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Fire Design of Mass Timber Structural Members

Demonstrating Fire-Resistance Ratings of Mass Timber Products

Traditionally, the role of the structural engineer on building projects has focused on structure-related tasks—member sizing, connection detailing, general notes, and specifications for structural components. Design criteria such as fire-resistance ratings (FRRs), acoustics, and aesthetics have primarily been the architect's domain. However, when it comes to mass timber, the structure often contributes to the building's passive fire resistance. This can happen when the structure is functioning as an exposed finish or when partial fire resistance is provided by a covering over the timber and the rest is provided by the timber itself. This combination of structure, finish, and fire resistance makes the mass timber design process a necessarily collaborative effort between architect and engineer.


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This paper presents several methods for demonstrating the FRR of a mass timber element, particularly when the mass timber structural members are required to be fire-resistance-rated. These elements include horizontal assemblies (floors, roofs) and walls, which serve both structural and fire containment purposes, and structural members such as beams and columns where the purpose is mainly structural. While much of the information is introductory, it covers how to evaluate the suitability of tested horizontal cross-laminated timber (CLT) assemblies with reduced load ratings for different spans and loading conditions and the different models for calculating structural FRRs of nail-laminated timber (NLT). Unless otherwise noted, references are to the 2024 IBC.

Sources of FRR Requirements

For buildings designed under the International Building Code (IBC), construction type is one of the major determinants of which timber products can be used, whether the timber products can be left exposed to view, and the FRR requirements for building elements, including those constructed with mass timber products. For information on selecting construction type and determining the FRR of building elements, see the IBC and the WoodWorks publication, *Fire Requirements for Mass Timber Elements – Code Applications, Construction Types, and Fire Ratings*. The latter provides a detailed review of the sources and types of fire requirements applicable to mass timber buildings.

Generally, the IBC requires lower FRRs for smaller buildings and higher FRRs for larger buildings. Using business occupancies (B) as an example, unrated construction is allowed in some buildings up to four stories (Type III-B), and 1-hour-rated construction is permitted in some buildings up to six stories (Type III-A). For the newer Type IV-C and IV-B construction types, which can be a maximum of nine or 12 stories respectively, the

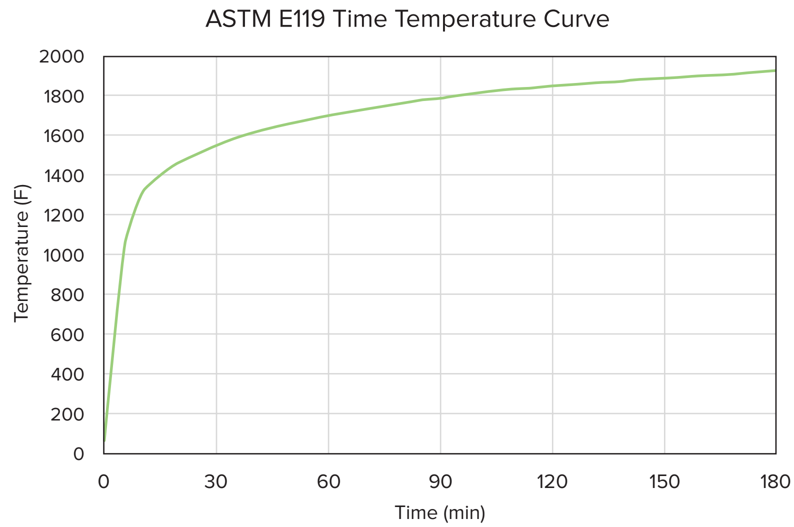

FIGURE 1: Standard fire-resistance test time-temperature relationship

requirement increases to a 2-hour FRR for many elements. Type IV-A construction can be up to 18 stories but requires a 3-hour FRR on the primary structural frame and bearing walls with noncombustible protection providing a portion of the required rating. To determine an appropriate construction type and associated FRR requirements, refer to the applicable building code or reach out to WoodWorks for assistance. For structural members, the FRR requirement is commonly from 0 to 3 hours.

Methods of Demonstrating the FRR of a Mass Timber Member

The two main routes for demonstrating FRRs are through testing, described in IBC Section 703.2.1, and analytical methods, described in Section 703.2.2.

Tested FRRs

The IBC recognizes fire-resistance testing of certain building elements in accordance with ASTM E119 Standard Test Methods for Fire Tests of Building Construction and Materials (ASTM, 2020) or UL 263 Standard for Fire Tests of Building Construction and Materials (ULSE, 2022). The Canadian standard for demonstrating FRRs, ULC S101 Standard Methods of Fire Endurance Tests of Building Construction and Materials (ULSE, 2014), is similar to the two U.S. standards and an FRR demonstrated via ULC S101 can be used for projects under the IBC if deemed acceptable to the project team and authority having jurisdiction (AHJ).

According to Section 4.2 of ASTM E119-20, the fire test procedure is described as:

> *The test exposes a test specimen to a standard fire controlled to achieve specified temperatures throughout a specified time period. […] The test provides a relative measure of the fire-test-response of comparable building elements under these fire exposure conditions. The exposure is not representative of all fire conditions because conditions vary with changes in the amount, nature, and distribution of fire loading, ventilation, compartment size and configuration, and heat sink characteristics of the compartment.*

In the standard test procedures, the specimen is placed on a side or within a fire chamber, instrumented, and exposed to the standard time-temperature curve shown in Figure 1.

These procedures have several pass/fail criteria, the following three of which may apply during the standard fire exposure time:

- **Structural fire resistance**, where a load-bearing member or assembly supports the applied load without structural failure
- **Integrity** (or **burn-through resistance**), where the member or assembly does not, at any location, allow transmission of flames or hot gases to the unexposed surface that would ignite cotton waste
- **Thermal insulation** (or **thermal separation**), where an average temperature rise on the unexposed surface of the member or assembly does not exceed 250° F (139° C)

Each criterion is not required of all fire-resistance-rated members and assemblies. Where a member or assembly is required to prevent the passage of fire, the burn-through and thermal separation criteria apply. As examples, this applies to fire barriers, fire partitions, fire walls, and fire-rated horizontal assemblies as defined in the IBC.

When a member or assembly is load-bearing, the structural fire-resistance criterion applies. Where a load-bearing member or assembly is not required to prevent passage of fire, *only* the structural fire resistance applies. Examples include load-bearing columns and beams completely contained within a single fire compartment of a building.

A fourth possible test criterion is a hose-stream test, in which an assembly must remain intact when exposed to a defined hose stream following a fire exposure. The hose stream test subjects the assembly to impact, erosion, and cooling effects of a hose stream, and is required on walls and partitions with a 1-hour or longer FRR.

To achieve a fire-resistance rating, the member or assembly must pass all applicable criteria. Stated another way, the first applicable criterion that fails determines the rating. In fire-resistance testing, it is common practice to run a test to a target rating and stop the test without testing to failure. For example, a floor test may be run to 2 hours then stopped, as the floor has achieved the targeted 2-hour FRR.

Successful fire-resistance tests have been completed on numerous mass timber elements and assemblies achieving FRRs of 3 hours or more. Additional tests by manufacturers and others are ongoing.

Standardized tests have been used to demonstrate the FRR of floor, roof, and wall assemblies using mass timber panels as the structural element. Some successful tests have included protection of the assembly with applied noncombustible protection such as Type X gypsum board, while others have left the mass timber exposed to view and the FRR is provided by the inherent fire resistance of the wood element.

In floor and wall assemblies that provide burn-through resistance and thermal separation, the panel-to-panel connection is often the assembly's weak point. This connection does not have continuous panel material and protection against burn-through and thermal separation at the gap between panels is provided by supplemental materials. Consequently, if only testing is used to justify the FRR, the referenced test(s) should include the panel-to-panel connection detail being used on the project.

