0.67	0.69	0.74	0.75	0.77	0.82	0.88	0.89	0.92	0.99	1.00	1.00	1.00	1.00
11	0.37	0.40	0.43	0.46	0.50	0.58	0.60	0.61	0.63	0.68	0.71	0.76	0.77	0.78	0.84	0.89	0.91	0.94	1.00	1.00	1.00	1.00	1.00
11-¼	0.38	0.41	0.44	0.47	0.51	0.59	0.61	0.63	0.64	0.70	0.72	0.77	0.78	0.80	0.85	0.91	0.92	0.96	1.00	1.00	1.00	1.00	1.00
12	0.40	0.43	0.46	0.49	0.53	0.61	0.64	0.66	0.68	0.73	0.76	0.81	0.82	0.83	0.89	0.95	0.97	1.00	1.00	1.00	1.00	1.00	1.00
12-¼	0.40	0.43	0.47	0.50	0.54	0.62	0.65	0.67	0.69	0.74	0.77	0.82	0.83	0.85	0.91	0.97	0.98	1.00	1.00	1.00	1.00	1.00	1.00
12-⅜	0.40	0.44	0.47	0.50	0.54	0.63	0.65	0.67	0.69	0.75	0.77	0.83	0.84	0.85	0.91	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00
13-¼	0.42	0.46	0.49	0.52	0.57	0.66	0.68	0.70	0.72	0.78	0.81	0.86	0.88	0.89	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-½	0.43	0.46	0.50	0.53	0.57	0.66	0.69	0.71	0.73	0.79	0.82	0.87	0.89	0.90	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
13-¾	0.43	0.47	0.50	0.53	0.58	0.67	0.70	0.72	0.74	0.80	0.83	0.88	0.90	0.91	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15	0.45	0.49	0.53	0.56	0.61	0.71	0.73	0.75	0.78	0.84	0.87	0.93	0.94	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
15-⅛	0.46	0.49	0.53	0.56	0.61	0.71	0.73	0.76	0.78	0.84	0.87	0.93	0.95	0.96	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
16-½	0.48	0.52	0.55	0.59	0.64	0.74	0.77	0.79	0.82	0.88	0.91	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17	0.48	0.52	0.56	0.60	0.65	0.75	0.78	0.80	0.83	0.89	0.92	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
17-⅞	0.49	0.54	0.57	0.61	0.66	0.77	0.79	0.82	0.85	0.91	0.94	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
18	0.50	0.54	0.58	0.61	0.66	0.77	0.80	0.82	0.85	0.92	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19	0.51	0.55	0.59	0.63	0.68	0.79	0.82	0.84	0.87	0.94	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-¼	0.51	0.55	0.59	0.63	0.68	0.79	0.82	0.85	0.87	0.94	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
19-½	0.51	0.55	0.59	0.63	0.69	0.80	0.82	0.85	0.88	0.95	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20-⅝	0.52	0.57	0.61	0.65	0.70	0.81	0.84	0.87	0.89	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
21	0.53	0.57	0.61	0.65	0.71	0.82	0.85	0.87	0.90	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22	0.53	0.58	0.62	0.66	0.72	0.83	0.86	0.89	0.91	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
22-½	0.54	0.58	0.63	0.67	0.72	0.84	0.87	0.89	0.92	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23	0.54	0.59	0.63	0.67	0.73	0.84	0.87	0.90	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
23-⅜	0.55	0.59	0.63	0.67	0.73	0.85	0.88	0.91	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
24	0.55	0.60	0.64	0.68	0.74	0.85	0.88	0.91	0.94	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
30	0.58	0.63	0.68	0.72	0.78	0.91	0.94	0.97	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
36	0.61	0.66	0.70	0.75	0.81	0.94	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
60	0.65	0.71	0.76	0.81	0.88	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Design load ratios (R_S) assume bending about the X-X axis, and continuous lateral support along the compression edge.
Design load ratios (R_S) may be interpolated for depths (d) other than those shown.
For the purposes of this table, the dimension d is measured in the direction normal to the axis about which bending occurs. The dimensions d and b are dressed, dry dimensions.
The design moment for fire, $F'_{b,f}S_f$, is approximated by multiplying the adjusted ASD design moment used in structural design, F'_bS, by R_S. ($F'_{b,f}S_f = F'_bS\ R_S$)

A.2 Compression Members

Structural:

$$D+L \leq R_s\, F_c\, A_s\, C_{p\text{-}s}\, C_D\, C_M\, C_t \qquad \text{Equation A-4}$$

Fire:

$$D+L \leq 2.58\, F_c\, A_f\, C_{p\text{-}f} \qquad \text{Equation A-5}$$

Where;

- D = Design dead load
- L = Design live load
- R_s = Design load ratio
- F_c = Tabulated compression parallel-to-grain design value
- $C_{p\text{-}s}$ = Column stability factor using full (initial) cross-section dimensions
- $C_{p\text{-}f}$ = Column stability factor using cross-section dimensions reduced from fire exposure, with a column buckling strength, FcE, multiplied by a factor of 2.03 per NDS Section 16.2.2.
- A_s = Area of full (initial) cross-section dimensions
- A_f = Area of cross-section reduced from fire exposure
- C_D = Load Duration factor
- C_M = Wet Service factor
- C_t = Temperature factor

Solve for Rs:

$$R_S = \frac{2.58\, A_f\, C_{p\text{-}f}}{A_s\, C_{p\text{-}s}\, C_D\, C_M\, C_t} \qquad \text{Equation A-6}$$

