



AMERICAN WOOD COUNCIL

MANUAL

for Engineered Wood Construction
2015 EDITION

Updates and Errata

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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.



AMERICAN WOOD COUNCIL

MANUAL

**for Engineered Wood Construction
2015 EDITION**

Manual for Engineered Wood Construction 2015 Edition

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FOREWORD

This *Manual for Engineered Wood Construction (Manual)* provides guidance for design of most wood-based structural products used in the construction of wood buildings. This *Manual* is intended for use with the *Wood Design Package* which includes the following documents:

- *ANSI/AWC NDS-2015 National Design Specification® (NDS®) for Wood Construction* – with Commentary,
- *NDS Supplement – Design Values for Wood Construction, 2015 Edition*,
- *ANSI/AWC SDPWS-2015 – Special Design Provisions for Wind and Seismic (SDPWS)* – with Commentary,

The American Wood Council (AWC) has developed this manual for design professionals. AWC and its predecessor organizations have provided engineering design information to users of structural wood products for over

75 years, first in the form of the *Wood Structural Design Data* series and then in the *National Design Specification (NDS) for Wood Construction*.

It is intended that this document be used in conjunction with competent engineering design, accurate fabrication, and adequate supervision of construction. AWC does not assume any responsibility for errors or omissions in the document, nor for engineering designs, plans, or construction prepared from it.

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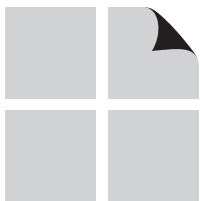


TABLE OF CONTENTS

Part/Title	Page	Part/Title	Page
M1: GENERAL REQUIREMENTS FOR STRUCTURAL DESIGN	1	M7: PREFABRICATED WOOD I-JOISTS	27
M1.1 Products Covered in This Manual		M7.1 General	
M1.2 General Requirements		M7.2 Reference Design Values	
M1.3 Design Procedures		M7.3 Adjustment of Reference Design Values	
M2: DESIGN VALUES FOR STRUCTURAL MEMBERS.....	3	M7.4 Special Design Considerations	
M2.1 General Information		M8: STRUCTURAL COMPOSITE LUMBER..	43
M2.2 Reference Design Values		M8.1 General	
M2.3 Adjustment of Reference Design Values		M8.2 Reference Design Values	
M3: DESIGN PROVISIONS AND EQUATIONS.....	5	M8.3 Adjustment of Reference Design Values	
M3.1 General		M8.4 Special Design Considerations	
M3.2 Bending Members - General		M9: WOOD STRUCTURAL PANELS	49
M3.3 Bending Members - Flexure		M9.1 General	
M3.4 Bending Members - Shear		M9.2 Reference Design Values	
M3.5 Bending Members - Deflection		M9.3 Adjustment of Reference Design Values	
M3.6 Compression Members		M9.4 Design Considerations	
M3.7 Solid Columns		M10: CROSS-LAMINATED TIMBER	59
M3.8 Tension Members		M10.1 General	
M3.9 Combined Bending and Axial Loading		M10.2 Reference Design Values	
M3.10 Design for Bearing		M10.3 Adjustment of Reference Design Values	
M4: SAWN LUMBER.....	11	M10.4 Design Considerations	
M4.1 General		M11: MECHANICAL CONNECTIONS	65
M4.2 Reference Design Values		M11.1 General	
M4.3 Adjustment of Reference Design Values		M11.2 Reference Design Values	
M4.4 Special Design Considerations		M11.3 Adjustment of Reference Design Values	
M5: STRUCTURAL GLUED LAMINATED TIMBER	17	M11.4 Typical Connection Details	
M5.1 General		M11.5 Pre-Engineered Metal Connectors	
M5.2 Reference Design Values		M12: DOWEL-TYPE FASTENERS.....	81
M5.3 Adjustment of Reference Design Values		M12.1 General	
M5.4 Special Design Considerations		M12.2 Reference Withdrawal Design Values	
M6: ROUND TIMBER POLES AND PILES... 	23	M12.3 Reference Lateral Design Values	
M6.1 General		M12.4 Combined Lateral and Withdrawal Loads	
M6.2 Reference Design Values		M12.5 Adjustment of Reference Design Values	
M6.3 Adjustment of Reference Design Values		M12.6 Multiple Fasteners	
M6.4 Special Design Considerations			

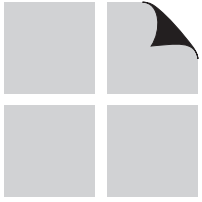
Part/Title	Page	Part/Title	Page
M13: SPLIT RING AND SHEAR PLATE CONNECTORS.....	85	M15: SPECIAL LOADING CONDITIONS	89
M13.1 General		M15.1 Lateral Distribution of Concentrated Loads	
M13.2 Reference Design Values		M15.2 Spaced Columns	
M13.3 Placement of Split Ring and Shear Plate Connectors		M15.3 Built-Up Columns	
M14: TIMBER RIVETS	87	M15.4 Wood Columns with Side Loads and Eccentricity	
M14.1 General		M16: FIRE DESIGN	91
M14.2 Reference Design Values		M16.1 General	
M14.3 Placement of Timber Rivets		M16.2 Design Procedures for Exposed Wood Members	
		M16.3 Wood Connections	

LIST OF TABLES

M4.3-1	Applicability of Adjustment Factors for Sawn Lumber	13	M9.2-4	Wood Structural Panel Rigidity and Shear Through-the-Thickness Capacities.....	55
M4.4-1	Approximate Moisture and Thermal Dimensional Changes.....	14	M9.3-1	Applicability of Adjustment Factors for Wood Structural Panels	56
M4.4-2	Coefficient of Moisture Expansion, e_{ME} , and Fiber Saturation Point, FSP, for Solid Woods.....	15	M9.4-1	Panel Edge Support	57
M4.4-3	Coefficient of Thermal Expansion, e_{TE} , for Solid Woods	16	M9.4-2	Minimum Nailing for Wood Structural Panel Applications.....	58
M5.1-1	Economical Spans for Structural Glued Laminated Timber Framing Systems	19	M10.2-1	Allowable Design Properties for the Parallel Layers of PRG 320 CLT (for use in the U.S.)	60
M5.3-1	Applicability of Adjustment Factors for Structural Glued Laminated Timber.....	21	M10.2-2	Allowable Bending Capacities for CLT Grades (for use in the U.S.).....	61
M5.4-1	Average Specific Gravity and Weight Factor	22	M10.3-1	Applicability of Adjustment Factors for Cross-Laminated Timber.....	63
M6.3-1	Applicability of Adjustment Factors for Round Timber Poles and Piles	25	M11.3-1	Applicability of Adjustment Factors for Mechanical Connections	67
M7.3-1	Applicability of Adjustment Factors for Prefabricated Wood I-Joists	30	M12.3-1	Applicability of Adjustment Factors for Dowel-Type Fasteners.....	83
M8.3-1	Applicability of Adjustment Factors for Structural Composite Lumber.....	46	M13.2-1	Applicability of Adjustment Factors for Split Ring and Shear Plate Connectors	86
M9.1-1	Guide to Panel Use.....	51	M14.2-1	Applicability of Adjustment Factors for Timber Rivets.....	88
M9.2-1	Wood Structural Panel Bending Stiffness and Strength Capacities.....	52	M16.1-1	Heavy Timber Size Equalivalencies.....	92
M9.2-2	Wood Structural Panel Axial Stiffness, Tension, and Compression Capacities.....	53	M16.1-2	Minimum Sizes to Qualify as Heavy Timber Construction	93
M9.2-3	Wood Structural Panel Shear-in-the-Plane Capacities	55	M16.1-3	Privacy Afforded According to STC Rating.....	99

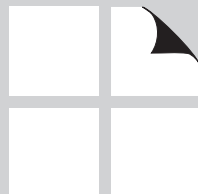
LIST OF FIGURES

M5.1-1	Unbalanced and Balanced Layup Combinations	18	M9.2-4	Shear-in-the-Plane for Wood Structural Panels	54
M5.2-1	Structural Glued Laminated Timber Axes Orientations	20	M9.2-5	Through-the-Thickness Shear for Wood Structural Panels.....	54
M7.4-1	Design Span Determination	31	M10.2-1	Example of CLT Panel in Bending.....	62
M7.4-2	Load Case Evaluations.....	33	M10.2-2	CLT Panel in Bending with Axial Compression Load in the Plane of the Panel.....	62
M7.4-3	End Bearing Web Stiffeners (Bearing Block).....	35	M10.2-3	Shear-in-the-Plane for CLT Panels.....	63
M7.4-4	Web Stiffener Bearing Interface	36	M10.2-4	Through-the-Thickness Shear for CLT Panels	63
M7.4-5	Beveled End Cut	36	M16.1-1	Cross Sections of Possible One-Hour Area Separations	98
M7.4-6	Sloped Bearing Conditions (Low End).....	37	M16.3-1	Beam to Column Connection - Connection Not Exposed to Fire	101
M7.4-7	Sloped Bearing Conditions (High End)	38	M16.3-2	Beam to Column Connection - Connection Exposed to Fire Where Appearance is a Factor	101
M7.4-8	Lateral Support Requirements for Joists in Hangers	39	M16.3-3	Beam to Column Connection - Connection Exposed to Fire Where Appearance is Not a Factor	101
M7.4-9	Top Flange Hanger Support	39	M16.3-4	Column Connections Covered	102
M7.4-10	Connection Requirements for Face Nail Hangers	40	M16.3-5	Beam to Girder - Concealed Connection	102
M7.4-11	Details for Vertical Load Transfer	41			
M9.2-1	Structural Panel with Strength Direction Across Supports	50			
M9.2-2	Example of Structural Panel in Bending....	51			
M9.2-3	Structural Panel with Axial Compression Load in the Plane of the Panel.....	54			



M1: GENERAL REQUIREMENTS FOR STRUCTURAL DESIGN

M1.1	Products Covered in This Manual	2
M1.2	General Requirements	2
	M1.2.1 Bracing	2
M1.3	Design Procedures	2



M1.1 Products Covered in This Manual

This Manual was developed with the intention of covering all structural applications of wood-based products and their connections that meet the requirements of the referenced standards. The Manual is a dual format document incorporating design provisions for both allowable stress design (ASD) and load and resistance factor design (LRFD). Design information is available for the following list of products. Each product chapter contains information for use with this Manual and the *National Design Specification® (NDS®) for Wood Construction*. Chapters are organized to parallel the chapter format of the *NDS*.

- Sawn Lumber Chapter 4
- Structural Glued Laminated Timber Chapter 5
- Round Timber Poles and Piles Chapter 6

- Prefabricated Wood I-Joists Chapter 7
- Structural Composite Lumber Chapter 8
- Wood Structural Panels Chapter 9
- Cross-Laminated Timber Chapter 10
- Mechanical Connections Chapter 11
- Dowel-Type Fasteners Chapter 12
- Split Ring and Shear Plate Connectors Chapter 13
- Timber Rivets Chapter 14

An additional Standard, titled *Special Design Provisions for Wind and Seismic (SDPWS)*, has been developed to cover materials, design, and construction of wood members, fasteners, and assemblies to resist wind and seismic forces.

M1.2 General Requirements

This Manual is organized for use with the Wood Design Package which includes the following documents:

- *NDS and Commentary*;
- *NDS Supplement: Design Values for Wood Construction*,
- *Special Design Provisions for Wind and Seismic (SDPWS) and Commentary*,

M1.2.1 Bracing

Design considerations related to both temporary and permanent bracing differ among product types. Specific discussion of bracing is included in the product chapter.

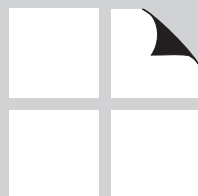
M1.3 Design Procedures

The *NDS* is a dual format specification incorporating design provisions for ASD and LRFD. Behavioral equations, such as those for member and connection design, are the same for both ASD and LRFD. Adjustment factor tables include applicable factors for determining an adjusted ASD design value or an adjusted LRFD design value. *NDS* Appendix N – (Mandatory) Load and Resistance Factor Design (LRFD) and product chapters outline requirements that are unique to LRFD and adjustment factors for LRFD.

The basic design equations for ASD or LRFD require that the specified product adjusted design value meet or exceed the actual (applied) stress or other effect imposed by the specified loads. Load combination equations for use with ASD and LRFD are given in the model building codes.

M2: DESIGN VALUES FOR STRUCTURAL MEMBERS

M2.1	General Information	4
M2.2	Reference Design Values	4
M2.3	Adjustment of Design Values	4



M2.1 General Information

Structural wood products are provided to serve a wide range of end uses. Some products are marketed through commodity channels where the products meet specific standards and the selection of the appropriate product is the responsibility of the user.

Other products are custom manufactured to meet the specific needs of a given project. Products in this category are metal plate connected wood trusses and custom structural glued laminated timbers. Design of the individual

members is based on criteria specified by the architect or engineer of record on the project. Manufacture of these products is performed in accordance with the product's manufacturing standards. Engineering of these products normally only extends to the design of the products themselves. Construction-related issues, such as load path analysis and erection bracing, remain the responsibility of the professional of record for the project.

M2.2 Reference Design Values

Reference design value designates the allowable stress design value based on normal load duration. To avoid confusion, the descriptor "reference" is used and serves as a reminder that design value adjustment factors are applicable for design values in accordance with referenced conditions specified in the *NDS* – such as normal load duration.

Reference design values for sawn lumber, structural glued laminated timber, and round timber poles and piles are contained in the *NDS Supplement: Design Values for Wood Construction*. Reference design values for dowel-type fasteners, split ring and shear plate connectors, and timber rivets are contained in the *NDS*. Reference design values for all other products are typically contained in the manufacturer's code evaluation report.

M2.3 Adjustment of Reference Design Values

Adjusted design value designates reference design values which have been multiplied by adjustment factors. Basic requirements for design use terminology applicable to both ASD and LRFD. In equation format, this takes the standard form $f_b \leq F_b'$ which is applicable to either ASD or LRFD. Reference design values ($F_b, F_t, F_v, F_c, F_{c\perp}, E, E_{min}$) are multiplied by adjustment factors to determine adjusted design values ($F_b', F_t', F_v', F_c', F_{c\perp}', E', E_{min}'$).

Reference conditions have been defined such that a majority of wood products used in interior or in protected environments will require no adjustment for moisture, temperature, or treatment effects.

Moisture content (MC) reference conditions are 19% or less for sawn lumber products. The equivalent limit for glued products (structural glued laminated timber, structural composite lumber, prefabricated wood I-joists, wood structural panels, and cross-laminated timber) is defined as 16% MC or less.

Temperature reference conditions include sustained temperatures up to 100°F. Note that it has been traditionally assumed that these reference conditions also include common building applications in desert locations where daytime temperatures will often exceed 100°F. Examples

of applications that may exceed the reference temperature range include food processing or other industrial buildings.

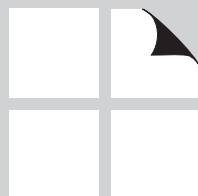
Tabulated design values and capacities are for untreated members. Tabulated design values and capacities also apply to wood products pressure treated by an approved process and preservative except as specified for load duration factors.

An unincised reference condition is assumed. For members that are incised to increase penetration of preservative chemicals, use the incising adjustment factors given in the product chapter.

The effects of fire retardant chemical treatment on strength shall be accounted for in the design. Reference design values, including connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service. The impact load duration factor shall not apply to structural members pressure-treated with fire retardant chemicals.

M3: DESIGN PROVISIONS AND EQUATIONS

M3.1	General	6
M3.2	Bending Members - General	6
M3.3	Bending Members - Flexure	6
M3.4	Bending Members - Shear	6
M3.5	Bending Members - Deflection	7
M3.6	Compression Members	7
M3.7	Solid Columns	8
M3.8	Tension Members	8
	M3.8.1 Tension Parallel to Grain	8
	M3.8.2 Tension Perpendicular to Grain	8
M3.9	Combined Bending and Axial Loading	8
M3.10	Design for Bearing	9



M3.1 General

This Chapter covers design of members for bending, compression, tension, combined bending and axial loads, and bearing.

M3.2 Bending Members - General

This section covers design of members stressed primarily in flexure (bending). Examples of such members include primary framing members (beams) and secondary framing members (purlins, joists). Products commonly used in these applications include glulam, solid sawn lumber, structural composite lumber, and prefabricated I-joists.

Bending members are designed so that no design capacity is exceeded under applied loads. Strength criteria for bending members include bending moment, shear, local buckling, lateral torsional buckling, and bearing.

See specific product chapters for moment and shear capacities (joist and beam selection tables) and reference bending and shear design values.

M3.3 Bending Members - Flexure

The basic equation for moment design of bending members is:

$$M' \geq M \quad (\text{M3.3-1})$$

where:

M' = adjusted moment capacity

M = bending moment

The equation for calculation of adjusted moment capacity is:

$$M' = F_b' S \quad (\text{M3.3-2})$$

where:

S = section modulus, in.³

F_b' = adjusted bending design value, psi. See product chapters for applicable adjustment factors.

M3.4 Bending Members - Shear

The basic equation for shear design of bending members is:

$$V' \geq V \quad (\text{M3.4-1})$$

where:

V' = adjusted shear capacity parallel to grain, lbs

V = shear force, lbs

The equation for calculation of shear capacity is:

$$V' = F_v' I_b/Q \quad (\text{M3.4-2})$$

which, for rectangular unnotched bending members, reduces to:

$$V' = 2/3 (F_v') A \quad (\text{M3.4-3})$$

where:

I = moment of inertia, in.⁴

A = area, in.²

F_v' = adjusted shear design value, psi. See product chapters for applicable adjustment factors.

M3.5 Bending Members - Deflection

Users should note that design of bending members is often controlled by serviceability limitations rather than strength. Serviceability considerations, such as deflection and vibration, are often designated by the authority having jurisdiction.

For a simple span uniformly loaded rectangular member, the equation used for calculating mid-span deflection is:

$$\Delta = \frac{5w\ell^4}{384EI} \quad (\text{M3.5-1})$$

where:

Δ = deflection, in.

w = uniform load, lb/in.

ℓ = span, in.

EI = stiffness of beam section, lb-in.²

Values of modulus of elasticity, E , and moment of inertia, I , for lumber and structural glued laminated timber for use in the preceding equation can be found in the *NDS Supplement*. Engineered wood products such as I-joists and structural composite lumber will have EI values published in individual manufacturer's product literature or evaluation reports. Some manufacturers might publish "true" E which would require additional computations to account for shear deflection. See *NDS* Appendix F for information on shear deflection. See product chapters for more details about deflection calculations.

M3.6 Compression Members

This section covers design of members stressed primarily in compression parallel to grain. Examples of such members include columns, truss members, and diaphragm chords.

Information in this section is limited to the case in which loads are applied concentrically to the member. Provisions of *NDS* 3.9 or *NDS* Chapter 15 should be used if loads are eccentric or if the compressive forces are applied in addition to bending forces.

The *NDS* differentiates between solid, built-up, and spaced columns. In this context built-up columns are assembled from multiple pieces of similar members connected in accordance with *NDS* 15.3.

A spaced column must comply with provisions of *NDS* 15.2. Note that this definition includes main column elements, spacer blocks with their connectors, and end blocks with shear plate or split ring connectors.

Compression Parallel to Grain

The basic equation for design of compression members is:

$$P' \geq P \quad (\text{M3.6-1})$$

where:

P' = adjusted compression parallel to grain capacity, lbs

P = compressive force, lbs

The complete equation for calculation of adjusted compression capacity is:

$$P' = F_c' A \quad (\text{M3.6-2})$$

where:

A = area, in.²

F_c' = adjusted compression parallel to grain design value, psi. See product chapters for applicable adjustment factors.

Special Considerations

Net Section Calculation

As in design of tension members, compression members should be checked both on a gross section and a net section basis (see *NDS* 3.6.3).

Bearing Capacity Checks

Design for bearing is addressed in *NDS* 3.10.

Radial Compression in Curved Members

Stresses induced in curved members under load include a component of stress in the direction of the radius of curvature. Radial compression is a specialized design consideration that is addressed in *NDS* 5.4.1.

M3.7 Solid Columns

Slenderness Considerations and Stability

The user is cautioned that stability calculations are highly dependent upon boundary conditions assumed in the analysis. For example, the common assumption of a pinned-pinned column is only accurate or conservative if

the member is restrained against sidesway. If sidesway is possible and a pinned-free condition exists, the value of K_c in *NDS* 3.7.1.2 doubles (see *NDS* Appendix Table G1 for recommended buckling length coefficients, K_c) and the computed adjusted compression parallel to grain capacity decreases.

M3.8 Tension Members

This section covers design of members stressed primarily in tension parallel to grain. Examples of such members include shear wall end posts, truss members, and diaphragm chords.

The designer is advised that use of wood members in applications that induce tension perpendicular to grain stresses should be avoided.

M3.8.1 Tension Parallel to Grain

The basic equation for design of tension members is:

$$T' \geq T \quad (\text{M3.8-1})$$

where:

T' = adjusted tension parallel to grain capacity, lbs

T = tensile force, lbs

The equation for calculation of adjusted tension capacity is:

$$T' = F_t' A \quad (\text{M3.8-2})$$

where:

A = area, in.²

F_t' = adjusted tension design value, psi. See product chapters for applicable adjustment factors.

Net Section Calculation

Design of tension members is often controlled by the ability to provide connections to develop tensile forces within the member. In the area of connections, one must design not only the connection itself (described in detail in Chapter M11) but also the transfer of force across the net section of the member. One method for determining these stresses is provided in *NDS* Appendix E.

M3.8.2 Tension Perpendicular to Grain

Radial Stress in Curved Members

Stresses induced in curved members under load include a component of stress in the direction of the radius of curvature. This stress is traditionally called radial tension. Radial stress design is a specialized consideration that is covered in *NDS* 5.4.1.

M3.9 Combined Bending and Axial Loading

This section covers design of members stressed under combined bending and axial loads. The applicable strength criteria for these members is explicit in the *NDS* equations – limiting the sum of various stress ratios to less than or equal to unity.

Bending and Axial Tension

For designs in which the axial load is in tension rather than compression, the designer should use *NDS* Equations 3.9-1 and 3.9-2.

Bending and Axial Compression

The equation for design of members under bending plus compression loads is given below in terms of load and moment ratios:

$$\left(\frac{P}{P'}\right)^2 + \frac{M_1}{M_1' \left(1 - \frac{P}{P_{E1}}\right)} + \frac{M_2}{M_2' \left[1 - \frac{P}{P_{E2}} - \left(\frac{M_1}{M_E}\right)^2\right]} \leq 1.0 \quad (\text{M3.9-1})$$

where:

- P' = adjusted compression capacity determined per M3.6, lbs
- P = compressive force determined per M3.6, lbs
- M_1' = adjusted moment capacity (strong axis) determined per M3.3, in.-lbs
- M_1 = bending moment (strong axis) determined per M3.3, in.-lbs
- M_2' = adjusted moment capacity (weak axis) determined per M3.3, in.-lbs
- M_2 = bending moment (weak axis) determined per M3.3, in.-lbs
- P_{E1} = $F_{cE1} A$ = critical column buckling capacity (strong axis) determined per NDS 3.9.2, lbs
- P_{E2} = $F_{cE2} A$ = critical column buckling capacity (weak axis) determined per NDS 3.9.2, lbs
- M_E = $F_{bE} S$ = critical beam buckling capacity determined per NDS 3.9.2, in.-lbs

Members must be designed by multiplying all applicable adjustment factors by the reference design values for the product. See M3.3 and M3.6 for discussion of applicable adjustment factors for bending or compression, respectively.

Equation 3.9-4 in the *NDS* is used to check the intermediate calculation for members subjected to flatwise bending in combination with axial compression, with or without edgewise bending. When a flatwise bending load is checked with the third term of the stress interaction equation (*NDS* Equation 3.9-3), the axial and edgewise bending interaction in the denominator can become a negative value. The occurrence of the negative value indicates an overstress. However, use of this negative term in the stress interaction equation (*NDS* Equation 3.9-3) overlooks the overstress in flatwise bending and incorrectly reduces the overall interaction.

Design Techniques

A key to understanding design of members under combined bending and axial loads is that components of the design equation are simple ratios of compressive force (or moment) to compression capacity (or moment capacity). Note that the compression term in this equation is squared. This is the result of empirical test data. Moderate compressive forces do not have as large an impact on capacity (under combined loads) as previously thought. It is believed that this is the result of compressive “reinforcing” of what would otherwise be a tensile failure mode in bending.

M3.10 Design for Bearing

Columns often transfer large forces within a structural system. While satisfaction of column strength criteria is usually the primary concern, the designer should also check force transfer at the column bearing.

For cases in which the column is bearing on another wood member, especially if bearing is perpendicular to grain, this calculation will often control the design.

The basic equation for bearing design is:

$$R' \geq R \quad (\text{M3.10-1})$$

where:

- R' = adjusted compression perpendicular to grain capacity, lbs
- R = compressive force or reaction, lbs

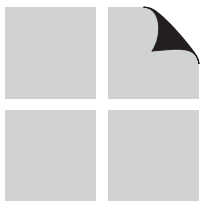
The equation for calculation of adjusted compression perpendicular to grain capacity is:

$$R' = F_{c\perp}' A \quad (\text{M3.10-2})$$

where:

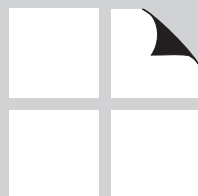
A = area, in.²

$F_{c\perp}'$ = adjusted compression perpendicular to grain design value, psi. See product chapters for applicable adjustment factors.



M4: SAWN LUMBER

M4.1	General	12
M4.2	Reference Design Values	12
M4.3	Adjustment of Reference Design Values	13
M4.4	Special Design Considerations	14



M4.1 General

Product Information

Structural lumber products are well-known throughout the construction industry. The economic advantages of lumber often dictate its choice as a preferred building material.

Lumber is available in a wide range of species, grades, sizes, and moisture contents. Structural lumber products are typically specified by either the stress level required or by the species, grade, and size required.

This Chapter provides information for designing structural lumber products in accordance with the *NDS*.

Common Uses

Structural lumber and timbers have been a primary construction material throughout the world for many centuries. They are the most widely used framing material for housing in North America.

In addition to use in housing, structural lumber finds broad use in commercial and industrial construction. Its high strength, universal availability, and cost saving attributes make it a viable option in most low- and mid-rise construction projects.

Structural lumber is used as beams, columns, headers, joists, rafters, studs, and plates in conventional construction. In addition to its use in lumber form, structural lumber is used to manufacture structural glued laminated beams, trusses, wood I-joists, and cross-laminated timber.

Availability

Structural lumber is a widely available construction material. However, to efficiently specify structural lumber for individual construction projects, the specifier should be aware of the species, grades, and sizes available locally. The best source of this information is your local lumber supplier.

M4.2 Reference Design Values

General

The *NDS Supplement* provides reference design values for design of sawn lumber members. These design values are used when manual calculation of member capacity is required and must be used in conjunction with the adjustment factors specified in *NDS* 4.3.

Reference Design Values

Reference design values are provided in the *NDS Supplement* as follows:

<i>NDS Supplement</i>	
Table Number	
4A and 4B	Visually graded dimension lumber
4C	Mechanically graded dimension lumber
4D	Visually graded timbers
4E	Visually graded decking
4F	Non-North American visually graded dimension lumber

M4.3 Adjustment of Reference Design Values

To generate member design capacities, reference design values for sawn lumber are multiplied by adjustment factors and section properties per Chapter M3. Applicable adjustment factors for sawn lumber are defined in *NDS* 4.3. Table M4.3-1 shows the applicability of adjustment factors for sawn lumber in a slightly different format for the designer.

Table M4.3-1 Applicability of Adjustment Factors for Sawn Lumber

Allowable Stress Design	Load and Resistance Factor Design
$F_b' = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r$	$F_b' = F_b C_M C_t C_L C_F C_{fu} C_i C_r (2.54)(0.85) \lambda$
$F_t' = F_t C_D C_M C_t C_F C_i$	$F_t' = F_t C_M C_t C_F C_i (2.70)(0.80) \lambda$
$F_v' = F_v C_D C_M C_t C_i$	$F_v' = F_v C_M C_t C_i (2.88)(0.75) \lambda$
$F_{c\perp}' = F_{c\perp} C_M C_t C_i C_b$	$F_{c\perp}' = F_{c\perp} C_M C_t C_i C_b (1.67)(0.90)$
$F_c' = F_c C_D C_M C_t C_F C_i C_P$	$F_c' = F_c C_M C_t C_F C_i C_P (2.40)(0.90) \lambda$
$E' = E C_M C_t C_i$	$E' = E C_M C_t C_i$
$E_{min}' = E_{min} C_M C_t C_i C_T$	$E_{min}' = E_{min} C_M C_t C_i C_T (1.76)(0.85)$

Bending Member Example

For unincised, straight, laterally supported bending members stressed in edgewise bending in single member use and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 4.3), the adjusted design values reduce to:

For ASD:

$$F_b' = F_b C_D C_F$$

$$F_v' = F_v C_D$$

$$F_{c\perp}' = F_{c\perp} C_b$$

$$E' = E$$

For LRFD:

$$F_b' = F_b C_F (2.54)(0.85) \lambda$$

$$F_v' = F_v (2.88)(0.75) \lambda$$

$$F_{c\perp}' = F_{c\perp} C_b (1.67)(0.90)$$

$$E' = E$$

Axially Loaded Member Example

For unincised axially loaded members used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 4.3) designed to resist tension or compression loads, the adjusted tension or compression design values reduce to:

For ASD:

$$F_c' = F_c C_D C_F C_P$$

$$F_t' = F_t C_D C_F$$

$$E_{min}' = E_{min}$$

For LRFD:

$$F_c' = F_c C_F C_P (2.40)(0.90) \lambda$$

$$F_t' = F_t C_F (2.70)(0.80) \lambda$$

$$E_{min}' = E_{min} (1.76)(0.85)$$

M4.4 Special Design Considerations

General

With proper detailing and protection, structural lumber can perform well in a variety of environments. One key to proper detailing is planning for the natural shrinkage and swelling of wood members as they are subjected to various drying and wetting cycles. While moisture changes have the largest impact on lumber dimensions, some designs must also check the effects of temperature on dimensions as well.

Dimensional Changes

Table M4.4-1 is extracted from more precise scientific and research reports on these topics. The coefficients are conservative (yielding more shrinkage and expansion than one might expect for most species). This level of information should be adequate for common structural applications. Equations are provided in this section for those designers who require more precise calculations.

Design of wood members and assemblies for fire resistance is discussed in Chapter M16.

Table M4.4-1 Approximate Moisture and Thermal Dimensional Changes

Description	Radial or Tangential Direction
Dimensional change due to moisture content change ¹	1% change in dimension per 4% change in MC
Dimensional change due to temperature change ²	20×10^{-6} in./in. per degree F

1. Corresponding longitudinal direction shrinkage/expansion is about 1% to 5% of that in radial and tangential directions.

2. Corresponding longitudinal direction coefficient is about 1/10 as large as radial and tangential.

Equations for Computing Moisture and Thermal Shrinkage/Expansion

Due to Moisture Changes

For more precise computation of dimensional changes due to changes in moisture, the change in radial (R), tangential (T), and volumetric (V) dimensions due to changes in moisture content can be calculated as:

$$X = X_o (\Delta MC) e_{ME} \quad (M4.4-1)$$

where:

X_o = initial dimension or volume

X = dimension or volume change

ΔMC = moisture content change (%)

e_{ME} = coefficient of moisture expansion:
linear (in./in./%MC) or
volumetric (in.³/in.³/%MC)

and:

$$\Delta MC = M - M_o \quad (M4.4-2)$$

where:

M_o = initial moisture content % ($M_o \leq FSP$)

M = new moisture content % ($M \leq FSP$)

FSP = fiber saturation point

Values for e_{ME} and FSP are shown in Table M4.4-2.

Due to Temperature Changes

For more precise calculation of dimensional changes due to changes in temperature, the shrinkage/expansion of solid wood including lumber and timbers can be calculated as:

$$X = X_o (\Delta T) e_{TE} \quad (M4.4-3)$$

where:

X_o = reference dimension at T_o

X = computed dimension at T

T_o = reference temperature (°F)

T = temperature at which the new dimension is calculated (°F)

e_{TE} = coefficient of thermal expansion
(in./in./°F)

and:

$$\Delta T = T - T_o \quad (M4.4-4)$$

where:

$$-60^\circ\text{F} \leq T_o \leq 130^\circ\text{F}$$

The coefficient of thermal expansion of oven-dry wood parallel to grain ranges from about 1.7×10^{-6} to 2.5×10^{-6} per °F.

The linear expansion coefficients across the grain (radial and tangential) are proportional to wood density. These coefficients are about five to ten times greater than the parallel-to-the-grain coefficients and are given as:

Radial:

$$e_{TE} = [18(G) + 5.5](10^{-6} \text{ in./in./}^{\circ}F) \quad (\text{M4.4-5})$$

Tangential:

$$e_{TE} = [18(G) + 10.2](10^{-6} \text{ in./in./}^{\circ}F) \quad (\text{M4.4-6})$$

where:

G = tabulated specific gravity for the species.