For convenience, WoodWorks maintains a web-based inventory of completed mass timber fire-resistance tests—the Mass Timber Fire & Acoustic Database. At the time of this writing, it includes over 50 fire-resistance-tested mass timber floor, roof, and wall assemblies.

Analytical Methods for FRRs

The IBC also allows the FRR to be demonstrated by recognized analytical methods instead of direct testing of the proposed design. The analytical methods are based on the same standard fire-resistance testing, and some of the options do not require analysis by the design team.

The five analytical methods from IBC Section 703.2.2 are:

1. **Designs documented in approved sources** – This can include publications from trusted industry organizations such as the Gypsum Association's *Fire Resistance and Sound Control Design Manual* (GA, 2024) and the American Wood Council's (AWC's) *DCA 3 – Fire-Resistance-Rated Wood-Frame Wall and Floor/Ceiling Assemblies* (AWC, 2024). There is little coverage of mass timber elements in these sources.
2. **Prescriptive designs in IBC Section 721** – Fire resistance is detailed within the tables of IBC Section 721. However, there is no coverage of mass timber in the referenced assemblies.
3. **Calculations as listed in IBC Section 722** – Discussed further below, fire resistance can be calculated by following the methods in Chapter 16 of AWC's National Design Specification® for Wood Construction (NDS®) (AWC, 2024) or by adding noncombustible layers in accordance with IBC Section 722.7.
4. **Engineering analysis based on designs with completed fire tests** – It is common to use a completed ASTM E119 or UL 263 fire-resistance test to justify a similar assembly by comparing the fire performance properties of materials in the tested and proposed assemblies. This type of analysis is usually performed by an expert in the fire performance of the materials being used.
5. **Fire resistance designs certified by an approved agency** – Approved agencies are not referenced in the IBC as they are *approved* by the AHJ, not the IBC. Commonly approved agencies include International Code Council – Evaluation Services (ICC-ES) and Underwriters Laboratories (UL). These agencies have presumably performed the analysis to justify the many compliant options contained within their certified designs.

Calculated Fire Resistance in IBC 722

The recognized methods for calculating FRRs are located or referenced in IBC Section 722. Section 722.1 references third-party standards for concrete, masonry, steel, and wood. The reference for wood, found in Section 722.1 Item 4, is to Chapter 16 of the NDS. Section 722.7 includes additional requirements and methods for using noncombustible protection to help achieve the FRR of mass timber.

Calculated Fire Resistance of Wood Members

Calculation methods are used in many cases when demonstrating FRRs for exposed timber beams and columns. For mass timber panels in fire-resistance-rated floors and walls, both calculations and fire-resistance test results are common.

Chapter 16 of the NDS includes methods for calculating the FRR of exposed wood members for up to 2 hours. The provisions apply to various wood products including sawn lumber, structural glue-laminated timbers (glulam), structural composite lumber (SCL), and CLT. NDS Sections 16.4 and 16.5 reference AWC's Fire Design Specification for Wood Construction (FDS) (AWC, 2024) for calculated fire resistance where added protective materials are used to protect wood members and connections.

See Figure 2 for a visual summary of the code path for calculating the FRR of exposed wood elements.

Calculated Char Depth

Wood is a combustible material and, when exposed to high enough temperatures, will ignite and burn. The process of pyrolysis releases products of combustion (heat, water, carbon dioxide) and converts the wood fiber to char, which is a zero-strength layer. Due to the insulative properties of wood, pyrolysis does not occur through an entire wood member at once but moves from the exposed surface deeper into the cross section of the wood member over time. The depth of the char front from the original, uncharred surface of the member is given the notation a_{char}. In fire tests measuring the temperature profile, this char front occurs in a temperature range of about 288° C to 315° C (550° F to 600° F). These zones in charred wood are shown in Figure 3.

When exposed to the standard time-temperature curve and conditions defined in the FRR test standard, the rate of char and strength degradation of the affected wood are repeatable and predictable behaviors. As described in Chapter 18 of the *Wood Handbook: Wood as an Engineering Material* (USDA FPL, 2021), the rate of char progression into wood members in experimental tests varies with factors such as moisture content, species, and grain orientation of the wood. However, based on evaluating the char depth and structural fire resistance of many different wood component types, Chapter 16 of the NDS (Section 16.2.1) and Chapter 3 of the FDS recognize a nominal char depth of 1.5 inches measured at 1 hour for design of wood-based structural components.

$$\beta_n = 1.5 \text{ in./hr}$$

The formation of char on the surface of a member protects the core cross section of the timber members, and there is a steep temperature gradient between the cool interior and char front. As the char layer increases in thickness, it insulates the inner materials, decreasing the rate of char progression into the wood.

IBC Section 703.2.2
Methods for determining fire resistance

- Prescriptive designs per IBC 721.1
- ***Calculations in accordance with IBC 722***
- Fire-resistance designs documented in sources
- Engineering analysis based on a comparison

IBC Section 722
Calculated Fire Resistance

"The calculated *fire resistance* of exposed wood members and wood decking shall be permitted in accordance with **Chapter 16 of the National Design Specification for Wood Construction (NDS)**

NDS Chapter 16
Fire Design of Wood Members

- Limited to calculating fire resistance up to 2 hours
- Char depth varies based on standard exposure time (i.e., FRR)

FIGURE 2: Code path for exposed wood fire-resistance calculations

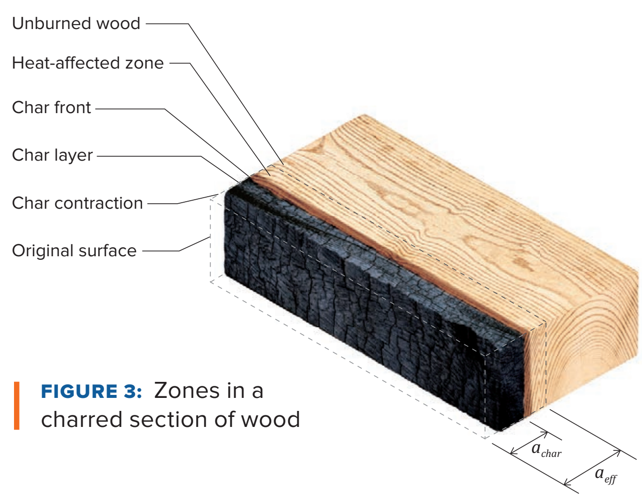

FIGURE 3: Zones in a charred section of wood

The NDS char model recognizes this behavior by providing a non-linear char depth model, shown in NDS Equation 16.2-2 as

$$a_{char} = \beta_t t^{0.813}$$

where:

β_t is the non-linear char rate, in in./hr$^{0.813}$

t is the time of standard exposure, in hours

The non-linear char rate is set so the calculated depth at 1 hour equals the nominal char rate.

$$1.5 \text{ in.} = \beta_t(1 \text{ “hr”})^{0.813} \Rightarrow \beta_t = 1.5\text{in./hr}^{0.813}$$

The exponent of the non-linear char rate is based on the reduction in char rate over time in experimental results.

The calculated char depth predicts the location of the char front where charred wood has no structural capacity. Further inside the member, beyond the char front, the temperature decreases to interior wood at ambient temperatures with full structural capacity. Accompanying this temperature gradient is a correlated change in structural capacity. At temperatures above ambient, the structural properties of wood fibers, such as strength and stiffness, degrade. The ratio of the reduced structural property to the original property, labeled R, decreases with increasing temperature. Figure 4 shows a conceptual example of how the temperature and wood fiber structural properties vary in wood under a 1-hour standard fire exposure. For more information on the temperature gradients and wood fiber properties within the fire-exposed wood members, see Chapter 18 of the Wood Handbook. Using models of temperature-dependent wood properties, a detailed structural analysis of a fire-affected member can be performed. Such methods are useful in research and product development, and beyond the scope of standardized engineering design methods.