While these relationships can be directly calculated using the NDS, they cannot be easily tabulated directly for columns with non-square cross sections. However, to address columns with non-square cross sections of dimensions d x b, where d is measured in the direction normal to the axis about which buckling is considered, design load ratios may be tabulated for columns having square cross sections of dimensions d x d, along with a multiplier to adjust for dimension b. The R_s ratio for the column under evaluation is calculated as the product of these two values, which are designated as R_{s1} and R_{s2}, respectively. Thus, for standard reference conditions (where $C_D = 1.0$, $C_M = 1.0$ and $C_t = 1.0$) and four-sided fire exposure, R_{s1} and R_{s2} are calculated as follows:

$$R_{s1} = \frac{2.58 C_{p\text{-}f}(d-2a)^2}{C_{p\text{-}s}\,(d)^2} \qquad \text{Equation A-7}$$

$$R_{s2} = \frac{(1-2^{a}/_{b})}{(1-2^{a}/_{d})} \qquad \text{Equation A-8}$$

$$R_s = R_{s1}R_{s2} \leq 1.0 \qquad \text{Equation A-9}$$

Where;

- d = Cross-sectional dimension measured in the direction normal to the axis about which buckling is considered (see *Figure A-2*)
- b = Cross-sectional dimension measured in the direction parallel to the axis about which buckling is considered (see *Figure A-2*)
- a = Effective char depth

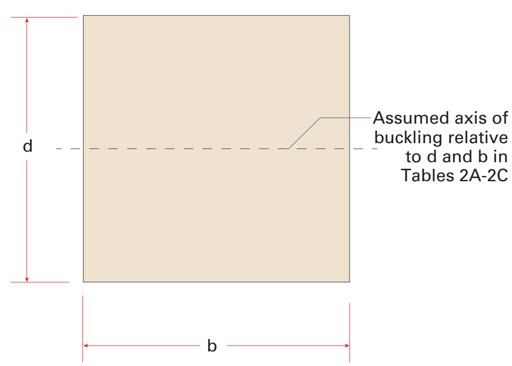

Figure A-2 Compression member cross-section

The R_{s1} and R_{s2} values given in *Table A2(1-hr)*, *Table A2(1.5-hr)* and *Table A2(2-hr)* were developed for columns under standard reference conditions ($C_D = 1.0$; $C_M = 1.0$; $C_t = 1.0$), assuming four-sided exposure for 1-hour, 1.5-hour and 2-hour ratings, respectively. They may also be conservatively applied to three-sided fire exposures. It should be noted that design load ratios calculated using the tabulated R_{s1} and R_{s2} values will usually yield a slightly conservative value. This is because they are derived based on the most conservative result using c = 0.8 and c = 0.9, which allows them to be used for sawn lumber, structural glued laminated timber, or structural composite lumber. Additionally, the R_{s1} values are also based on the assumption that $E'_{min}/F^*_c = 350$. Because of this, the design load ratios, R_s, may conservatively be used for all species and grades where the ratio of E'_{min} to F^*_c is greater than or equal to 350.

Table A2(1-hr) Design Load Ratios, $R_s = (R_{s1})(R_{s2}) \leq 1.0$, for Compression Members Exposed on Four Sides, 1 – HOUR RATING
(Structural Calculations at Standard Reference Conditions: $C_D = 1.0$, $C_M = 1.0$, $C_t = 1.0$, $C_i = 1.0$)