Table M4.4-2 Coefficient of Moisture Expansion, e_{ME} , and Fiber Saturation Point, FSP, for Solid Woods

Species	e_{ME}			
	Radial (in./in./%)	Tangential (in./in./%)	Volumetric (in. ³ /in. ³ /%)	FSP (%)
Alaska Cedar	0.0010	0.0021	0.0033	28
Douglas Fir-Larch	0.0018	0.0033	0.0050	28
Englemann Spruce	0.0013	0.0024	0.0037	30
Redwood	0.0012	0.0022	0.0032	22
Red Oak	0.0017	0.0038	0.0063	30
Southern Pine	0.0020	0.0030	0.0047	26
Western Hemlock	0.0015	0.0028	0.0044	28
Yellow Poplar	0.0015	0.0026	0.0041	31

Table M4.4-3 provides the numerical values for e_{TE} for the most commonly used commercial species or species groups.

Wood that contains moisture reacts to varying temperature differently than does dry wood. When moist wood is heated, it tends to expand because of normal thermal expansion and to shrink because of loss in moisture content. Unless the wood is very dry initially (perhaps 3% or 4% MC or less), the shrinkage due to moisture loss on heating will be greater than the thermal expansion, so the net dimensional change on heating will be negative. Wood at intermediate moisture levels (about 8% to 20%) will expand when first heated, then gradually shrink to a volume smaller than the initial volume, as the wood gradually loses water while in the heated condition.

Even in the longitudinal (grain) direction, where dimensional change due to moisture change is very small, such changes will still predominate over corresponding dimensional changes due to thermal expansion unless

the wood is very dry initially. For wood at usual moisture levels, net dimensional changes will generally be negative after prolonged heating.

Calculation of actual changes in dimensions can be accomplished by determining the equilibrium moisture content of wood at the temperature value and relative humidity of interest. Then the relative dimensional changes due to temperature change alone and moisture content change alone are calculated. By combining these two changes the final dimension of lumber and timber can be established.

Table M4.4-3 Coefficient of Thermal Expansion, e_{TE} , for Solid Woods

Species	e_{TE}	
	Radial (10^{-6} in./in./°F)	Tangential (10^{-6} in./in./°F)
California Redwood	13	18
Douglas Fir-Larch ¹	15	19
Douglas Fir-South	14	19
Eastern Spruce	13	18
Hem-Fir ¹	13	18
Red Oak	18	22
Southern Pine	15	20
Spruce-Pine-Fir	13	18
Yellow Poplar	14	18

1. Also applies when species name includes the designation "North."

Durability

Designing for durability is a key part of the architectural and engineering design of the building. Many design conditions can be detailed to minimize the potential for decay; for other problem conditions, preservative-treated wood or naturally durable species should be specified.

This section does not cover the topic of designing for durability in detail. There are many excellent texts on the topic, including AWC's *Design of Wood Structures for Permanence, WCD No. 6*. Designers are advised to use this type of information to assist in designing for "difficult" design areas, such as:

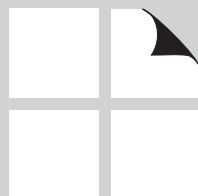
- structures in moist or humid conditions
- where wood comes in contact with concrete or masonry
- where wood members are supported in steel hangers or connectors in which condensation could collect
- anywhere that wood is directly or indirectly exposed to the elements
- where wood, if it should ever become wet, could not naturally dry out.

This list is not intended to be all-inclusive – it is merely an attempt to alert designers to special conditions that may cause problems when durability is not considered in the design.

M5: STRUCTURAL GLUED LAMINATED TIMBER

5

M5.1	General	18
M5.2	Reference Design Values	20
M5.3	Adjustment of Reference Design Values	21
M5.4	Special Design Considerations	22



M5.1 General

Products Description

Structural glued laminated timber (glulam) is a structural member glued up from suitably selected and prepared pieces of wood either in a straight or curved form with the grain of all of the pieces parallel to the longitudinal axis of the member. The reference design values given in the *NDS Supplement* are applicable only to structural glued laminated timber members produced in accordance with *American National Standard for Wood Products — Structural Glued Laminated Timber*, ANSI A190.1.

Structural glued laminated timber members are produced in laminating plants by gluing together dry lumber, normally of 2-inch or 1-inch nominal thickness, under controlled temperature and pressure conditions. Members with a wide variety of sizes, profiles, and lengths can be produced having superior characteristics of strength, serviceability, and appearance. Structural glued laminated timber beams are manufactured with the strongest laminations on the bottom and top of the beam, where the greatest tension and compression stresses occur in bending. This allows a more efficient use of the lumber resource by placing higher grade lumber in zones that have higher stresses and lumber with less structural quality in lower stressed zones.

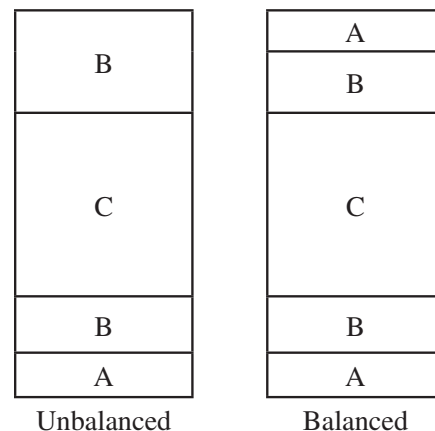
Structural glued laminated timber members are manufactured from several softwood species, primarily Douglas fir-larch, southern pine, hem-fir, spruce-pine-fir, eastern spruce, and Alaska cedar. Other species including redwood and various hardwoods are occasionally used. Standard structural glued laminated timber sizes are given in the *NDS Supplement*. Any length, up to the maximum length permitted by transportation and handling restrictions, is available.

A structural glued laminated timber member can be manufactured using a single grade or multiple grades of lumber, depending on intended use. In addition, a mixed-species structural glued laminated timber member is also possible. When the member is intended to be primarily loaded either axially or in bending with the loads acting parallel to the wide faces of the laminations, a single grade combination is recommended. On the other hand, a multiple grade combination provides better cost-effectiveness when the member is primarily loaded in bending due to loads applied perpendicular to the wide faces of the laminations.

On a multiple grade combination, a structural glued laminated timber member can be produced as either a balanced or unbalanced combination, depending on the

geometrical arrangement of the laminations about the mid-depth of the member. As shown in Figure M5.1-1, a balanced combination is symmetrical about the mid-depth, so it has the same reference bending design values for both positive and negative bending. Unbalanced combinations are asymmetrical about the mid-depth, so they have lower reference design values for negative bending than for positive bending. The top face of an unbalanced member is required to be marked “TOP” by the manufacturer. The balanced combinations are intended for use in continuous or cantilevered applications to provide equal capacity in both positive and negative bending. Whereas unbalanced combinations are primarily for use in simple span applications, they can also be used for short cantilever applications (cantilever less than 20% of the back span) or for continuous span applications when the design is controlled by shear or deflection.

Figure M5.1-1 Unbalanced and Balanced Layup Combinations



Structural glued laminated timber members can be used as primary or secondary load-carrying components in structures. Table M5.1-1 lists economical spans for selected timber framing systems using structural glued laminated timber members in buildings. Other common uses of structural glued laminated timber members are for utility structures, pedestrian bridges, highway bridges, railroad bridges, marine structures, noise barriers, and towers. Table M5.1-1 may be used for preliminary design purposes to determine the economical span ranges for the selected framing systems. However, all systems require a more extensive analysis for final design.

Table M5.1-1 Economical Spans for Structural Glued Laminated Timber Framing Systems

Type of Framing System	Economical Spans (ft)
ROOF	
Simple Span Beams	
Straight or slightly cambered	10 to 100
Tapered, double tapered-pitched, or curved	25 to 105
Cantilevered Beams (Main span)	up to 90
Continuous Beams (Interior spans)	10 to 50
Girders	40 to 100
Three-Hinged Arches	
Gothic	40 to 100
Tudor	40 to 140
A-Frame	20 to 100
Three-centered, Parabolic, or Radial	40 to 250
Two-Hinged Arches	
Radial or Parabolic	50 to 200
Trusses (Four or more ply chords)	
Flat or parallel chord	50 to 150
Triangular or pitched	50 to 150
Bowstring (Continuous chord)	50 to 200
Trusses (Two or three ply chords)	
Flat or parallel chord	20 to 75
Triangular or pitched	20 to 75
Tied arches	50 to 200
Dome structures	200 to 500+
FLOOR	
Simple Span Beams	10 to 40
Continuous Beams (Individual spans)	10 to 40
HEADERS	
Windows and Doors	< 10
Garage Doors	9 to 18

Appearance Classifications

Structural glued laminated timber members are typically produced in four appearance classifications: Premium, Architectural, Industrial, and Framing. Premium and Architectural beams are higher in appearance qualities and are surfaced for a smooth finish ready for staining or painting. Industrial classification beams are normally used in concealed applications or in construction where appearance is not important. Framing classification beams are typically used for headers and other concealed applications in residential construction. Design values for structural glued laminated timber members are independent of the appearance classifications.

For more information and detailed descriptions of these appearance classifications and their typical uses, refer to APA EWS Technical Note Y110 or AITC Standard 110.

Availability

Structural glued laminated timber members are available in both custom and stock sizes. Custom beams are manufactured to the specifications of a specific project, while stock beams are made in common dimensions, shipped to distribution yards, and cut to length when the beam is ordered. Stock beams are available in virtually every major metropolitan area. Although structural glued laminated timber members can be custom fabricated to provide a nearly infinite variety of forms and sizes, the best economy is generally realized by using standard-size members as noted in the *NDS Supplement*. When in doubt, the designer is advised to check with the structural glued laminated timber supplier or manufacturer concerning the availability of a specific size prior to specification.

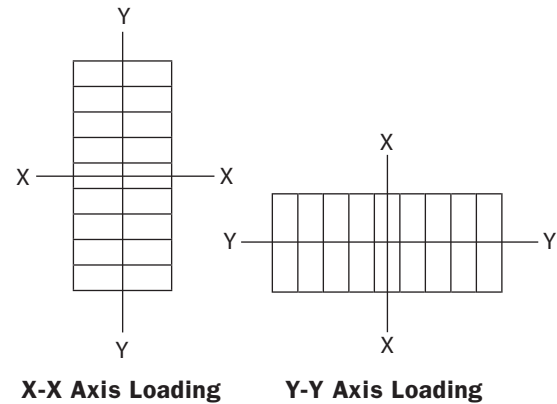
M5.2 Reference Design Values

Reference design values of structural glued laminated timber are affected by the layout of members composed of various grades of lumber, as well as the orientation of the laminations relative to the applied loads. As a result, different design values are assigned for structural glued laminated timber used primarily in bending (*NDS Supplement Table 5A*) and primarily in axial loading (*NDS Supplement Table 5B*). The reference design values are used in conjunction with the dimensions provided in Table 1C (western species) and Table 1D (southern pine) of the *NDS Supplement*, but are applicable to any size of structural glued laminated timber when the appropriate modification factors discussed in M5.3 are applied.

Reference design values are given in *NDS Supplement Table 5A* for bending about the X-X axis (see Figure M5.2-1). Although permitted, axial loading or bending about the Y-Y axis (also see Figure M5.2-1) is not efficient in using the structural glued laminated timber combinations given in *NDS Supplement Table 5A*. In such cases, the designer should select structural glued laminated timber from *NDS Supplement Table 5B*. Similarly, structural glued laminated timber combinations in *NDS Supplement Table 5B* are inefficiently utilized if the primary use is bending about the X-X axis.

The reference design values given in *NDS Supplement Tables 5A and 5B* are based on use under normal duration of load (10 years) and dry conditions (less than 16% moisture content). When used under other conditions, see *NDS Chapter 5* for adjustment factors. The reference bending design values are based on members loaded as simple beams. When structural glued laminated timber is used in continuous or cantilevered beams, the reference bending design values given in *NDS Supplement Table 5A* for top of beam stressed in tension should be used for the design of stress reversal.

Figure M5.2-1 Structural Glued Laminated Timber Axes Orientations



M5.3 Adjustment of Reference Design Values

The adjustment factors provided in the *NDS* are for non-reference end-use conditions and material modification effects. These factors are used to modify the reference design values when one or more of the specific end uses or material modification conditions are beyond the limits of the reference conditions given in the *NDS*.

Adjustment factors unique to structural glued laminated timber include the volume factor, C_v , the curvature factor, C_c , stress interaction factor, C_t , and the shear reduction factor, C_{vr} . All are defined in Chapter 5 of the *NDS*.

To generate member design capacities, reference design values for structural glued laminated timber are multiplied by adjustment factors and section properties per Chapter M3. Applicable adjustment factors for structural glued laminated timber are defined in *NDS* 5.3. Table M5.3-1 shows the applicability of adjustment factors for structural glued laminated timber in a slightly different format for the designer.

Table M5.3-1 Applicability of Adjustment Factors for Structural Glued Laminated Timber¹

Allowable Stress Design	Load and Resistance Factor Design
$F_b' = F_b C_D C_M C_t C_L C_V C_{fu} C_c C_t$	$F_b' = F_b C_M C_t C_L C_V C_{fu} C_c C_t (2.54)(0.85) \lambda$
$F_t' = F_t C_D C_M C_t$	$F_t' = F_t C_M C_t (2.70)(0.80) \lambda$
$F_v' = F_v C_D C_M C_t C_{vr}$	$F_v' = F_v C_M C_t C_{vr} (2.88)(0.75)$
$F_{rt}' = F_{rt} C_D C_M C_t$	$F_{rt}' = F_{rt} C_M C_t (2.88)(0.75) \lambda$
$F_c' = F_c C_D C_M C_t C_p$	$F_c' = F_c C_M C_t C_p (2.40)(0.90) \lambda$
$F_{c\perp}' = F_{c\perp} C_M C_t C_b$	$F_{c\perp}' = F_{c\perp} C_M C_t C_b (1.67)(0.90)$
$E' = E C_M C_t$	$E' = E C_M C_t$
$E_{min}' = E_{min} C_M C_t$	$E_{min}' = E_{min} C_M C_t (1.76)(0.85)$

1. The beam stability factor, C_L , shall not apply simultaneously with the volume factor, C_v , for structural glued laminated timber bending members (see *NDS* 5.3.6). Therefore, the lesser of these adjustment factors shall apply.

Bending Member Example

For straight, prismatic, untapered, laterally supported bending members loaded perpendicular to the wide face of the laminations and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 5.3), the adjusted design values reduce to:

For ASD:

$$F_b' = F_b C_D C_v$$

$$F_v' = F_v C_D$$

$$F_{c\perp}' = F_{c\perp} C_b$$

$$E' = E$$

For LRFD:

$$F_b' = F_b C_v (2.54)(0.85) \lambda$$

$$F_v' = F_v (2.88)(0.75) \lambda$$

$$F_{c\perp}' = F_{c\perp} C_b (2.40)(0.90)$$

$$E' = E$$

Axially Loaded Member Example

For axially loaded members used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 5.3) designed to resist tension or compression loads, the adjusted tension or compression design values reduce to:

For ASD:

$$F_c' = F_c C_D C_p$$

$$F_t' = F_t C_D$$

$$E_{min}' = E_{min}$$

For LRFD:

$$F_c' = F_c C_p (2.40)(0.90) \lambda$$

$$F_t' = F_t (2.70)(0.80) \lambda$$

$$E_{min}' = E_{min} (1.76)(0.85)$$

M5.4 Special Design Considerations

General

This section contains information concerning physical properties of structural glued laminated timber members including specific gravity and response to moisture or temperature.

In addition to designing to accommodate dimensional changes and detailing for durability, another significant design consideration is that of fire performance, which is discussed in Chapter M16.

Note that the information contained in this section addresses design considerations separate from those described in *NDS* 5.4.

Specific Gravity

Table M5.4-1 provides specific gravity values for some of the most common wood species used for structural glued laminated timber. These values are used in determining various physical and connection properties. Further, weight factors are provided at four moisture contents. When the

cross-sectional area (in.²) is multiplied by the appropriate weight factor, it provides the weight of the structural glued laminated timber member per linear foot of length. For other moisture contents, the tabulated weight factors can be interpolated or extrapolated.

Structural glued laminated timber members often are manufactured using different species at different portions of the cross section. In this case the weight of the structural glued laminated timber may be computed by the sum of the products of the cross-sectional area and the weight factor for each species.

Dimensional Changes

See M4.4 for information on calculating dimensional changes due to moisture or temperature.

Durability

See M4.4 for information on detailing for durability.

Table M5.4-1 Average Specific Gravity and Weight Factor

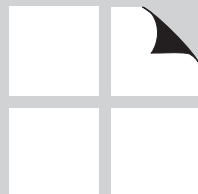
Species Combination	Specific Gravity ¹	Weight Factor ²			
		12%	15%	19%	25%
California Redwood (close grain)	0.44	0.195	0.198	0.202	0.208
Douglas Fir-Larch	0.50	0.235	0.238	0.242	0.248
Douglas Fir (South)	0.46	0.221	0.225	0.229	0.235
Eastern Spruce	0.41	0.191	0.194	0.198	0.203
Hem-Fir	0.43	0.195	0.198	0.202	0.208
Red Maple	0.58	0.261	0.264	0.268	0.274
Red Oak	0.67	0.307	0.310	0.314	0.319
Southern Pine	0.55	0.252	0.255	0.259	0.265
Spruce-Pine-Fir (North)	0.42	0.195	0.198	0.202	0.208
Yellow Poplar	0.43	0.213	0.216	0.220	0.226

1. Specific gravity is based on weight and volume when oven-dry.

2. Weight factor shall be multiplied by net cross-sectional area in in.² to obtain weight in pounds per lineal foot.

M6: ROUND TIMBER POLES AND PILES

M6.1	General	24
M6.2	Reference Design Values	24
M6.3	Adjustment of Reference Design Values	25
M6.4	Special Design Considerations	26



M6.1 General

Product Information

Timber poles and piles offer many advantages relative to competing materials. As with other wood products, timber poles and piles offer the unique advantage of being the only major construction material that is a renewable resource.

Common Uses

Timber poles are used extensively in post-frame construction and are also used architecturally. This Chapter is not for use with poles used in the support of utility

lines. Timber piles are generally used as part of foundation systems.

Availability

Timber poles are supplied to the utility industry in a variety of grades and species. Because these poles are graded according to ANSI 05.1, *Specifications and Dimensions for Wood Poles*, they must be regraded according to ASTM D3200 if they are to be used with the *NDS*.

Timber piles are typically available in three species: Pacific Coast Douglas-fir, southern pine, and red pine. However, local pile suppliers should be contacted because availability is dependent upon geographic location.

M6.2 Reference Design Values

General

The tables in the *NDS Supplement* provide reference design values for timber pole and pile members. These reference design values are used when manual calculation of member strength is required and shall be used in conjunction with adjustment factors specified in *NDS* Chapter 6.

Pole and Pile Reference Design Values

Reference design values for piles and poles are provided in *NDS Supplement* Tables 6A and 6B, respectively.

These values, with the exception of F_c , are applicable for all locations in the pole or pile. The F_c values can be increased at other locations in accordance with *NDS* 6.3.9.

Reference design values for poles and piles are based on air dried conditioning. Kiln-drying, steam conditioning, or boultonizing prior to treatment will have the effect of reducing the reference design value per *NDS* 6.3.5.

M6.3 Adjustment of Reference Design Values

To generate member design capacities, reference design values are multiplied by adjustment factors and section properties. Adjustment factors unique to round timber poles and piles include the condition treatment factor, C_{ct} , the critical section factor, C_{cs} , and the load sharing factor, C_{ls} . All are defined in Chapter 6 of the *NDS*.

To generate member design capacities, reference design values for round timber poles and piles are multiplied by adjustment factors and section properties per Chapter M3. Applicable adjustment factors for round timber poles and piles are defined in *NDS* 6.3. Table M6.3-1 shows the applicability of adjustment factors for round timber poles and piles in a slightly different format for the designer.

Table M6.3-1 Applicability of Adjustment Factors for Round Timber Poles and Piles

Allowable Stress Design	Load and Resistance Factor Design
$F_c' = F_c C_D C_t C_{ct} C_P C_{cs} C_{ls}$	$F_c' = F_c C_t C_{ct} C_P C_{cs} C_{ls} (2.40)(0.90) \lambda$
$F_b' = F_b C_D C_t C_{ct} C_F C_{ls}$	$F_b' = F_b C_t C_{ct} C_F C_{ls} (2.54)(0.85) \lambda$
$F_v' = F_v C_D C_t C_{ct}$	$F_v' = F_v C_t C_{ct} (2.88)(0.75) \lambda$
$F_{c\perp}' = F_{c\perp} C_t C_{ct} C_b$	$F_{c\perp}' = F_{c\perp} C_t C_{ct} C_b (1.67)(0.90)$
$E' = E C_t$	$E' = E C_t$
$E_{min}' = E_{min} C_t$	$E_{min}' = E_{min} C_t (1.76)(0.85)$

Axially Loaded Pole or Pile Example

For single, axially loaded, air dried poles or piles, fully laterally supported in two orthogonal directions, used in a normal environment (meeting the reference conditions of *NDS* 2.3 and 6.3), designed to resist compression loads only, and less than 13.5" in diameter, the adjusted compression design values reduce to:

For ASD:

$$F_c' = F_c C_D$$

For LRFD:

$$F_c' = F_c (2.40)(0.90) \lambda$$

M6.4 Special Design Considerations

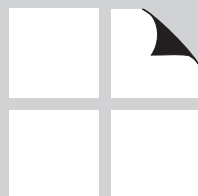
With proper detailing and protection, timber poles and piles can perform well in a variety of environments. One key to proper detailing is planning for the natural shrinkage and swelling of wood products as they are subjected to various drying and wetting cycles. While moisture changes have the largest impact on product dimensions, some designs must also check the effects of temperature. See M4.4 for design information on dimensional changes due to moisture and temperature.

Durability issues related to piles are generally both more critical and more easily accommodated. Since piles are in constant ground contact, they cannot be “insulated” from contact with moisture – thus, the standard condition for piles is preservatively treated. The importance of proper treatment processing of piles cannot be overemphasized. See M4.4 for more information about durability.

In addition to designing to accommodate dimensional changes and detailing for durability, another significant issue in the planning of wood structures is that of fire performance, which is discussed in Chapter M16.

M7: PREFABRICATED WOOD I-JOISTS

M7.1	General	28
M7.2	Reference Design Values	28
M7.3	Adjustment of Reference Design Values	30
M7.4	Special Design Considerations	31



M7.1 General

Product Information

Prefabricated wood I-joists are exceptionally stiff, lightweight, and capable of long spans. Holes may be easily cut in the web according to manufacturer's recommendations, allowing ducts and utilities to be run through the joist. I-joists are dimensionally stable and uniform in size, with no crown. This keeps floors quieter, reduces field modifications, and eliminates rejects in the field. I-joists may be field cut to proper length using conventional methods and tools.

Manufacturing of I-joists utilizes the geometry of the cross section and high strength components to maximize the strength and stiffness of the wood fiber. Flanges are manufactured from solid sawn lumber or structural composite lumber, while webs typically consist of plywood or oriented strand board. The efficient utilization of raw materials, along with high-quality exterior adhesives and state of the art quality control procedures, result in an extremely consistent product that maximizes environmental benefits as well.

Prefabricated wood I-joists are produced as proprietary products which are covered by code evaluation reports. Evaluation reports and product literature should be consulted for current design information.

Common Uses

Prefabricated wood I-joists are widely used as a framing material for housing in North America. I-joists are made in different grades and with various processes and can be utilized in various applications. Proper design is required to optimize performance and economics.

In addition to use in housing, I-joists find increasing use in commercial and industrial construction. The high strength, stiffness, wide availability, and cost saving attributes make them a viable alternative in most low-rise construction projects.

Prefabricated wood I-joists are typically used as floor and roof joists in conventional construction. In addition, I-joists are used as studs where long lengths and high strengths are required.

Availability

To efficiently specify I-joists for individual construction projects, consideration should be given to the size and the required strength of the I-joist. Sizes vary with each individual product. The best source of this information is your local lumber supplier, distribution center, or I-joist manufacturer. Proper design is facilitated through the use of manufacturer's literature and specification software available from I-joist manufacturers.

M7.2 Reference Design Values

Introduction to Design Values

As stated in *NDS 7.2*, each wood I-joist manufacturer develops its own proprietary design values. The derivation of these values is reviewed by the applicable building code authority. Since materials, manufacturing processes, and product evaluations may differ between the various manufacturers, selected design values are only appropriate for the specific product and application.

To generate the design capacity of a given product, the manufacturer of that product evaluates test data. The design capacity is then determined per ASTM D5055.

The latest model building code agency evaluation reports are a reliable source for wood I-joist design values. These reports list accepted design values for shear, moment, stiffness, and reaction capacity based on minimum bearing. In addition, evaluation reports note the limitations

on web holes, concentrated loads, and requirements for web stiffeners.

Bearing/Reaction Design

Tabulated design capacities reflect standard conditions and must be modified as discussed in *NDS Chapter 7* to obtain adjusted design values.

Bearing lengths at supports often control the design capacity of an I-joist. Typically minimum bearing lengths are used to establish design parameters. In some cases additional bearing is available and can be verified in an installation. Increased bearing length means that the joist can support additional loading, up to the value limited by the shear capacity of the web material and web joint. Both interior and exterior reactions must be evaluated.

Use of web stiffeners may be required and typically increase the bearing capacity of the joist. Correct

installation is required to obtain the specified capacities. Additional loading from walls above will load the joist in bearing, further limiting the capacity of the joist if proper end detailing is not followed. Additional information regarding design considerations for joist bearing can be found in M7.4.

Adjusted design reactions (bearing capacities), R_r' , are determined in the same empirical fashion as is allowable shear.

Shear Design

At end bearing locations, critical shear is the vertical shear at the ends of the design span. The practice of neglecting all uniform loads within a distance from the end support equal to the joist depth, commonly used for other wood materials, is not applicable to end supports for wood I-joists. At locations of continuity, such as interior supports of multi-span I-joists, the critical shear location for several wood I-joist types is located a distance equal to the depth of the joist from the centerline of bearing (uniform loads only). A cantilevered portion of a wood I-joist is generally not considered a location of continuity (unless the cantilever length exceeds the joist depth) and vertical shear at the cantilever bearing is the critical shear. Individual manufacturers, or the appropriate evaluation reports, should be consulted for reference to shear design at locations of continuity.

Often, the adjusted shear design values, V_r' , are based on other considerations such as bottom flange bearing length or the installation of web stiffeners or bearing blocks.

Moment Design

Reference moment design values, M_r' , of prefabricated wood I-joists are determined from empirical testing of a completely assembled joist or by engineering analysis supplemented by tension testing the flange component. If the flange contains end jointed material, the allowable tension value is the lesser of the joint capacity or the material capacity.

Because flanges of a wood I-joist can be highly stressed, field notching of the flanges is not allowed. Similarly, excessive nailing or the use of improper nail sizes can cause flange splitting that will also reduce capacity. The manufacturer should be contacted when evaluating a damaged flange.

Deflection Design

Wood I-joists, due to their optimized web materials, are susceptible to the effects of shear deflection. This component of deflection can account for as much as 15% to 30% of the total deflection. For this reason, both bending and shear deflection are considered in deflection design. A typical deflection calculation for simple span wood I-joists under uniform load is shown in Equation M7.2-1.

Deflection = Bending Component + Shear Component

$$\Delta = \frac{5w\ell^4}{384EI} + \frac{w\ell^2}{k} \quad (\text{M7.2-1})$$

where:

w = uniform load in pounds per lineal inch

ℓ = design span, in.

EI = joist moment of inertia times flange modulus of elasticity

k = shear deflection coefficient

Individual manufacturers provide equations in a similar format. Values for use in the preceding equation can be found in the individual manufacturer's evaluation reports. For other load and span conditions, an approximate answer can be found by using conventional bending deflection equations adjusted as follows:

$$\text{Deflection} = \text{Bending Deflection} \left(1 + \frac{384 EI}{5\ell^2 k} \right)$$

Since wood I-joists can have long spans, the model building code maximum live load deflection criteria may not be appropriate for many floor applications. Many wood I-joist manufacturers recommend using stiffer criteria, such as L/480 for residential floor construction and L/600 for public access commercial applications such as office floors. The minimum code required criteria for storage floors and roof applications is normally adequate.

M7.3 Adjustment of Reference Design Values

General

Member design capacity is the product of reference design values and adjustment factors. Reference design values for I-joists are discussed in M7.2.

The design values listed in the evaluation reports are generally applicable to dry use conditions. Less typical conditions, such as high moisture, high temperatures, or pressure impregnated chemical treatments, typically result in strength and stiffness adjustments different from those

used for sawn lumber. *NDS* 7.3 outlines adjustments to reference design values for I-joists; however, individual wood I-joist manufacturers should be consulted to verify appropriate adjustments. Table M7.3-1 shows the applicability of adjustment factors for prefabricated wood I-joists in a slightly different format for the designer.

The user is cautioned that manufacturers may not permit the use of some applications and/or treatments. Unauthorized treatments can void a manufacturer's warranty and may result in structural deficiencies.

Table M7.3-1 Applicability of Adjustment Factors for Prefabricated Wood I-Joists

Allowable Stress Design	Load and Resistance Factor Design
$M_r' = M_r C_D C_M C_t C_L C_r$	$M_r' = M_r C_M C_t C_L C_r K_F (0.85) \lambda$
$V_r' = V_r C_D C_M C_t$	$V_r' = V_r C_D C_M C_t K_F (0.75) \lambda$
$R_r' = R_r C_D C_M C_t$	$R_r' = R_r C_M C_t K_F (0.75) \lambda$
$EI' = EI C_M C_t$	$EI' = EI C_M C_t$
$EI_{\min}' = EI_{\min} C_M C_t$	$EI_{\min}' = EI_{\min} C_M C_t K_F (0.85)$
$K' = K C_M C_t$	$K' = K C_M C_t$

Bending Member Example

For fully laterally supported bending members loaded in strong axis bending and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 7.3), the adjusted design values reduce to:

For ASD:

$$M_r' = M_r C_D$$

$$V_r' = V_r C_D$$

$$R_r' = R_r C_D$$

$$EI' = EI$$

$$K' = K$$

For LRFD:

$$M_r' = M_r K_F (0.85) \lambda$$

$$V_r' = V_r K_F (0.75) \lambda$$

$$R_r' = R_r K_F (0.75) \lambda$$

$$EI' = EI$$

$$K' = K$$

Lateral Stability

The design values contained in the evaluation reports for prefabricated wood I-joists assume the compression flange is supported throughout its length to prevent lateral displacement, and the ends at points of bearing have lateral support to prevent rotation (twisting). Lateral restraint is generally provided by diaphragm sheathing or bracing spaced at 16" on center or less (based on 1½" width joist flanges) nailed to the joist's compression flange.

Applications without continuous lateral bracing will generally have reduced moment design capacities. The reduced capacity results from the increased potential for lateral buckling of the joist's compression flange. *NDS* 7.3.5.3 provides guidance for design of an unbraced I-joist compression flange.

Special Loads or Applications

Wood I-joists are configured and optimized to act primarily as joists to resist bending loads supported at the bearing by the bottom flange. Applications that result in significant axial tension or compression loads, require web holes, special connections, or other unusual conditions should be evaluated only with the assistance of the individual wood I-joist manufacturers.

M7.4 Special Design Considerations

Introduction

The wood I-joist is similar to conventional lumber in that it is based on the same raw materials, but differs in how the material is composed. For this reason, conventional lumber design practices are not always compatible with the unique configuration and wood fiber orientation of the wood I-joist. Designers using wood I-joists should develop solutions in accordance with the following guidelines.