TABLE 1: Effective char rates and char depths (for β_n = 1.5 in./hr)

Required Fire Endurance (hrs)	Char Depth, a_{char} (in.)	Effective Char Depth a_{eff} (in.)
1	1.5	1.8
1-1/2	2.1	2.5
2	2.6	3.2

Source: NDS Table 13.3.1, AWC

To simplify engineering analysis of structural properties of the charred wood members, the NDS defines an effective char depth, a_{eff}, greater than a_{char}. For calculating structural capacity, wood fiber deeper than a_{eff} is assumed to have full strength, while wood fiber within a_{eff} is assumed to have zero strength. The wood fiber between a_{eff} and a_{char} can be called the heat-affected zone or zero-strength layer.

Based on structural fire-resistance testing of many wood components under load, the relationship between a_{char} and a_{eff} is defined in the NDS Equation 16.3-1 as:

$$a_{eff} = 1.2\, a_{char}$$

The above relationships have been validated for calculating the structural FRR of various exposed wood products, including sawn lumber, structural glulam, and SCL. The applicable char depth and effective char depth for these products following the NDS is shown in Table 1 .

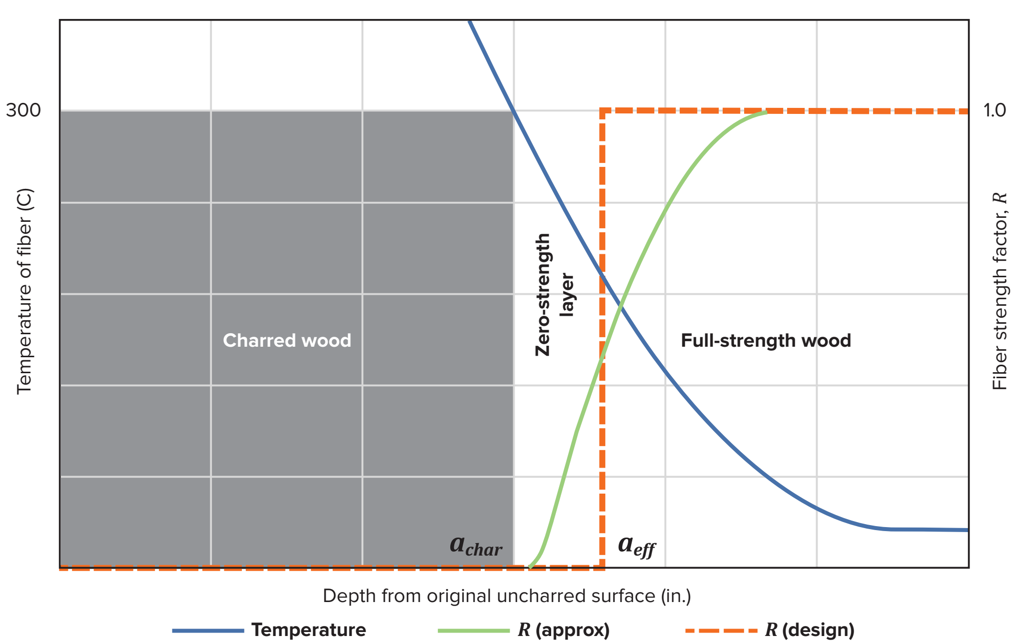

FIGURE 4: Idealized temperature and wood fiber strength profile for 1-hour standard exposure

Strength Checks for Structural FRRs

Checking the structural adequacy of a wood member for structural FRRs is a different design scenario than normal gravity design checks. Calculated structural FRRs predict whether the structural component under the applied loads can pass the fire-resistance test for the required duration. The calculated structural fire-resistance method uses the same loading and acceptance criteria in ASTM E119. The ASTM E119 fire-resistance test requires that the assembly or component does not fail under the applied load ($D+L$) for the required duration.

For wood components, the calculated structural fire-resistance method uses the calculated equivalent of the "full capacity" of the component, with no safety factors or similar strength reduction factors applied. In more technical terms, the calculated capacity is an estimate of the average ultimate strength. This average ultimate strength value is calculated by increasing the reference design values using the "design stress to member strength factor," K, as shown in NDS Table 16.3.3 and FDS Table 3.2.2. The strength adjustment factors range from 1.67 to 2.85 above the reference design values as replicated in Table 2. A more detailed explanation of this adjustment is found in NDS Section C16.3.2.

TABLE 2: Member strength adjustment factors

Member Strength	Adjustment Factor, K, for Fire Design
Bending strength	2.85
Beam bucking strength	2.03
Tensile strength	2.85
Compressive strength	2.58
Bearing strength	1.67*
Shear strength	2.75*
Column buckling strength	2.03

Source: NDS Table 16.3.3
**FDS Table 3.3.2*

In summary, the steps to a calculated structural fire-resistance check of a wood member are:

- Determine required FRR and applied loading from the design requirements.
- Calculate the reduced section size of the wood member based on a_{eff} from the required FRR and section properties.
- Determine the adjusted fire design strength of the reduced section.
- Verify the adjusted fire design strength is not less than the demands for the applied loading ($D+L$).

What load combinations are used with structural FRR calculations?

The structural fire-resistance calculations in NDS Section 16.3 are used with allowable stress design (ASD) load combinations. ASTM E119 specifically calls for loading equal to 100% of the ASD load combination: $D+L$. The standard fire-resistance test procedures have been used in the U.S. since before 1918, and while engineering material standards have evolved to include load and resistance factor design (LRFD) and ultimate limit state design procedures, the historic FRR requirements and comparable calculations remain benchmarked to ASD load combination loads. For elements with snow or roof live loads, the maximum ASD load combination of dead, live, roof live, snow, and rain should be used.

Both ASTM E119 and the FDS provide instructions on how to convert LRFD design loads to design loads for testing or calculating structural fire resistance. However, as this conversion requires knowing the dead and live load components of the applied load, it may be simpler to use the ASD load combinations.

ASCE 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE, 2022) Section 2.5.4 also contains load combinations for extraordinary events, which, according to the commentary, "are not intended to supplant traditional approaches to ensure fire endurance based on standardized time-temperature curves and code-specified endurance times. Current code-specified endurance times are based on the ASTM E119 time-temperature curve under full allowable design load." The ASCE 7 load combinations for extraordinary events are not to be used to calculate superimposed loads for ASTM E119 tests, nor to calculate the loads used to determine the FRR to meet provisions of the IBC and NDS Chapter 16.