	Depth of Member, d (measured in the direction normal to the axis about which buckling is considered)																					
	5-½	**6**	**6-¾**	**6-⅞**	**7-¾**	**7-½**	**8-¼**	**8-½**	**8-¾**	**9**	**9-¼**	**9-⅝**	**10-½**	**10-¾**	**11**	**11-¼**	**12**	**15**	**18**	**21**	**24**	**36**
L_e/d	**R_{s1} - Design Stress Adjustment for L_e/d**																					
0	0.31	0.41	0.56	0.59	0.65	0.70	0.82	0.86	0.89	0.93	0.96	1.01	1.11	1.14	1.17	1.19	1.26	1.49	1.65	1.77	1.86	2.09
2	0.30	0.40	0.55	0.58	0.65	0.69	0.81	0.85	0.89	0.92	0.96	1.00	1.11	1.14	1.16	1.19	1.26	1.49	1.65	1.77	1.86	2.09
4	0.26	0.37	0.52	0.55	0.62	0.66	0.79	0.83	0.86	0.90	0.93	0.98	1.09	1.11	1.14	1.17	1.24	1.47	1.63	1.75	1.85	2.08
6	0.19	0.29	0.45	0.48	0.55	0.60	0.73	0.77	0.81	0.85	0.88	0.93	1.04	1.07	1.10	1.12	1.20	1.43	1.60	1.72	1.82	2.05
8	0.12	0.21	0.35	0.38	0.45	0.50	0.63	0.67	0.71	0.75	0.79	0.84	0.96	0.99	1.01	1.04	1.12	1.37	1.54	1.67	1.77	2.01
10	0.084	0.15	0.26	0.28	0.35	0.39	0.51	0.55	0.59	0.63	0.66	0.72	0.83	0.86	0.89	0.92	1.00	1.26	1.45	1.58	1.69	1.95
12	0.061	0.11	0.20	0.22	0.27	0.30	0.41	0.44	0.48	0.51	0.55	0.59	0.70	0.73	0.76	0.79	0.87	1.13	1.32	1.46	1.58	1.85
14	0.048	0.086	0.16	0.17	0.21	0.24	0.33	0.36	0.39	0.42	0.45	0.49	0.59	0.61	0.64	0.67	0.74	1.00	1.19	1.33	1.45	1.73
16	0.040	0.072	0.13	0.14	0.18	0.20	0.28	0.30	0.33	0.35	0.38	0.42	0.50	0.53	0.55	0.57	0.64	0.87	1.06	1.20	1.32	1.61
18	0.036	0.064	0.12	0.13	0.16	0.18	0.25	0.27	0.29	0.32	0.34	0.38	0.45	0.48	0.50	0.52	0.58	0.80	0.97	1.11	1.22	1.50
20	0.033	0.060	0.11	0.12	0.15	0.17	0.23	0.25	0.28	0.30	0.32	0.35	0.43	0.45	0.47	0.49	0.55	0.75	0.92	1.05	1.16	1.44
22	0.032	0.057	0.11	0.12	0.14	0.16	0.22	0.25	0.27	0.29	0.31	0.34	0.41	0.43	0.45	0.47	0.53	0.73	0.89	1.02	1.13	1.41
24	0.031	0.056	0.10	0.11	0.14	0.16	0.22	0.24	0.26	0.28	0.30	0.33	0.40	0.42	0.44	0.46	0.52	0.71	0.87	1.00	1.11	1.38
26	0.031	0.055	0.10	0.11	0.14	0.16	0.22	0.24	0.26	0.28	0.30	0.33	0.40	0.42	0.44	0.45	0.51	0.71	0.86	0.99	1.10	1.37
28	0.030	0.054	0.10	0.11	0.14	0.16	0.21	0.23	0.25	0.27	0.29	0.32	0.39	0.41	0.43	0.45	0.51	0.70	0.86	0.98	1.09	1.36
30	0.030	0.054	0.10	0.11	0.14	0.15	0.21	0.23	0.25	0.27	0.29	0.32	0.39	0.41	0.43	0.45	0.50	0.70	0.85	0.98	1.08	1.36
40	0.029	0.053	0.098	0.11	0.13	0.15	0.21	0.23	0.25	0.27	0.29	0.32	0.38	0.40	0.42	0.44	0.49	0.69	0.84	0.97	1.07	1.34
50	0.029	0.053	0.097	0.11	0.13	0.15	0.21	0.23	0.25	0.27	0.29	0.31	0.38	0.40	0.42	0.44	0.49	0.68	0.84	0.96	1.07	1.34
b	**R_{s2} - Design Stress Adjustment for Width, b, of Column**																					
5-½	1.00	0.86	0.74	0.73	0.69	0.66	0.61	0.60	0.59	0.58	0.57	0.55	0.53	0.52	0.51	0.51	0.49	0.45	0.43	0.42	0.41	0.38
6	1.16	1.00	0.86	0.84	0.79	0.77	0.71	0.69	0.68	0.67	0.65	0.64	0.61	0.60	0.59	0.59	0.57	0.53	0.50	0.48	0.47	0.44
6-¾	1.35	1.17	1.00	0.98	0.93	0.90	0.83	0.81	0.79	0.78	0.76	0.75	0.71	0.70	0.69	0.69	0.67	0.61	0.58	0.56	0.55	0.52
6-⅞	1.38	1.19	1.02	1.00	0.95	0.92	0.85	0.83	0.81	0.79	0.78	0.76	0.72	0.72	0.71	0.70	0.68	0.63	0.60	0.57	0.56	0.53
7-¼	1.46	1.26	1.08	1.06	1.00	0.97	0.89	0.87	0.86	0.84	0.82	0.80	0.77	0.76	0.75	0.74	0.72	0.66	0.63	0.61	0.59	0.56
7-½	1.51	1.30	1.11	1.09	1.03	1.00	0.92	0.90	0.88	0.87	0.85	0.83	0.79	0.78	0.77	0.76	0.74	0.68	0.65	0.63	0.61	0.58
8-¼	1.63	1.41	1.21	1.18	1.12	1.08	1.00	0.98	0.96	0.94	0.92	0.90	0.86	0.85	0.84	0.83	0.81	0.74	0.70	0.68	0.66	0.63
8-½	1.67	1.44	1.24	1.21	1.15	1.11	1.02	1.00	0.98	0.96	0.94	0.92	0.88	0.87	0.86	0.85	0.82	0.76	0.72	0.70	0.68	0.64
8-¾	1.70	1.47	1.26	1.24	1.17	1.13	1.04	1.02	1.00	0.98	0.96	0.94	0.90	0.88	0.87	0.87	0.84	0.77	0.74	0.71	0.69	0.65
9	1.74	1.50	1.29	1.26	1.19	1.15	1.06	1.04	1.02	1.00	0.98	0.96	0.91	0.90	0.89	0.88	0.86	0.79	0.75	0.72	0.71	0.67
9-¼	1.77	1.53	1.31	1.28	1.21	1.17	1.08	1.06	1.04	1.02	1.00	0.98	0.93	0.92	0.91	0.90	0.87	0.80	0.76	0.74	0.72	0.68
9-⅝	1.81	1.56	1.34	1.31	1.24	1.20	1.11	1.09	1.06	1.04	1.02	1.00	0.95	0.94	0.93	0.92	0.89	0.82	0.78	0.76	0.74	0.70
10-½	1.90	1.64	1.41	1.38	1.31	1.26	1.17	1.14	1.12	1.10	1.08	1.05	1.00	0.99	0.98	0.97	0.94	0.86	0.82	0.79	0.77	0.73
10-¾	1.93	1.66	1.43	1.40	1.32	1.28	1.18	1.15	1.13	1.11	1.09	1.06	1.01	1.00	0.99	0.98	0.95	0.88	0.83	0.80	0.78	0.74
11	1.95	1.68	1.44	1.41	1.34	1.29	1.19	1.17	1.14	1.12	1.10	1.07	1.02	1.01	1.00	0.99	0.96	0.89	0.84	0.81	0.79	0.75
11-¼	1.97	1.70	1.46	1.43	1.35	1.31	1.21	1.18	1.16	1.13	1.11	1.09	1.03	1.02	1.01	1.00	0.97	0.89	0.85	0.82	0.80	0.76
12	2.03	1.75	1.50	1.47	1.39	1.35	1.24	1.21	1.19	1.17	1.15	1.12	1.07	1.05	1.04	1.03	1.00	0.92	0.88	0.84	0.82	0.78
15	2.20	1.90	1.63	1.60	1.51	1.46	1.35	1.32	1.29	1.27	1.24	1.21	1.16	1.14	1.13	1.12	1.09	1.00	0.95	0.92	0.89	0.84
18	2.32	2.00	1.71	1.68	1.59	1.54	1.42	1.39	1.36	1.33	1.31	1.28	1.22	1.20	1.19	1.18	1.14	1.05	1.00	0.97	0.94	0.89
21	2.40	2.07	1.78	1.74	1.65	1.59	1.47	1.44	1.41	1.38	1.36	1.32	1.26	1.25	1.23	1.22	1.18	1.09	1.04	1.00	0.97	0.92
24	2.46	2.13	1.82	1.78	1.69	1.63	1.51	1.47	1.44	1.42	1.39	1.36	1.29	1.28	1.26	1.25	1.21	1.12	1.06	1.03	1.00	0.94
36	2.61	2.25	1.93	1.89	1.79	1.73	1.60	1.56	1.53	1.50	1.47	1.44	1.37	1.35	1.34	1.32	1.29	1.18	1.13	1.09	1.06	1.00