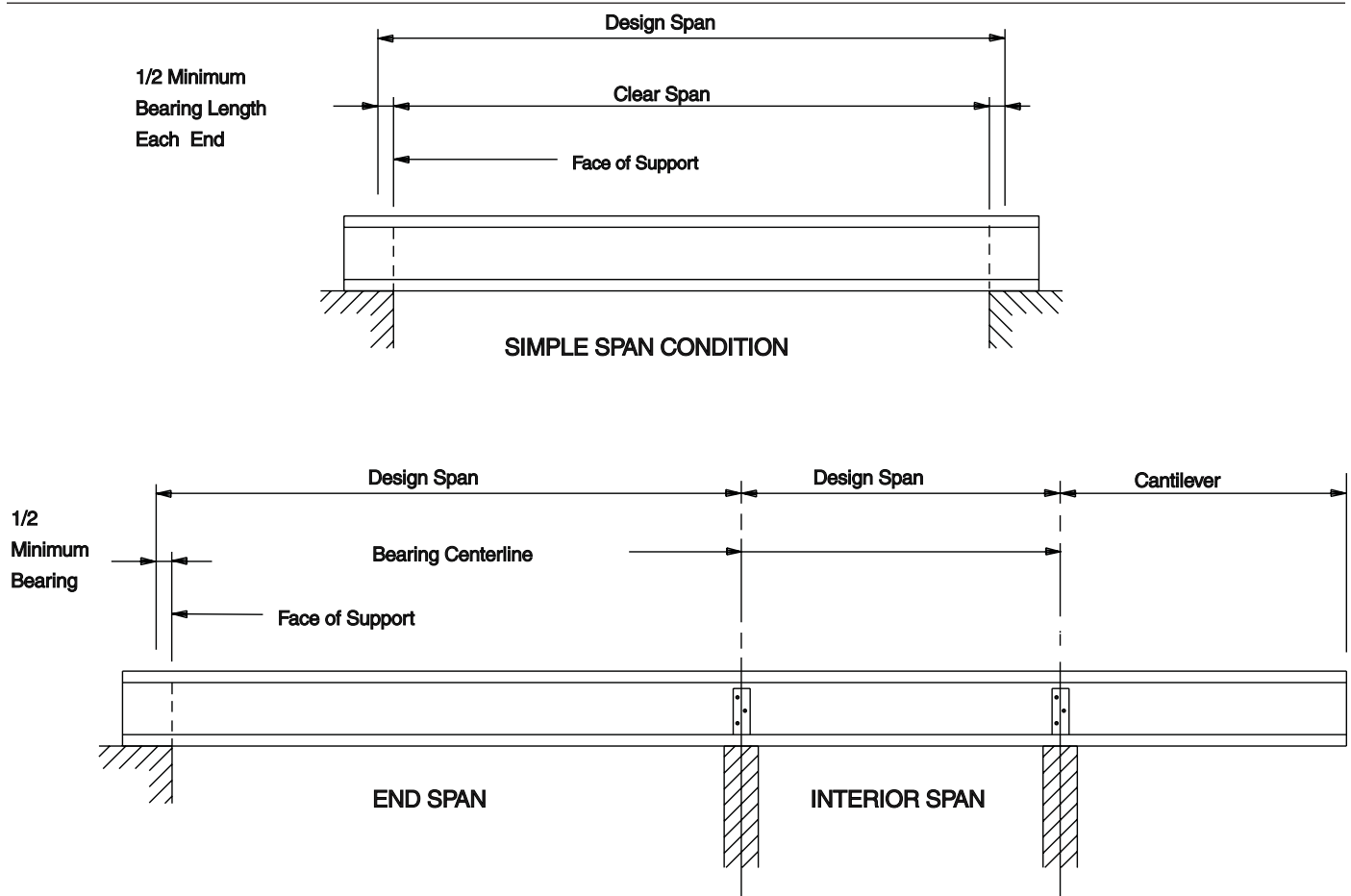
Durability issues cannot be overemphasized. See M4.4 for more information about durability.

In addition to detailing for durability, another significant issue in the planning of wood structures is that of fire performance, which is discussed in Chapter M16.

Design Span

The design span used for determining critical shears and moments is defined as the clear span between the faces of support plus one-half the minimum required bearing on each end (see Figure M7.4-1). For most wood I-joists, the minimum required end bearing length varies from 1½" to 3½" (adding 2" to the clear span dimension is a good estimate for most applications). At locations of continuity over intermediate bearings, the design span is measured from the centerline of the intermediate support to the face of the bearing at the end support, plus one-half the minimum required bearing length. For interior spans of a continuous joist, the design span extends from centerline to centerline of the intermediate bearings.

Figure M7.4-1 Design Span Determination



Load Cases

Most building codes require consideration of a critical distribution of loads. Due to the long length and continuous span capabilities of the wood I-joist, these code provisions have particular meaning. Considering a multiple span member, the following design load cases should be considered:

- All spans with total loads
- Alternate span loading
- Adjacent span loading
- Partial span loading (joists with holes)
- Concentrated load provisions (as occurs)

A basic description of each of these load cases follows:

All spans with total loads – This load case involves placing all live and dead design loads on all spans simultaneously.

Alternate span loading – This load case places the L, L_R, S, or R load portion of the design loads on every other span and can involve two loading patterns. The first pattern results in the removal of the live loads from all even numbered spans. The second pattern removes live loads from all odd numbered spans. For roof applications, some building codes require removal of only a portion of the live loads from odd or even numbered spans. The alternate span load case usually generates maximum end reactions, mid-span moments, and mid-span deflections. Illustrations of this type of loading are shown in Figure M7.4-2.

Adjacent span loading – This load case (see Figure M7.4-2) removes L, L_R, S or R loads from all but two adjoining spans. All other spans, if they exist, are loaded with dead loads only. Depending on the number of spans

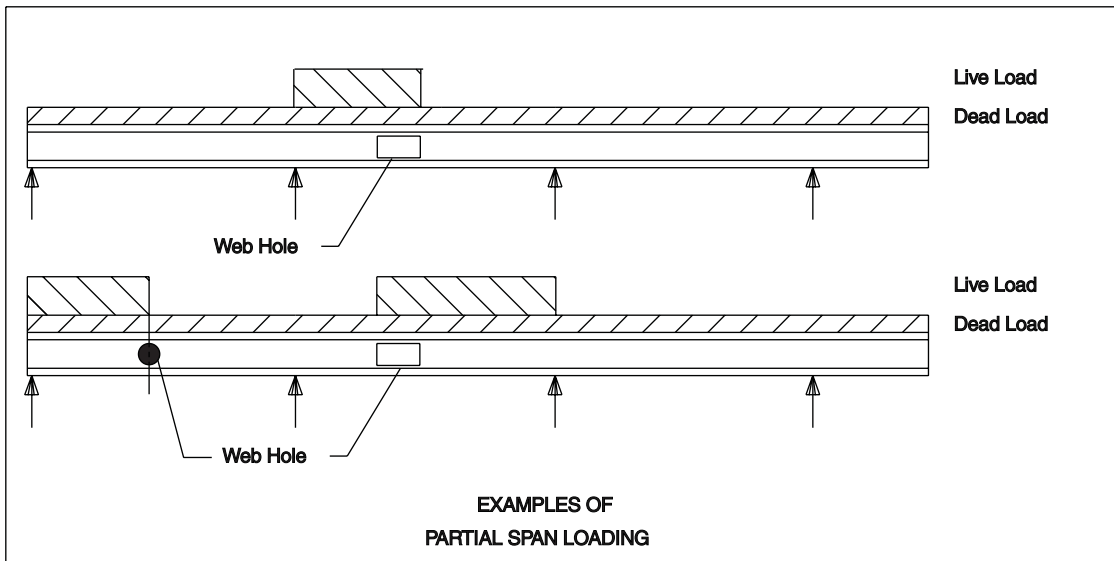
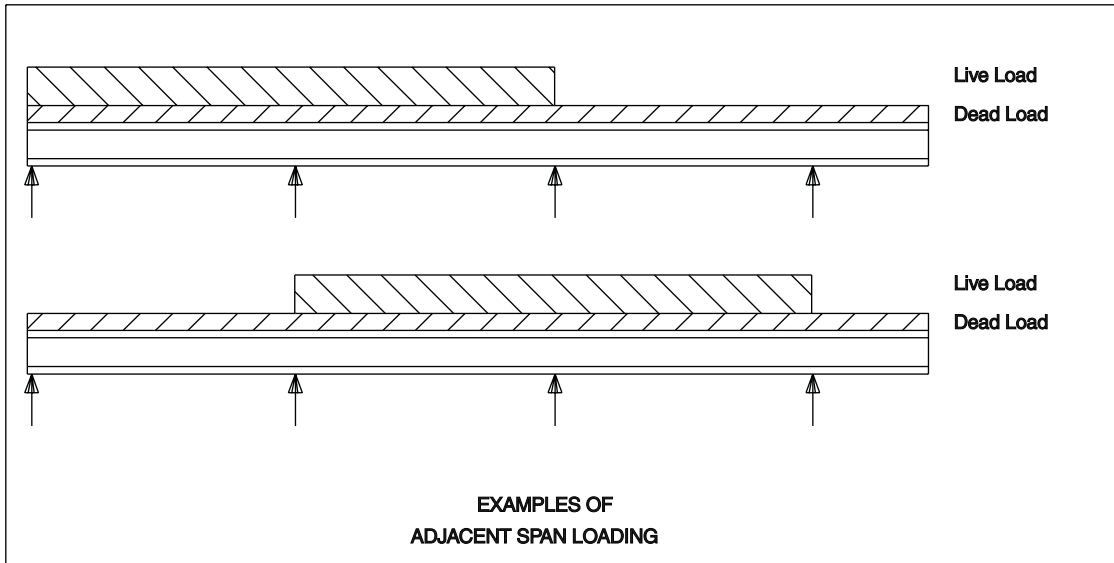
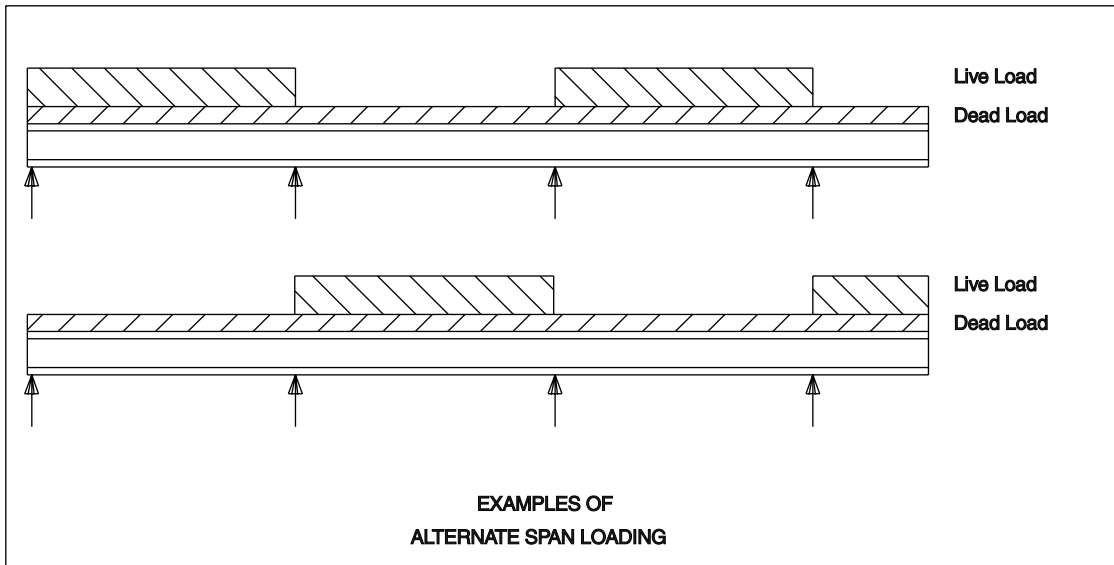
involved, this load case can lead to a number of load patterns. All combinations of adjacent spans become separate loadings. This load case is used to develop maximum shears and reactions at internal bearing locations.

Partial span loading – This load case involves applying L, L_R, S or R loads to less than the full length of a span (see Figure M7.4-2). For wood I-joists with web holes, this case is used to develop shear at hole locations. When this load case applies, uniform L, L_R, S, R load is applied only from an adjacent bearing to the opposite edge of a rectangular hole (centerline of a circular hole). For each hole within a given span, there are two corresponding load cases. Live loads other than the uniform application load, located within the span containing the hole, are also applied simultaneously. This includes all special loads such as point or tapered loads.

Concentrated load provisions – Most building codes have a concentrated load (live load) provision in addition to standard application design loads. This load case considers this concentrated load to act in combination with the system dead loads on an otherwise unloaded floor or roof. Usually, this provision applies to non-residential construction. An example is the “safe” load applied over a 2½ square foot area for office floors. This load case helps insure the product being evaluated has the required shear and moment capacity throughout its entire length and should be considered when analyzing the effect of web holes.

A properly designed multiple span member requires numerous load case evaluations. Most wood I-joist manufacturers have developed computer programs, load and span tables, or both that take these various load cases into consideration.

Figure M7.4-2 Load Case Evaluations



Floor Performance

Designing a floor system to meet the minimum requirements of a building code may not always provide acceptable performance to the end user. Although minimum criteria help assure a floor system can safely support the imposed loads, the system ultimately must perform to the satisfaction of the end user. Since expectancy levels may vary from one person to another, designing a floor system becomes a subjective issue requiring judgment as to the sensitivity of the intended occupant.

Joist deflection is often used as the primary means for designing floor performance. Although deflection is a factor, there are other equally important variables that can influence the performance of a floor system. A glue-nailed floor system will generally have better deflection performance than a nailed only system. Selection of the decking material is also an important consideration. Deflection of the sheathing material between joists can be reduced by placing the joists at a closer on center spacing or increasing the sheathing thickness.

Proper installation and job site storage are important considerations. All building materials, including wood I-joists, need to be kept dry and protected from exposure to the elements. Proper installation includes correct spacing of sheathing joints, care in fastening of the joists and sheathing, and providing adequate and level supports. All of these considerations are essential for proper system performance.

Vibration may be a design consideration for floor systems that are stiff and where very little dead load (i.e., partition walls, ceilings, furniture, etc.) exists. Vibration can generally be damped with a ceiling system directly attached to the bottom flange of the wood I-joists. Effective bridging or continuous bottom flange nailers (i.e., 2x4 nailed flat-wise and perpendicular to the joist and tied off to the end walls) can also help minimize the potential for vibration in the absence of a direct applied ceiling. Limiting the span/depth ratio of the I-joist may also improve floor performance.

Joist Bearing

Bearing design for wood I-joists requires more than consideration of perpendicular to grain bearing values. Minimum required bearing lengths take into account a number of considerations. These include: cross grain bending and tensile forces in the flanges, web stiffener connection to the joist web, adhesive joint locations and strength, and perpendicular to grain bearing stresses. The model building code evaluation reports provide a source

for bearing design information, usually in the form of minimum required bearing lengths.

Usually, published bearing lengths are based on the maximum allowable shear capacity of the particular product and depth or allowable reactions are related to specific bearing lengths. Bearing lengths for wood I-joists are most often based on empirical test results rather than a calculated approach. Each specific manufacturer should be consulted for information when deviations from published criteria are desired.

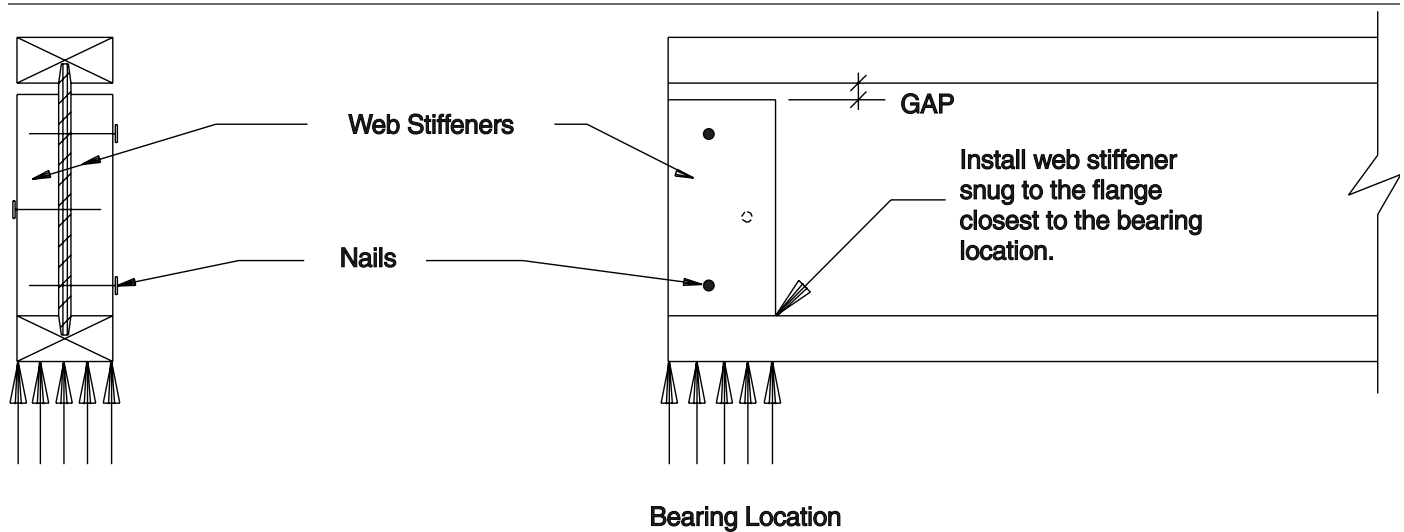
To better understand the variables involved in a wood I-joist bearing, it's convenient to visualize the member as a composition of pieces, each serving a specific task. For a typical simple span joist, the top flange is a compression member, the bottom flange is a tension member, and the web resists the vertical shear forces. Using this concept, shear forces accumulate in the web member at the bearing locations and must be transferred through the flanges to the support structure. This transfer involves two critical interfaces: between the flange and support member and between the web and flange materials.

Starting with the support member, flange to support bearing involves perpendicular to grain stresses. The lowest design value for either the support member or flange material is usually used to develop the minimum required bearing area.

The second interface to be checked is between the lower joist flange and the bottom edge of the joist web, assuming a bottom flange bearing condition. This connection, usually a routed groove in the flange and a matching shaped profile on the web, is a glued joint secured with a waterproof structural adhesive. The contact surfaces include the sides and bottom of the routed flange.

In most cases, the adhesive line stresses at this joint control the bearing length design. The effective bearing length of the web into the flange is approximately the length of flange bearing onto the support plus an incremental length related to the thickness and stiffness of the flange material.

Since most wood I-joists have web shear capacity in excess of the flange to web joint strength, connection reinforcement is sometimes utilized. The most common method of reinforcement is the addition of web stiffeners (also commonly referred to as bearing blocks). Web stiffeners are vertically oriented wood blocks positioned on both sides of the web. Web stiffeners should be cut so that a gap of at least 1/8" is between the stiffener and the flange to avoid a force fit. Stiffeners are positioned tight to the bottom flange at bearing locations and snug to the bottom of the top flange beneath heavy point loads within a span. Figure M7.4-3 provides an illustration of a typical end bearing assembly.

Figure M7.4-3 End Bearing Web Stiffeners (Bearing Block)

END VIEW

SIDE VIEW

Web Stiffeners

When correctly fastened to the joist web, web stiffeners transfer some of the load from the web into the top of the bottom flange. This reduces the loads on the web to flange joint. A pair of web stiffeners (one on each side) is usually mechanically connected to the web with nails or staples loaded in double shear. For some of the higher capacity wood I-joists, nailing and supplemental gluing with a structural adhesive is required. The added bearing capacity achievable with web stiffeners is limited by the allowable bearing stresses where the stiffeners contact the bearing flange and by their mechanical connection to the web.

Web stiffeners also serve the implied function of reinforcing the web against buckling. Since shear capacity usually increases proportionately with the depth, web stiffeners are very important for deep wood I-joists. For example, a 30" deep wood I-joist may only develop 20%

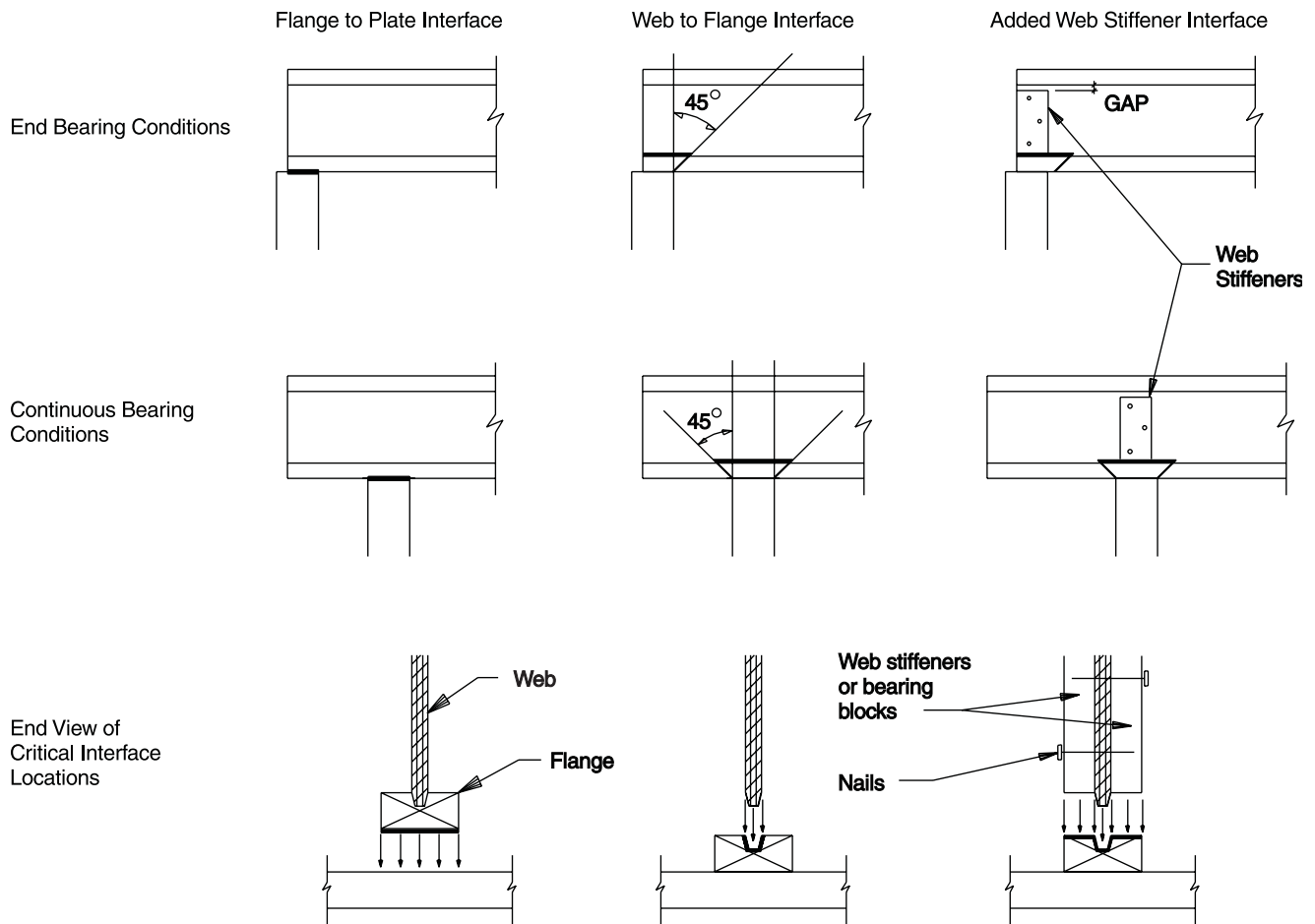
to 30% of its shear and bearing capacity without properly attached web stiffeners at the bearing locations. This is especially important at continuous span bearing locations, where reaction magnitudes can exceed simple span reactions by an additional 25%.

Web stiffeners should be cut so that a gap of at least 1/8" is between the stiffener and the top or bottom of the flange to avoid a force fit. Web stiffeners should be installed snug to the bottom flange for bearing reinforcement or snug to the top flange if under concentrated load from above.

For shallow depth joists, where relatively low shear capacities are required, web stiffeners may not be needed. When larger reaction capacities are required, web stiffener reinforcement may be needed, especially where short bearing lengths are desired. Figure M7.4-4 illustrates the bearing interfaces.

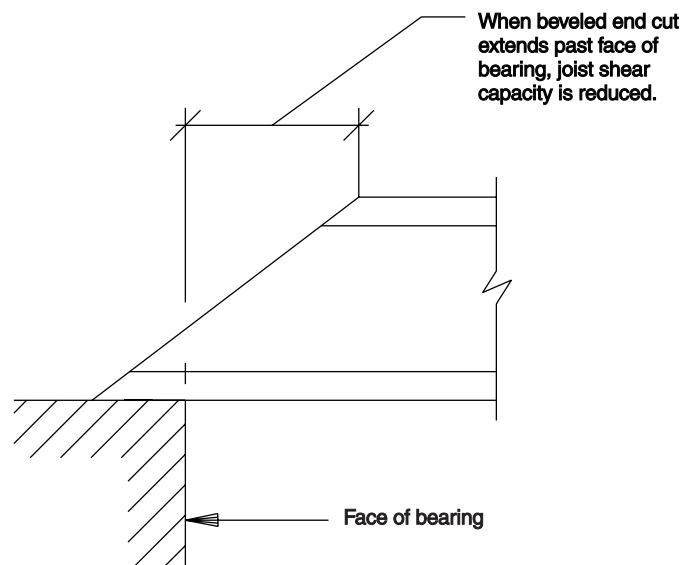
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M7: PREFABRICATED WOOD I-JOISTS

Figure M7.4-4 Web Stiffener Bearing Interface**Beveled End Cuts**

Beveled end cuts, where the end of the joist is cut on an angle (top flange does not project over the bearing, much like a fire cut), also requires special design consideration. Again the severity of the angle, web material, location of web section joints, and web stiffener application criteria effect the performance of this type of bearing condition. The specific wood I-joist manufacturers should be consulted for limits on this type of end cut.

It is generally accepted that if a wood I-joist has the minimum required bearing length, and the top flange of the joist is not cut beyond the face of bearing (measured from a line perpendicular to the joist's bottom flange), there is no reduction in shear or reaction capacity. This differs from the conventional lumber provision that suggests there is no decrease in shear strength for beveled cuts of up to an angle of 45°. The reason involves the composite nature of the wood I-joist and how the member fails in shear and or bearing. Figure M7.4-5 provides an illustration of the beveled end cut limitation.

Figure M7.4-5 Beveled End Cut

Sloped Bearing Conditions

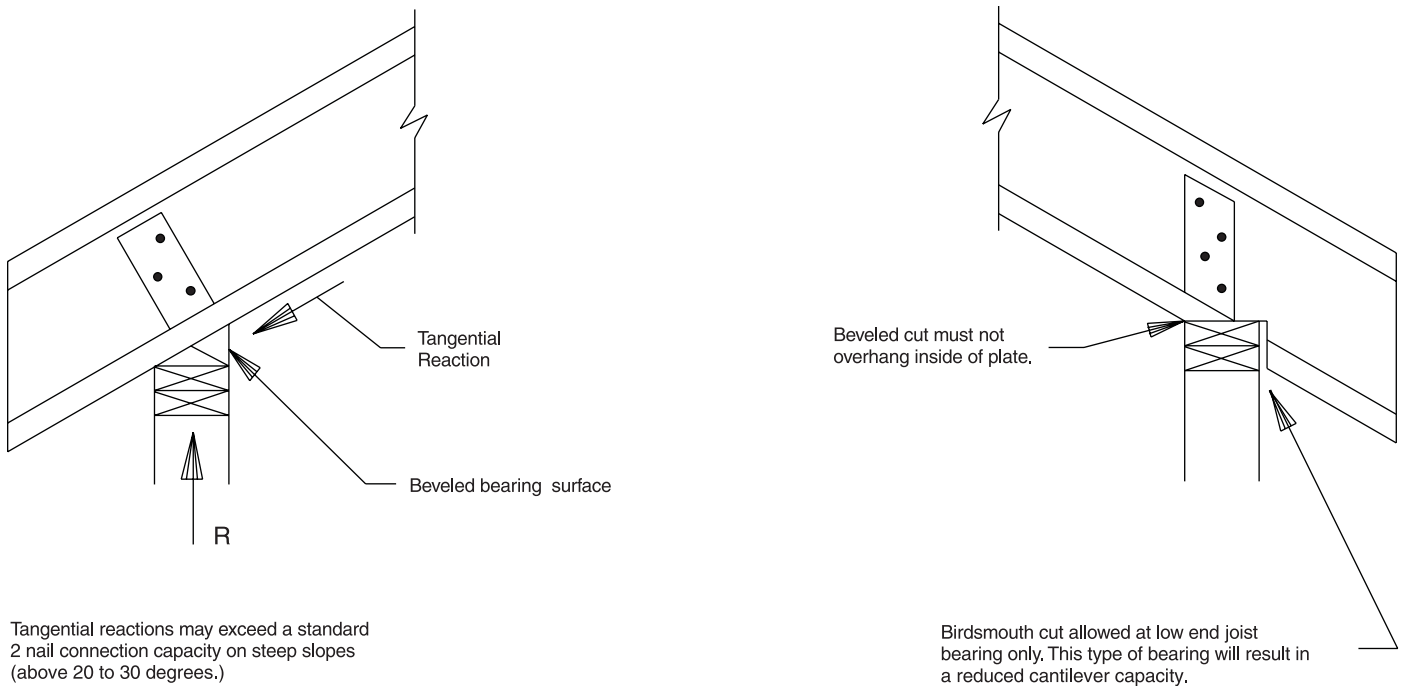
Sloped bearing conditions require design considerations different from conventional lumber. An example is a birdsmouth bearing cut (notches in the bottom flange, see Figure M7.4-6). This type of bearing should only be used on the low end bearing for wood I-joists. Another example is the use of metal joist support connectors that attach only to the web area of the joist and do not provide a bottom seat in which to bear. In general, this type of connector is not recommended for use with wood I-joists without consideration for the resulting reduced capacity.

The birdsmouth cut is a good solution for the low end bearing when the slope is steep and the tangential loads are high (loads along the axis of the joist member). This assumes the quality of construction is good and the cuts are made correctly and at the right locations. This type of

bearing cut requires some skill and is not easy to make, particularly with the wider flange joists. The bearing capacity, especially with high shear capacity members, may be reduced as a result of the cut since the effective flange bearing area is reduced. The notched cut will also reduce the member's shear and moment capacity at a cantilever location.

An alternative to a birdsmouth cut is a beveled bearing plate matching the joist slope or special sloped seat bearing hardware manufactured by some metal connector suppliers. These alternatives also have special design considerations with steep slope applications. As the member slope increases, so does the tangential component of reaction, sometimes requiring additional flange to bearing nailing or straps to provide resistance. Figure M7.4-6 shows some examples of acceptable low end bearing conditions.

Figure M7.4-6 Sloped Bearing Conditions (Low End)



Tangential reactions may exceed a standard 2 nail connection capacity on steep slopes (above 20 to 30 degrees.)

Birdsmouth cut allowed at low end joist bearing only. This type of bearing will result in a reduced cantilever capacity.

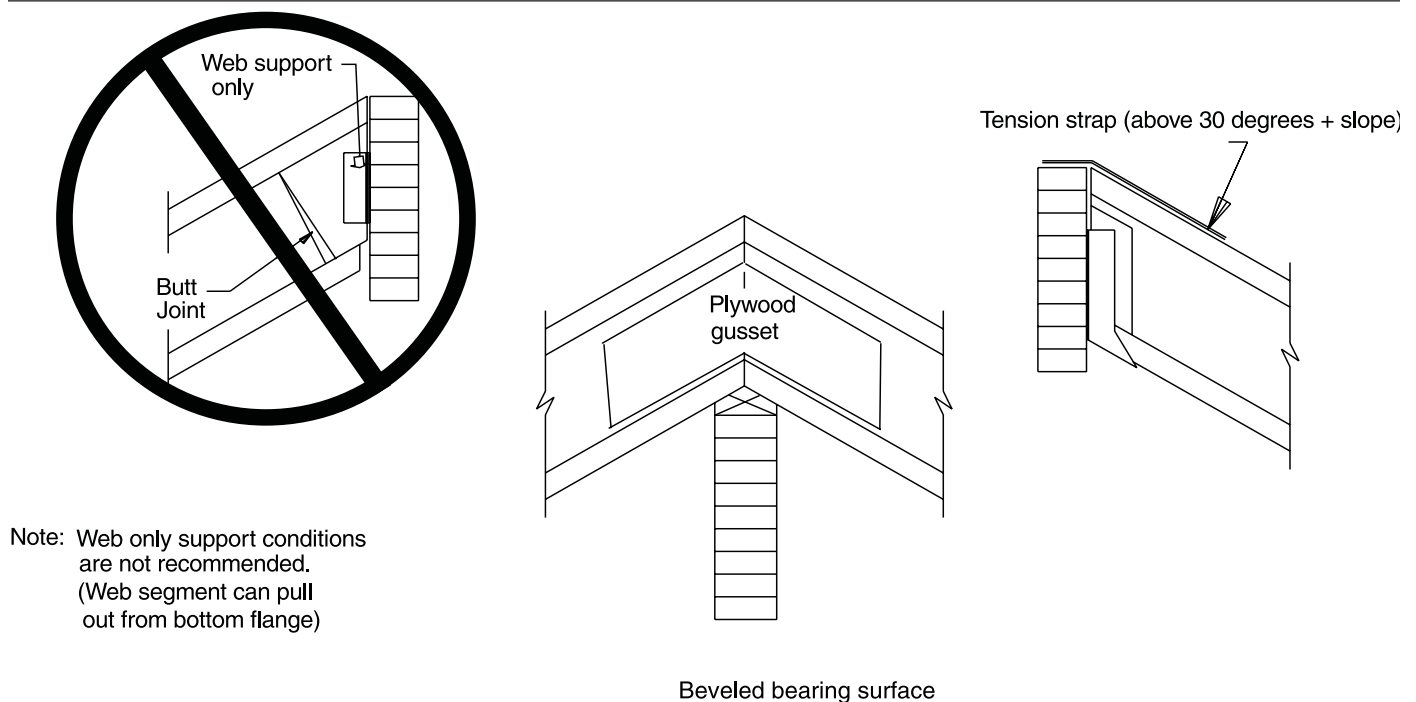
For the high end support, bottom flange bearing in a suitable connector or on a beveled plate is recommended. When slopes exceed 30°, straps or gussets may be needed to resist the tangential component of the reaction.