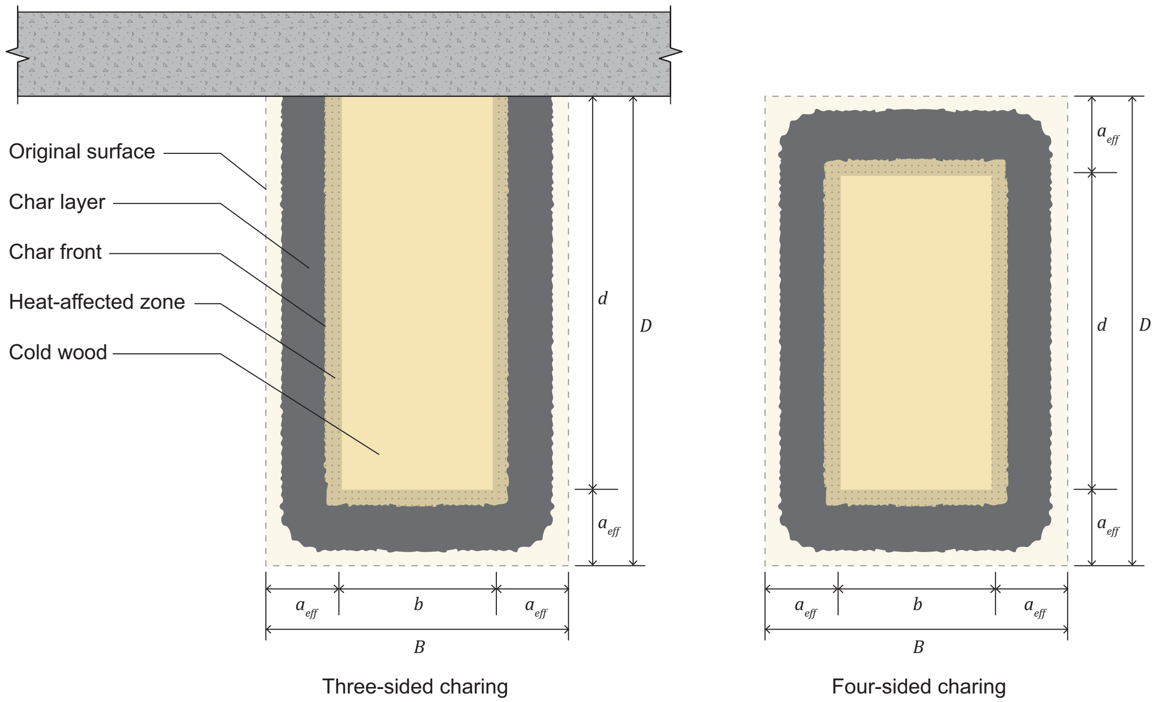

FIGURE 5: Reduction in structural size for three-sided and four-sided exposures

If a structural capacity is sensitive to deformations developed during the calculated fire exposure, the designer should consider the impact of deformations on capacity. For example, second order effects from internal bending (P-δ) should be considered in the structural FRR calculations of eccentrically-loaded wall or column elements. However, building serviceability requirements, such as deflection limits in IBC Section 1604.3, and objectives such as acceptable floor vibrations for occupant comfort, are only applicable to the unreduced section and not the reduced section and structural FRR criteria.

To calculate the net section properties of the reduced size of the wood member, two approaches are commonly used for exposed framing elements. A three-sided char model is used for beams and similar elements protected from exposure on one side, where columns and beams exposed to the fire on all sides use a four-sided char model. The impacts of these models are shown in Figure 5.

Simple Sawn Lumber Beam Example

Consider an 8x12-inch nominal Douglas fir #2 sawn lumber floor beam spanning 15 feet from support to support with an ASD design load of 400 plf. NDS Supplement (AWC, 2024) Table 4D provides an unadjusted allowable bending stress, F_b, of 875 psi, as this size of timber is considered a beam and stringer.

The maximum induced moment for the D+L load combination is:

$$M_{max} = wL^2/8 = (400 \text{ lb/ft})\ (15 \text{ ft})^2 \ / \ 8 = 11{,}250 \text{ ft-lb}$$

The (non-fire) adjusted bending stress value, per NDS Table 4.3.1 is:

$$F_b' = F_b\ C_D\ C_M\ C_t\ C_L\ C_F\ C_{fu}\ C_i\ C_r$$

Let's assume the design conditions are such that:

$$F_b' = F_b\ (1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0) = 875 \text{ psi}$$

The section modulus of the full member is

$$S = 7.5 \text{ in. } (11.5 \text{ in.})^2 / 6 = 165.3 \text{ in.}^3$$

The induced bending stress is:

$$f_b = M_{max}/S = 11{,}250 \text{ ft-lb } (12 \text{ in./ft}) \ / \ 165.3 \text{ in.}^3 = 817 \text{ psi}$$

$$f_b = 817 \text{ psi} <= F_b' = 875 \text{ psi}$$

So the bending stress check passes in the normal D+L gravity case.

If this beam were required to have a 1-hour FRR and is attached to a continuous floor assembly above, protecting the top side of the beam and bracing it for lateral-torsional stability for the full 1-hour exposure, the effective char depth on the two sides and the bottom is 1.8 inches (per NDS Table 16.3.1). Its cross-sectional dimensions under 1-hour fire check are:

Width, b = 7.50 – (2)(1.80) = 3.90 in.

Depth, d = 11.50 – 1.80 = 9.70 in.

This reduced cross section is then checked under fire conditions, with allowable design stresses increased by the factors given in NDS Table 16.3.3.

The section modulus of the net section is:

$S_{1\text{-}hr}$ = 3.90 in. (9.7 in.)^2 / 6 = 61.2 in.^{3}

The induced bending stress is:

$f_{b,\,1\text{-hr}} = M_{max}/S_{1\text{-}hr}$ = 11,250 ft-lb (12 in./ft) / 61.2 in.^{3} = 2,206 psi

The fire condition adjusted bending stress value, using the adjustment factor for fire design per NDS Table 16.3.3, is:

$F_b'{}_{1\text{-}hr} = F_b\ (2.85)\ C_F\ C_V\ C_{fu}\ C_L$ = 875 psi (2.85) (1.0) (1.0) (1.0) = 2,494 psi

$f_{b,1\text{-}hr}$ = 2,206 psi <= $F_b'{}_{1\text{-}hr}$ = 2,494 psi

The bending stress check passes in the 1-hour calculated fire-resistance check.

Alternatively, engineers can reference the design tools in Appendix A of AWC's *Calculating the Fire Resistance of Wood Members and Assemblies Technical Report No. 10* (TR-10) (AWC, 2021). Table A1 shows design load ratios, R_s, which are the ratios of moment capacity in the fire condition, $F_b'{}_f\ S_f$ to the ASD D+L moment capacities, $F_b'\,S$.

For a 1-hour-rated wood member, should the beam be upsized by 1.5 inches on each exposed side?

Not necessarily. Many structural mass timber elements will not need any size increase to achieve a 1-hour rating. As seen in the example calculation, the 8x12-inch nominal sawn lumber beam does not need any size increase to handle the flexural demands for the 1-hour structural fire checks. This is because the recognized bending capacity for the fire check in this case is not reduced below the bending capacity for non-fire design checks.

Additionally, sizes of mass timber elements are often selected because of deflection or floor vibration performance, resulting in a larger member than needed for normal strength criteria. **There is no faster way to design an inefficient mass timber structure than by simply adding 1.5 inches to the dimensions of the element at each exposed face for a 1-hour fire-resistance rating.** An efficient mass timber structure will have FRRs justified by tests or structural fire calculations using the fire design adjustment factors in NDS Section 16.3.3 and FDS Section 3.3.2.

TR-10 Table A1 (1 hour) is for the bending capacity of rectangular members exposed on three sides, such as a typical floor or roof beam, to a 1-hour rating. Table A1 does not have an entry for a beam depth of exactly 11.5 inches; however, the R_s for beams with a width of 7.5 inches and depths of 11-1/4 inches and 12 inches is 1.0. If the values were different, R_s could be found by interpolation between the tabulated values. If R_s = 1.0, the flexural capacity of the beam at a 1-hour FRR is not reduced from the initial ASD capacity.

$F_b'{}_{1\text{-}hr}S_{1\text{-}hr} = R_s\,F_b'\,S$ = (1.0)(875 psi)(165.3 in.^{3}) (1 ft/12 in.) = 12,050 ft-lb > 11,250 ft-lb

The flexural capacity of the beam is sufficient in the 1-hour fire condition.

TR-10 Table A2 provides a similar design table for columns exposed on all four sides.

Other relevant structural capacity checks, such as shear in the beam and bearing stresses at supports, would also need to pass for the beam to satisfy the 1-hour structural fire-resistance requirements via calculations.