Design load ratios are calculated as the product of R_{S1} and R_{S2}, but should not be taken as greater than 1.0. ($R_S = [R_{S1}][R_{S2}] \leq 1.0$)

For the purposes of this table, the dimension d is measured in the direction normal to the axis about which buckling is considered. The designer should consider buckling about both axes and use the lesser design value. The dimensions d and b are dressed, dry dimensions.

Tabulated values may be used for sawn lumber, structural glued laminated timber, or structural composite lumber where $E'_{min}/F'_c \geq 350$.

Values of R_{S1} and R_{S2} may be interpolated for values of d, L_e/d, and b other than those shown.

Table A2(1.5-hr) Design Load Ratios, $R_s = (R_{s1})(R_{s2}) \le 1.0$, for Compression Members Exposed on Four Sides, 1.5 – HOUR RATING (Structural Calculations at Standard Reference Conditions: C_D = 1.0, C_M = 1.0, C_t = 1.0, C_i = 1.0)

	Depth of Member, d (measured in the direction normal to the axis about which buckling is considered)																					
	6-¾	**6-⅞**	**7-¼**	**7-½**	**8-¼**	**8-½**	**8-¾**	**9**	**9-¼**	**9-⅝**	**10-½**	**10-¾**	**11**	**11-¼**	**12**	**12-¼**	**15**	**16-½**	**18**	**21**	**24**	**36**
L_e/d	**R_{s1} - Design Stress Adjustment for L_e/d**																					
0	0.17	0.19	0.25	0.29	0.40	0.44	0.47	0.51	0.54	0.59	0.71	0.74	0.77	0.79	0.88	0.90	1.15	1.25	1.34	1.50	1.62	1.91
2	0.16	0.18	0.24	0.28	0.39	0.43	0.46	0.50	0.54	0.59	0.70	0.73	0.76	0.79	0.87	0.90	1.14	1.25	1.34	1.49	1.61	1.91
4	0.12	0.14	0.20	0.24	0.36	0.39	0.43	0.47	0.50	0.56	0.67	0.70	0.73	0.76	0.84	0.87	1.12	1.23	1.32	1.47	1.60	1.90
6	0.066	0.080	0.13	0.16	0.28	0.32	0.36	0.40	0.43	0.49	0.61	0.64	0.67	0.70	0.79	0.82	1.07	1.18	1.28	1.44	1.56	1.87
8	0.041	0.049	0.081	0.11	0.20	0.23	0.26	0.30	0.33	0.39	0.51	0.54	0.57	0.60	0.69	0.72	0.99	1.11	1.21	1.37	1.50	1.82
10	0.027	0.033	0.055	0.073	0.14	0.16	0.19	0.22	0.25	0.29	0.40	0.43	0.45	0.48	0.57	0.60	0.87	0.99	1.09	1.27	1.41	1.75
12	0.019	0.024	0.040	0.053	0.10	0.12	0.14	0.16	0.19	0.22	0.31	0.33	0.36	0.39	0.46	0.49	0.74	0.85	0.96	1.14	1.28	1.63
14	0.015	0.018	0.031	0.041	0.080	0.095	0.11	0.13	0.15	0.18	0.25	0.27	0.29	0.31	0.37	0.39	0.62	0.73	0.83	1.01	1.15	1.51
16	0.013	0.015	0.026	0.034	0.067	0.080	0.094	0.11	0.12	0.15	0.21	0.22	0.24	0.26	0.32	0.33	0.53	0.63	0.72	0.88	1.01	1.38
18	0.011	0.014	0.023	0.031	0.060	0.071	0.084	0.097	0.11	0.13	0.19	0.20	0.22	0.23	0.28	0.30	0.48	0.57	0.65	0.80	0.93	1.28
20	0.010	0.013	0.022	0.029	0.056	0.067	0.078	0.090	0.10	0.12	0.17	0.19	0.20	0.22	0.27	0.28	0.45	0.54	0.62	0.76	0.88	1.22
22	0.010	0.012	0.021	0.028	0.054	0.064	0.075	0.087	0.099	0.12	0.17	0.18	0.20	0.21	0.26	0.27	0.43	0.52	0.60	0.74	0.85	1.18
24	0.010	0.012	0.020	0.027	0.052	0.062	0.073	0.085	0.097	0.12	0.16	0.18	0.19	0.21	0.25	0.27	0.43	0.51	0.58	0.72	0.84	1.17
26	0.010	0.012	0.020	0.026	0.051	0.061	0.072	0.083	0.095	0.11	0.16	0.17	0.19	0.20	0.25	0.26	0.42	0.50	0.58	0.71	0.83	1.15
28	0.010	0.012	0.020	0.026	0.051	0.061	0.071	0.082	0.094	0.11	0.16	0.17	0.19	0.20	0.24	0.26	0.42	0.50	0.57	0.71	0.82	1.14
30		0.012	0.019	0.026	0.050	0.060	0.071	0.082	0.093	0.11	0.16	0.17	0.19	0.20	0.24	0.26	0.41	0.49	0.57	0.70	0.82	1.14
40		0.011	0.019	0.025	0.049	0.059	0.069	0.080	0.092	0.11	0.15	0.17	0.18	0.20	0.24	0.25	0.41	0.48	0.56	0.69	0.81	1.13