Support connections only to the web area of a wood I-joist, especially at the high end of a sloped application, are not generally recommended. Since a wood I-joist is comprised of a number of pieces, joints between web sections occurring near the end of the member may reduce the joist's shear capacity when not supported from the bottom flange.

When a wood I-joist is supported from the web only, the closest web to web joint from the end may be stressed in tension. This could result in a joint failure with the web section pulling out of the bottom flange. Locating these internal joints away from the end of the member or applying joint reinforcements are potential remedies, but generally are not practical in the field.

The best bearing solution is to provide direct support to the joist's bottom flange to avoid reductions in capacity. Figure M7.4-7 shows typical high end bearing conditions.

Figure M7.4-7 Sloped Bearing Conditions (High End)



Connector Design/Joist Hangers

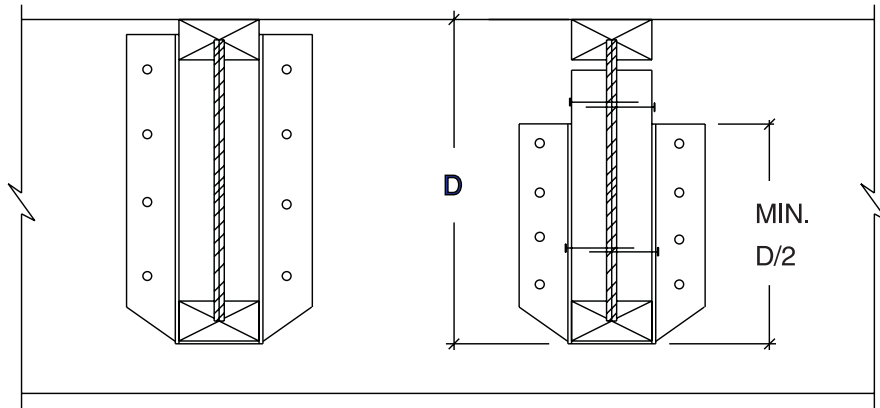
Although there are numerous hangers and connectors available that are compatible with wood I-joists, many are not. Hangers developed for conventional lumber or glulam beams often use large nails and space them in a pattern that will split the joist flanges and web stiffeners. Hanger selection considerations for wood I-joists should include nail length and diameter, nail location, wood I-joist bearing capacity, composition of the supporting member, physical fit, and load capacity. For example, hangers appropriate for a wood I-joist to glulam beam support may not be compatible for an I-joist to I-joist connection.

In general, nails into the flanges should not exceed the diameter of a 10d common nail, with a recommended length no greater than 1½". Nails into web stiffeners should not exceed the diameter of a 16d common nail. Nails through

the sides of the hanger, when used in combination with web stiffeners, can be used to reduce the joist's minimum required bearing length. Nails help transfer loads directly from the I-joist web into the hanger, reducing the load transferred through direct bearing in the bottom hanger seat.

Hangers should be capable of providing lateral support to the top flange of the joist. This is usually accomplished by a hanger flange that extends the full depth of the joist. As a minimum, hanger support should extend to at least mid-height of a joist used with web stiffeners. Some connector manufacturers have developed hangers specifically for use with wood I-joists that provide full lateral support without the use of web stiffeners. Figure M7.4-8 illustrates lateral joist support requirements for hangers. *NDS Commentary* section C7.3.5 also provides additional guidance on direct attachment of sheathing to the I-joist compression flange to provide lateral support.

Figure M7.4-8 Lateral Support Requirements for Joists in Hangers



Top and bottom joist flanges must be laterally restrained against rotation.

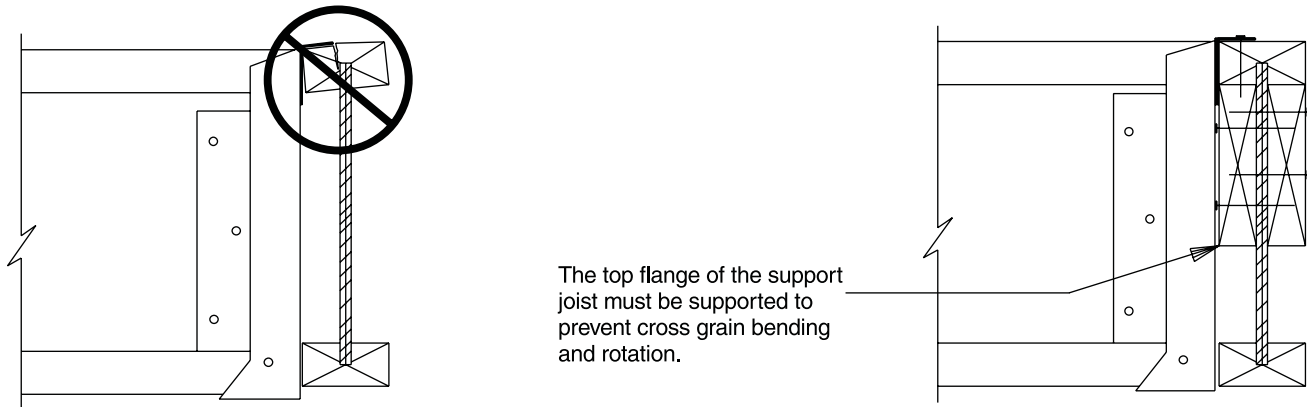
When top flange style hangers are used to support one I-joist from another, especially the wider flange I-joists, web stiffeners need to be installed tight to the bottom side of the support joist's top flanges. This prevents cross grain bending and rotation of the top flange (see Figure M7.4-9).

When face nail hangers are used for joist to joist connections, nails into the support joist should extend through and beyond the web element (Figure M7.4-10). Filler blocks should also be attached sufficiently to provide

vide support for the hanger. Again, nail diameter should be considered to avoid splitting the filler block material.

Multiple I-joists need to be adequately connected together to achieve desired performance. This requires proper selection of a nailing or bolting pattern and attention to web stiffener and blocking needs. Connections should be made through the webs of the I-joists and never through the flanges.

Figure M7.4-9 Top Flange Hanger Support



The top flange of the support joist must be supported to prevent cross grain bending and rotation.

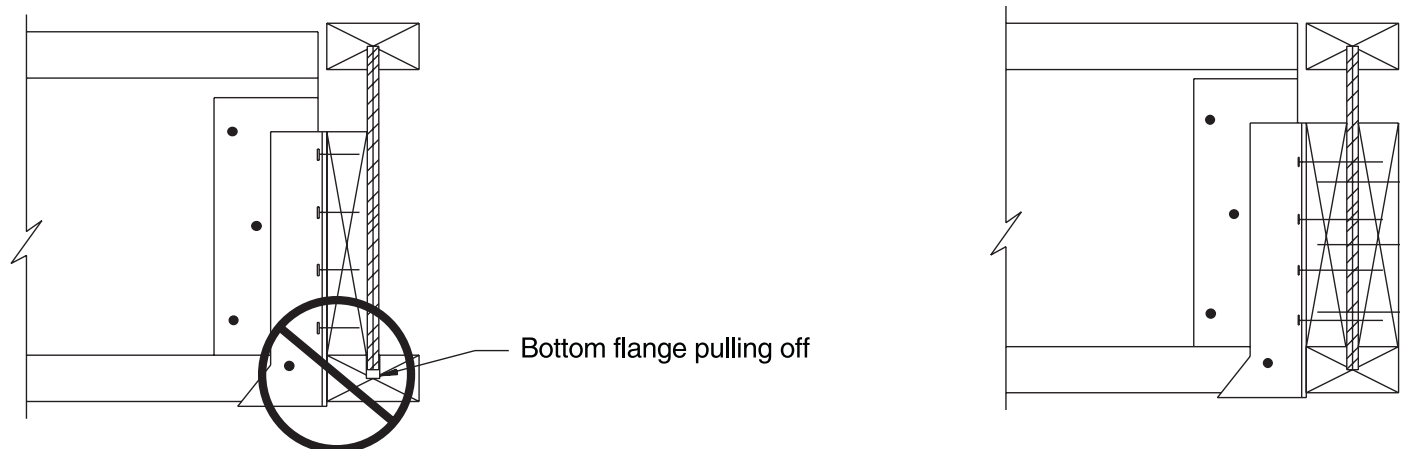
Caution: Large diameter nails can cause splitting.

For a double I-joint member loaded from one side only, the minimum connection between members should be capable of transferring at least 50% of the applied load. Likewise, for a triple member loaded from one side only, the minimum connection between members must be capable of transferring at least 2/3 of the applied load. The actual connection design should consider the potential slip and differential member stiffness. Many manufacturers recommend limiting multiple members to three joists. Multiple I-joists with 3½" wide flanges may be further limited to two members.

The low torsional resistance of most wood I-joists is also a design consideration for joist to joist connections. Eccentrically applied side loads, such as a top flange hanger hung from the side of a double joist, create the potential for joist rotation. Bottom flange restraining straps, blocking, or directly applied ceiling systems may be needed on heavily loaded eccentric connections to resist rotation. Figure M7.4-10 shows additional I-joint connection considerations for use with face nail hangers.

Figure M7.4-10 Connection Requirements for Face Nail Hangers

Hanger nails must extend past the supporting joist's web member into the backer blocking



Vertical Load Transfer

Bearing loads originating above the joists at the bearing location require blocking to transfer these loads around the wood I-joist to the supporting wall or foundation. This is typically the case in a multi-story structure where bearing walls stack and platform framing is used. Usually, the available bearing capacity of the joist is needed to support its reaction, leaving little if any excess capacity to support additional bearing wall loads from above.

The most common type of blocking uses short pieces of wood I-joist, often referred to as blocking panels, positioned directly over the lower bearing and cut to fit in between the joists. These panels also provide lateral

support for the joists and an easy means to transfer lateral diaphragm shears.

The ability to transfer lateral loads (due to wind, seismic, construction loads, etc.) to shear walls or foundations below is important to the integrity of the building design. Compared with dimension lumber blocking, which usually is toe-nailed to the bearing below, wood I-joist blocking can develop higher diaphragm transfer values because of a wider member width and better nail values.

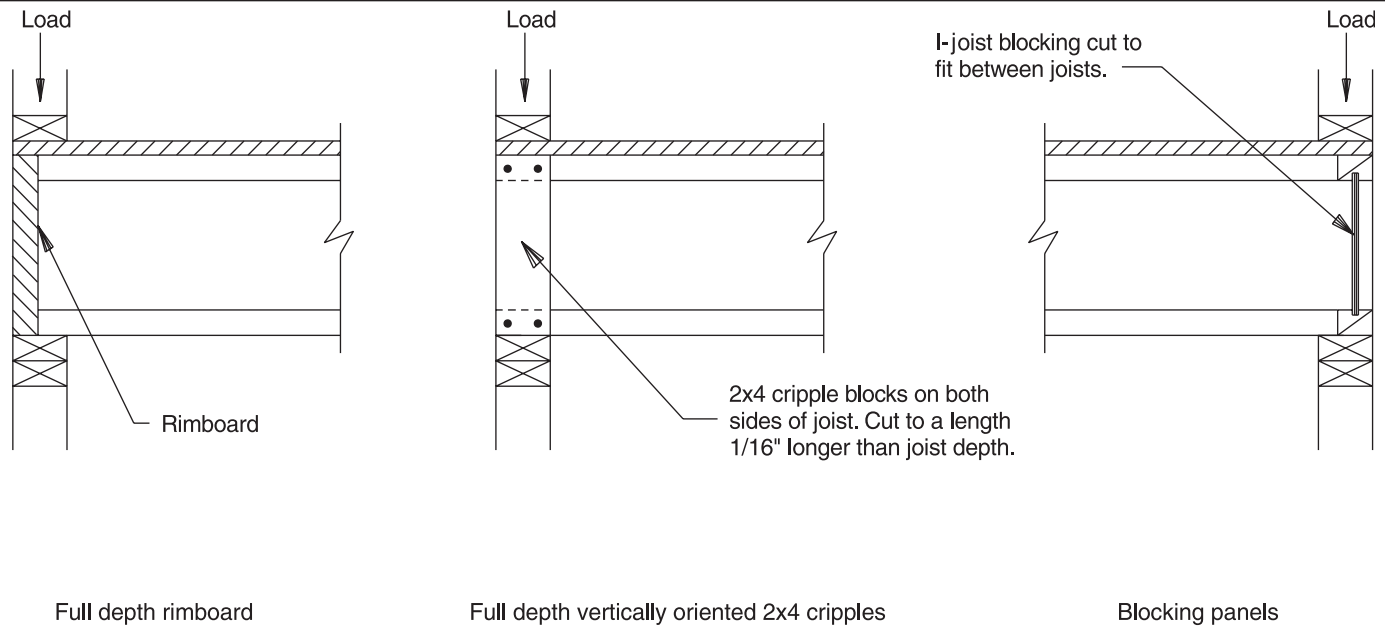
Specialty products designed specifically for rim boards are pre-cut in strips equal to the joist depth and provide support for the loads from above. This solution may also provide diaphragm boundary nailing for lateral loads.

A third method uses vertically oriented short studs, often called squash blocks or cripple blocks, on each side of the joist and cut to a length slightly longer than the depth of the joist. This method should be used in combination with some type of rim joist or blocking material when lateral stability or diaphragm transfer is required.

The use of horizontally oriented sawn lumber as a blocking material is unacceptable. Wood I-joists generally

do not shrink in the vertical direction due to their panel type web, creating the potential for a mismatch in height as sawn lumber shrinks to achieve equilibrium. When conventional lumber is used in the vertical orientation, shrinkage problems are not a problem because changes in elongation due to moisture changes are minimal. Figure M7.4-11 shows a few common methods for developing vertical load transfer.

Figure M7.4-11 Details for Vertical Load Transfer



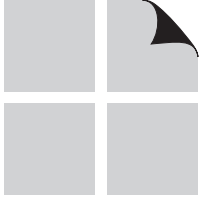
Web Holes

Holes cut in the web area of a wood I-joist affect the member's shear capacity. Usually, the larger the hole, the greater the reduction in shear capacity. For this reason, holes are generally located in areas where shear stresses are low. This explains why the largest holes are generally permitted near mid-span of a member. The required spacing between holes and from the end of the member is dependent upon the specific materials and processes used during manufacturing.

The allowable shear capacity of a wood I-joist at a hole location is influenced by a number of variables. These include: percentage of web removed, proximity to a vertical joint between web segments, the strength of the web to flange glue joint, the stiffness of the flange, and the shear strength of the web material. Since wood I-joists are manufactured using different processes and materials, each manufacturer should be consulted for the proper web hole design.

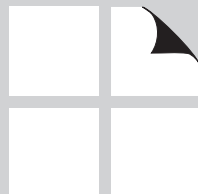
The methodology used to analyze application loads is important in the evaluation of web holes. All load cases that will develop the highest shear at the hole location should be considered. Usually, for members resisting simple uniform design loads, the loading condition that develops the highest shear loads in the center area of a joist span involves partial span loading.

Web holes contribute somewhat to increased deflection. The larger the hole the larger the contribution. Provided not too many holes are involved, the contribution is negligible. In most cases, if the manufacturer's hole criteria are followed and the number of holes is limited to three or less per span, the additional deflection does not warrant consideration.



M8: STRUCTURAL COMPOSITE LUMBER

M8.1	General	44
M8.2	Reference Design Values	45
M8.3	Adjustment of Reference Design Values	46
M8.4	Special Design Considerations	47



M8.1 General

Product Information

Structural composite lumber (SCL) products are well known throughout the construction industry. The advantages of SCL include environmental benefits from better wood fiber utilization along with higher strength, stiffness, and consistency from fiber orientation and manufacturing process control.

SCL is manufactured from strips or full sheets of veneer. The process typically includes alignment of stress graded fiber, application of adhesive, and pressing the material together under heat and pressure. By redistributing natural defects and through state of the art quality control procedures, the resulting material is extremely consistent and maximizes the strength and stiffness of the wood fiber.

The material is typically produced in a long length continuous or fixed press in a billet form. This is then resawn into required dimensions for use. Material is currently available in a variety of depths from 4-3/8" to 24" and thicknesses from 3/4" to 7".

SCL is available in a wide range of sizes and grades. When specifying SCL products, a customer may specify on the basis of size, stress (strength), or appearance.

SCL products are proprietary and are covered by code evaluation reports. Such reports should be consulted for current design information while manufacturer's literature can be consulted for design information, sizing tables, and installation recommendations.

Common Uses

SCL is widely used as a framing material for housing. SCL is made in different grades and with various processes and can be utilized in numerous applications. Proper design is required to optimize performance and economics.

In addition to use in housing, SCL finds increasing use in commercial and industrial construction. Its high strength, stiffness, universal availability, and cost saving attributes make it a viable alternative in most low-rise construction projects.

SCL is used as beams, headers, joists, rafters, studs, and plates in conventional construction. In addition, SCL is used to fabricate structural glued laminated beams, trusses, prefabricated wood I-joists, and cross-laminated timber.

Availability

SCL is regarded as a premium construction material and is widely available. To efficiently specify SCL for individual projects, the customer should be aware of the species and strength availability. Sizes vary with each individual product. The best source of this information is your local lumber supplier, distribution center, or SCL manufacturer. Proper design is facilitated through the use of manufacturer's literature, code reports, and software available from SCL manufacturers.

M8.2 Reference Design Values

General

As stated in *NDS* 8.2, SCL products are proprietary and each manufacturer develops design values appropriate for their products. These values are reviewed by the model building codes and published in evaluation reports and manufacturer's literature.

Reference design values are used in conjunction with the adjustment factors in M8.3.

Shear Design

SCL is typically designed and installed as a rectangular section. Loads near supports may be reduced per *NDS* 3.4.3.1. However, such load must be included in bearing calculations. Shear values for SCL products often change with member orientation.

Bearing

SCL typically has high F_c and $F_{c\perp}$ properties. With the higher shear and bending capacities, shorter or continuous spans are often controlled by bearing. The user is cautioned to ensure the design accounts for compression of the support material (i.e., plate) as well as the beam material. Often the plate material is of softer species and will control the design.

Bending

Published bending capacities of SCL beams are determined from testing of production specimens. Adjustment for the size of the member is also determined by test.

Field notching or drilling of holes is typically not allowed. Similarly, excessive nailing or the use of improper nail sizes can cause splitting that will also reduce capacity. The manufacturer should be contacted when evaluating a damaged beam.

Deflection Design

Deflection calculations for SCL typically are similar to provisions for other rectangular wood products (see M3.5). Values for use in deflection equations can be found in the individual manufacturer's product literature or evaluation reports. Some manufacturers might publish "true" E values which would require additional calculations to account for shear deflection (see *NDS* Appendix F).

M8.3 Adjustment of Reference Design Values

Member design capacity is the product of reference design values, adjustment factors, and section properties. Reference design values for SCL are discussed in M8.2.

Adjustment factors are provided for applications outside the reference end-use conditions and for member configuration effects as specified in *NDS* 8.3. When one or more of the specific end use or member configuration conditions are beyond the range of the reference conditions, these adjustment factors shall be used to modify the appropriate property. Adjustment factors for the effects of moisture, temperature, member configuration, and size are

provided in *NDS* 8.3. Additional adjustment factors can be found in the manufacturer's product literature or code evaluation report. Table M8.3-1 shows the applicability of adjustment factors for SCL in a slightly different format for the designer.

Certain products may not be suitable for use in some applications or with certain treatments. Such conditions can result in structural deficiencies and may void manufacturer warranties. The manufacturer or code evaluation report should be consulted for specific information.

Table M8.3-1 Applicability of Adjustment Factors for Structural Composite Lumber¹

Allowable Stress Design	Load and Resistance Factor Design
$F_b' = F_b C_D C_M C_t C_L C_V C_r$	$F_b' = F_b C_M C_t C_L C_V C_r (2.54)(0.85) \lambda$
$F_t' = F_t C_D C_M C_t$	$F_t' = F_t C_M C_t (2.70)(0.80) \lambda$
$F_v' = F_v C_D C_M C_t$	$F_v' = F_v C_M C_t (2.88)(0.75) \lambda$
$F_c' = F_c C_D C_M C_t C_P$	$F_c' = F_c C_M C_t C_P (2.40)(0.90) \lambda$
$F_{c\perp}' = F_{c\perp} C_M C_t C_b$	$F_{c\perp}' = F_{c\perp} C_M C_t C_b (1.67)(0.90)$
$E' = E C_M C_t$	$E' = E C_M C_t$
$E_{min}' = E_{min} C_M C_t$	$E_{min}' = E_{min} C_M C_t (1.76)(0.85)$

1. See *NDS* 8.3.6 for information on simultaneous application of the volume factor, C_v , and the beam stability factor, C_L .

Bending Member Example

For fully laterally supported members stressed in strong axis bending and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 8.3), the adjusted design values reduce to:

For ASD:

$$F_b' = F_b C_D C_V$$

$$F_v' = F_v C_D$$

$$F_{c\perp}' = F_{c\perp} C_b$$

$$E' = E$$

For LRFD:

$$F_b' = F_b C_V (2.54)(0.85) \lambda$$

$$F_v' = F_v (2.88)(0.75) \lambda$$

$$F_{c\perp}' = F_{c\perp} C_b (1.67)(0.90)$$

$$E' = E$$

Axially Loaded Member Example

For axially loaded members used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 8.3) designed to resist tension or compression loads, the adjusted tension or compression design values reduce to:

For ASD:

$$F_c' = F_c C_D C_P$$

$$F_t' = F_t C_D$$

$$E_{min}' = E_{min}$$

For LRFD:

$$F_c' = F_c C_P (2.40)(0.90) \lambda$$

$$F_t' = F_t (2.70)(0.80) \lambda$$

$$E_{min}' = E_{min} (1.76)(0.85)$$

M8.4 Special Design Considerations

General

With proper detailing and protection, SCL can perform well in a variety of environments. One key to proper detailing is planning for the natural shrinkage and swelling of wood members as they are subjected to various drying and wetting cycles. While moisture changes have the largest impact on lumber dimensions, some designs must also check the effects of temperature. While SCL is typically produced using dry veneer, some moisture accumulation may occur during storage. If the product varies significantly from specified dimensions, the user is cautioned from using such product as it will “shrink” as it dries.

In addition to designing to accommodate dimensional changes and detailing for durability, another significant issue in the planning of wood structures is that of fire performance, which is covered in Chapter M16.

Dimensional Changes

The dimensional stability and response to temperature effects of engineered lumber is similar to that of solid sawn lumber of the same species.

Some densification of the wood fiber can occur in various manufacturing processes. SCL that is densified will result in a product that has more wood fiber in a given volume and can, therefore, hold more water than a solid sawn equivalent. When soaked these products expand and dimensional changes can occur.

Adhesive applied during certain processes tends to form a barrier to moisture penetration. Therefore, the material will typically take longer to reach equilibrium than its solid sawn counterpart.

For given temperatures and applications, different levels of relative humidity are present. This will cause the material to move toward an equilibrium moisture content (EMC). Eventually all wood products will reach their EMC for a given environment. SCL will typically equilibrate at a lower EMC (typically 3% to 4% lower) than solid sawn lumber and will take longer to reach an ambient EMC. Normal swings in humidity during the service life of the structure should not produce noticeable dimensional changes in SCL members.

More information on designing for moisture and temperature change is included in M4.4.

Durability

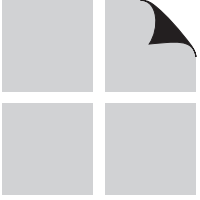
Designing for durability is a key part of the architectural and engineering design of the building. Wood exposed to high levels of moisture can decay over time. While there are exceptions – such as naturally durable species, preservative-treated wood, and those locations that can completely air-dry between moisture cycles – prudent design calls for a continuing awareness of the possibility of moisture accumulation. Awareness of the potential for decay is the key – many design conditions can be detailed to minimize the accumulation of moisture; for other problem conditions, preservative-treated wood or naturally durable species should be specified.

This section cannot cover the topic of designing for durability in detail. There are many excellent texts that devote entire chapters to the topic, and designers are advised to use this information to assist in designing “difficult” design areas, such as:

- structures in high moisture or humid conditions,
- where wood comes in contact with concrete or masonry,
- where wood members are supported in steel hangers or connectors in which condensation could collect,
- anywhere that wood is directly or indirectly exposed to the elements,
- where wood, if it should ever become wet, could not naturally dry out.

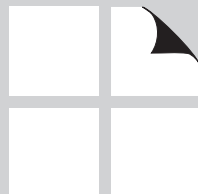
This list is not intended to be all-inclusive – it is merely an attempt to alert designers to special conditions that may cause problems when durability is not considered in the design.

More information on detailing for durability is included in M4.4.



M9: WOOD STRUCTURAL PANELS

M9.1	General	50
M9.2	Reference Design Values	50
M9.3	Adjustment of Reference Design Values	56
M9.4	Design Considerations	57



M9.1 General

Product Description

Wood structural panels are wood-based panel products that have been rated for use in structural applications. Common applications for wood structural panels include roof sheathing, wall sheathing, subflooring, and single-layer flooring (combination subfloor-underlayment). Structural plywood is also manufactured in various sanded grades.

Wood structural panels are classified by span ratings. Panel span ratings identify the maximum recommended support spacings in inches for specific end uses. The span rating applies when the long panel dimension or strength axis is across supports, unless the strength axis is otherwise identified. Design capacities are provided on the basis of span ratings.

Sanded grades are classed according to grade and performance category and design capacities are provided on that basis.

Designers must specify wood structural panels by the span ratings, performance category, grades, and construc-

tions associated with tabulated design recommendations. Exposure durability classification must also be identified.

Wood structural panel adhesive bond classification relates to moisture resistance of the glue bond, and thus to structural integrity of the panel. Bond classification of the panel does not relate to physical (erosion, ultraviolet, etc.) or biological (mold, fungal decay, insect, etc.) resistance of the panel. Exposure 1 panels may be used for applications where construction delays may be expected prior to providing protection or where exposure to the outdoors is on the underside only. Exterior bond classification indicates the glue bond is suitable for applications subject to long-term exposure to weather or moisture.

Single-floor panels typically have tongue-and-groove edges. If square edge panels are specified for single-floor applications, the specification shall require lumber blocking between supports.

Table M9.1-1 provides descriptions and typical uses for various panel grades and types.

M9.2 Reference Design Values

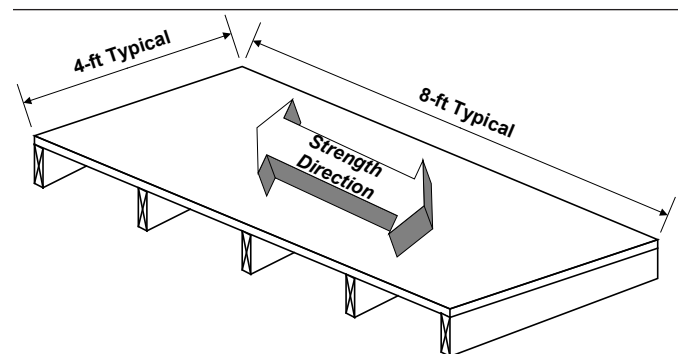
General

Wood structural panel design capacities listed in Tables M9.2-1 and M9.2-2 are minimum for grade and span rating. Multipliers shown in each table provide adjustments in capacity for Structural I panel grades. To take advantage of these multipliers, the specifier must insure that the correct panel is used in construction.

The tabulated capacities and adjustment factors are based on data from tests of panels manufactured in accordance with industry standards and which bear the trademark of a qualified inspection and testing agency.

Structural panels have a strength axis direction and a cross panel direction. The direction of the strength axis is defined as the axis parallel to the orientation of OSB face strands or plywood face veneer grain and is typically the long dimension of the panel unless otherwise indicated by the manufacturer. This is illustrated in Figure M9.2-1.

Figure M9.2-1 Structural Panel with Strength Direction Across Supports



Panel Stiffness and Strength

Panel design capacities listed in Table M9.2-1 are based on flat panel bending (Figure M9.2-2) as measured by testing according to principles of ASTM D3043 Method C (large panel testing).

Stiffness (EI)

Panel bending stiffness is the capacity to resist deflection and is represented as EI. E is the reference modulus of elasticity of the material, and I is the moment of inertia

of the cross section. The units of EI are lbf-in.² per foot of panel width.

Strength (F_bS)

Bending strength capacity is the design maximum moment, represented as F_bS. F_b is the reference extreme fiber bending stress of the material, and S is the section modulus of the cross section. The units of F_bS are lbf-in. per foot of panel width.

Figure M9.2-2 Example of Structural Panel in Bending

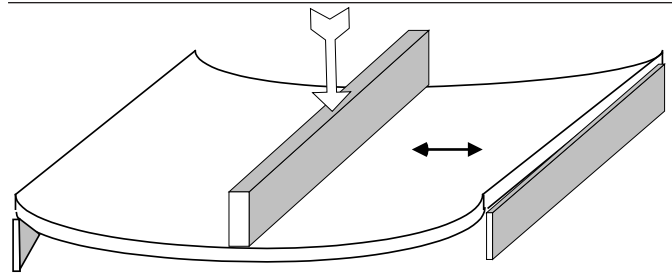


Table M9.1-1 Guide to Panel Use

Panel Grade and Bond Classification	Description & Use	Common Performance Category (in.)	Panel Construction	
			OSB	Plywood Minimum Veneer Grade
Sheathing EXP 1	Unsanded sheathing grade for wall, roof, subflooring, and industrial applications such as pallets and for engineering design with proper capacities.	5/16, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4	Yes	Yes
Structural I Sheathing EXP 1	Panel grades to use where shear and cross-panel strength properties are of maximum importance. Plywood Structural I is made from all Group 1 woods.	3/8, 7/16, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4	Yes	Yes
Single Floor EXP 1	Combination subfloor-underlayment. Provides smooth surface for application of carpet and pad. Possesses high concentrated and impact load resistance during construction and occupancy. Touch-sanded. Available with tongue-and-groove edges.	19/32, 5/8, 23/32, 3/4, 7/8, 1, 1-3/32, 1-1/8	Yes	Yes
Underlayment EXP 1 or EXT	For underlayment under carpet and pad. Available with exterior glue. Touch-sanded or sanded. Panels with performance category of 19/32 or greater may be available with tongue-and-groove edges.	1/4, 11/32, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4	No	Yes, face C-Plugged, back D, inner D
C-D-Plugged EXP 1	For built-ins, wall and ceiling tile backing. Not for underlayment. Touch-sanded.	1/2, 19/32, 5/8, 23/32, 3/4	No	Yes, face C-Plugged, back D, inner D
Sanded Grades EXP 1 or EXT	Generally applied where a high-quality surface is required. Includes A-A, A-C, A-D, B-B, B-C, and B-D grades.	1/4, 11/32, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4	No	Yes, face B or better, back D or better, inner C & D
Marine EXT	Superior Exterior-type plywood made only with Douglas-fir or western larch. Special solid-core construction. Available with medium density overlay (MDO) or high density overlay (HDO) face. Ideal for boat hull construction.	1/4, 11/32, 3/8, 15/32, 1/2, 19/32, 5/8, 23/32, 3/4	No	Yes, face A or face B, back A or inner B

Table M9.2-1 Wood Structural Panel Bending Stiffness and Strength Capacities

Span Rating	Stress Parallel to Strength Axis ¹				Stress Perpendicular to Strength Axis ¹			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply		3-ply	4-ply	5-ply	
PANEL BENDING STIFFNESS, EI (lbf-in.²/ft of panel width)								
24/0	66,000	66,000	66,000	60,000	3,600	7,900	11,000	11,000
24/16	86,000	86,000	86,000	78,000	5,200	11,500	16,000	16,000
32/16	125,000	125,000	125,000	115,000	8,100	18,000	25,000	25,000
40/20	250,000	250,000	250,000	225,000	18,000	39,500	56,000	56,000
48/24	NA	440,000	440,000	400,000	NA	65,000	91,500	91,500
16oc	165,000	165,000	165,000	150,000	11,000	24,000	34,000	34,000
20oc	230,000	230,000	230,000	210,000	13,000	28,500	40,500	40,500
24oc	NA	330,000	330,000	300,000	NA	57,000	80,500	80,500
32oc	NA	NA	715,000	650,000	NA	NA	235,000	235,000
48oc	NA	NA	1,265,000	1,150,000	NA	NA	495,000	495,000
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.5	1.5	1.6	1.6
PANEL BENDING STRENGTH, F_bS (lbf-in./ft of panel width)								
24/0	250	275	300	300	54	65	97	97
24/16	320	350	385	385	64	77	115	115
32/16	370	405	445	445	92	110	165	165
40/20	625	690	750	750	150	180	270	270
48/24	NA	930	1,000	1,000	NA	270	405	405
16oc	415	455	500	500	100	120	180	180
20oc	480	530	575	575	140	170	250	250
24oc	NA	705	770	770	NA	260	385	385
32oc	NA	NA	1,050	1,050	NA	NA	685	685
48oc	NA	NA	1,900	1,900	NA	NA	1,200	1,200
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.3	1.4	1.5	1.5

1. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

NA - Not applicable. Atypical panel construction.