AWC FDS and TR-10

The FDS is a code-referenced standard developed following an ANSI process and expands on much of the design information contained in TR-10. However, TR-10 also includes a comparison of fire-resistance test results with design method predictions, demonstrating the validity of the fire design models contained within the NDS and FDS.

Photo: David Barber, ARUP

Glulam column before and after 2-hour fire test

Thermal Separation and Burn-Through Resistance of a Structural Member

When considering an assembly that needs to provide fire containment, the thermal separation and burn-through criteria of ASTM E119 need to be met. For a single mass timber structural element meeting the structural FRR requirements described above, there will be neither a thermal separation nor a burn-through criteria failure *through* the mass timber element. However, connections and intersections at the edges of these timber elements may need to be checked for thermal separation and burn-through where they are part of the fire containment design.

Noncombustible Protective Covering

When using a calculated route to determine the FRR of mass timber elements, it is possible to combine the fire resistance inherent in the timber member, described above, with protection from added coverings.

IBC Section 722.7 provides the prescriptive methods to protect mass timber using Type X gypsum board. Table 722.7.1.(2) defines the noncombustible protection provided by this material as shown in Table 3. If the installation instructions of Section 722.7.2 are met, multiple layers of Type X gypsum board may be installed and their protection time is additive.

IBC Section 722.7 also allows the FRR of a mass timber member to be the additive combination of the rating of the timber and protection time provided by the noncombustible protection:

> *The fire-resistance rating of the mass timber elements shall consist of the fire resistance of the unprotected element added to the protection time of the noncombustible protection.*

Consider a timber member with a 2-hour FRR requirement. If two layers of 5/8-inch Type X gypsum board are installed in accordance with IBC Section 722.7.2, the structural FRR of the timber member itself only needs to be 120 – (2) 40 = 40 minutes.

TABLE 3: Prescriptive noncombustible protection contributions to FRR

Type of Protection	Contribution per Layer (min)
1/2" Type X gypsum board	25
5/8" Type X gypsum board	40

Source: 2021 IBC Table 722.7.1(2)

Structural Fire Resistance of Glulam

The structural fire resistance of exposed structural glulam is commonly found using the calculated fire-resistance methods of NDS Chapter 16 as described previously. Additionally, NDS Section 16.3.4 requires glulam layups primarily used for bending members to have one or more lower grade core laminations replaced with higher grade laminations used at the outermost layers of the glulam. Glulam layups primarily used for bending are non-homogenous, meaning the outer laminations of a beam are stronger and stiffer than those in the interior. This allows cost-effective use of the laminating material. For a fire-resistance-rated beam, this substitution maintains the high-stress grade laminations as the edges of the reduced cross section.

Glulam beams with a 1-hour FRR must substitute one outer tension lamination for a core lamination on the tension side in unbalanced beams, and on both sides for balanced beams. Figure 6 shows an example layup modification made by a glulam manufacturer to provide a beam for a 1-hour unbalanced application. Similarly, beams with a 1.5-hour or 2-hour FRR must substitute two outer tension laminations for two core laminations on the tension side in unbalanced beams, and on both sides for balanced beams. While relatively uncommon, beams with a 30-minute FRR could follow the same substitution as the 1-hour FRR.

Further discussion of this can be found in NDS Commentary C16.3.4.

Where the modified layup is properly specified and provided by the manufacturer, the structural strength checks for the fire condition use the same reference design values from the beam properties used in normal design, which may be found in NDS Supplement Table 5A.

Structural glulam columns typically use homogenous layups with all laminations of the same structural grade. Properties for these can be found in NDS Supplement Table 5B, labeled layups “primarily used in axial tension or compression.” As all the laminations are the same structural grade, no modifications are needed for these layups to be structurally fire-resistance rated.

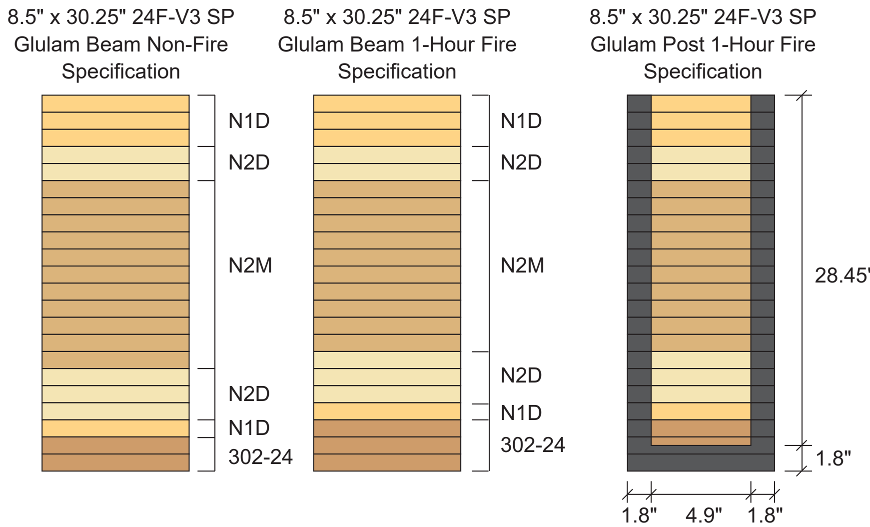

FIGURE 6: Glulam beam layups and calculated post-fire dimensions

For examples showing how to calculate the structural FRR of an exposed glulam element, see Section 5.2.3 of WoodWorks' *Structural Design of Mass Timber Elements: Gravity Design Examples* and TR-10 Examples 1 and 2.

When used as a floor or roof panel in a plank-like orientation, structural glue-laminated timber, is sometimes called GLT. These elements are used in bending in the narrow dimension of the glulam and typically use homogenous layups from the "members stressed primarily in axial tension or compression" table (NDS Supplement Table 5B), even though they are used in bending. As GLT panels are bending in the weak direction of the member, there is little benefit to adding high-strength laminations at the edges of the panels.

Structural FRR of CLT

The FRR of CLT assemblies has been demonstrated on building projects using both tested assemblies and calculation methods.

Calculated Char Depth of CLT Using the NDS Model

The NDS calculation method for char depth in exposed CLT is different than other wood products in that the calculation of the effective char depth of CLT is dependent on both the FRR and lamination thicknesses. The NDS model for char advancing through CLT is based on the one dimensional (1-D) non-linear char rate described previously; however, the char rate resets back to the initial rate when the char reaches the interface between two layers of the CLT.

The char depth, a_{char}, can be calculated using NDS Section 16.2 and effective char depth, a_{eff}, can be calculated using NDS Section 16.3, or pre-calculated and tabulated for constant CLT layer thicknesses and common FRRs as shown in Table 4.

In the NDS model for char depth in CLT, the lamination thickness influences the char depth. For example, a CLT panel with 1-3/8-inch laminations and subject to a 1-hour FRR would have an effective char depth of 1.9 inches. Compare this to the effective char depth of 1.8 inches for sawn lumber, SCL, and glulam.

TABLE 4: Effective char depth for CLT (for β_n = 1.5 inches/hour)

Required Fire Endurance (hrs)	Effective Char Depths, a_{eff} (in.)								
	Lamination thicknesses, h_{lam} (in.)								
	5/8	3/4	7/8	1	1-1/4	1-3/8	1-1/2	1-3/4	2
1	2.2	2.2	2.1	2.0	2.0	1.9	1.8	1.8	1.8
1-1/2	3.4	3.2	3.1	3.0	2.9	2.8	2.8	2.8	2.6
2	4.4	4.3	4.1	4.0	3.9	3.8	3.6	3.6	3.6

Source: NDS Table 16.3.2

1-Hour FRR for 5-Ply, 6-7/8-Inch CLT

Consider a 5-ply, 6-7/8-inch V1 CLT panel with a 1-hour FRR requirement, as shown in Figure 7. Per ANSI/APA PRG 320-2019: Standard for Performance-Rated Cross-Laminated Timber (ANSI/APA, 2019) Table A2, this panel is constructed with 1-3/8-inch-thick laminations. Per Table 4, the effective char depth is 1.9 inches. This effective char depth includes the bottom-most layer (1.375 inches) and 0.525 inches (1.9 to 1.375 inches) of the second-from-bottom layer, leaving 0.85 inches of the second-from-bottom layer unaffected. The net residual thickness of the panel is 4.975 inches comprised of the upper three layers and partial thickness of the second-from-bottom layer.