50		0.011	0.019	0.025	0.049	0.059	0.069	0.080	0.091	0.11	0.15	0.17	0.18	0.19	0.24	0.25	0.40	0.48	0.56	0.69	0.80	1.12
b	**R_{s2} - Design Stress Adjustment for Width, b, of Column**																					
6-¾	1.00	0.95	0.83	0.78	0.66	0.63	0.60	0.58	0.56	0.54	0.49	0.48	0.47	0.47	0.44	0.44	0.39	0.37	0.36	0.34	0.33	0.10
6-⅞	1.05	1.00	0.88	0.82	0.69	0.66	0.64	0.61	0.59	0.57	0.52	0.51	0.50	0.49	0.47	0.46	0.41	0.39	0.38	0.36	0.34	0.19
7-¼	1.20	1.14	1.00	0.93	0.79	0.75	0.72	0.70	0.67	0.64	0.59	0.58	0.57	0.56	0.53	0.52	0.46	0.44	0.43	0.41	0.39	0.30
7-½	1.29	1.22	1.07	1.00	0.85	0.81	0.78	0.75	0.72	0.69	0.64	0.62	0.61	0.60	0.57	0.56	0.50	0.48	0.46	0.44	0.42	0.32
8-¼	1.52	1.45	1.27	1.18	1.00	0.96	0.92	0.89	0.86	0.82	0.75	0.74	0.72	0.71	0.67	0.66	0.59	0.56	0.54	0.52	0.50	0.36
8-½	1.59	1.51	1.33	1.24	1.05	1.00	0.96	0.93	0.90	0.86	0.79	0.77	0.75	0.74	0.71	0.70	0.62	0.59	0.57	0.54	0.52	0.39
8-¾	1.66	1.57	1.38	1.29	1.09	1.04	1.00	0.96	0.93	0.89	0.82	0.80	0.79	0.77	0.73	0.72	0.64	0.61	0.59	0.56	0.54	0.46
9	1.72	1.63	1.43	1.33	1.13	1.08	1.04	1.00	0.97	0.92	0.85	0.83	0.81	0.80	0.76	0.75	0.67	0.64	0.61	0.58	0.56	0.48
9-¼	1.78	1.69	1.48	1.38	1.17	1.12	1.07	1.03	1.00	0.96	0.88	0.86	0.84	0.83	0.79	0.78	0.69	0.66	0.64	0.60	0.58	0.50
9-⅝	1.86	1.77	1.55	1.44	1.22	1.17	1.12	1.08	1.05	1.00	0.92	0.90	0.88	0.86	0.82	0.81	0.72	0.69	0.66	0.63	0.61	0.52
10-½	2.03	1.92	1.69	1.57	1.33	1.27	1.22	1.18	1.14	1.09	1.00	0.98	0.96	0.94	0.90	0.88	0.79	0.75	0.72	0.69	0.66	0.53
10-¾	2.07	1.97	1.73	1.61	1.36	1.30	1.25	1.20	1.16	1.11	1.02	1.00	0.98	0.96	0.92	0.90	0.80	0.77	0.74	0.70	0.68	0.56
11	2.11	2.00	1.76	1.64	1.39	1.33	1.27	1.23	1.19	1.14	1.04	1.02	1.00	0.98	0.93	0.92	0.82	0.78	0.75	0.72	0.69	0.61
11-¼	2.15	2.04	1.79	1.67	1.41	1.35	1.30	1.25	1.21	1.16	1.06	1.04	1.02	1.00	0.95	0.94	0.83	0.80	0.77	0.73	0.70	0.62
12	2.26	2.14	1.88	1.75	1.48	1.42	1.36	1.31	1.27	1.21	1.11	1.09	1.07	1.05	1.00	0.99	0.87	0.84	0.81	0.77	0.74	0.63
12-¼	2.29	2.18	1.91	1.78	1.50	1.44	1.38	1.33	1.29	1.23	1.13	1.11	1.09	1.07	1.01	1.00	0.89	0.85	0.82	0.78	0.75	0.64
15	2.58	2.45	2.15	2.00	1.69	1.62	1.56	1.50	1.45	1.39	1.27	1.25	1.22	1.20	1.14	1.13	1.00	0.96	0.92	0.87	0.84	0.68
16-½	2.70	2.56	2.25	2.09	1.77	1.69	1.63	1.57	1.52	1.45	1.33	1.30	1.28	1.26	1.20	1.18	1.05	1.00	0.96	0.91	0.88	0.77
18	2.79	2.66	2.33	2.17	1.84	1.76	1.69	1.63	1.57	1.50	1.38	1.35	1.32	1.30	1.24	1.22	1.08	1.04	1.00	0.95	0.91	0.84
21	2.95	2.80	2.46	2.29	1.94	1.85	1.78	1.72	1.66	1.59	1.46	1.43	1.40	1.37	1.31	1.29	1.14	1.09	1.06	1.00	0.96	0.88
24	3.06	2.91	2.56	2.38	2.01	1.93	1.85	1.78	1.72	1.65	1.51	1.48	1.45	1.43	1.36	1.34	1.19	1.14	1.10	1.04	1.00	0.92
36	3.33	3.17	2.78	2.59	2.19	2.09	2.01	1.94	1.88	1.79	1.65	1.61	1.58	1.55	1.48	1.46	1.29	1.24	1.19	1.13	1.09	1.00

Design load ratios are calculated as the product of R_{s1} and R_{s2}, but should not be taken as greater than 1.0. ($R_s = [R_{s1}][R_{s2}] \le 1.0$)
For the purposes of this table, the dimension d is measured in the direction normal to the axis about which buckling is considered. The designer should consider buckling about both axes and use the lesser design value. The dimensions d and b are dressed, dry dimensions.
Tabulated values may be used for sawn lumber, structural glued laminated timber, or structural composite lumber where $E'_{min}/F'_c \ge 350$.
Values of R_{s1} and R_{s2} may be interpolated for values of d, L_e/d, and b other than those shown.