Axial Capacities

Axial Stiffness (EA)

Panel axial stiffnesses listed in Table M9.2-2 are based on testing according to the principles of ASTM D3501 Method B. Axial stiffness is the capacity to resist axial strain and is represented as EA. E is the reference axial modulus of elasticity of the material, and A is the area of the cross section. The units of EA are pounds force per foot of panel width.

Tension (F_tA)

Tension capacities listed in Table M9.2-2 are based on testing according to the principles of ASTM D3500 Method B. Tension capacity is given as F_tA. F_t is the reference axial tensile stress of the material, and A is the area of the cross section. The units of F_tA are pounds force per foot of panel width.

Table M9.2-2 Wood Structural Panel Axial Stiffness, Tension, and Compression Capacities

Span Rating	Stress Parallel to Strength Axis ¹				Stress Perpendicular to Strength Axis ¹			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply		3-ply	4-ply	5-ply	
PANEL TENSION, F_tA (lbf/ft of panel width)								
24/0	2,300	2,300	3,000	2,300	600	600	780	780
24/16	2,600	2,600	3,400	2,600	990	990	1,300	1,300
32/16	2,800	2,800	3,650	2,800	1,250	1,250	1,650	1,650
40/20	2,900	2,900	3,750	2,900	1,600	1,600	2,100	2,100
48/24	NA	4,000	5,200	4,000	NA	1,950	2,550	2,550
16oc	2,600	2,600	3,400	2,600	1,450	1,450	1,900	1,900
20oc	2,900	2,900	3,750	2,900	1,600	1,600	2,100	2,100
24oc	NA	3,350	4,350	3,350	NA	1,950	2,550	2,550
32oc	NA	NA	5,200	4,000	NA	NA	3,250	3,250
48oc	NA	NA	7,300	5,600	NA	NA	4,750	4,750
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
PANEL COMPRESSION, F_cA (lbf/ft of panel width)								
24/0	2,850	4,300	4,300	2,850	2,500	3,750	3,750	2,500
24/16	3,250	4,900	4,900	3,250	2,500	3,750	3,750	2,500
32/16	3,550	5,350	5,350	3,550	3,100	4,650	4,650	3,100
40/20	4,200	6,300	6,300	4,200	4,000	6,000	6,000	4,000
48/24	NA	7,500	7,500	5,000	NA	7,200	7,200	4,300
16oc	4,000	6,000	6,000	4,000	3,600	5,400	5,400	3,600
20oc	4,200	6,300	6,300	4,200	4,000	6,000	6,000	4,000
24oc	NA	7,500	7,500	5,000	NA	7,200	7,200	4,300
32oc	NA	NA	9,450	6,300	NA	NA	9,300	6,200
48oc	NA	NA	12,150	8,100	NA	NA	10,800	6,750
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
PANEL AXIAL STIFFNESS, EA (lbf/ft of panel width)								
24/0	3,350,000	3,350,000	3,350,000	3,350,000	2,900,000	2,900,000	2,900,000	2,500,000 ²
24/16	3,800,000	3,800,000	3,800,000	3,800,000	2,900,000	2,900,000	2,900,000	2,700,000 ²
32/16	4,150,000	4,150,000	4,150,000	4,150,000	3,600,000	3,600,000	3,600,000	2,700,000
40/20	5,000,000	5,000,000	5,000,000	5,000,000	4,500,000	4,500,000	4,500,000	2,900,000 ³
48/24	NA	5,850,000	5,850,000	5,850,000	NA	5,000,000	5,000,000	3,300,000 ³
16oc	4,500,000	4,500,000	4,500,000	4,500,000	4,200,000	4,200,000	4,200,000	2,700,000
20oc	5,000,000	5,000,000	5,000,000	5,000,000	4,500,000	4,500,000	4,500,000	2,900,000 ³
24oc	NA	5,850,000	5,850,000	5,850,000	NA	5,000,000	5,000,000	3,300,000 ³
32oc	NA	NA	7,500,000	7,500,000	NA	NA	7,300,000	4,200,000
48oc	NA	NA	8,200,000	8,200,000	NA	NA	7,300,000	4,600,000
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

1. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (veneer), which is generally the long panel direction, unless otherwise marked.
 2. The values shall be permitted to be increased to 2,900,000 lbf/ft for the calculation of the bending stiffness (EI_{joist}) of prefabricated wood I-joists.
 3. The values shall be permitted to be increased to 4,500,000 lbf/ft for the calculation of the composite floor bending stiffness (EI_{composite}) of prefabricated wood I-joists.

NA - Not applicable. Atypical panel construction.

Compression ($F_c A$)

Compression (Figure M9.2-3) capacities listed in Table M9.2-2 are based on testing according to the principles of ASTM D3501 Method B. Compression capacity is given as $F_c A$. F_c is the reference axial compression stress of the material, and A is the area of the cross section. The units of $F_c A$ are pounds force per foot of panel width.

Shear Capacities

Shear-in-the-Plane of the Panel ($F_s [Ib/Q]$)

Shear-in-the-plane of the panel (innerlaminar or rolling shear) capacities listed in Table M9.2-3 are based on testing according to the principles of ASTM D2718. Shear strength in the plane of the panel is the capacity to resist horizontal shear breaking loads when loads are applied or developed on opposite faces of the panel (Figure M9.2-4), as in flat panel bending. Planar shear capacity is given as $F_s [Ib/Q]$. F_s is the reference material innerlaminar shear stress, and Ib/Q is the panel cross-sectional shear constant. The units of $F_s [Ib/Q]$ are pounds force per foot of panel width.

Panel Rigidity Through-the-Thickness ($G_v t_v$)

Panel rigidities listed in Table M9.2-4 are based on testing according to the principles of ASTM D2719. Panel rigidity is the capacity to resist deformation under shear through the thickness stress (Figure M9.2-5). Rigidity is given as $G_v t_v$. G_v is the reference modulus of rigidity, and t_v is the effective panel thickness for shear. The units of $G_v t_v$ are pounds force per inch of panel depth (for vertical applications). Multiplication of $G_v t_v$ by panel depth gives GA , used by designers for some applications.

Panel Shear Through-the-Thickness ($F_v t_v$)

Through-the-thickness shear capacities listed in Table M9.2-4 are based on testing according to the principles of ASTM D2719. Allowable shear through-the-thickness is the capacity to resist horizontal shear breaking loads when loads are applied or developed on opposite edges of the panel (Figure M9.2-5), such as in an I-joint web. Where additional support is not provided to prevent buckling, design capacities in Table M9.2-4 are limited to sections 2 ft or less in depth. Deeper sections may require additional reductions. F_v is the reference shear through-the-thickness stress of the material, and t_v is the effective panel thickness for shear. The units of $F_v t_v$ are pounds force per inch of shear resisting panel length.

Figure M9.2-3 Structural Panel with Axial Compression Load in the Plane of the Panel

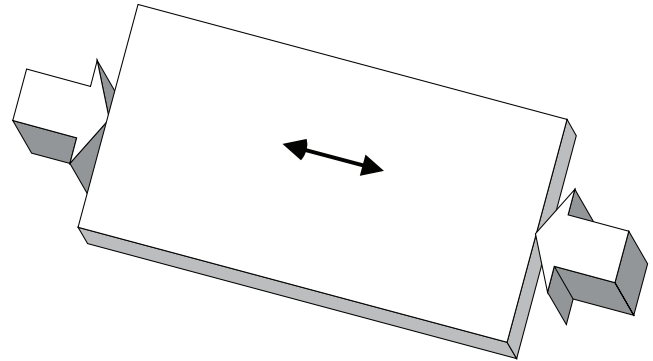


Figure M9.2-4 Shear-in-the-Plane for Wood Structural Panels

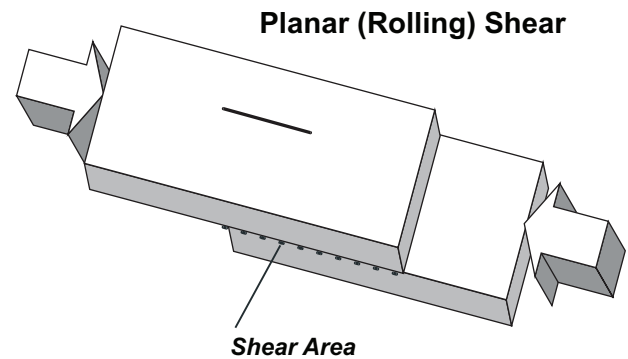


Figure M9.2-5 Through-the-Thickness Shear for Wood Structural Panels

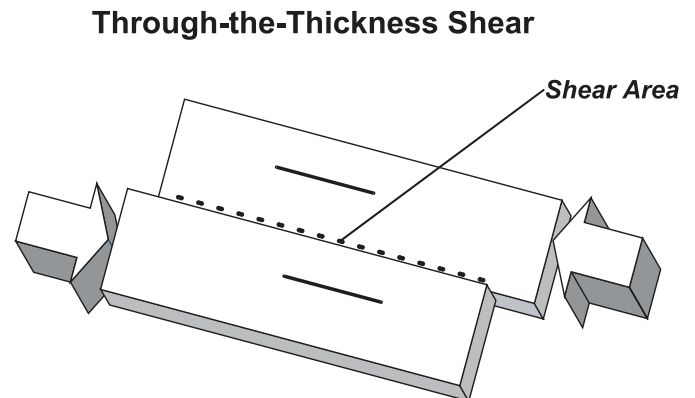


Table M9.2-3 Wood Structural Panel Shear-in-the-Plane Capacities

Span Rating	Stress Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply		3-ply	4-ply	5-ply	
PANEL SHEAR-IN-THE-PLANE, F_s (lbf/Q) (lbf/ft of panel width)								
24/0	155	155	170	130	275	375	130	130
24/16	180	180	195	150	315	435	150	150
32/16	200	200	215	165	345	480	165	165
40/20	245	245	265	205	430	595	205	205
48/24	NA	300	325	250	NA	725	250	250
16oc	245	245	265	205	430	595	205	205
20oc	245	245	265	205	430	595	205	205
24oc	NA	300	325	250	NA	725	250	250
32oc	NA	NA	390	300	NA	NA	300	300
48oc	NA	NA	500	385	NA	NA	385	385
Multiplier for Structural I Panels	1.4	1.4	1.4	1.0	1.4	1.4	1.0	1.0

NA - Not applicable. Atypical panel construction

Table M9.2-4 Wood Structural Panel Rigidity and Shear Through-the-Thickness Capacities

Span Rating	Stress Parallel to Strength Axis				Stress Perpendicular to Strength Axis			
	Plywood			OSB	Plywood			OSB
	3-ply	4-ply	5-ply ¹		3-ply	4-ply	5-ply ¹	
PANEL RIGIDITY THROUGH-THE-THICKNESS, $G_{t,t}$ (lbf/in. of panel depth)								
24/0	25,000	32,500	37,500	77,500	25,000	32,500	37,500	77,500
24/16	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
32/16	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
40/20	28,500	37,000	43,000	88,500	28,500	37,000	43,000	88,500
48/24	NA	40,500	46,500	96,000	NA	40,500	46,500	96,000
16oc	27,000	35,000	40,500	83,500	27,000	35,000	40,500	83,500
20oc	28,000	36,500	42,000	87,000	28,000	36,500	42,000	87,000
24oc	NA	39,000	45,000	93,000	NA	39,000	45,000	93,000
32oc	NA	NA	54,000	110,000	NA	NA	54,000	110,000
48oc	NA	NA	76,000	155,000	NA	NA	76,000	155,000
Multiplier for Structural I Panels	1.3	1.3	1.1	1.0	1.3	1.3	1.1	1.0
PANEL SHEAR THROUGH-THE-THICKNESS, $F_{v,t}$ (lbf/in. of shear-resisting panel length)								
24/0	53	69	80	155	53	69	80	155
24/16	57	74	86	165	57	74	86	165
32/16	62	81	93	180	62	81	93	180
40/20	68	88	100	195	68	88	100	195
48/24	NA	98	115	220	NA	98	115	220
16oc	58	75	87	170	58	75	87	170
20oc	67	87	100	195	67	87	100	195
24oc	NA	96	110	215	NA	96	110	215
32oc	NA	NA	120	230	NA	NA	120	230
48oc	NA	NA	160	305	NA	NA	160	305
Multiplier for Structural I Panels	1.3	1.3	1.1	1.0	1.3	1.3	1.1	1.0

1. 5-ply applies to plywood with five or more layers. For 5-ply plywood with three layers, use $G_{t,t}$ values for 4-ply panels.

NA - Not applicable. Atypical panel construction.

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M9.3 Adjustment of Reference Design Values

General

Adjusted panel design capacities are determined by multiplying reference capacities, as given in Tables M9.2-1 through M9.2-4, by the adjustment factors in *NDS* 9.3. Some adjustment factors should be obtained from the manufacturer or other approved source. In the *NDS Commentary*, C9.3 provides additional information on typical adjustment factors.

Tabulated capacities provided in this Chapter are suitable for reference end-use conditions. Reference end-use conditions are consistent with conditions typically associated with light-frame construction. For wood

structural panels, these typical conditions involve the use of full-sized untreated panels in moderate temperature and moisture exposures.

Appropriate adjustment factors are provided for applications in which the conditions of use are inconsistent with reference conditions. In addition to temperature and moisture, this includes consideration of panel treatment and size effects.

NDS Table 9.3.1 lists applicability of adjustment factors for wood structural panels. Table M9.3-1 shows the applicability of adjustment factors for wood structural panels in a slightly different format for the designer.

Table M9.3-1 Applicability of Adjustment Factors for Wood Structural Panels

Allowable Stress Design	Load and Resistance Factor Design
$F_b S' = F_b S C_D C_M C_t C_s$	$F_b S' = F_b S C_M C_t C_s (2.54)(0.85) \lambda$
$F_t A' = F_t A C_D C_M C_t C_s$	$F_t A' = F_t A C_M C_t C_s (2.70)(0.80) \lambda$
$F_v t_v' = F_v t_v C_D C_M C_t$	$F_v t_v' = F_v t_v C_M C_t (2.88)(0.75) \lambda$
$F_s (Ib/Q)' = F_s (Ib/Q) C_D C_M C_t$	$F_s (Ib/Q)' = F_s (Ib/Q) C_M C_t (2.88)(0.75) \lambda$
$F_c A' = F_c A C_D C_M C_t$	$F_c A' = F_c A C_M C_t (2.40)(0.90) \lambda$
$F_{c\perp}' = F_{c\perp} C_M C_t$	$F_{c\perp}' = F_{c\perp} C_M C_t (1.67)(0.90)$
$EI' = EI C_M C_t$	$EI' = EI C_M C_t$
$EA' = EA C_M C_t$	$EA' = EA C_M C_t$
$G_v t_v' = G_v t_v C_M C_t$	$G_v t_v' = G_v t_v C_M C_t$

Bending Member Example

For non-Structural I grade wood structural panels, greater than 24" in width, loaded in bending, and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 9.3), the adjusted design values reduce to:

For ASD:

$$F_b S' = F_b S C_D$$

$$EI' = EI$$

For LRFD:

$$F_b S' = F_b S (2.54)(0.85) \lambda$$

$$EI' = EI$$

Axially Loaded Member Example

For non-Structural I grade wood structural panels, greater than 24" in width, axially loaded, and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 4.3) designed to resist tension or compression loads, the adjusted tension or compression design values reduce to:

For ASD:

$$F_c A' = F_c A C_D$$

$$F_t A' = F_t A C_D$$

$$EA' = EA$$

For LRFD:

$$F_c A' = F_c A (2.40)(0.90) \lambda$$

$$F_t A' = F_t A (2.70)(0.80) \lambda$$

$$EA' = EA$$

Preservative Treatment

Capacities given in Tables M9.2-1 through M9.2-4 apply, without adjustment, to plywood pressure-impregnated with preservatives and redried in accordance with American Wood Protection Association (AWPA) Standard U1. OSB panels are currently recommended only for non-pressure applications of preservative treating in accordance with AWPA Standard T1.

Fire Retardant Treatment

The information provided in this Chapter does not apply to fire retardant treated panels. All capacities and end-use conditions for fire retardant treated panels shall be in accordance with the recommendations of the company providing the treating and redrying service.

M9.4 Design Considerations

Panel Edge Support

For certain span ratings, the maximum recommended roof span for sheathing panels is dependent upon panel edge support. Edge support may be provided by lumber blocking, tongue and groove, or panel clips when edge support is required. Table M9.4-1 summarizes the relationship between panel edge support and maximum recommended spans.

Table M9.4-1 Panel Edge Support²

Sheathing Span Rating	Maximum Recommended Span (in.)	
	With Edge Support	Without Edge Support
24/0	24	19.2 ¹
24/16	24	24
32/16	32	28
40/20	40	32
48/24	48	36

- 20 in. for 3/8 and 7/16 performance category panels, 24 in. for 15/32 and 1/2 performance category panels.
- Additional edge support is recommended when panel widths are less than 24 inches. Edge support requirements should be obtained from the manufacturer.

Long-Term Loading

Wood structural panels under constant load will creep (deflection will increase) over time. Where total deflection under long-term loading must be limited, provisions of *NDS* 3.5.2 are applicable. For wood structural panels in dry-service conditions, the immediate deflection due to long term loading is increased by a factor of 2.0 to account for deformation under long-term loading. For long-term loading under high moisture and/or high temperature, appropriate adjustments should be obtained from the manufacturer or an approved source.

Panel Spacing

Wood-based panel products expand and contract slightly as a natural response to changes in panel moisture content. To provide for in-plane dimensional changes, panels should be installed with a 1/8" spacing at all panel end and edge joints at the time of installation. A standard 10d box nail may be used to check panel edge and panel end spacing.

Minimum Nailing

Minimum nailing for wood structural panel applications is shown in Table M9.4-2.

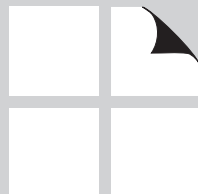
Table M9.4-2 Minimum Nailing for Wood Structural Panel Applications

Application	Recommended Nail Size & Type	Nail Spacing (in.)	
		Panel Edges	Intermediate Supports
Single Floor—Glue-nailed installation⁵			
Ring- or screw-shank			
16, 20, 24 oc, 3/4 performance category or less	6d ¹	6	12
24 oc, 7/8 or 1 performance category	8d ¹	6	12
32, 48 oc, (32-in. span (c-c) application)	8d ¹	6	12
48 oc, (48-in. span (c-c) application)	8d ²	6	6
Single Floor—Nailed-only installation			
Ring- or screw-shank			
16, 20, 24 oc, 3/4 performance category or less	6d	6	12
24 oc, 7/8 or 1 performance category	8d	6	12
32, 48 oc, (32-in. span application)	8d	6	12
48 oc, (48-in. span application)	8d ²	6	6
Sheathing—Subflooring³			
Common smooth, ring- or screw-shank			
7/16 to 1/2 thick performance category	6d	6	12
7/8 performance category or less	8d	6	12
Thicker panels	10d	6	6
Sheathing—Wall sheathing			
Common smooth, ring- or screw-shank or galvanized box³			
7/16 performance category or less	6d	6	12
Over 7/16 performance category	8d	6	12
Sheathing—Roof sheathing			
Common smooth, ring- or screw-shank³			
5/16 to 1 performance category	8d	6	12 ⁴
Thicker panels	8d ring- or screw-shank or 10d common smooth	6	12 ⁴

1. 8d common nails may be substituted if ring- or screw-shank nails are not available.
2. 10d ring-shank, screw-shank, or common nails may be substituted if supports are dry in accordance with *NDS*.
3. Other code-approved fasteners may be used.
4. For spans 48 in. or greater, space nails 6 in. at all supports.
5. Use only adhesives conforming to ASTM D3498.

M10: CROSS- LAMINATED TIMBER

M10.1	General	60
M10.2	Reference Design Values	60
M10.3	Adjustment of Reference Design Values	63
M10.4	Design Considerations	64



M10.1 General

Product Description

Cross-Laminated Timber (CLT) is a prefabricated engineered wood product consisting of at least three lay-

ers of solid-sawn lumber or structural composite lumber where the adjacent layers are cross-oriented and bonded with structural adhesive to form a solid wood element.

M10.2 Reference Design Values

General

CLT design properties shown in Tables M10.2-1 and M10.2-2 are excerpted from ANSI/APA PRG 320 Annex A and are based on products manufactured in accordance with the ANSI standard and which bear the trademark of a qualified inspection and testing agency.

Table M10.2-1 provides allowable design properties for the parallel layers of CLT grades listed, which represent the production intended for use by the CLT manufacturers in North America and are based on the following layups:

- E1: 1950f-1.7E Spruce-pine-fir MSR lumber in all parallel layers and No. 3 Spruce-pine-fir lumber in all perpendicular layers
- E2: 1650f-1.5E Douglas fir-Larch MSR lumber in all parallel layers and No. 3 Douglas fir-Larch lumber in all perpendicular layers

- E3: 1200f-1.2E Eastern Softwoods, Northern Species, or Western Woods MSR lumber in all parallel layers and No. 3 Eastern Softwoods, Northern Species, or Western Woods lumber in all perpendicular layers
- E4: 1950f-1.7E Southern pine MSR lumber in all parallel layers and No. 3 Southern pine lumber in all perpendicular layers
- V1: No. 2 Douglas fir-Larch lumber in all parallel layers and No. 3 Douglas fir-Larch lumber in all perpendicular layers
- V2: No. 1/No. 2 Spruce-pine-fir lumber in all parallel layers and No. 3 Spruce-pine-fir lumber in all perpendicular layers
- V3: No. 2 Southern pine lumber in all parallel layers and No. 3 Southern pine lumber in all perpendicular layers

TABLE M10.2-1 Allowable Design Properties^(a,b,c) for the Parallel Layers of PRG 320 CLT (for use in the U.S.) Reprinted with permission of APA-The Engineered Wood Association

CLT Grades	Major Strength Direction						Minor Strength Direction					
	F _{b,0} (psi)	E ₀ (10 ⁶ psi)	F _{t,0} (psi)	F _{c,0} (psi)	F _{v,0} (psi)	F _{s,0} (psi)	F _{b,90} (psi)	E ₉₀ (10 ⁶ psi)	F _{t,90} (psi)	F _{c,90} (psi)	F _{v,90} (psi)	F _{s,90} (psi)
E1	1,950	1.7	1,375	1,800	135	45	500	1.2	250	650	135	45
E2	1,650	1.5	1,020	1,700	180	60	525	1.4	325	775	180	60
E3	1,200	1.2	600	1,400	110	35	350	0.9	150	475	110	35
E4	1,950	1.7	1,375	1,800	175	55	575	1.4	325	825	175	55
V1	900	1.6	575	1,350	180	60	525	1.4	325	775	180	60
V2	875	1.4	450	1,150	135	45	500	1.2	250	650	135	45
V3	975	1.6	550	1,450	175	55	575	1.4	325	825	175	55

For SI: 1 psi = 0.006895 MPa

(a) See APA PRG 320 Section 4 for symbols.

(b) Tabulated values are allowable lumber design values and not permitted to be increased for the lumber size adjustment factor in accordance with the NDS. The design values shall be used in conjunction with the section properties provided by the CLT manufacturer based on the actual layup used in manufacturing the CLT panel (see Table M10.2-2).

(c) Custom CLT grades that are not listed in this table shall be permitted in accordance with ANSI/APA PRG 320 Section 7.2.1.

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The allowable design capacities for these CLT grades with layups of 3, 5, and 7 layers are provided in Table M10.2-2. These capacities are deemed achievable using the standard laminations tabulated in the PRG 320 standard. These capacities were derived analytically using the Shear Analogy Model¹ (the calculated moment capacities in the

major strength direction were further multiplied by a factor of 0.85 for conservatism) and validated by testing. The lamination thicknesses are as tabulated. Allowable tensile, compressive, and shear capacities will be developed and added to future editions of the PRG 320 standard.

TABLE M10.2-2. Allowable Bending Capacities^(a,b,c) for CLT Grades (for use in the U.S.) Reprinted with permission of APA-The Engineered Wood Association

CLT Grade	Lamination Thickness (in.) in CLT Layup								Major Strength Direction			Minor Strength Direction			
	CLT t (in.)			⊥				⊥		F _b S _{eff,0} (lbf-ft/ft)	EI _{eff,0} (10 ⁶ lbf-in. ² /ft)	GA _{eff,0} (10 ⁶ lbf/ft)	F _b S _{eff,90} (lbf-ft/ft)	EI _{eff,90} (10 ⁶ lbf-in. ² /ft)	GA _{eff,90} (10 ⁶ lbf/ft)
			⊥		⊥		⊥		⊥						
E1	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	4,525	115	0.46	160	3.1	0.61	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	10,400	440	0.92	1,370	81	1.2	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,375	1,089	1.4	3,125	309	1.8	
E2	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	3,825	102	0.53	165	3.6	0.56	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	8,825	389	1.1	1,430	95	1.1	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	15,600	963	1.6	3,275	360	1.7	
E3	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	2,800	81	0.35	110	2.3	0.44	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	6,400	311	0.69	955	61	0.87	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	11,325	769	1.0	2,180	232	1.3	
E4	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	4,525	115	0.53	180	3.6	0.63	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	10,425	441	1.1	1,570	95	1.3	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,400	1,090	1.6	3,575	360	1.9	
V1	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	2,090	108	0.53	165	3.6	0.59	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	4,800	415	1.1	1,430	95	1.2	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,500	1,027	1.6	3,275	360	1.8	
V2	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	2,030	95	0.46	160	3.1	0.52	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	4,675	363	0.91	1,370	81	1.0	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,275	898	1.4	3,125	309	1.6	
V3	4 1/8	1 3/8	1 3/8	1 3/8	-	-	-	-	2,270	108	0.53	180	3.6	0.59	
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	-	-	5,200	415	1.1	1,570	95	1.2	
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	9,200	1,027	1.6	3,575	360	1.8	

For SI: 1 in. = 25.4 mm; 1 ft = 304.8 mm; 1 lbf = 4.448 N

(a) See ANSI/APA PRG 320 Section 4 for symbols.

(b) This table represents one of many possibilities that the CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the layup.

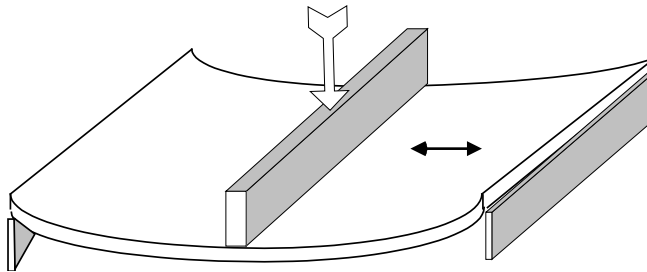
(c) Custom CLT grades that are not listed in this table shall be permitted in accordance with ANSI/APA PRG 320 Section 7.2.1.

¹ Gagnon, S. and M. Popovski. 2011. *Structural Design of Cross-Laminated Timber Elements*. In Chapter 3, CLT Handbook. FPInnovations, Canada.

CLT Stiffness and Strength

CLT stiffness and strength design capacities are based on flat panel bending (Figure M10.2-1) according to principles of ANSI/APA PRG 320.

Figure M10.2-1 Example of CLT Panel in Bending



Strength, $F_b(S_{\text{eff}})$

Bending strength capacity is the design maximum moment, represented as $(F_b S_{\text{eff}})$. F_b is the reference extreme fiber bending stress, and S_{eff} is the effective section modulus of the cross section based on the layup used in the manufacturing process. The units of $F_b S_{\text{eff}}$ are lbf-in. per foot of panel width.

Bending Stiffness for Deflection Calculations, $(EI)_{\text{app}}$

CLT apparent bending stiffness is the capacity to resist deflection and is represented as $(EI)_{\text{app}}$. The apparent stiffness is based on the layup used in the manufacturing process and is calculated using published $(EI)_{\text{eff}}$ values adjusted per the NDS. The units of EI are lbf-in.² per foot of panel width.

CLT Axial Capacities

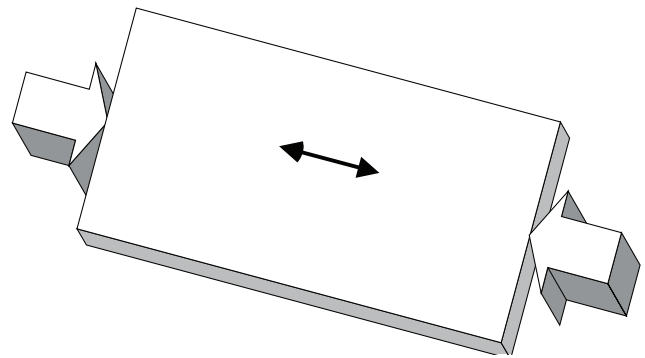
Tension, $F_t(A_{\text{parallel}})$

Tension capacities are based on the principles of ANSI/APA PRG 320. Tension capacity is given as $F_t(A_{\text{parallel}})$. F_t is the reference axial tensile stress of the material, and A_{parallel} is the area of the cross section with fibers parallel to the load direction. The units of $F_t(A_{\text{parallel}})$ are pounds force per foot of panel width. This property can be obtained from the CLT manufacturer or an approved source.

Compression, $F_c(A_{\text{parallel}})$

Compression capacities (Figure M10.2-2) are based on principles of APA PRG 320. Compression capacity is given as $F_c(A_{\text{parallel}})$. F_c is the reference axial compression stress of the material and A_{parallel} is the area of the cross section with fibers parallel to the load direction. The units of $F_c(A_{\text{parallel}})$ are pounds force per foot of panel width.

Figure M10.2-2 CLT Panel with Axial Compression Load in the Plane of the Panel



Bending Stiffness for Buckling Stability Calculations, $(EI)_{\text{app-min}}$

CLT apparent bending stiffness for panel buckling stability calculations is represented as $(EI)_{\text{app-min}}$. The apparent bending stiffness is based on the layup used in the manufacturing process. The units of EI are lbf-in.² per foot of panel width. This property can be obtained from the CLT manufacturer or an approved source.

Bearing, $F_c \perp (A)$

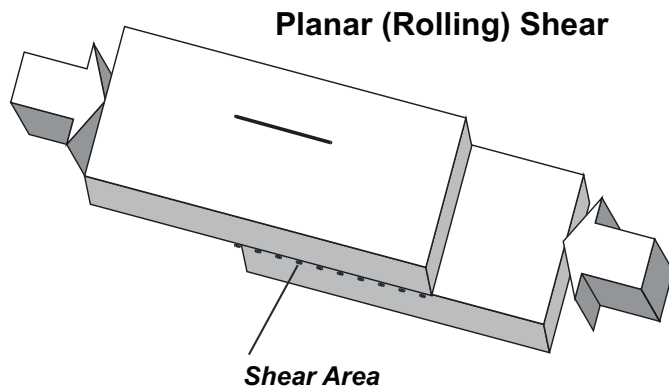
Bearing capacities are based on compression perpendicular to grain values of the side grain of the CLT panels and the total bearing area of all wood. This property can be obtained from the CLT manufacturer or an approved source. As with solid sawn lumber, compression perpendicular to grain design values are permitted to be multiplied by the bearing area factor specified in NDS 3.10.4.

CLT Shear Capacities

Shear-in-the-Plane of the Panel, $F_s(Ib/Q)_{\text{eff}}$

Shear-in-the-plane of the panel (planar or rolling shear) capacities are based on the principles of ANSI/APA PRG 320. Shear strength in the plane of the panel is the capacity to resist horizontal shear breaking loads when loads are applied or developed on opposite faces of the panel (Figure M10.2-3), as in flat panel bending. Planar shear capacity is given as $F_s(Ib/Q)_{\text{eff}}$. F_s is the reference material innerlaminar shear stress, and $(Ib/Q)_{\text{eff}}$ is the effective CLT cross-sectional shear constant. The units of $F_s(Ib/Q)_{\text{eff}}$ are pounds force per foot of panel width.

Figure M10.2-3 Shear-in-the-Plane for CLT Panels

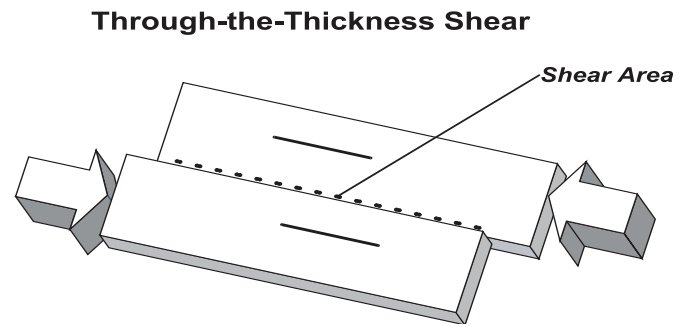


Shear Through-the-Thickness, $F_v(t_v)$

Through-the-thickness shear capacities are based on the principles of ANSI/APA PRG 320. Allowable shear through-the-thickness is the capacity to resist horizontal shear breaking loads when loads are applied or developed on opposite edges of the panel (Figure M10.2-4).