In this example, the second-from-bottom layer is oriented in the panel's weak direction and provides no strength in the strong direction. Therefore, the remaining structural portion of the 5-ply, 6-7/8-inch CLT panel is limited to the upper three layers. For the V1 5-ply panel, the upper three layers are the same as all the layers of a V1 3-ply, 4-1/8-inch panel, so the design properties of the V1 3-ply, 4-1/8-inch panel can be used directly.

The reference bending capacity of a 3-ply, 4-1/8-inch, V1 panel, is $F_bS_{eff,0}$ = 2,090 lb-ft/ft as found in PRG 320-2019 Table A2. The reference shear capacity of the same panel is $V_{S,0}$ = 1,980 lb/ft.

Calculating the adjusted bending strength of the panel in the structural fire check using the adjustment factor for fire design from NDS Table 16.3.3 results in:

$$F_bS_{eff,0,fire} = 2.85\ (2{,}090\ \text{lb-ft/ft}) = 5{,}957\ \text{lb-ft/ft}$$

This adjusted bending strength of the panel in structural fire check is greater than the ASD design bending value of 4,800 lb-ft/ft for the full 5-ply panel. This is a common result where the 1-hour FRR adjusted bending strength of the 5-ply, 6-7/8-inch CLT panel is higher than the ASD design bending value of the full panel.

Calculating the adjusted shear strength of the panel in the structural fire check, again using the adjustment factor for fire design from Table 16.3.3, results in:

$$V_{S,0,fire} = 2.75\ (1{,}980\ \text{lb-ft}) = 5{,}445\ \text{lb-ft}$$

This adjusted shear strength of the panel in the structural fire check is also stronger than the ASD design shear value of 3,300 lb-ft/ft for the full 5-ply panel.

In horizontal applications where the ceilings of CLT floors and roofs are exposed with a 1-hour FRR, it is common to use 5-ply, 6-7/8-inch CLT. In this scenario, the bending and shear strength of the CLT in the structural fire check condition do not commonly control the allowable panel spans.

Notice that, in this structural fire condition check, we do not check deflections of the reduced-depth panel. The ASTM E119 fire-resistance test and NDS Chapter 16 structural capacity checks do not include any limits on deflection for fire events. Examining the $EI_{eff,f,0}$ of the 3-ply vs. 5-ply V1 CLT, the deflections in the fire condition will be approximately four times the deflection of the full panel under the same load. Provided the structural configuration isn't exceptionally sensitive to such deflection in a way that would cause structural failure, such deflections are deemed acceptable.

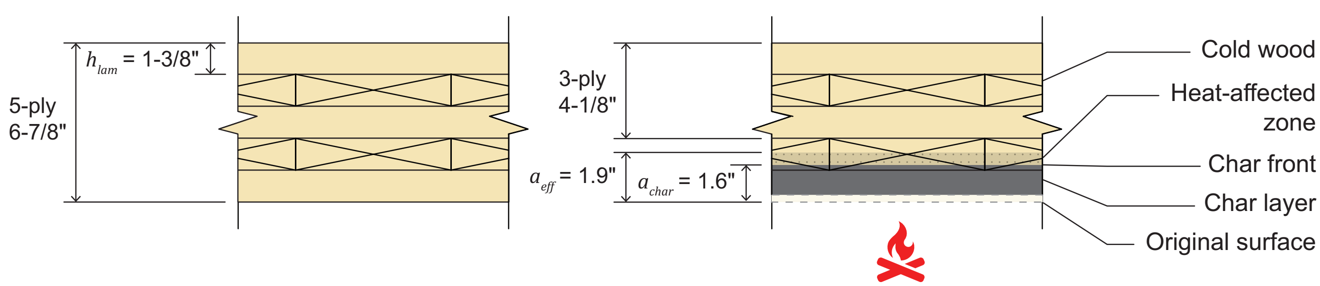

FIGURE 7: Calculated char depth on 5-ply CLT for 1-hour FRR

Other Calculated CLT Fire Design Conditions

Other calculated fire design cases for CLT are not as straightforward. In lightly loaded conditions, 3-ply, 4-1/8-inch CLT floors can be designed to have a 1-hour structural FRR. In this scenario, the structural capacity is provided by 2.23 inches of timber, which leaves only one structural layer spanning in the primary strength direction. In Type IV-B and IV-C construction, it is common to have a 2-hour FRR for the CLT floor panels. The a_{eff} for a 2-hour FRR for CLT made from 1-3/8-inch laminations is 3.8 inches. For 5-ply, 6-7/8-inch CLT the 2-hour FRR calculations show 3.08 inches as structurally available, which is two full layers, plus 0.33 inches of a third layer. In both cases, there are no full-depth CLT panels matching the net sections for the fire cases, and the reference design values of the net sections need to be calculated from the lamination properties.

While PRG 320-2019 is referenced in the 2024 IBC, Appendix X3 of PRG 320-2025 contains an engineering model to help when applying the shear analogy method to asymmetric/unbalanced panels to calculate reference design values of CLT. Alternatively, the CLT manufacturer may have information on design capacities of their CLT under different FRRs.

When using CLT manufactured from SCL, the procedures are the same as described above (for sawn lumber CLT), modified for the actual layer thicknesses and lamination properties. CLT products made from SCL laminations are commonly known by their brand names, such as Mass Ply Panels (MPP) and VersaWorks VLT.

Tested Fire Resistance of CLT

While the calculation methods of demonstrating fire resistance are often used, there are also many fire-resistance test results of CLT assemblies. WoodWorks' Mass Timber Fire & Acoustic Database can help find an appropriate fire-resistance-tested assembly. For example, the database includes approximately 15 tested 2-hour-rated horizontal floor assemblies with 5-ply CLT, most of which include CLT exposed at the ceiling. Other tested CLT assemblies in the database include 1-hour FRR floors and 1-hour and 2-hour FRR walls.

When a fire-resistance test is used for proving FRR, the following details need to be confirmed:

1. The tested assembly matches that of the proposed project construction.
2. The test was completed with an applied load that is consistent with the proposed project forces.
3. Details such as connections between panels, presence of caulks or taps, connectors, CLT grade and CLT layup are consistent with the proposed project.
4. All fire-resistance test limitations have been reviewed and understood.

Regarding the applied load, many tests are performed using a superimposed load selected to simulate a maximum load condition in the tested assembly (100% ASD D+L). If the test report states a 100% ASD load or unreduced load was applied during the test, then further verification of the project-specific design loads to the tested loading is not required. However, if this is not the case, then further verification of the applicability of the test is needed to verify structural adequacy.

The process to verify the structural adequacy of a loaded fire-resistance test for the project specifics is as follows:

- Find the maximum structural demands on the assembly during the test.
- Calculate the similar demands on the assembly being designed at ASD D+L.
- Verify that the structural demands for the design are not greater than the structural demands induced during the fire test.

As an example, a report for a successful 2-hour test on a 5-ply, 6-7/8-inch-thick V1 grade CLT panel notes the test configuration was a 12-foot clear span and a total applied load, including self-weight of the CLT, of 130 psf.

If the candidate design under consideration has the same span of 12 feet in a simply-supported configuration, then a design load, D+L, of 130 psf maximum is acceptable when using this test report. However, for candidate designs with different spans, the allowable load decreases with longer spans and increases with shorter spans.