Table A2(2-hr) Design Load Ratios, $R_s = (R_{s1})(R_{s2}) \leq 1.0$, for Compression Members Exposed on Four Sides, 2 – HOUR RATING
(Structural Calculations at Standard Reference Conditions: $C_D = 1.0$, $C_M = 1.0$, $C_t = 1.0$, $C_i = 1.0$)

	Depth of Member, d (measured in the direction normal to the axis about which buckling is considered)																					
	8-¼	**8-½**	**8-¾**	**9**	**9-¼**	**9-⅝**	**10-½**	**10-¾**	**11**	**11-¼**	**12**	**12-¼**	**13-¼**	**13-½**	**13-¾**	**15**	**16-½**	**17**	**18**	**21**	**24**	**36**
L_e/d	**R_{s1} - Design Stress Adjustment for L_e/d**																					
0	0.14	0.17	0.20	0.23	0.26	0.30	0.41	0.44	0.47	0.49	0.58	0.60	0.70	0.73	0.75	0.86	0.98	1.02	1.09	1.26	1.40	1.75
2	0.13	0.16	0.19	0.22	0.25	0.29	0.40	0.43	0.46	0.49	0.57	0.60	0.70	0.72	0.75	0.86	0.97	1.01	1.08	1.25	1.39	1.75
4	0.085	0.11	0.14	0.18	0.21	0.26	0.37	0.40	0.43	0.45	0.54	0.57	0.67	0.69	0.72	0.83	0.95	0.99	1.06	1.23	1.38	1.73
6	0.045	0.064	0.086	0.11	0.14	0.18	0.29	0.32	0.35	0.38	0.47	0.50	0.61	0.63	0.66	0.78	0.90	0.94	1.01	1.19	1.34	1.70
8	0.027	0.039	0.053	0.070	0.088	0.12	0.20	0.23	0.26	0.28	0.37	0.40	0.50	0.53	0.56	0.68	0.81	0.85	0.92	1.12	1.27	1.65
10	0.018	0.026	0.036	0.047	0.060	0.082	0.15	0.17	0.19	0.21	0.28	0.30	0.39	0.42	0.44	0.56	0.68	0.72	0.80	1.00	1.16	1.56
12	0.013	0.019	0.026	0.034	0.043	0.060	0.11	0.12	0.14	0.16	0.21	0.23	0.31	0.33	0.35	0.45	0.56	0.60	0.67	0.86	1.02	1.44
14	0.010	0.015	0.020	0.026	0.034	0.047	0.084	0.096	0.11	0.12	0.17	0.18	0.24	0.26	0.28	0.36	0.46	0.50	0.56	0.74	0.89	1.31
16		0.012	0.017	0.022	0.028	0.039	0.070	0.080	0.091	0.10	0.14	0.15	0.21	0.22	0.23	0.31	0.39	0.42	0.48	0.64	0.78	1.18
18		0.011	0.015	0.020	0.025	0.035	0.063	0.072	0.082	0.092	0.12	0.14	0.18	0.20	0.21	0.28	0.35	0.38	0.43	0.58	0.71	1.08
20		0.010	0.014	0.018	0.023	0.032	0.058	0.067	0.076	0.086	0.12	0.13	0.17	0.18	0.20	0.26	0.33	0.36	0.41	0.54	0.67	1.03
22		0.010	0.013	0.018	0.023	0.031	0.056	0.064	0.073	0.082	0.11	0.12	0.17	0.18	0.19	0.25	0.32	0.34	0.39	0.52	0.64	1.00
24			0.013	0.017	0.022	0.030	0.055	0.063	0.071	0.080	0.11	0.12	0.16	0.17	0.19	0.24	0.31	0.34	0.38	0.51	0.63	0.98
26			0.013	0.017	0.022	0.030	0.054	0.062	0.070	0.079	0.11	0.12	0.16	0.17	0.18	0.24	0.31	0.33	0.38	0.51	0.62	0.97
28			0.013	0.017	0.021	0.029	0.053	0.061	0.069	0.078	0.11	0.12	0.16	0.17	0.18	0.24	0.31	0.33	0.37	0.50	0.62	0.96
30			0.013	0.017	0.021	0.029	0.053	0.061	0.069	0.078	0.11	0.12	0.16	0.17	0.18	0.24	0.30	0.33	0.37	0.50	0.61	0.96
40			0.012	0.016	0.021	0.029	0.052	0.059	0.068	0.076	0.10	0.11	0.15	0.16	0.18	0.23	0.30	0.32	0.36	0.49	0.61	0.95
50			0.012	0.016	0.021	0.028	0.051	0.059	0.067	0.075	0.10	0.11	0.15	0.16	0.17	0.23	0.30	0.32	0.36	0.49	0.60	0.94
b	**R_{s2} - Design Stress Adjustment for Width, b, of Column**																					
8-¼	1.00	0.91	0.84	0.79	0.74	0.68	0.59	0.57	0.55	0.53	0.49	0.48	0.45	0.44	0.43	0.40	0.38	0.37	0.36	0.33	0.32	0.28
8-½	1.10	1.00	0.92	0.86	0.81	0.75	0.64	0.62	0.60	0.58	0.54	0.53	0.49	0.48	0.47	0.44	0.42	0.41	0.39	0.37	0.35	0.31
8-¾	1.19	1.08	1.00	0.93	0.88	0.81	0.70	0.67	0.65	0.63	0.59	0.57	0.53	0.52	0.51	0.48	0.45	0.44	0.43	0.40	0.38	0.34
9	1.27	1.16	1.07	1.00	0.94	0.87	0.75	0.72	0.70	0.68	0.63	0.61	0.57	0.56	0.55	0.51	0.48	0.47	0.46	0.43	0.40	0.36
9-¼	1.35	1.24	1.14	1.06	1.00	0.92	0.80	0.77	0.74	0.72	0.67	0.65	0.61	0.60	0.59	0.55	0.51	0.50	0.49	0.45	0.43	0.38
9-⅝	1.47	1.34	1.24	1.15	1.08	1.00	0.86	0.83	0.81	0.78	0.73	0.71	0.66	0.65	0.64	0.59	0.56	0.55	0.53	0.49	0.47	0.42
10-½	1.70	1.55	1.43	1.34	1.26	1.16	1.00	0.97	0.94	0.91	0.84	0.82	0.76	0.75	0.74	0.69	0.64	0.63	0.61	0.57	0.54	0.48
10-¾	1.76	1.61	1.48	1.38	1.30	1.20	1.04	1.00	0.97	0.94	0.87	0.85	0.79	0.77	0.76	0.71	0.67	0.66	0.63	0.59	0.56	0.50
11	1.82	1.66	1.53	1.43	1.34	1.24	1.07	1.03	1.00	0.97	0.90	0.88	0.81	0.80	0.79	0.73	0.69	0.68	0.66	0.61	0.58	0.52
11-¼	1.88	1.71	1.58	1.47	1.38	1.28	1.10	1.06	1.03	1.00	0.93	0.91	0.84	0.82	0.81	0.76	0.71	0.70	0.68	0.63	0.59	0.53
12	2.03	1.85	1.71	1.59	1.50	1.38	1.19	1.15	1.11	1.08	1.00	0.98	0.90	0.89	0.88	0.82	0.77	0.75	0.73	0.68	0.64	0.57
12-¼	2.07	1.89	1.74	1.63	1.53	1.41	1.22	1.17	1.14	1.10	1.02	1.00	0.93	0.91	0.90	0.84	0.78	0.77	0.75	0.69	0.66	0.59
13-¼	2.24	2.04	1.89	1.76	1.65	1.52	1.31	1.27	1.23	1.19	1.11	1.08	1.00	0.98	0.97	0.90	0.85	0.83	0.81	0.75	0.71	0.63
13-½	2.28	2.08	1.92	1.79	1.68	1.55	1.34	1.29	1.25	1.21	1.12	1.10	1.02	1.00	0.98	0.92	0.86	0.85	0.82	0.76	0.72	0.64
13-¾	2.31	2.11	1.95	1.82	1.71	1.57	1.36	1.31	1.27	1.23	1.14	1.12	1.03	1.02	1.00	0.93	0.88	0.86	0.83	0.77	0.73	0.66
15	2.48	2.26	2.09	1.95	1.83	1.69	1.45	1.40	1.36	1.32	1.22	1.20	1.11	1.09	1.07	1.00	0.94	0.92	0.89	0.83	0.79	0.70
16-½	2.64	2.41	2.22	2.07	1.95	1.80	1.55	1.50	1.45	1.41	1.30	1.27	1.18	1.16	1.14	1.07	1.00	0.98	0.95	0.88	0.84	0.75
17	2.69	2.45	2.27	2.11	1.99	1.83	1.58	1.53	1.48	1.43	1.33	1.30	1.20	1.18	1.16	1.09	1.02	1.00	0.97	0.90	0.85	0.76
18	2.78	2.53	2.34	2.18	2.05	1.89	1.63	1.58	1.53	1.48	1.37	1.34	1.24	1.22	1.20	1.12	1.05	1.03	1.00	0.93	0.88	0.79
21	2.99	2.73	2.52	2.35	2.21	2.04	1.76	1.70	1.64	1.60	1.48	1.44	1.34	1.31	1.29	1.21	1.13	1.11	1.08	1.00	0.95	0.85
24	3.15	2.88	2.66	2.48	2.33	2.15	1.85	1.79	1.73	1.68	1.56	1.52	1.41	1.39	1.36	1.27	1.19	1.17	1.14	1.05	1.00	0.89
36	3.53	3.22	2.97	2.77	2.61	2.40	2.07	2.00	1.94	1.88	1.74	1.70	1.58	1.55	1.53	1.43	1.34	1.31	1.27	1.18	1.12	1.00