F_v is the reference shear through-the-thickness stress of the material, and t_v is the panel thickness for shear. The units of $F_v(t_v)$ are pounds force per inch of shear resisting panel length. This property can be obtained from the CLT manufacturer or an approved source.

Figure M10.2-4 Through-the-Thickness Shear for CLT Panels



M10.3 Adjustment of Reference Design Values

General

Adjusted CLT design capacities are determined by multiplying reference capacities by the adjustment factors in NDS 10.3. Some adjustment factors should be obtained from the manufacturer or other approved source. In the *NDS Commentary*, C10.3 provides additional information on typical adjustment factors.

Tabulated capacities provided in the manufacturer's literature are suitable for reference end-use conditions. For CLT, these typical conditions involve the use of untreated

products in moderate temperature and moisture exposures. Appropriate adjustment factors are provided for applications in which the conditions of use are inconsistent with reference conditions. In addition to temperature and moisture, this includes load duration, beam stability, column stability, and bearing area adjustments.

NDS Table 10.3.1 lists applicability of adjustment factors for cross-laminated timber. Table M10.3-1 shows the applicability of adjustment factors for cross-laminated timber in an equation format.

Table M10.3-1 Applicability of Adjustment Factors for Cross-Laminated Timber

Allowable Stress Design	Load and Resistance Factor Design
$F_b(S_{eff})' = F_b(S_{eff}) C_D C_M C_t C_L$	$F_b(S_{eff})' = F_b(S_{eff}) C_M C_t C_L (2.54)(0.85) \lambda$
$F_t(A_{parallel})' = F_t(A_{parallel}) C_D C_M C_t$	$F_t(A_{parallel})' = F_t(A_{parallel}) C_M C_t (2.70)(0.80) \lambda$
$F_v t_v' = F_v t_v C_D C_M C_t$	$F_v t_v' = F_v t_v C_M C_t (2.88)(0.75) \lambda$
$F_s(Ib/Q)_{eff}' = F_s(Ib/Q)_{eff} C_M C_t$	$F_s(Ib/Q)_{eff}' = F_s(Ib/Q)_{eff} C_M C_t (2.88)(0.75)$
$F_c(A_{parallel})' = F_c(A_{parallel}) C_D C_M C_t C_P$	$F_c(A_{parallel})' = F_c(A_{parallel}) C_M C_t C_P (2.40)(0.90) \lambda$
$F_{c\perp}(A)' = F_{c\perp}(A) C_M C_t C_b$	$F_{c\perp}(A)' = F_{c\perp}(A) C_M C_t C_b (1.67)(0.90)$
$(EI)_{app}' = (EI)_{app} C_M C_t$	$(EI)_{app}' = (EI)_{app} C_M C_t$
$(EI)_{app-min}' = (EI)_{app-min} C_M C_t$	$(EI)_{app-min}' = (EI)_{app-min} C_M C_t (1.76)(0.85)$

Bending Member Example

For cross-laminated timber, loaded in bending, and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 10.3), the adjusted design values reduce to:

For ASD:

$$F_b(S_{\text{eff}})' = F_b(S_{\text{eff}}) C_D$$

$$F_{c\perp}(A)' = F_{c\perp}(A) C_b$$

$$(EI)_{\text{app}}' = (EI)_{\text{app}}$$

For LRFD:

$$F_b(S_{\text{eff}})' = F_b(S_{\text{eff}}) (2.54)(0.85) \lambda$$

$$F_{c\perp}(A)' = F_{c\perp}(A) C_b (1.67)(0.90)$$

$$(EI)_{\text{app}}' = (EI)_{\text{app}}$$

Axially Loaded Member Example

For cross-laminated timber, axially loaded, and used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 10.3) designed to resist tension or compression loads, the adjusted tension or compression design values reduce to:

For ASD:

$$F_c(A_{\text{parallel}})' = F_c(A_{\text{parallel}}) C_D C_P$$

$$F_t(A_{\text{parallel}})' = F_t(A_{\text{parallel}}) C_D$$

For LRFD:

$$F_c(A_{\text{parallel}})' = F_c(A_{\text{parallel}}) C_P (2.40)(0.90) \lambda$$

$$F_t(A_{\text{parallel}})' = F_t(A_{\text{parallel}}) (2.70)(0.80) \lambda$$

Preservative Treatment

The information provided in this chapter does not apply to preservative treated CLT. All capacities and end-use conditions for preservative treated CLT shall be in accordance with the recommendations of the manufacturer.

Fire Retardant Treatment

The information provided in this chapter does not apply to fire retardant treated CLT. All capacities and end-use conditions for fire retardant treated CLT shall be in accordance with the recommendations of the company providing the treating and redrying service.

M10.4 Design Considerations

Connections

Connections used in cross-laminated timber shall be in accordance with applicable *NDS* connection chapters or manufacturers' recommendations.

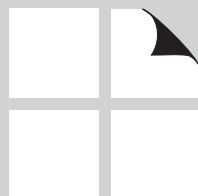
Long-Term Loading

Cross-laminated timber under constant load will creep (deflection will increase) over time. Where total deflection

under long-term loading must be limited, provisions of *NDS* 3.5.2 are applicable. For CLT in dry-service conditions, the immediate deflection due to long-term loading is increased by a factor of 2.0 to account for deformation under long-term loading. For long-term loading under high moisture and/or high temperature, appropriate adjustments should be obtained from the manufacturer or an approved source.

M11: MECHANICAL CONNECTIONS

M11.1	General	66
M11.2	Reference Design Values	67
M11.3	Adjustment of Reference Design Values	67
M11.4	Typical Connection Details	68
M11.5	Pre-Engineered Metal Connectors	76



M11.1 General

This Chapter covers design of connections between wood members using metal fasteners. Several common connection types are outlined below.

Dowel-Type (Nails, Bolts, Screws, Pins)

These connectors rely on metal-to-wood bearing for transfer of lateral loads and on friction or mechanical interfaces for transfer of axial (withdrawal) loads. They are commonly available in a wide range of diameters and lengths. More information is provided in Chapter M12.

Split Rings and Shear Plates

These connectors rely on their geometry to provide larger metal-to-wood bearing areas per connector. Both are installed into precut grooves or daps in the members. More information is provided in Chapter M13.

Timber Rivets

Timber rivets are a dowel-type connection, however, because the ultimate load capacity of such connections are limited by rivet bending and localized crushing of wood at the rivets or by the tension or shear strength of the wood at the perimeter of the rivet group, a specific design procedure is required. Timber rivet design loads are based on the lower of the maximum rivet bending load and the maximum load based on wood strength. Chapter M13 contains more information on timber rivet design.

Structural Framing Connections

Structural framing connections provide a single-piece connection between two framing members. They generally consist of bent or welded steel, carrying load from the supported member (through direct bearing) into the supporting member (by hanger flange bearing, fastener shear, or a combination of the two). Structural framing connections are proprietary connectors and are discussed in more detail in M11.4.

Other Connectors

Just as the number of possible building geometries is limitless, so too is the number of possible connection geometries. In addition to providing custom fabrication of connectors to meet virtually any geometry that can be

designed, metal connector manufacturers have several categories of connectors that do not fit the categories above, including:

- framing anchors
- hold down devices
- straps and ties

These connectors are also generally proprietary connectors. See the manufacturer's literature or M11.4 for more information regarding design.

Connections are designed so that no applicable capacity is exceeded under loads. Strength criteria include lateral or withdrawal capacity of the connection, and tension or shear in the metal components. Some types of connections also include compression perpendicular to grain as a design criteria.

Users should note that design of connections may also be controlled by serviceability limitations. These limitations are product specific and are discussed in specific product chapters.

Stresses in Members at Connections

Local stresses in connections using multiple fasteners can be evaluated in accordance with *NDS* Appendix E.

M11.2 Reference Design Values

Reference design values for mechanical connections are provided in various sources. The *NDS* contains reference design values for dowel-type connections such as nails, bolts, lag screws, wood screws, split rings, shear plates, drift bolts, drift pins, and timber rivets.

Pre-engineered metal connectors are proprietary and reference design values are provided in code evaluation reports. More information on their use is provided in M11.5.

Metal connector plates are proprietary connectors for trusses, and reference design values are provided in code evaluation reports.

Staples and many power-driven fasteners are proprietary, and reference design values are provided in code evaluation reports.

M11.3 Adjustment of Reference Design Values

To generate connection design capacities, reference design values for connections are multiplied by adjustment factors per *NDS* 11.3. Applicable adjustment factors for connections are defined in *NDS* Table 11.3.1. Table

M11.3-1 shows the applicability of adjustment factors for connections in a slightly different format for the designer.

Table M11.3-1 Applicability of Adjustment Factors for Mechanical Connections¹

	Allowable Stress Design	Load and Resistance Factor Design
Lateral Loads		
Dowel-Type Fasteners	$Z' = Z C_D C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn}$	$Z' = Z C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn} (3.32)(0.65) \lambda$
Split Ring and Shear Plate Connectors	$P' = P C_D C_M C_t C_g C_{\Delta} C_d C_{st}$	$P' = P C_M C_t C_g C_{\Delta} C_d C_{st} (3.32)(0.65) \lambda$
	$Q' = Q C_D C_M C_t C_g C_{\Delta} C_d$	$Q' = Q C_M C_t C_g C_{\Delta} C_d (3.32)(0.65) \lambda$
Timber Rivets	$P' = P C_D C_M C_t C_{st}$	$P' = P C_M C_t C_{st} (3.32)(0.65) \lambda$
	$Q' = Q C_D C_M C_t C_{\Delta} C_{st}$	$Q' = Q C_M C_t C_{\Delta} C_{st} (3.32)(0.65) \lambda$
Spike Grids	$Z' = Z C_D C_M C_t C_{\Delta}$	$Z' = Z C_M C_t C_{\Delta} (3.32)(0.65) \lambda$
Withdrawal Loads		
Nails, Spikes, Lag Screws, Wood Screws, and Drift Pins	$W' = W C_D C_M C_t C_{eg} C_{tn}$	$Z' = Z C_M C_t C_{eg} C_{tn} (3.32)(0.65) \lambda$

1. See *NDS* Table 11.3.1 footnotes for additional guidance on application of adjustment factors for mechanical connections.

The following connection product chapters contain examples of the application of adjustment factors to reference design values:

- Chapter M12 – dowel-type fasteners,
- Chapter M13 – split ring and shear plate connectors,
- Chapter M14 – timber rivets.

M11.4 Typical Connection Details

General Concepts of Well-Designed Connections

Connections must obviously provide the structural strength necessary to transfer loads. Well-designed connections hold the wood members in such a manner that shrinkage/swelling cycles do not induce splitting across the grain. Well-designed connections also minimize regions that might collect moisture – providing adequate clearance for air movement to keep the wood dry. Finally, well-designed connections minimize the potential for tension perpendicular to grain stresses – either under design conditions or under unusual loading conditions.

The following connection details (courtesy of the Canadian Wood Council) are organized into nine groups:

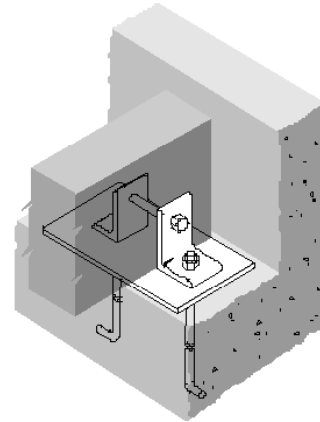
1. Beam to concrete or masonry wall connections
2. Beam to column connections
3. Column to base connections
4. Beam to beam connections
5. Cantilever beam connections
6. Arch peak connections
7. Arch base to support
8. Moment splice
9. Problem connections

Many of the detail groups begin with a brief discussion of the design challenges pertinent to the specific type of connection. Focusing on the key design concepts of a broad class of connections often leads to insights regarding a specific detail of interest.

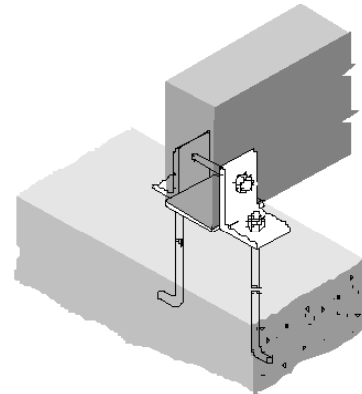
Group 1. Beam to Concrete or Masonry Wall Connections

Design concepts. Concrete is porous and “wicks” moisture. Good detailing never permits wood to be in direct contact with concrete.

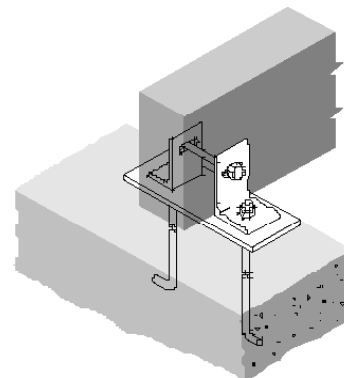
1. Beam on shelf in wall. The bearing plate distributes the load and keeps the beam from direct contact with the concrete. Steel angles provide uplift resistance and can also provide some lateral resistance. The end of the beam should not be in direct contact with the concrete.



2. Similar to detail 1 with a steel bearing plate only under the beam.



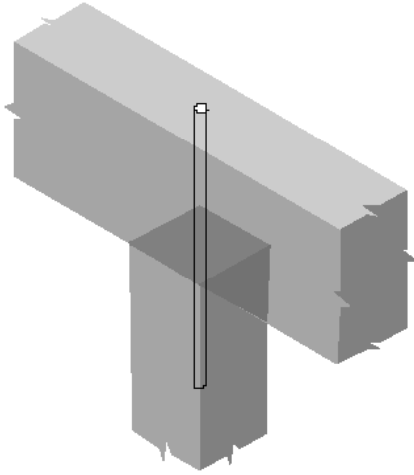
3. Similar to detail 1 with slotted holes to accommodate slight lateral movement of the beam under load. This detail is more commonly used when the beam is sloped, rather than flat.



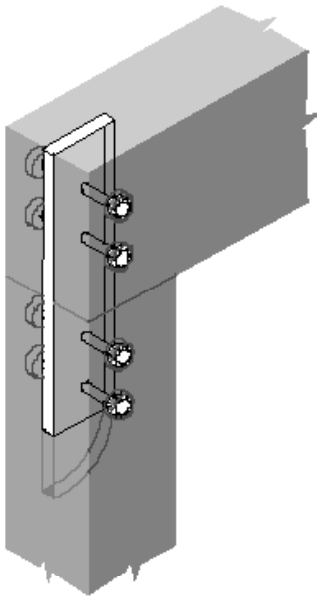
Group 2. Beam to Column Connections

Design concepts. All connections in the group must hold the beam in place on top of the column. Shear transfer is reasonably easy to achieve. Some connections must also resist some beam uplift. Finally, for cases in which the beam is spliced, rather than continuous over the column, transfer of forces across the splice may be required.

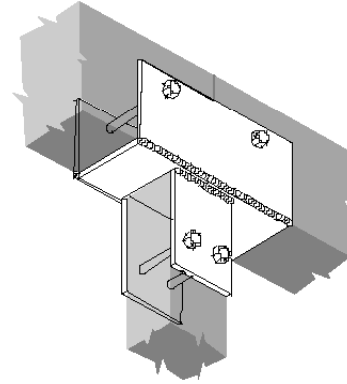
4. Simple steel dowel for shear transfer.



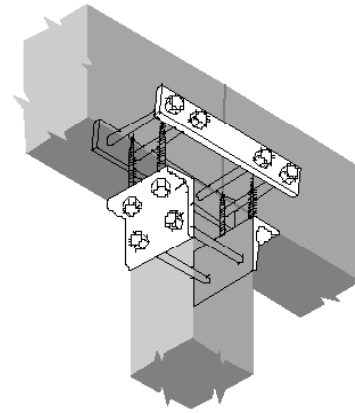
5. Concealed connection in which a steel plate is inserted into a kerf in both the beam and the column. Transverse pins or bolts complete the connection.



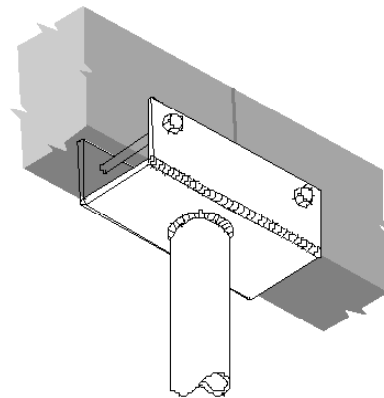
6. Custom welded column caps can be designed to transfer shear, uplift, and splice forces. Note design variations to provide sufficient bearing area for each of the beams and differing plate widths to accommodate differences between the column and the beam widths.



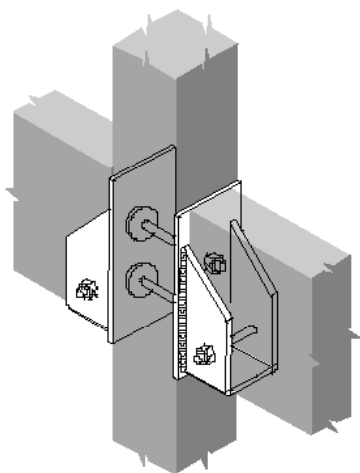
7. Combinations of steel angles and straps, bolted and screwed, to transfer forces.



8. A very common connection – beam seat welded to the top of a steel column.



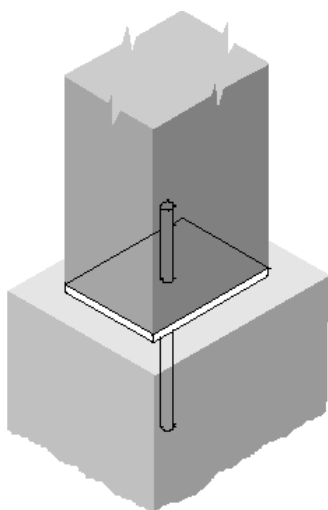
9. When both beams and columns are continuous and the connection must remain in-plane, either the beam or the column must be spliced at the connection. In this detail the column continuity is maintained. Optional shear plates may be used to transfer higher loads. Note that, unless the bolt heads are completely recessed into the back of the bracket, the beam end will likely require slotting. In a building with many bays, it may be difficult to maintain dimensions in the beam direction when using this connection.



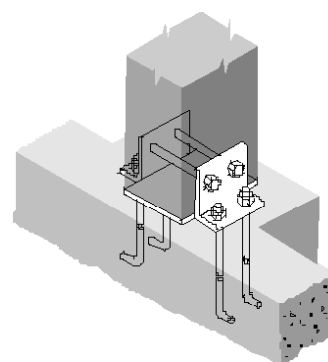
Group 3. Column to Base Connections

Design concepts. Since this is the bottom of the structure, it is conceivable that moisture from some source might run down the column. Experience has shown that base plate details in which a steel “shoe” is present can collect moisture that leads to decay in the column.

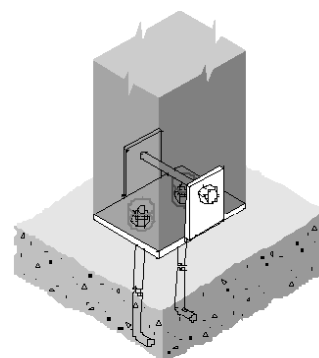
10. Similar to detail 4, with a bearing plate added.



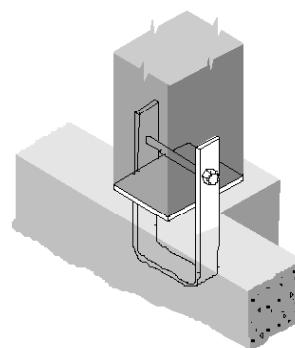
11A. Similar to details 1 and 2.



11B. Alternate to detail 11A.



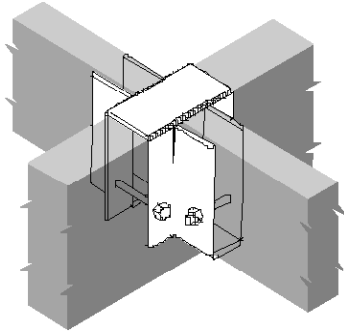
12. Similar to detail 3.



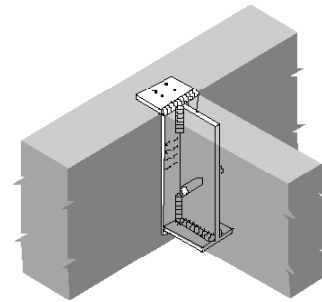
Group 4. Beam to Beam Connections

Design concepts. Many variations of this type of connection are possible. When all members are flat and their tops are flush, the connection is fairly straightforward. Slopes and skews require special attention to fabrication dimensions – well-designed connections provide adequate clearance to insert bolts or other connectors and also provide room to grip and tighten with a wrench. Especially for sloped members, special attention is required to visualize the stresses induced as the members deflect under load – some connections will induce large perpendicular to grain stresses in this mode.

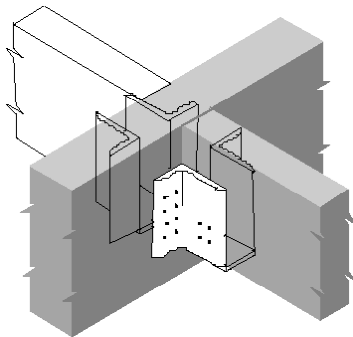
13. Bucket-style welded bracket at a “cross” junction. The top of the support beam is sometimes dapped to accommodate the thickness of the steel.



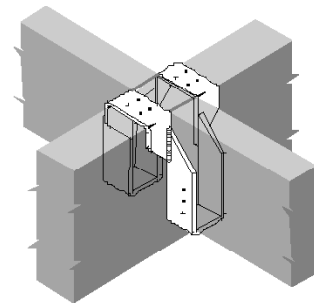
suspended beam, permitting plugging of the holes after the pin is installed. Note that the kerf in the suspended beam must accommodate not only the width of the steel plate, but also the increased width at the fillet welds.



14. Face-mounted hangers are commonly used in beam to beam connections. In a “cross” junction special attention is required to fastener penetration length into the carrying beam (to avoid interference from other side).

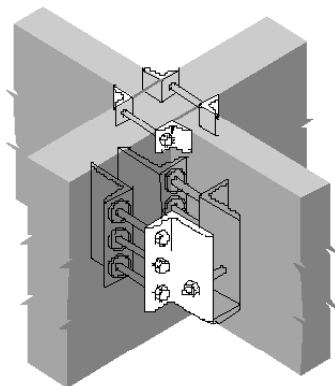
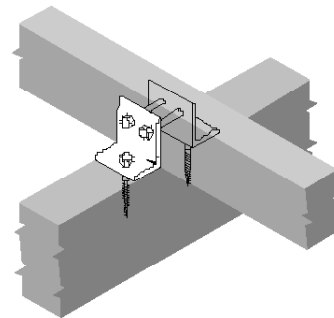


17. Similar to detail 13, with somewhat lower load capacity.



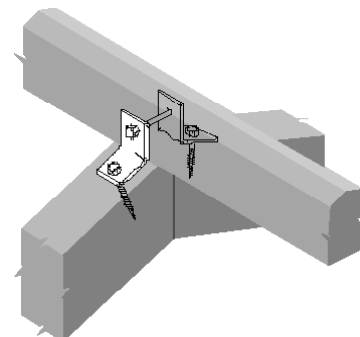
15. Deep members may be supported by fairly shallow hangers – in this case, through-bolted with shear plates. Clip angles are used to prevent rotation of the top of the suspended beam. Note that the clip angles are not connected to the suspended beam – doing so would restrain a deep beam from its natural across-the-grain shrinking and swelling cycles and would lead to splits.

18. Clip angle to connect crossing beam.

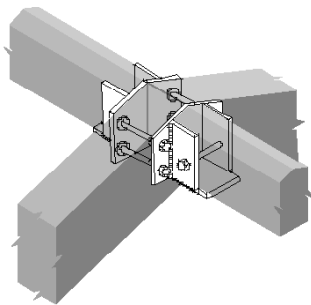


16. Concealed connections similar to detail 5. The suspended beam may be dapped on the bottom for a flush connection. The pin may be slightly narrower than the

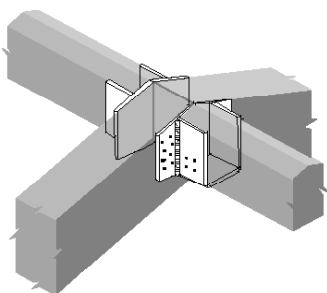
19. Special detail to connect the ridge purlin to sloped members or to the peak of arch members.



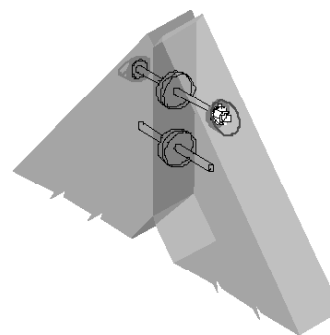
20A. Similar to detail 19, but with the segments of the ridge purlin set flush with the other framing.



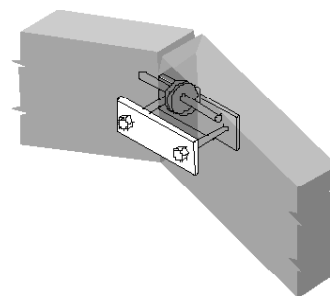
20B. Alternate to detail 20A.



23. Similar to detail 22, with added shear plate.

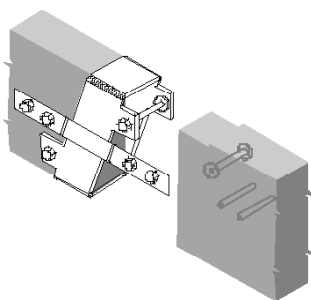


24. Similar to detail 22 for low slope arches. Side plates replace the threaded rod.



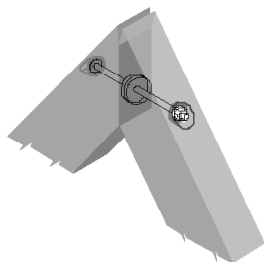
Group 5. Cantilever Beam Connections

21. Hinge connector transfers load without need to slope cut member ends. Beams are often dapped top and bottom for a flush fit.



Group 6. Arch Peak Connections

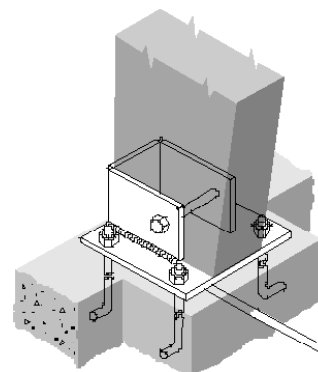
22. Steep arches connected with a rod and shear plates.



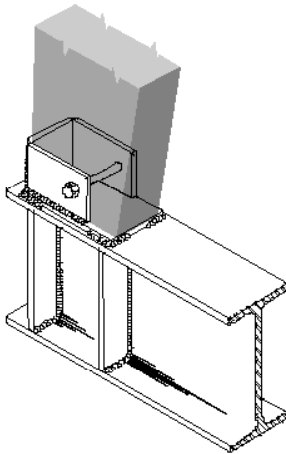
Group 7. Arch Base to Support

Design concepts. Arches transmit thrust into the supporting structure. The foundation may be designed to resist this thrust or tie rods may be used. The base detail should be designed to accommodate the amount of rotation anticipated in the arch base under various loading conditions. Elastomeric bearing pads can assist somewhat in distributing stresses. As noted earlier, the connection should be designed to minimize any perpendicular to grain stresses during the deformation of the structure under load.

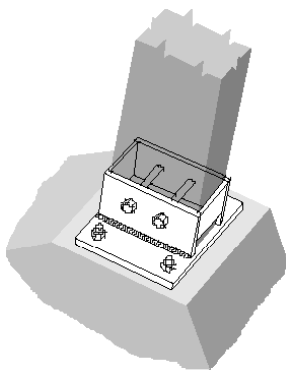
25. Welded shoe transmits thrust from arch to support. Note that inside edge of shoe is left open to prevent collection of moisture.



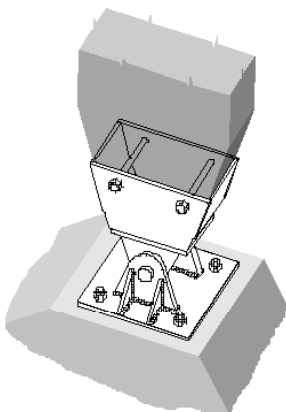
26. Arch base fastened directly to a steel tie beam in a shoe-type connection.



27. Similar to detail 25. This more rigid connection is suitable for spans where arch rotation at the base is small enough to not require the rotational movement permitted in detail 25. Note that, although the shoe is “boxed” a weep slot is provided at the inside face.



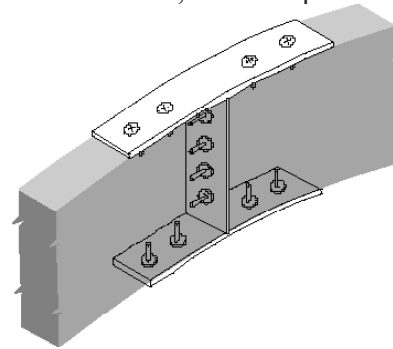
28. For very long spans or other cases where large rotations must be accommodated, a true hinge connection may be required.



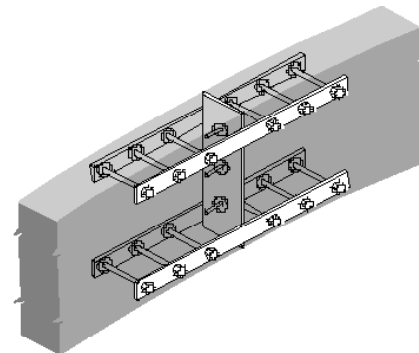
Group 8. Moment Splice

Design concepts. Moment splices must transmit axial tension, axial compression, and shear. They must serve these functions in an area of the structure where structural movement may be significant – thus, they must not introduce cross-grain forces if they are to function properly.

29. Separate pieces of steel each provide a specific function. Top and bottom plate transfer axial force, pressure plates transfer direct thrust, and shear plates transmit shear.



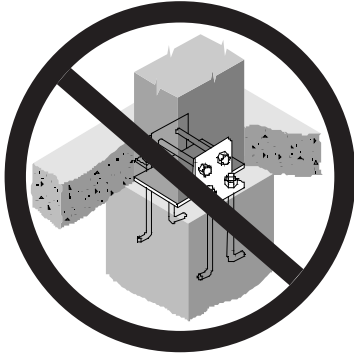
30. Similar to detail 29. Connectors on side faces may be easier to install, but forces are higher because moment arm between steel straps is less than in detail 29.



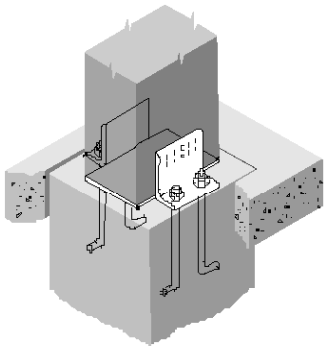
Group 9. Problem Connections

Hidden column base. It is sometimes preferable architecturally to conceal the connection at the base of the column. In any case it is crucial to detail this connection to minimize decay potential.

31A. Similar to detail 11, but with floor slab poured over the top of the connection. **THIS WILL CAUSE DECAY AND IS NOT A RECOMMENDED DETAIL!**

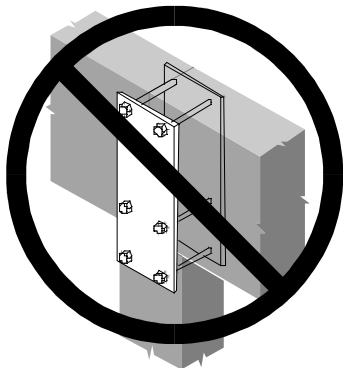


31B. Alternate to detail 31A.

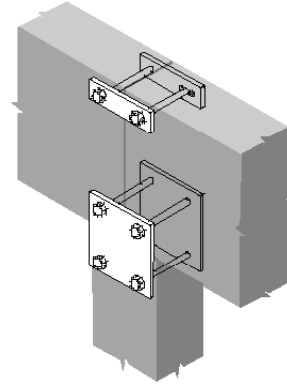


Full-depth side plates. It is sometimes easier to fabricate connections for deep beams from large steel plates rather than having to keep track of more pieces. Lack of attention to wood's dimensional changes as it "breathes" may lead to splits.

32A. Full-depth side plates may appear to be a good connection option. Unfortunately, the side plates will remain fastened while the wood shrinks over the first heating season. Since it is restrained by the side plates, the beam may split. **THIS DETAIL IS NOT RECOMMENDED!**

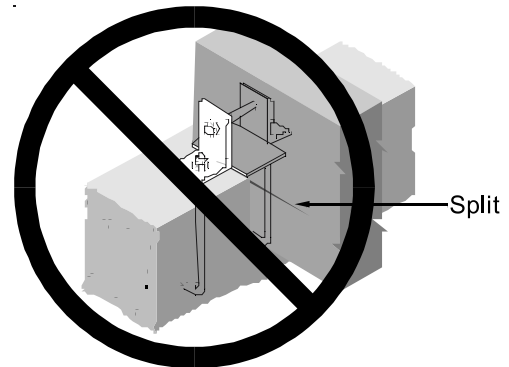


32B. As an alternative to detail 32A, smaller plates will transmit forces, but they do not restrain the wood from its natural movements.