Examining the loading and spans of the test under consideration, the maximum flexural demand mid-span, as tested, is calculated as:

$$M_{max,test} = wL^2 / 8 = 130 \text{ psf } (12 \text{ ft})^2 / 8 = 2{,}340 \text{ lb-ft/ft.}$$

For 5-ply, 6-7/8-inch V1 CLT, the reference flexural design capacity is 4,800 lb-ft/ft, so this test used a reduced load of 49% (2,340/4,800) of the flexural capacity.

The peak shear demand on the test configuration at the support is:

$$V_{max,test} = wL / 2 = 130 \text{ psf } (12 \text{ ft}) / 2 = 780 \text{ lb/ft.}$$

For 5-ply, 6-7/8-inch V1 CLT, the reference shear design capacity is 3,300 lb-ft/ft, so this test used a reduced load of 24% (780/3,300) of the shear capacity.

Consequently, acceptable designs using the same 5-ply, 6-7/8-inch V1 CLT and this fire-resistance test are those that keep the peak flexural demand at or less than 2,340 lb-ft/ft and peak shear demand at or less than 780 lb/ft, for the ASD D+L load combination.

Interestingly, the calculated structural fire resistance flexural strength for the same condition is 1,789 lb-ft/ft (37% of the original). The tested flexural capacity in this test is significantly greater than the calculated structural fire capacity. This is not an uncommon result as the calculation methods are inherently conservative. Tested fire results can be useful to achieve a longer span or higher load rating for 2-hour FRR CLT floor and roof assemblies than calculations alone.

While flexural capacity may control many horizontal CLT fire design scenarios, the shear capacity also needs to be verified as acceptable. This is particularly the case in multi-span configurations where the peak shear demand increases in the multi-span configuration compared to single-span configurations. See Section 2.6 of WoodWorks' *Structural Design of Mass Timber Elements: Gravity Design Examples*.

This approach of evaluating the critical limit states of the tested configuration and then using the same limit states for different design scenarios is not specifically discussed within ASTM E119 or NDS Chapter 16 but is consistent with the design approach in the NDS. It is a rational engineering approach to extend a reduced load test to spans and support configurations that have not been tested, and the engineer should use judgment and present the approach to the AHJ. The above calculations and examples only consider bending and shear demands. The designer should also assess other structural design criteria such as bearing at supports, likely using the calculated structural fire-resistance method of the NDS.

Structural FRR of NLT

With the surge of interest in mass timber buildings, it is common to consider nail-laminated timber panels for floors, roofs, and walls. An NLT style of construction is recognized in IBC Section 2304.9.3 under the historical name mechanically-laminated decking. NLT floor and roof panels are typically installed with wood structural panel (WSP) sheathing applied to the top surface of the NLT. Among its purposes, the sheathing serves as a flat sub-floor, providing diaphragm shear capacity, and, for fire performance, a draftstop that prevents air penetration through small gaps between individual laminated pieces of timber in the NLT and at panel joints.

There are several potential methods to calculate the reduced section for the structural FRR of NLT.

Calculation Method 1: NDS Provisions for Lumber Decking

NDS Section 16.3.5 provides a method to calculate the structural fire resistance of NLT using a two dimensional (2-D) char model. This model assumes the full char rate advancing from the exposed side of the NLT, and 33% of the normal char rate advancing from each side of each lamination into the width of the lamination. For a 1-hour rating, this method results in a calculated reduction in net section of the lamination of a_{eff} of 1.8 inches from the exposed side and 0.6 inch into each side of each lamination. For typical 2-by nominal lumber laminations, with a 1.5-inch actual width, this method only leaves 1.5 – (2) 0.6 = 0.3 in. width of structural material as shown in Figure 8.

Using this calculated reduction in the net section for structural fire resistance, the assumed structural material is quite small for a 1-hour FRR. When including the stress increase factors in NDS Table 16.3.3, NLT made from full-size dimensional lumber has 1-hour structural bending strength of about 13%, 25%, and 32% for 2x4, 2x6, and 2x8 laminations, respectively. NLT from 2-by lumber laminations cannot achieve a 2-hour FRR under this simplified design method.

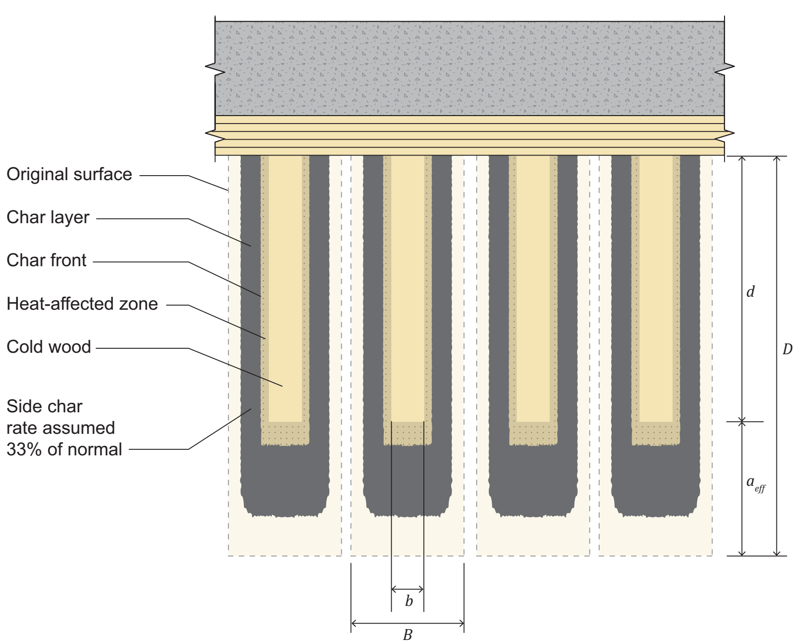

FIGURE 8: Reduced NLT section using 100%/33% char model

Calculation Method 2: FDS Char Penetration at Abutting Edges

FDS Section 3.3.3.1 recognizes method 1 above as an option for NLT, as well as the validity of FDS Section 3.2.3 regarding char penetration at intersections and abutting edges. Most NLT panels will meet the requirements of FDS 3.2.3.2—namely that air flow does not occur between the individual laminations of NLT and that gaps between individual laminations are not greater than 1/8 inch. WSP sheathing that is at least 3/8-inch thick on the unexposed side of an NLT panel qualifies as draftstopping in FDS Section 2.5.3.1. When the individual laminations in NLT are touch-tight at the time of manufacturing, gaps between them will generally be less than 1/8 inch.

Applying FDS Section 3.2.3.2 to the structural fire design of NLT consists of using the main char depth on the exposed side of the NLT, progressing the normal a_{eff} distance from the exposed edge, with the char front penetrating into the abutting edges to a char penetration of 2 a_{char}. To assess the structural impact of the char penetration, the loss of net section of the NLT is to be determined with important dimensional information provided in FDS Commentary Figure C3-7, recreated in Figure 9 with additional dimensional information.

Applying this char model to NLT creates a non-rectangular net section, similar to that shown in Figure 10. When running through the net section analysis of this char pattern applied to 2-by sawn laminations and including the 2.85 bending stress increase factor of NDS Table 16.3.3, NLT made from full-size dimensional 2-by lumber has a 1-hour FRR structural bending strength of around 24% and 73% of the original for 2x4 and 2x6 laminations, respectively. Similarly, NLT made from 2x8 laminations has a 1-hour FRR structural bending strength of 100% of the original 2x8 laminations. In 2-hour structural FRR calculations, NLT made from 2x8 laminations retains about 28% moment capacity.