Design load ratios are calculated as the product of R_{s1} and R_{s2}, but should not be taken as greater than 1.0. ($R_s = [R_{s1}][R_{s2}] \leq 1.0$)
For the purposes of this table, the dimension d is measured in the direction normal to the axis about which buckling is considered. The designer should consider buckling about both axes and use the lesser design value. The dimensions d and b are dry dressed dimensions.
Tabulated values may be used for sawn lumber, structural glued laminated timber, or structural composite lumber where $E'_{min}/F'_c \geq 350$.
Values of R_{s1} and R_{s2} may be interpolated for values of d, L_e/d, and b other than those shown.

A.3 Timber Decks

Structural:

$$D+L \le R_s F_b S_s C_D C_M C_t \qquad \text{Equation A-10}$$

Fire:

$$D+L \le 2.85 F_b S_f \qquad \text{Equation A-11}$$

Where;

- D = Design dead load
- L = Design live load
- R_s = Design load ratio
- F_b = Tabulated bending design value
- S_s = Section modulus using full (initial) cross-section dimensions
- S_f = Section modulus using cross-section dimensions reduced from fire exposure
- C_D = Load Duration factor
- C_M = Wet Service factor
- C_t = Temperature factor

Solve for R_s:

$$R_s = \frac{2.85 S_f}{S_s C_D C_M C_t} \qquad \text{Equation A-12}$$

For butt-jointed timber decks, NDS Section 16.2.5 states that the char rate on the butt-jointed sides of the timber decking shall be taken as 33% (one-third) of the effective char rate. Thus, the charred section modulus is calculated as $S_f = (b - 2a/3)(d - a)^2/6$, and the design load ratio, R_s, for butt-jointed timber decking is calculated as follows:

$$R_s = \frac{2.85(b - 2^a/_3)(d - a)^2}{bd^2 C_D C_M C_t} \qquad \text{Equation A-13}$$