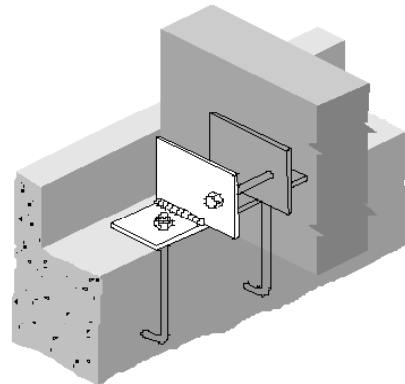


Notched beam bearing. Depth limitations sometimes cause detailing difficulties at the beam supports. A simple solution is to notch the beam at the bearing. This induces large tension perpendicular to grain stresses and leads to splitting of the beam at the root of the notch.

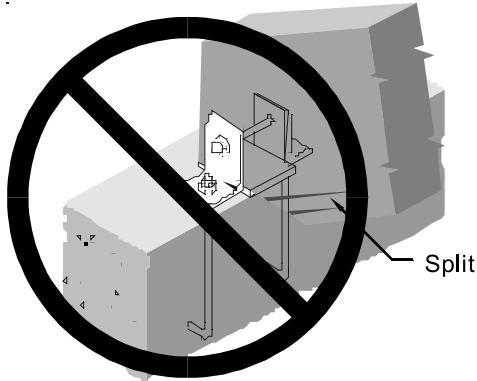
33A. Notching a beam at its bearing may cause splits. **THIS DETAIL IS NOT RECOMMENDED!**



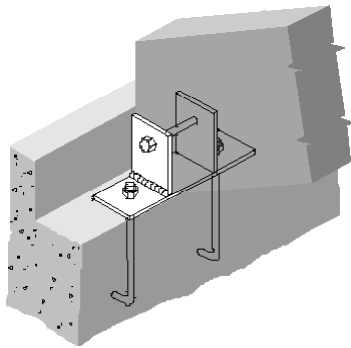
33B. Alternate to detail 33A.



34A. This sloped bearing with a beam that is not fully supported may also split under load. **THIS DETAIL IS NOT RECOMMENDED!**

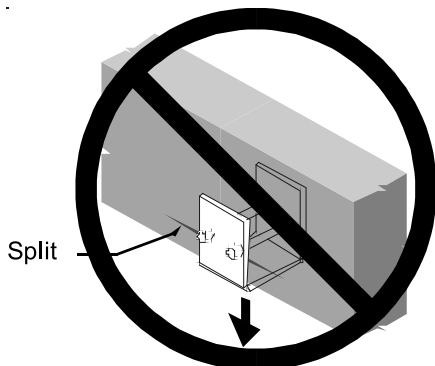


34B. Alternate to detail 34A.

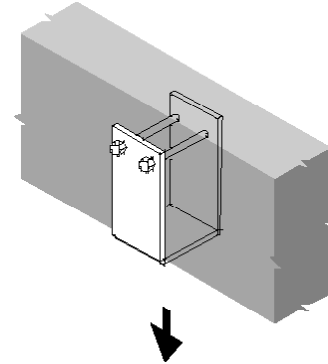


Hanging to underside of beam. Sometimes it is advantageous to hang a load from the underside of a beam. This is acceptable as long as the hanger is fastened to the upper half of the beam. Fastening to the lower half of the beam may induce splits.

35A. Connecting a hanger to the lower half of a beam that pulls downward may cause splits. **THIS DETAIL IS NOT RECOMMENDED!**

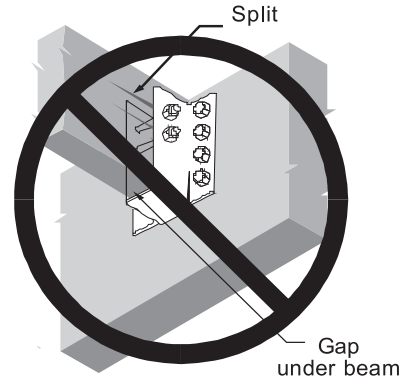


35B. As an alternative to detail 35A, the plates may be extended and the connection made to the upper half of the beam.

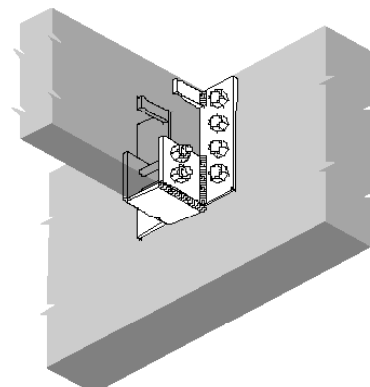


Hanger to side of beam. See full-depth side plates discussion.

36A. Deep beam hangers that have fasteners installed in the side plates toward the top of the supported beam may promote splits at the fastener group should the wood member shrink and lift from the bottom of the beam hanger because of the support provided by the fastener group. **THIS DETAIL IS NOT RECOMMENDED!**



36B. Alternate to detail 36A.



M11.5 Pre-Engineered Metal Connectors

Product Information

Pre-engineered metal connectors for wood construction are commonly used in all types of wood construction. There are numerous reasons for their widespread use. Connectors often make wood members easier and faster to install. They increase the safety of wood construction, not only from normal loads, but also from natural disasters such as earthquakes and high winds. Connectors make wood structures easier to design by providing simpler connections of known load capacity. They also allow for the use of more cost-effective engineered wood members by providing the higher capacity connections often required by the use of such members. In certain locations, model building codes specifically require connectors or hangers.

Metal connectors are usually manufactured by stamping sheets or strips of steel, although some heavy hangers are welded together. Different thicknesses and grades of steel are used, depending on the required capacity of the connector.

Some metal connectors are produced as proprietary products which are covered by evaluation reports. Such reports should be consulted for current design information, while the manufacturer's literature can be consulted for additional design information and detailed installation instructions.

Common Uses

Pre-engineered metal connectors for wood construction are used throughout the world. Connectors are used to resist vertical dead, live, and snow loads; uplift loads from wind; and lateral loads from ground motion or wind. Almost any type of wood member may be fastened to another using a connector. Connectors may also be used to fasten wood to other materials, such as concrete, masonry, or steel.

Availability

Connectors are manufactured in varying load capacities, sizes, and configurations to fit a wide range of applications. A variety of connectors are widely available through lumber suppliers.

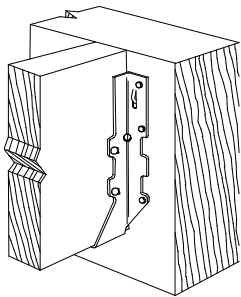
Because of the wide variety of available connectors, a generic design document such as this must be limited in its scope for simplicity's sake. Design values for specific connectors are available from the connector manufacturer.

Types of Connectors

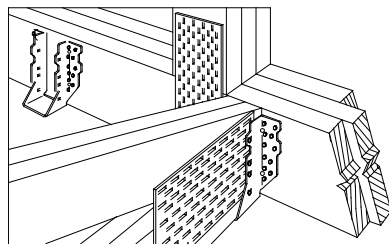
There are many different types of connectors, due to the many different applications in which connectors may be used. The following sections list the most common types of connectors.

Face Mount Joist Hangers

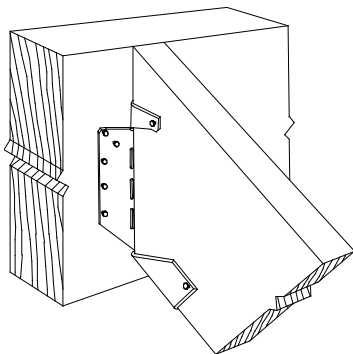
Face mount joist hangers install on the face of the supporting member, and rely on the shear capacity of the nails to provide holding power. Although referred to as joist hangers, these connectors may support other horizontal members subject to vertical loads, such as beams or purlins.



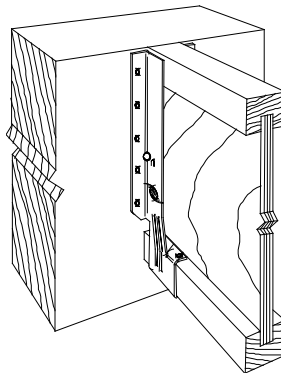
Face Mount Joist Hanger



Heavy Face Mount Joist Hanger



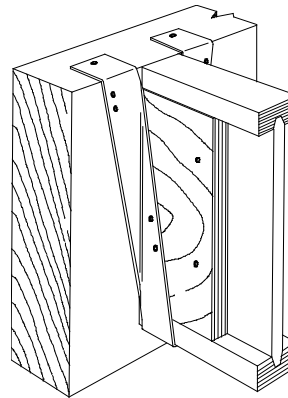
Slope- and Skew-Adjustable Joist Hanger



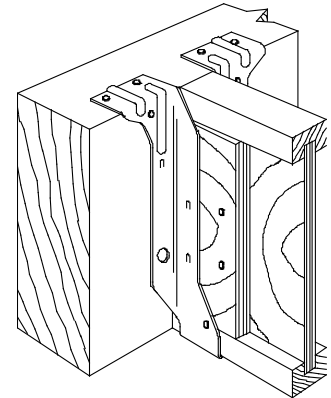
Face Mount I-Joist Hanger*

Top Flange Joist Hangers

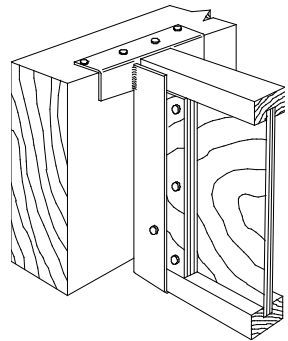
Top flange joist, beam, and purlin hangers rely on bearing of the top flange onto the top of the supporting member, along with the shear capacity of any fasteners that are present in the face. Although referred to as joist hangers, these connectors may support other horizontal members subject to vertical loads, such as beams or purlins.



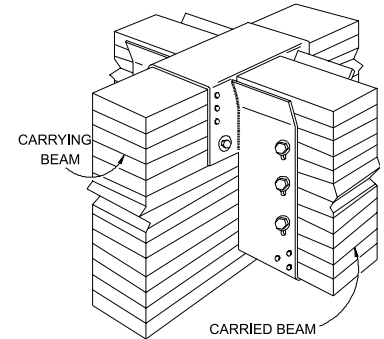
Bent-style Joist, Beam, and Purlin Hanger*



Top Flange I-joist Hanger*



Welded-type Purlin, Beam, and Joist Hanger*

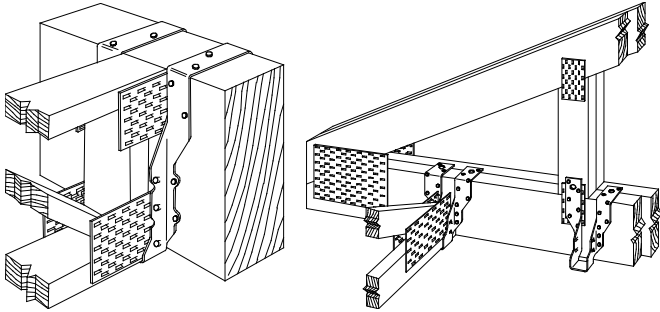


Heavy Welded Beam Hanger (Saddle Type)

* Joists shall be provided with lateral support at points of bearing to prevent rotation. Sometimes this restraint can be provided by the hangers.

Adjustable Style Hanger

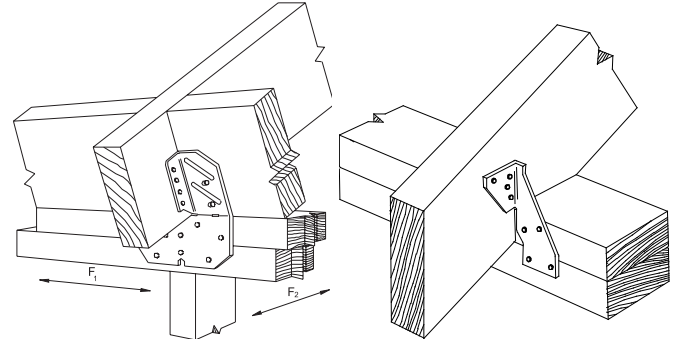
Adjustable style joist and truss hangers have straps which can be either fastened to the face of a supporting member, similar to a face mount hanger, or wrapped over the top of a supporting member, similar to a top flange hanger.



Adjustable Style Truss Hangers

Seismic and Hurricane Ties

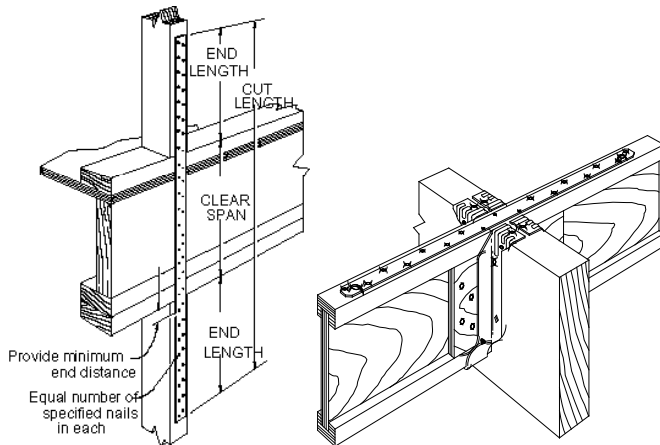
Seismic and hurricane ties are typically used to connect two members that are oriented 90° from each other. These ties resist forces through the shear capacity of the nails in the members. These connectors may provide resistance in three dimensions.



Seismic and Hurricane Ties
Connecting Roof Framing to Top Plates

Flat Straps

Flat straps rely on the shear capacity of the nails in the wood members to transfer load.

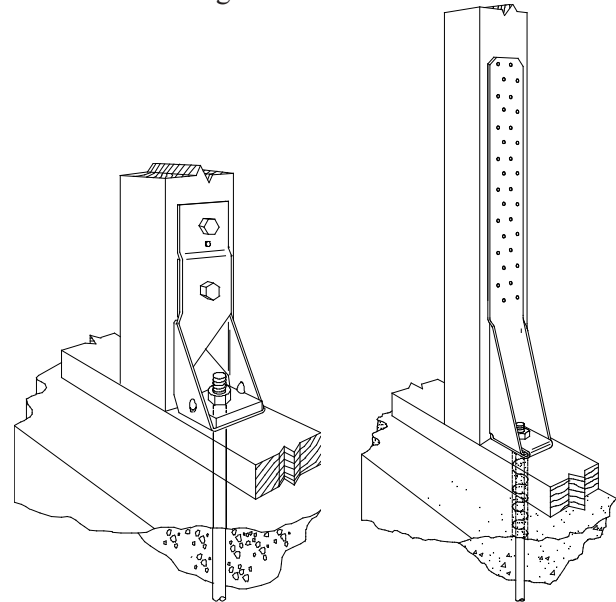


Strap Used to
Transfer Uplift Forces

Strap Used to
Transfer Lateral Forces

Hold Downs and Tension Ties

Hold downs and tension ties usually bolt to concrete or masonry, and connect wood members to the concrete or masonry through the shear resistance of either nails, screws, or bolts. They may also be used to connect two wood members together.

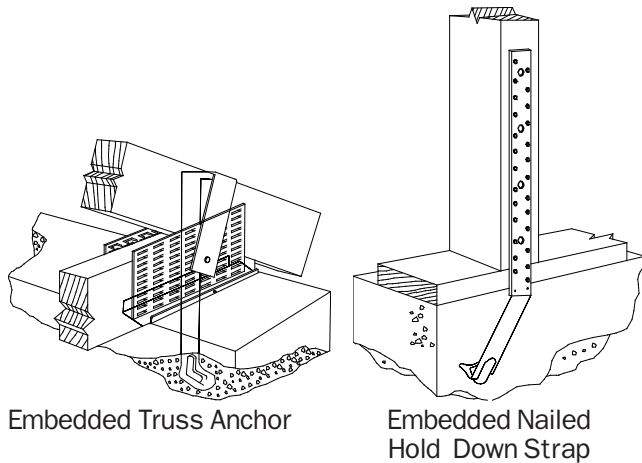


Hold Down

Tension Tie

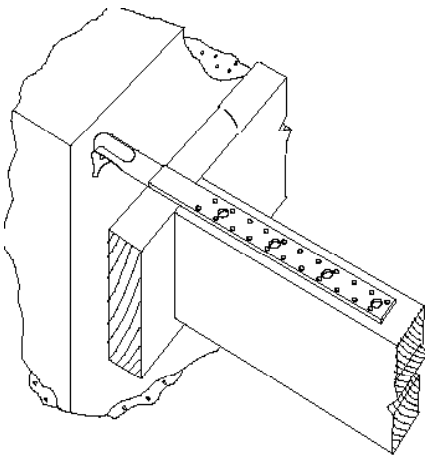
Embedded Type Anchors

Embedded anchors connect a wood member to concrete or masonry. One end of the connector embeds in the concrete or masonry, and the other end connects to the wood through the shear resistance of the nails or bolts.



Embedded Truss Anchor

Embedded Nailed Hold Down Strap



Purlin Anchor

Product Selection

Proper choice of connectors is required to optimize performance and economics. The selection of a connector will depend on several variables. These include the following:

- capacity required
- size and type of members being connected
- species of wood being connected
- slope and/or skew of member
- connector type preference

- type of fasteners to be used
- corrosion resistance desired
- appearance desired

Once the listed information is known, proper selection is facilitated through the use of manufacturer's literature, code evaluation reports, and software available from connector manufacturers.

This Manual provides guidance for specifying pre-engineered metal connectors to satisfy specific design criteria for a given application.

Connection Details

Connections, including pre-engineered metal connections, must provide the structural strength necessary to transfer loads. Well-designed connections hold wood members in such a manner that shrinkage/swelling cycles do not induce splitting across the grain. Well-designed connections also minimize collection of moisture – providing adequate clearance for air movement to keep the wood dry. Finally, well-designed connections minimize the potential for tension perpendicular to grain stresses – either under design conditions or under unusual loading conditions. Section M10.4 contains general concepts of well-designed connections, including over 40 details showing acceptable and unacceptable practice.

Other Considerations

With proper selection and installation, structural connectors will perform as they were designed. However, proper selection and installation involves a variety of items that both the designer and the installer must consider including the general topics of: the wood members being connected; the fasteners used; and the connectors themselves. These items are discussed in the following sections. This Manual does not purport to address these topics in an all-inclusive manner – it is merely an attempt to alert designers to the importance of selection and installation details for achieving the published capacity of the connector.

Wood Members

The wood members being connected have an impact on the capacity of the connection. The following are important items regarding the wood members themselves:

- The species of wood must be the same as that for which the connector was rated by the manufacturer. Manufacturers test and publish allowable design values only for certain species of wood. For other species, consult with the connector manufacturer.

- The wood must not split when the fastener is installed. A fastener that splits the wood will not take the design load. If wood tends to split, consider pre-boring holes using a diameter not exceeding 3/4 of the nail diameter. Pre-boring requirements for screws and bolts are provided in the *NDS*.
- Wood can shrink and expand as it loses and gains moisture. Most connectors are manufactured to fit common dry lumber dimensions. Other dimensions may be available from the manufacturer.
- Where built-up lumber (multiple members) is installed in a connector, the members must be fastened together prior to installation of the connector so that the members act as a single unit.
- The dimensions of the supporting member must be sufficient to receive the specified fasteners. Most connectors are rated based on full penetration of all specified fasteners. Refer to the connector manufacturer for other situations.
- Bearing capacity of the joist or beam should also be evaluated to ensure adequate capacity.

Fasteners

Most wood connectors rely on the fasteners to transfer the load from one member to the other. Therefore, the choice and installation of the fasteners is critical to the performance of the connector.

The following are important items regarding the fasteners used in the connector:

- All fasteners specified by the manufacturer must be installed to achieve the published value.
- The size of fastener specified by the manufacturer must be installed. Most manufacturers specify common nails, unless otherwise noted.
- The fastener must have at least the same corrosion resistance as the connector.
- Bolts must generally be structural quality bolts, equal to or better than ANSI/ASME Standard B18.2.1.
- Bolt holes must be a minimum of 1/32" and a maximum of 1/16" larger than the bolt diameter.
- Fasteners must be installed prior to loading the connection.
- Power-driven fasteners may deflect and injure the operator or others. Nail tools may be used to install connectors, provided the correct quantity and type of nails are properly installed in the manufacturer's nail holes. Nail tools with nail hole-locating mechanisms should be used. Follow the nail tool manufacturer's instructions and use the appropriate safety equipment.

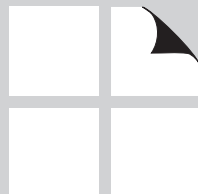
Connectors

Finally, the condition of the connector itself is critical to how it will perform. The following are important items regarding the connector itself:

- Connectors may not be modified in the field unless noted by the manufacturer. Bending steel in the field may cause fractures at the bend line, and fractured steel will not carry the rated load.
- Modified connectors may be available from the manufacturer. Not all modifications are tested by all manufacturers. Contact the manufacturer to verify loads on modified connectors.
- In general, all holes in connectors should be filled with the nails specified by the manufacturer. Contact the manufacturer regarding optional nail holes and optional loads.
- Different environments can cause corrosion of steel connectors. Always evaluate the environment where the connector will be installed. Connectors are available with differing corrosion resistances. Contact the manufacturer for availability. Fasteners must be at least the same corrosion resistance as that chosen for the connector.

M12: DOWEL- TYPE FASTENERS

M12.1 General	82
M12.2 Reference Withdrawal Design Values	82
M12.3 Reference Lateral Design Values	82
M12.4 Combined Lateral and Withdrawal Loads	82
M12.5 Adjustment of Reference Design Values	83
M12.6 Multiple Fasteners	83



M12.1 General

This Chapter covers design of connections between wood members using metal dowel-type (nails, bolts, lag screws, wood screws, drift pins) fasteners.

These connectors rely on metal-to-wood bearing for transfer of lateral loads and on friction or mechanical interfaces for transfer of axial (withdrawal) loads. They are commonly available in a wide range of diameters and lengths.

M12.2 Reference Withdrawal Design Values

The basic design equation for dowel-type fasteners under withdrawal loads is:

$$W'p \geq R_w$$

where:

W' = adjusted withdrawal design value

R_w = axial (withdrawal) force

p = depth of fastener penetration into wood member

Reference withdrawal design values are tabulated in *NDS* Chapter 12.

M12.3 Reference Lateral Design Values

The basic equation for design of dowel-type fasteners under lateral load is:

$$Z' \geq R_z$$

where:

Z' = adjusted lateral design value

R_z = lateral force

Reference lateral design values are tabulated in *NDS* Chapter 12.

M12.4 Combined Lateral and Withdrawal Loads

Lag screws, wood screws, nails, and spikes resisting combined lateral and withdrawal loads shall be designed in accordance with *NDS* 12.4.

M12.5 Adjustment of Reference Design Values

Dowel-type connections must be designed by applying all applicable adjustment factors to the reference withdrawal design value or reference lateral design value for the connection. *NDS* Table 11.3-1 lists all applicable adjustment factors for dowel-type connectors. Table

M12.3-1 shows the applicability of adjustment factors for dowel-type fasteners in a slightly different format for the designer.

Table M12.3-1 Applicability of Adjustment Factors for Dowel-Type Fasteners¹

	Allowable Stress Design	Load and Resistance Factor Design
Lateral Loads		
Dowel-Type Fasteners	$Z' = Z C_D C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn}$	$Z' = Z C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn} (3.32)(0.65) \lambda$
Withdrawal Loads		
Nails, Spikes, Lag Screws, Wood Screws, and Drift Pins	$W' = W C_D C_M C_t C_{eg} C_{tn}$	$Z' = Z C_M C_t C_{eg} C_{tn} (3.32)(0.65) \lambda$

1. See *NDS* Table 11.3.1 footnotes for additional guidance on application of adjustment factors for dowel-type fasteners.

Example of a Dowel-Type Fastener Loaded Laterally

For a single dowel-type fastener installed in side grain perpendicular to the length of the wood member, meeting the end and edge distance and spacing requirements of *NDS* 12.5.1, used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 11.3), and not a nail or spike in diaphragm construction, the general equation for Z' reduces to:

for ASD:

$$Z' = Z C_D$$

for LRFD:

$$Z' = Z (3.32)(0.65) \lambda$$

Example of a Dowel-Type Fastener Loaded in Withdrawal

For a single dowel-type fastener installed in side grain perpendicular to the length of the wood member, used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 11.3), the general equation for W' reduces to:

for ASD:

$$W' = W C_D$$

for LRFD:

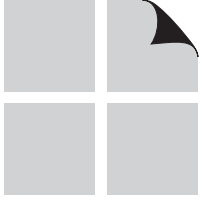
$$W' = W (3.32)(0.65) \lambda$$

Installation Requirements

To achieve stated design values, connectors must comply with installation requirements such as spacing of connectors, minimum edge and end distances, proper drilling of lead holes, and minimum fastener penetration.

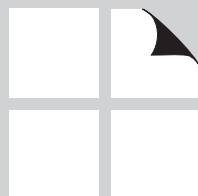
M12.6 Multiple Fasteners

Local stresses in connections using multiple fasteners can be evaluated in accordance with *NDS* Appendix E.



M13: SPLIT RING AND SHEAR PLATE CONNECTORS

M13.1	General	86
M13.2	Reference Design Values	86
M13.3	Placement of Split Ring and Shear Plate Connectors	86



M13.1 General

This Chapter covers design for split rings and shear plates. These connectors rely on their geometry to provide

larger metal-to-wood bearing areas per connector. Both are installed into precut grooves or daps in the members.

M13.2 Reference Design Values

Reference lateral design values (P, Q) are tabulated in the split ring and shear plate tables in *NDS* 13.2.

Design Adjustment Factors

Split ring and shear plate connections must be designed by applying all applicable adjustment factors to the refer-

ence lateral design value for the connection. *NDS* Table 11.3.1 provides all applicable adjustment factors for split ring and shear plate connectors. Table M13.2-1 shows the applicability of adjustment factors for dowel-type fasteners in a slightly different format for the designer.

Table M13.2-1 Applicability of Adjustment Factors for Split Ring and Shear Plate Connectors¹

	Allowable Stress Design	Load and Resistance Factor Design
Split Ring and Shear Plate Connectors	$P' = P C_D C_M C_t C_g C_{\Delta} C_d C_{st}$	$P' = P C_M C_t C_g C_{\Delta} C_d C_{st} (3.32)(0.65) \lambda$
	$Q' = Q C_D C_M C_t C_g C_{\Delta} C_d$	$Q' = Q C_M C_t C_g C_{\Delta} C_d (3.32)(0.65) \lambda$

1. See *NDS* Table 11.3.1 footnotes for additional guidance on application of adjustment factors for split ring and shear plate connectors.

For a single split ring or shear plate connection installed in side grain perpendicular to the length of the wood members, meeting the end and edge distance and spacing requirements of *NDS* 13.3, used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 11.3), and meeting the penetration requirements of *NDS* 13.2.3, the general equations for P' and Q' reduce to:

for ASD:

$$P' = P C_D$$

$$Q' = Q C_D$$

for LRFD:

$$P' = P (3.32)(0.65) \lambda$$

$$Q' = Q (3.32)(0.65) \lambda$$

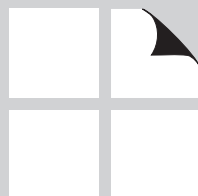
M13.3 Placement of Split Ring and Shear Plate Connectors

Installation Requirements

To achieve stated design values, connectors must comply with installation requirements such as spacing of connectors, minimum edge and end distances, proper dapping and grooving, drilling of lead holes, and minimum fastener penetration as specified in *NDS* 13.3.

M14: TIMBER RIVETS

M14.1 General	88
M14.2 Reference Design Values	88
M14.3 Placement of Timber Rivets	88



M14.1 General

This Chapter covers design for timber rivets. Timber rivets are hardened steel nails that are driven through pre-drilled holes in steel side plates (typically 1/4" thickness)

to form an integrated connection where the plate and rivets work together to transfer load to the wood member.

M14.2 Reference Design Values

Reference wood capacity design values parallel to grain, P_w , are tabulated in the timber rivet Tables 14.2.1A through 14.2.1F in the *NDS*.

Reference design values perpendicular to grain are calculated per *NDS* 14.2.2.

Design Adjustment Factors

Connections must be designed by applying all applicable adjustment factors to the reference lateral design value for the connection. *NDS* Table 11.3-1 lists all applicable adjustment factors for timber rivets. Table M14.2-1 shows the applicability of adjustment factors for timber rivets in a slightly different format for the designer.

Table M14.2-1 Applicability of Adjustment Factors for Timber Rivets¹

	Allowable Stress Design	Load and Resistance Factor Design
Timber Rivets	$P' = P C_D C_M C_t C_{st}$	$P' = P C_M C_t C_{st} (3.32)(0.65) \lambda$
	$Q' = Q C_D C_M C_t C_{\Delta} C_{st}$	$Q' = Q C_M C_t C_{\Delta} C_{st} (3.32)(0.65) \lambda$

1. See *NDS* Table 11.3.1 footnotes for additional guidance on application of adjustment factors for timber rivets.

For a timber rivet connection installed in side grain perpendicular to the length of the wood members, with metal side plates 1/4" or greater, used in a normal building environment (meeting the reference conditions of *NDS* 2.3 and 11.3), and where wood capacity perpendicular to grain, Q_w , does not control, the general equations for P' and Q' reduce to:

for ASD:

$$P' = P C_D$$

$$Q' = Q C_D$$

for LRFD:

$$P' = P (3.32)(0.65) \lambda$$

$$Q' = Q (3.32)(0.65) \lambda$$

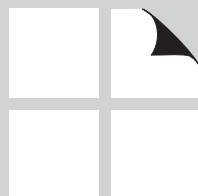
M14.3 Placement of Timber Rivets

Installation Requirements

To achieve stated design values, connectors must comply with installation requirements such as spacing of connectors, minimum edge and end distances per *NDS* 14.3; and drilling of lead holes, minimum fastener penetration, and other fabrication requirements per *NDS* 14.1.2.

M15: SPECIAL LOADING CONDITIONS

M15.1 Lateral Distribution of Concentrated Loads	90
M15.1.1 Lateral Distribution of a Concentrated Load for Moment	90
M15.1.2 Lateral Distribution of a Concentrated Load for Shear	90
M15.2 Spaced Columns	90
M15.3 Built-Up Columns	90
M15.4 Wood Columns with Side Loads and Eccentricity	90



M15.1 Lateral Distribution of Concentrated Loads

M15.1.1 Lateral Distribution of a Concentrated Load for Moment

The lateral distribution factors for moment in *NDS* Table 15.1.1 are keyed to the nominal thickness of the flooring or decking involved (2" to 6" thick). Spacing of the stringers or beams is based on recommendations of the American Association of State Highway and Transportation Officials.

Lateral distribution factors determined in accordance with *NDS* Table 15.1.1 can be used for any type of fixed or moving concentrated load.

M15.1.2 Lateral Distribution of a Concentrated Load for Shear

The lateral distribution factors for shear in *NDS* Table 15.1.2 relate the lateral distribution of concentrated load at the center of the beam or stringer span as determined under *NDS* 15.1.1, or by other means, to the distribution of load at the quarter points of the span. The quarter points are considered to be near the points of maximum shear in the stringers for timber bridge design.

M15.2 Spaced Columns

As used in the *NDS*, spaced columns refer to two or more individual members oriented with their longitudinal axis parallel, separated at the ends and in the middle portion of their length by blocking and joined at the ends by split ring or shear plate connectors capable of developing required shear resistance.

The end fixity developed by the connectors and end blocks increases the load-carrying capacity in compression parallel to grain of the individual members only in the direction perpendicular to their wide faces.

AWC's Wood Structural Design Data (WSD) provides load tables for spaced columns.

M15.3 Built-Up Columns

As with spaced columns, built-up columns obtain their efficiency by increasing the buckling resistance of individual laminations. The closer the laminations of a mechanically fastened built-up column deform together (the

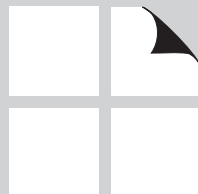
smaller the amount of slip occurring between laminations) under compressive load, the greater the relative capacity of that column compared to a simple solid column of the same slenderness ratio made with the same quality of material.

M15.4 Wood Columns with Side Loads and Eccentricity

The eccentric load design provisions of *NDS* 15.4.1 are not generally applied to columns supporting beam loads where the end of the beam bears on the entire cross section of the column. It is standard practice to consider such loads to be concentrically applied to the supporting column. This practice reflects the fact that the end fixity provided by the end of the column is ignored when the usual pinned end condition is assumed in column design. In applications where the end of the beam does not bear on the full cross section of the supporting column, or in special critical loading cases, use of the eccentric column loading provisions of *NDS* 15.4.1 may be considered appropriate by the designer.