Compared to method 1, method 2 results in much larger calculated structural fire capacities for NLT. Method 1, originally developed from tests of shallow decking, is very conservative when applied to the deep NLT configurations.

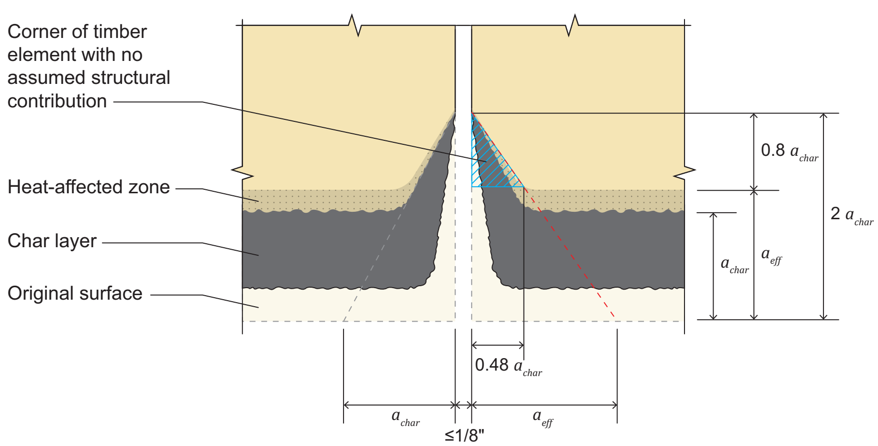

FIGURE 9: Net section impact of accelerated char on abutting edge based on FDS Figure C3-7

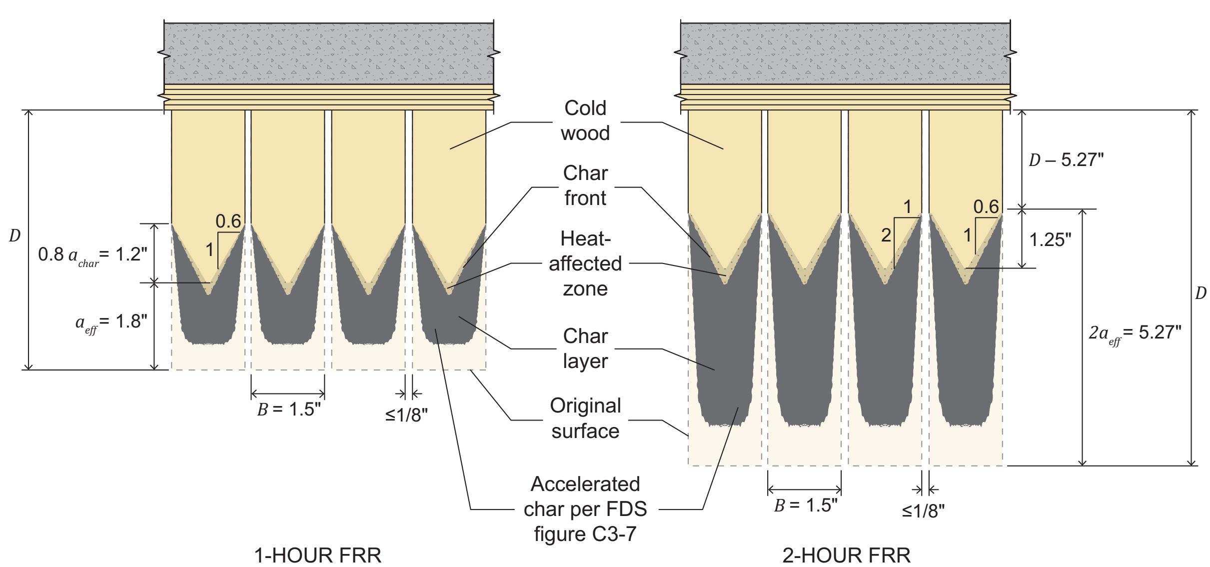

FIGURE 10: NLT char depth using FDS accelerated char at abutting edges model

Calculation Method 3: One-Dimensional Char Model

While not recognized in the NDS or FDS, there are projects that have used a 1-D char model, treating NLT panels as single pieces of timber. The *Nail Laminated Timber U.S. Design and Construction Guide* (BSLC, 2017) states that, when NLT has a monolithic topping layer (e.g., concrete or gypcrete) on top of the WSP sheathing, the charring is primarily 1-D. Recent research reported in *Evaluating Fire Performance of Nail-Laminated Timber: Influence of Gaps* (Ranger & Dagenais, 2019) quantified the influence of gap sizes on char rates through NLT panels. Researchers built NLT panels with varying gap sizes, which were instrumented and exposed to the standard fire for 1 hour and 30 minutes. Based on this and previous research, the 1-D char model of solid timber might be reasonably accurate for structural FRR calculations of NLT when the panels are manufactured tight with dry lumber; kept dry during transportation, construction, and use; and topped with a continuous draftstop material. Because of the challenges in achieving these conditions, the use of a 1-D char model is not recognized in the NDS or FDS for NLT and should be used only with stringent quality and construction control and approval of the AHJ.

Fire-Resistance-Tested NLT Assemblies

The Mass Timber Fire & Acoustic Database includes several NLT assemblies with successful fire-resistance tests. When using test results for loading and span conditions different than those tested, the procedures described above can be followed to verify the structural adequacy of the proposed design.

Lumber Decking

Multiple types of lumber decking are recognized in IBC Section 2304.9, and tongue-and-groove decking is recognized for use in heavy timber construction in Section 2304.11. When a fire-resistance rating is required, a calculated structural FRR for square-edged decking can be established following NDS Section 16.3.5, using the 100%/33% char depth method described in method 1 above, or the 2-D char penetration model described in method 2. Per NDS Section 16.3.5a, tongue-and-groove decking is permitted to be designed using a 1-D char rate with no char penetration between the individual pieces of decking.

Structural Composite Lumber

SCL includes a variety of engineered wood products, including laminated veneer lumber (LVL), laminated strand lumber (LSL), and parallel strand lumber (PSL). These products have been used for years in both residential and commercial light-frame wood construction, but they also have significant applications in mass timber buildings.

SCL products are recognized as accepted building materials in IBC Section 2303.1.10 when manufactured in accordance with ASTM D5456-21e1 Standard Specification for Evaluation of Structural Composite Lumber Products (ASTM, 2021). As noted in NDS Section 16.2.1, the nominal char rate of 1.5 inches per hour used for sawn lumber and glulam members is also applicable to SCL members.

Connections Between Mass Timber Panels

In assemblies that need to provide fire containment, evaluation of burn-through protection and thermal separation criteria is required. The critical locations for meeting these criteria are often the connections between panels in mass timber assemblies. Similar to structural FRRs, calculation methods can be used to justify connection performance. Calculation methods for evaluating burn-through and thermal separation in the FDS are appropriate for many mass timber connections.

FDS Section 3.6 provides a method to calculate thermal separation through a wood element (3.6.1.1) and around the wood element at intersections and abutting edges (3.6.1.2). Section 3.7 provides direction on the calculation of burn-through time. At abutting edges of wood elements, Section 3.7 points back to Section 3.6.1.2, which refers to Section 3.2.3 for char penetration at intersections and abutting edges. If calculations following Section 3.2.3 show that the char penetration at abutting edges of wood members does not fully penetrate the assembly to the unexposed side, the thermal separation and burn-through resistance requirements are met at the abutting edges. Fire-resistance tests can also be used to justify an FRR for a connection, where the connection is part of the tested assembly.

Conclusion

Mass timber is uniquely positioned as a structural building material that can be left exposed while also achieving an FRR. The IBC provides multiple commonly used methods of demonstrating FRRs through testing and analytical provisions, and engineers should keep each of these options open until they determine which will be more advantageous based on the specific FRR requirements, member sizes, spans, and loading conditions.

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