NDS Section 16.2.5 states that tongue-and-groove timber decks shall be designed as an assembly of wood beams fully exposed on the bottom face only. Thus, the charred section modulus is calculated as $S_f = b(d - a)^2/6$, and the design load ratio, R_s, for tongue-and-groove timber decking is calculated as follows:

$$R_s = \frac{2.85(d - a)^2}{d^2 C_D C_M C_t} \qquad \text{Equation A-14}$$

The design load ratios, R_s, given in *Tables A3.1* and *A3.2* were developed for butt-jointed and tongue-and-groove timber decks, respectively, under standard reference conditions ($C_D = 1.0$; $C_M = 1.0$; $C_t = 1.0$).

Table A3.1 Design Load Ratios, R_s, for Butt-Jointed Timber Decks (Protected on Top Face; Partially Protected on Sides per NDS Section 16.2.5) (Structural Calculations at Standard Reference Conditions: $C_D = 1.0$, $C_M = 1.0$, $C_t = 1.0$, $C_i = 1.0$, $C_L = 1.0$)

	1-HOUR				**1.5-HOUR**			**2-HOUR**	
Width, b	**1-½**	**2-½**	**3-½**	**5-½**	**2-½**	**3-½**	**5-½**	**3-½**	**5-½**
Depth, d	**Design Load Ratio, R_s**								
2-½	0.05	0.12	0.15	0.18	-	-	-	-	-
3	0.10	0.24	0.30	0.36	0.03	0.04	0.05	-	-
3-½	0.14	0.35	0.44	0.53	0.08	0.12	0.16	-	-
4	0.18	0.45	0.57	0.68	0.14	0.21	0.28	0.05	0.08
4-½	0.21	0.54	0.68	0.80	0.19	0.30	0.39	0.10	0.16
5	0.24	0.61	0.77	0.92	0.24	0.38	0.50	0.16	0.24
5-½	0.27	0.68	0.85	1.00	0.29	0.45	0.59	0.21	0.32

Table A3.2 Design Load Ratios, R_s, for Tongue-and-Groove Timber Decks (Protected on Top Face and Sides per NDS Section 16.2.5) (Structural Calculations at Standard Reference Conditions: C_D = 1.0, C_M = 1.0, C_t = 1.0, C_i = 1.0, C_L = 1.0)

	1-HOUR	**1.5-HOUR**	**2-HOUR**
Depth, d		**Design Load Ratio, R_s**	
2-½	0.22	-	-
3	0.46	0.08	-
3-½	0.67	0.23	0.03
4	0.86	0.40	0.12
4-½	1.00	0.56	0.25
5	1.00	0.71	0.38
5-½	1.00	0.85	0.51

APPENDIX B: Fire Resistance Calculation for Sawn Wood Joists

Starting with the standard char rate for wood equal to 1.5 in./hr:

$$\beta_{eff} = \frac{1.2\beta_n}{t^{0.187}} = \frac{(1.2)(1.5)}{t^{0.187}} = \frac{1.8}{t^{0.187}} \quad \text{Equation B-1}$$

Calculate the depth of the char layer:

$$a = \beta_{eff}t = \frac{1.8t}{t^{0.187}} = 1.8t^{0.813} \quad \text{Equation B-2}$$

Where :

β_{eff} = effective char rate (in./hr.) adjusted for exposure time, t

β_n = nominal char rate (in./hr.) linear char rate based on 1-hour E119 exposure

t = exposure time (hr.)

a = char layer depth (in.)

For a fully-braced bending member used in a floor or ceiling assembly, the relationship between the allowable bending stress for structural design and the bending member strength for fire design can be estimated using K = 2.85:

$$R_s = \frac{2.85S_f}{S_s} \quad \text{Equation B-3}$$

Expanding Equation B-3 in terms of bending section properties:

$$R_s = \frac{2.85(b-2a)(d-a)^2/6}{bd^2/6} = 2.85\left(\frac{b-2a}{b}\right)\left(\frac{d-a}{d}\right)^2 \quad \text{Equation B-4}$$

Rearranging Eqn. B-4 to solve for a:

$$a^3 - a\frac{(b+4d)}{2} + a(bd+d^2) - \frac{bd^2}{2}\left(1 - \frac{R_s}{2.85}\right) = 0 \quad \text{Equation B-5}$$

Where :

R_s = design stress ratio for structural design (0 – 100% of full design load)

S_f = bending section modulus of remaining cross-section after reducing for char on all exposed surfaces.

S_s = bending section modulus of initial cross-section.

b = breadth of rectangular bending member, in.

d = depth of rectangular bending member, in.

Calculating a direct solution for the char depth, a, as a function of time and load ratio is very complicated, but it can be solved relatively quickly by iteratively solving for the maximum char depth, then back-calculating the fire resistance time can be determined by substituting the char depth, a, back into Equation. B-2. The following cases for standard lumber dimensions have been determined:

Table B1 Design Load Ratio Limits for Wood Joists

Load Ratio R_s	2x6	2x8	2x10	2x12
	Structural Fire Resistance Time (minutes)			
0.00	20.4	20.4	20.4	20.4
0.10	19.3	19.4	19.4	19.4
0.20	18.2	18.3	18.4	18.5
0.30	17.1	17.3	17.4	17.5
0.40	16.1	16.3	16.5	16.6
0.50	15.1	15.3	15.5	15.6
0.60	14.1	14.4	14.6	14.7
0.70	13.2	13.5	13.7	13.8
0.80	12.3	12.6	12.8	13.0
0.90	11.4	11.8	12.0	12.1
1.00	10.6	10.9	11.2	11.3

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10-15