M16: FIRE DESIGN

M16.1 General	92
Lumber	94
Structural Glued Laminated Timber	94
Poles and Piles	95
Structural Composite Lumber	95
Wood I-Joists	96
Metal Plate Connected Wood Trusses	96
Transitory Floor Vibration and Sound Transmission	99
M16.2 Design Procedures for Exposed Wood Members	100
M16.3 Wood Connections	101



M16.1 General

This Chapter outlines fire considerations including design requirements and fire-rated assemblies for various wood products. Lumber, glued laminated timber, poles and piles, wood I-joists, structural composite lumber, metal plate connected wood trusses, and cross-laminated timber are discussed.

Planning

As a first step, the authority having jurisdiction where a proposed building is to be constructed must be consulted for the requirements of the specific design project. This normally concerns the type of construction desired as well as allowable building areas and heights for each construction type.

Wood building construction is generally classified into types such as wood frame (Type V), noncombustible or fire-retardant-treated wood wall-wood joist (Type III), and heavy timber (Type IV). Type V construction is defined as having exterior walls, bearing walls, partitions, floors and roofs of wood stud and joist framing of 2" nominal dimension. These are divided into two subclasses that are either protected or unprotected construction. Protected construction calls for having load-bearing assemblies of 1-hour fire endurance.

Type III construction has exterior walls of noncombustible materials and roofs, floors, and interior walls and partitions of wood frame. As in Type V construction, these are divided into two subclasses that are either protected or unprotected.

Type IV construction includes exterior walls of noncombustible materials or fire-retardant-treated wood and columns, floors, roofs, and interior partitions of wood of a minimum size, as shown in Tables M16.1-1 and M16.1-2.

In addition to having protected and unprotected subclasses for each building type, increases in floor area and height of the building are allowed when active fire protection, such as sprinkler protection systems, are included. For example, protected wood-frame business occupancies can be increased from three to four stories in height because of the presence of sprinklers. Also, the floor area may be further increased under some conditions. Additional information is available at www.awc.org.

Table M16.1-1 Heavy Timber Size Equalivalencies

Minimum Nominal Solid Sawn Size		Minimum Glued Laminated Timber Net Size		Minimum Structural Composite Lumber Net Size	
Width, inch	Depth, inch	Width, inch	Depth, inch	Width, inch	Depth, inch
8	8	6-3/4	8-1/4	7	7-1/2
6	10	5	10-1/2	5-1/4	9-1/2
6	8	5	8-1/4	5-1/4	7-1/2
6	6	5	6	5-1/4	5-1/2
4	6	3	6-7/8	3-1/2	5-1/2

Table M16.1-2 Minimum Sizes to Qualify as Heavy Timber Construction

Material		Minimum Size (all sizes nominal unless noted)
Roof Decking	<ul style="list-style-type: none"> • Sawn or glued laminated lumber planks, splined or T&G • Structural-use panels • Planks set on edge close together • Cross-laminated timber 	2 in. thickness 1-1/8 in. thickness ¹ 3 in. width 3 in. thickness (actual)
Floor Decking	<ul style="list-style-type: none"> • Sawn or glued laminated lumber planks, splined or T&G, and topped with <ul style="list-style-type: none"> - Tongue-and-groove flooring, or - Wood Structural panel, or - Particleboard • Planks set on edge close together, and topped with <ul style="list-style-type: none"> - Flooring, or - Wood Structural panel, or - Particleboard • Cross-laminated timber 	3 in. thickness 1 in. thickness ¹ 15/32 in. thickness ¹ 1/2 in. thickness 4 in. width 1 in. thickness ¹ 15/32 in. thickness ¹ 1/2 in. thickness 4 in. thickness (actual)
Roof Framing	<ul style="list-style-type: none"> • Wood-frame or glued laminated arches from the floor line or grade not supporting floors <ul style="list-style-type: none"> - Lower half of the height - Upper half of the height • Wood-frame or glued laminated arches from the top of walls, not supporting floors • Framed timber trusses and other roof framing • Spaced members shall be permitted to be composed of two or more pieces where <ul style="list-style-type: none"> - Blocked solidly throughout the intervening spaces - Spaces are tightly closed by a continuous wood cover plate secured to the underside of the members - Splice plates • Roof framing members protected by approved automatic sprinklers under the roof deck 	6 in. width by 8 in. width 6 in. width by 6 in. width 4 in. width by 6 in. width 4 in. width by 6 in. width 3 in. thickness 2 in. thickness 2 in. thickness 3 in. thickness 3 in. width
Floor Framing	<ul style="list-style-type: none"> • Sawn or glued-laminated beams and girders • Wood-frame or glulam arches from the floor line or grade supporting floors • Framed timber trusses 	6 in. width by 10 in. depth 8 in. width by 8 in. depth 8 in. width by 8 in. depth
Columns	<ul style="list-style-type: none"> • Sawn or glued-laminated beams and girders <ul style="list-style-type: none"> - Supporting floor loads - Supporting only roof and ceiling loads 	8 in. width by 8 in. depth 6 in. width by 8 in. depth
Exterior Walls	<ul style="list-style-type: none"> • Cross-laminated timber with fire-resistance rating up to 2 hours and covered with <ul style="list-style-type: none"> - Fire-retardant-treated wood sheathing, or - Gypsum board 	6 in. total wall thickness (actual) 15/32 in. thickness ¹ 1/2 in. thickness

1. Performance Category value for wood structural panels produced in accordance with PS 1 or PS 2.

Lumber

Building Code Requirements

For occupancies such as stores, apartments, offices, and other commercial and industrial uses, building codes commonly require floor/ceiling and wall assemblies to be fire-resistance rated in accordance with standard fire tests.

Depending on the application, wall assemblies may need to be rated either from one side or both sides. For specific exterior wall applications, the *International Building Code (IBC)* allows wood-frame, wood-sided walls to be tested for exposure to fire from the inside only. Rating for both interior and exterior exposure is only required when the wall has a fire separation distance of less than 5 feet. Code recognition of 1- and 2-hour wood-frame wall systems is also predicated on successful fire and hose stream testing in

accordance with ASTM E119, *Standard Test Methods for Fire Tests of Building Construction Materials*.

Fire Tested Assemblies

Fire-rated wood-frame assemblies can be found in a number of sources including the *IBC*, Underwriters Laboratories (UL) *Fire Resistance Directory*, Intertek Testing Services' *Directory of Listed Products*, and the Gypsum Association's *Fire Resistance Design Manual*. The American Wood Council (AWC) and its members have tested a number of wood-frame fire-rated assemblies. Descriptions of these successfully tested assemblies are provided in AWC's *Design for Code Acceptance (DCA) No. 3, Fire Rated Wood Floor and Wall Assemblies* which is available at www.awc.org.

Structural Glued Laminated Timber

Fires do not normally start in structural framing, but rather in the building's contents. These fires generally reach temperatures of between 1,290°F and 1,650°F. Glued laminated timber members perform very well under these conditions. Unprotected steel members typically suffer severe buckling and twisting during fires, often collapsing catastrophically.

Wood ignites at about 480°F, but charring may begin as low as 300°F. Wood typically chars at 1/40 in. per minute. Thus, after 30 minutes of fire exposure, only the outer 3/4 in. of the structural glued laminated timber will be damaged. Char insulates a wood member and hence raises the temperature it can withstand. Most of the cross section will remain intact, and the member will continue supporting loads during a typical building fire.

It is important to note that neither building materials alone, nor building features alone, nor detection and fire extinguishing equipment alone can provide adequate safety from fire in buildings. To ensure a safe structure in the event of fire, authorities base fire and building code requirements on research and testing, as well as fire histo-

ries. The model building codes classify heavy timber as a specific type of construction and give minimum sizes for roof and floor beams.

The requirements set out for heavy timber construction in model building codes do not constitute 1-hour fire resistance. However, procedures are available to calculate the structural glued laminated timber size required for projects in which 1-hour fire resistance is required (see *NDS 16.2* and AWC's *Technical Report 10* available at www.awc.org).

To achieve a 1-hour fire rating for beams whose dimensions qualify them for this rating, the basic layup must be modified – one core lamination must be removed from the center and the tension face augmented with the addition of a tension lamination. For more information concerning the effects of fire on structural glued laminated timber, refer to APA EWS Technical Note Y245 or AITC Technical Note 7. For determining fire resistance other than 1 hour, see *NDS 16.2* and AWC's *Technical Report 10* available at www.awc.org.

Poles and Piles

Very few elements of modern structures can be called “fire proof.” Even in buildings where the major structural members are noncombustible, most of the furnishings are flammable. It is for this reason that much of the attention in modern building codes addresses issues related to containing and limiting fires, rather than simply calling materials combustible and noncombustible.

While this topic is fairly complex for other types of products, fire performance is relatively straightforward for

poles and piles. Poles are generally used in cross-sectional sizes that qualify as heavy timber construction in the model building codes. On this basis, timber poles compare favorably with other construction materials in their performance under fire conditions. Piles are generally not exposed to fire conditions unless they extend substantially above the groundline.

Structural Composite Lumber

Engineered wood products have fire resistive characteristics very similar to conventional wood frame members. Since many engineered wood products are proprietary, they are usually recognized in a code evaluation report published by an evaluation service. Each evaluation report usually contains fire resistance information.

Very few elements of modern structures can be called “fire proof.” Even in buildings where the major structural members are noncombustible, most of the furnishings are flammable. It is for this reason that much of the attention in model building codes addresses issues related to containing and limiting fires, rather than simply calling materials combustible and noncombustible. The primary intent of the building codes is to ensure structural stability to allow exiting, evacuation, and fire fighting.

As with the previous topic of durability, this Manual cannot cover the topic of designing for optimal structural performance in fire conditions in detail. There are many excellent texts on the topic, and designers are advised to use this information early in the design process. To assist the designer in understanding several ways in which fire performance can be addressed, the following overview is provided.

Fire sprinklers are probably the most effective method to enhance fire resistance of engineered wood systems (as well as other systems). They are designed to control the fire while protecting the occupants and the building until the fire department arrives. They are the ultimate way to improve fire safety.

Heavy timber construction has proven to be acceptable in many areas where fire safety is of utmost concern. These applications have proven to be not only reliable, but economical in certain structures – many wider width SCL products can be used in heavy timber construction. Consult manufacturer’s literature or code evaluation reports for specific information.

The fire performance of wood structures can be enhanced in the same ways as that of structures of steel, concrete, or masonry:

- Fire sprinkler systems have proven to be effective in a variety of structures, both large and small
- Protection of the structural members with materials such as properly attached gypsum sheathing can provide greatly improved fire performance. Fire ratings, as established from test procedures specified in ASTM E-119, of up to 2 hours can be achieved through the use of gypsum sheathing
- Where surface burning characteristics are critical, fire-retardant treatments can be used to reduce the flamespread for some products

To reiterate, this Manual does not purport to address this topic in an all-inclusive manner – it is merely an attempt to alert designers to the need to address fire performance issues in the design of the structure.

Wood I-Joists

The wood I-joist industry has actively supported the following projects to establish fire performance of systems using wood I-joist products:

- ASTM E-119 fire tests have been conducted by the wood I-joist industry to establish fire resistance ratings for generic I-joist systems. Detailed descriptions of these systems are shown in AWC's *Design for Code Acceptance (DCA) No. 3, Fire Rated Wood Floor and Wall Assemblies* available at www.awc.org.
- ASTM E-119 fire tests have been conducted by wood I-joist manufacturers to establish fire resistance ratings for proprietary I-joist systems. Detailed descriptions of these systems are available from the individual I-joist manufacturer.
- National Fire Protection Research Foundation Report titled "National Engineered Lightweight Construction Fire Research Project." This report documents an extensive literature search of the fire performance of engineered lightweight construction.
- A video, *I-JOISTS: FACTS ABOUT PROGRESS*, has been produced by the Wood I-Joist Manufacturer's Association (WIJMA). This video describes some basic facts about changes taking place within the construction industry and the fire service. Along with this video is a document that provides greater details on fire performance issues.
- Industry research in fire endurance modeling for I-joist systems.

Metal Plate Connected Wood Trusses

Generally, a fire endurance rating of 1 hour is mandated by code for many of the applications where trusses could be used. All testing on these assemblies is performed in accordance with the ASTM's *Standard Methods for Fire Tests of Building Construction and Materials* (ASTM E119).

The two primary source documents for fire endurance assembly results are the *Fire Resistance Design Manual*, published by the Gypsum Association (GA), and the *Fire Resistance Directory*, published by Underwriters' Laboratories, Inc. (UL). Warnock Hersey (WH) assemblies are now listed in the *ITS Directory of Listed Products*. These tested assemblies are available for specification by architects or building designers, and for use by all truss manufacturers where a rated assembly is required, and can generally be applied to both floor and roof assembly applications.

According to the UL Directory's Design Information Section: "Ratings shown on individual designs apply to equal or greater height or thickness of the assembly, and to larger structural members, when both size and weight are equal or larger than specified, and when the thickness of the flanges, web or diameter of chords is equal or greater." Thus, larger and deeper trusses can be used under the auspices of the same design number. This approach has often been used in roof truss applications since roof trusses are usually much deeper than the tested assemblies.

Thermal and/or acoustical considerations at times may require the installation of insulation in a floor-ceiling

or roof-ceiling assembly that has been tested without insulation. As a general 'rule,' experience indicates that it is allowable to add insulation to an assembly, provided that the depth of the truss is increased by the depth of the insulation. And as a general 'rule,' assemblies that were tested with insulation may have the insulation removed.

To make a rational assessment of any modification to a tested assembly, one must look at the properties of the insulation and the impact that its placement inside the assembly will have on the fire endurance performance of the assembly. Insulation retards the transfer of heat, is used to retain heat in warm places, and reduces the flow of heat into colder areas. As a result, its addition to a fire endurance assembly will affect the flow of heat through and within an assembly. One potential effect of insulation placed directly on the gypsum board is to retard the dissipation of heat through the assembly, concentrating heat in the protective gypsum board.

In some cases specific branded products are listed in the test specifications. Modifications or substitutions to fire endurance assemblies should be reviewed with the building designer and code official, preferably with the assistance of a professional engineer. This review is required because the final performance of the assembly is a result of the composite of the materials used in the construction of the assembly.

The Structural Building Components Association provides summaries of wood truss fire endurance assemblies and sound transmission ratings. For more information, visit

www.sbcindustry.com. Also, several truss plate manufacturers have developed proprietary fire resistant assemblies. These results apply only to the specific manufacturer's truss plates and referenced fire endurance assembly system. For more detailed information on these assemblies, the individual truss plate manufacturer should be contacted. Complete specifications on the UL, GA, and WH assemblies are available on their respective websites.

Area Separation Assemblies

It is of great concern when fire-rated assemblies are designed and specified without consideration of sound structural principles. Should a fire develop, these structural inadequacies could cause the assemblies to fail unexpectedly, increasing the risk of loss of life. There are a number of ways to provide sound structural and fire endurance details that maintain 1-hour rated area separation assemblies.

Figure M16.1-1 shows several possible assemblies that can be used to make up the 1-hour rated system for separation between occupancies for a) floor trusses parallel to the wall assembly; and b) perpendicular to the wall assembly. A 2x4 firestop is used between the walls next to the wall top plates. This effectively prevents the spread of fire inside the wall cavity. A minimum 1/2-inch gypsum wallboard attached to one side of the floor truss system, and located between the floor trusses, also provides a draftstop and fire protection barrier between occupancy spaces if a fire starts in the floor truss concealed space, which is a rare occurrence. The tenant separation in the roof is maintained through the use of a 1/2-inch gypsum wallboard draftstop attached to the ends of one side of the monopitch trusses and provided for the full truss height. Figure M16.1-1 effectively provides 1-hour compartmentation for all the occupied spaces using listed 1-hour rated assemblies and the appropriate draftstops for the concealed spaces as prescribed by the model building codes.

If a fire-resistive assembly, rather than draftstopping, is required within concealed attic spaces, UL U338, U339, and U377, provide approved 1-hour and 2-hour rated assemblies that may be used within the roof cavity and that may be constructed with gable end frames.

The critical aspects for fire endurance assemblies include:

- Ensuring that the wall and ceiling assemblies of the room use 1-hour rated assemblies. These are independent assemblies. The wall assembly does not have to be continuous from the floor to the roof to meet the intent of the code or the fire endurance performance of the structure. The intent of the code is that the building be broken into compartments to contain a fire to a given area. Fire resistance assemblies are tested to provide code-complying

fire endurance to meet the intent of the code. The foregoing details meet the intent of the code.

- Properly fastening the gypsum wallboard to the wall studs and trusses. This is critical for achieving the desired fire performance from a UL or GA assembly.
- Ensuring that the detail being used is structurally sound, particularly the bearing details. All connection details are critical to assembly performance. When a fire begins, if the structural detail is poor, the system will fail at the poor connection detail earlier than expected.
- Accommodating both sound structural details with appropriate fire endurance details. Since all conceivable field conditions have not been and cannot be tested, rational engineering judgment needs to be used.

The foregoing principles could also be applied to structures requiring 2-hour rated area separation assemblies. In this case, acceptable 2-hour wall assemblies would be used in conjunction with the 2-hour floor-ceiling and roof-ceiling fire endurance assemblies.

Through-Penetration Fire Stops

Because walls and floors are penetrated for a variety of plumbing, ventilation, electrical, and communication purposes, ASTM E814 *Standard Method of Fire Tests of Through-Penetration Fire Stops* uses fire-resistive assemblies (rated per ASTM E119) and penetrates them with cables, pipes, and ducts, etc., before subjecting the assembly to ASTM E119's fire endurance tests.

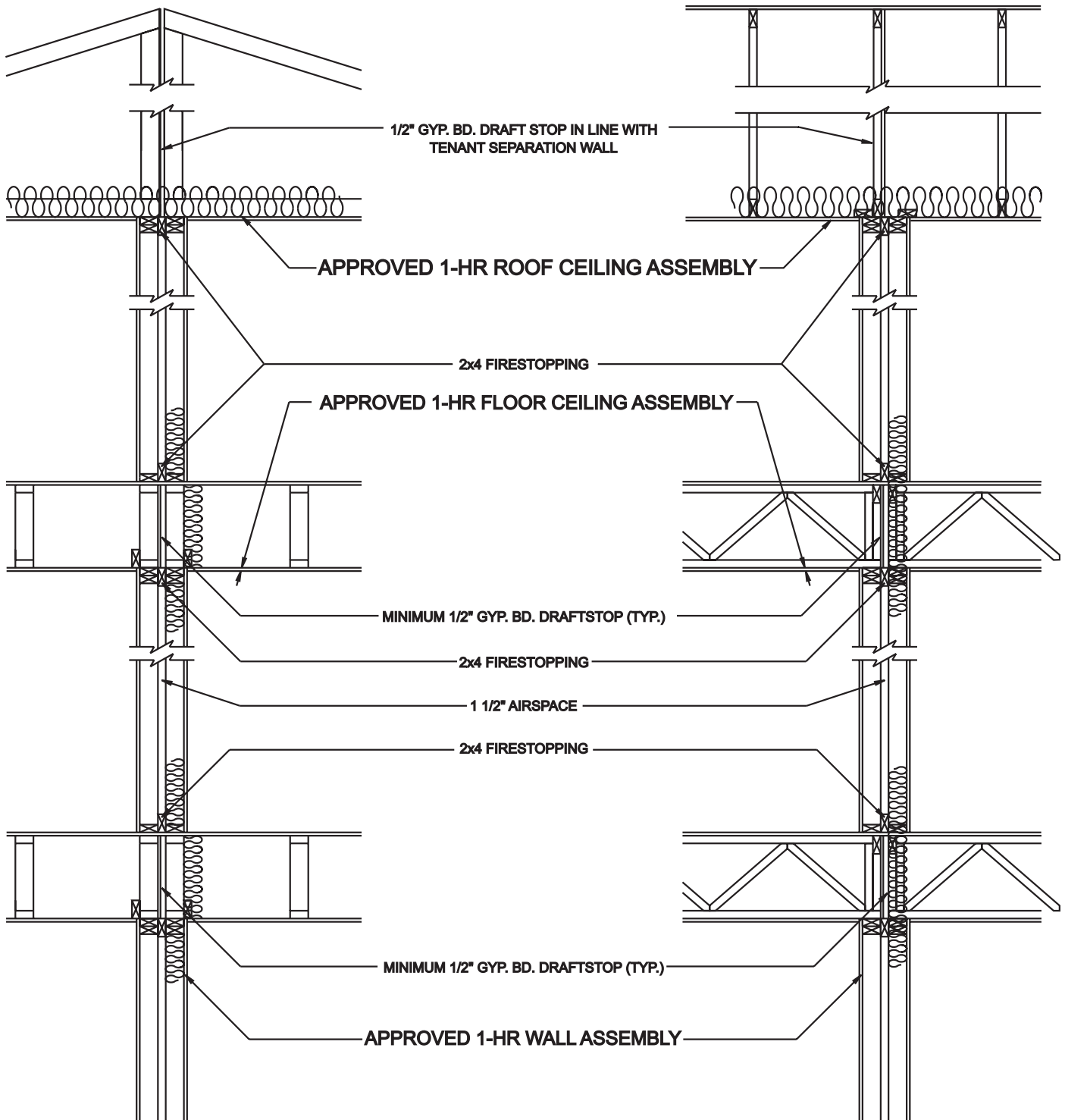
Despite the penetrations, firestop assemblies tested according to ASTM E814 must not significantly lose their fire containment properties in order to be considered acceptable. Properties are measured according to the passage of any heat, flame, hot gases, or combustion through the fire stop to the test assembly's unexposed surface.

Upon successful completion of fire endurance tests, fire stop systems are given F&T ratings. These ratings are expressed in hourly terms, in much the same fashion as fire-resistive barriers.

To obtain an F-Rating, a fire stop must remain in the opening during the fire and hose stream test, withstanding the fire test for a prescribed rating period without permitting the passage of flame on any element of its unexposed side. During the hose stream test, a fire stop must not develop any opening that would permit a projection of water from the stream beyond the unexposed side.

To obtain a T-Rating, a fire stop must meet the requirements of the F-Rating. In addition, the fire stop must prevent the transmission of heat during the prescribed rating period which would increase the temperature of any

Figure M16.1-1 Cross Sections of Possible One-Hour Area Separations



thermocouple on its unexposed surface, or any penetrating items, by more than 325°F.

The UL Fire Resistance Directory lists literally thousands of tested systems. They have organized the systems with an alpha-alpha-numeric identification number. The first alpha is either a F, W, or C. These letters signify the type of assembly being penetrated: F signifies a floor, W signifies a wall, and C signifies a ceiling. The second alpha signifies a limiting description, for example: C signifies a

framed floor, and L signifies a framed wall. The numeric portion is also significant: 0000-0999 signifies no penetrating items, 1000-1999 signifies metallic pipe, 2000-2999 signifies nonmetallic pipe, 3000-3999 signifies electrical cable, 4000-4999 signifies cable trays, 5000-5999 signifies insulated pipes, 6000-6999 signifies miscellaneous electrical penetrants, 7000-7999 signifies miscellaneous mechanical penetrants, and 8000-8999 signifies a combination of penetrants.

Transitory Floor Vibration and Sound Transmission

Sound Transmission

Sound transmission ratings are closely aligned with fire endurance ratings for assemblies. This is due to the fact that flame and sound penetrations follow similar paths of least resistance.

Sound striking a wall or ceiling surface is transmitted through the air in the wall or ceiling cavity. It then strikes the opposite wall surface, causing it to vibrate and transmit the sound into the adjoining room. Sound also is transmitted through any openings into the room, such as air ducts, electrical outlets, window openings, and doors. This is airborne sound transmission. The Sound Transmission Class (STC) method of rating airborne sounds evaluates the comfort ability of a particular living space. The higher the STC, the better the airborne noise control performance of the structure. An STC of 50 or above is generally considered a good airborne noise control rating. Table M16.1-3 describes the privacy afforded according to the STC rating.

Impact Sound Transmission is produced when a structural element is set into vibration by direct impact – someone walking, for example. The vibrating surface generates sound waves on both sides of the element. The

Impact Insulation Class (IIC) is a method of rating the impact sound transmission performance of an assembly. The higher the IIC, the better the impact noise control of the element. An IIC of 55 is generally considered a good impact noise control.

Table M16.1-3 Privacy Afforded According to STC Rating

STC Rating	Privacy Afforded
25	Normal speech easily understood
30	Normal speech audible, but not intelligible
35	Loud speech audible and fairly understandable
40	Loud speech barely audible, but not intelligible
45	Loud speech barely audible
50	Shouting barely audible
55	Shouting inaudible

M16.2 Design Procedures for Exposed Wood Members

For members stressed in one principle direction, simplifications can be made which allow the tabulation of load factor tables for fire design. 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. AWC's *Technical Report 10 Calculating the Fire Resistance of Exposed Wood Members* provides derivation of load ratios and load ratio tables for bending, compression, and tension members. TR10 is available at www.awc.org.

M16.3 Wood Connections

Where 1-hour fire endurance is required, connectors and fasteners must be protected from fire exposure by 1.5" of wood, fire-rated gypsum board, or any coating approved for a 1-hour rating. Typical details for commonly used fasteners and connectors in timber framing are shown in Figure M16.3-1 through Figure M16.3-5.

Figure M16.3-1 Beam to Column Connection - Connection Not Exposed to Fire

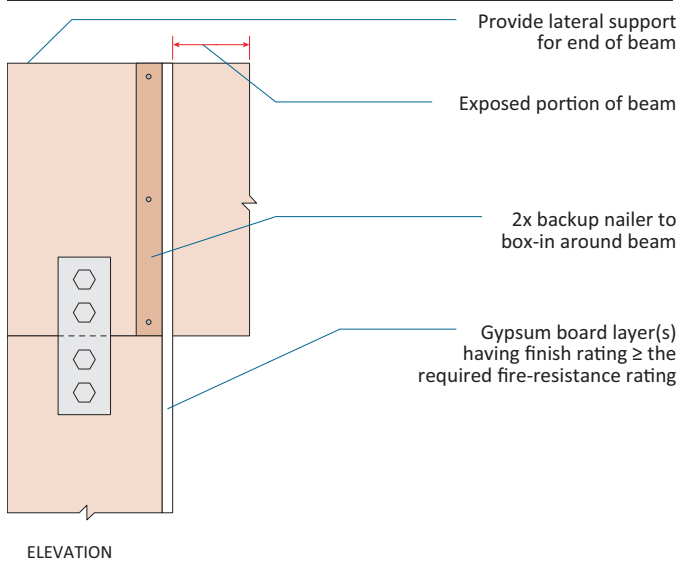


Figure M16.3-2 Beam to Column Connection - Connection Exposed to Fire Where Appearance is a Factor

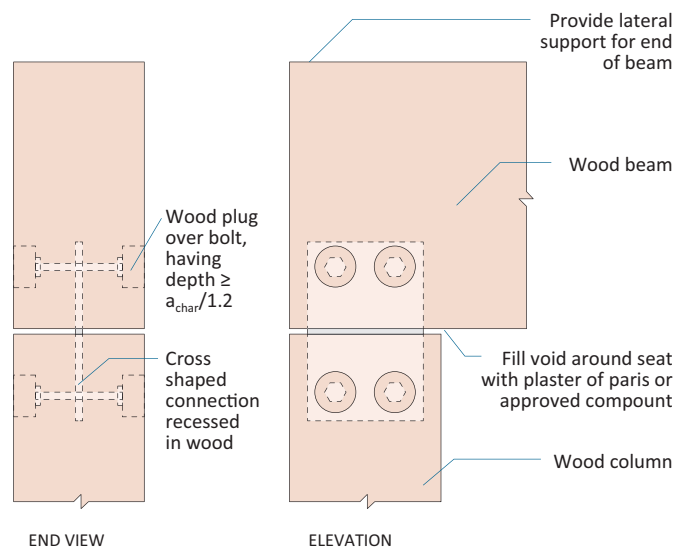


Figure M16.3-3 Beam to Column Connection - Connection Exposed to Fire Where Appearance is Not a Factor

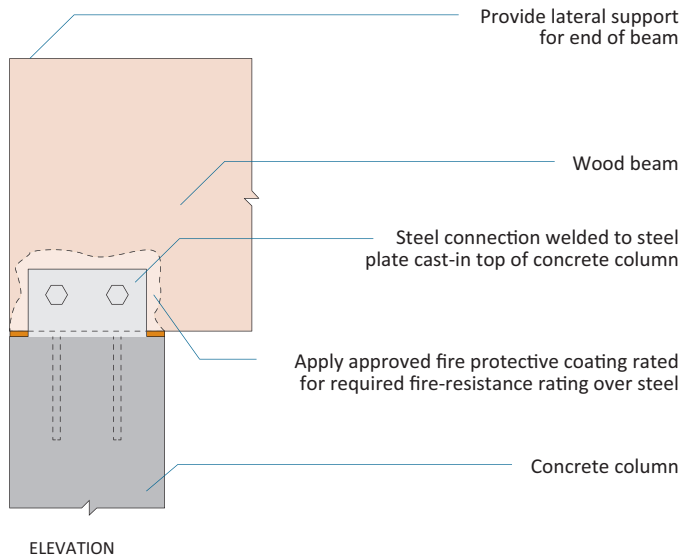


Figure M16.3-4 Column Connections Covered

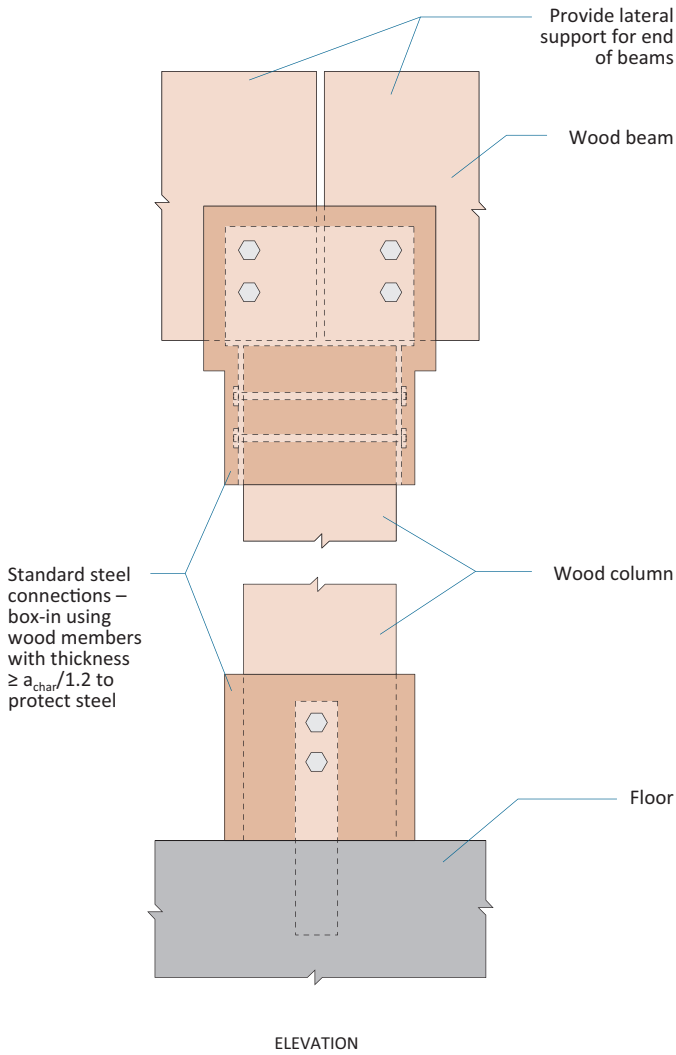
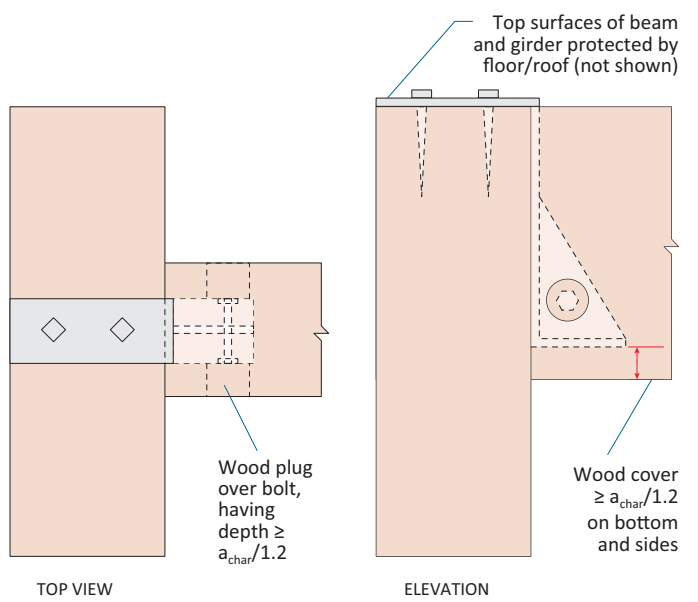


Figure M16.3-5 Beam to Girder - Concealed Connection



American Wood Council

AWC Mission Statement

To increase the use of wood by assuring the broad regulatory acceptance of wood products, developing design tools and guidelines for wood construction, and influencing the development of public policies affecting the use and manufacture of wood products.

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