AMERICAN WOOD COUNCIL

January 2024

ERRATA

to the 2015 Edition of Commentary for the *National Design Specification (NDS) for Wood Construction*

(All prior PDF and print versions)

Page **Revision**

249 Revise equation C12.2.2-2 as shown in red below:

$$K_W = 1.2\left(\frac{14250}{6}\right) \qquad \text{(C12.2.2-2)}$$

AMERICAN WOOD COUNCIL

January 2021

ERRATA
to the 2018 and Prior Editions of
the National Design Specification® (NDS®) for Wood Construction

Page Revision

91 Revise footnote 1 in Table 12.5.1D as follows:

1. The ℓ/D ratio used to determine the minimum ~~edge distance~~ spacing between rows shall be the lesser of:
   (a) length of fastener in wood main member/D = ℓ_m/D
   (b) total length of fastener in wood side member(s)/D = ℓ_s /D

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AMERICAN WOOD COUNCIL

July 2020

ERRATA
to the 2018 and Prior Editions of
the National Design Specification® (NDS®) for Wood Construction

Page	**Revision**
166	Clarifies that the following calculations in Example E.7 Sample Solution of Row of Bolts is intended for a single-row bolted connection with a 3-1/2″ thick main member and 1-1/2″ thick side member:

E.7 Sample Solution of Row of Bolts

Calculate the net section area tension and row tear-out adjusted ASD design capacities for the single-shear single-row bolted connection represented in Figure E2.

Main and Side Members:
#2 grade Hem-Fir ~~2x4~~ lumber. See *NDS Supplement* Table 4A – Visually Graded Dimension Lumber for reference design values. Adjustment factors C_D, C_T, C_M, and C_i are assumed to equal 1.0 in this example for calculation of adjusted design values.

$F_t' = 525 \text{ psi } (C_F) = 525(1.5) = 788 \text{ psi}$

$F_v' = 150 \text{ psi}$

Connection Details:
Bolt diameter, D: 1/2 in.
Bolt hole diameter, D_h: 0.5625 in.
Adjusted ASD bolt design value, $Z_{\parallel}'$: 550 lbs (See NDS Table 12A for 3-1/2″ main member thickness and 1-1/2″ side member thickness. For this trial design, the group action factor, C_g, is taken as 1.0).

Adjusted ASD Connection Capacity, n $Z_{\parallel}'$:

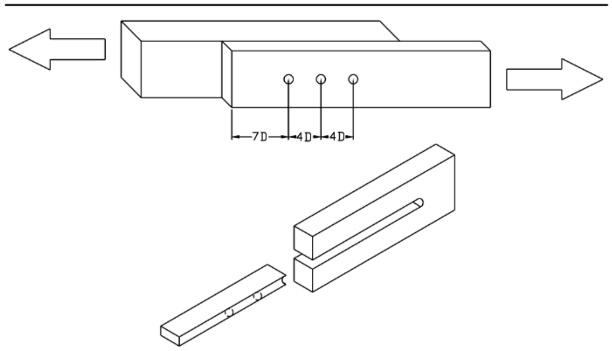
$nZ_{\parallel}' = (3 \text{ bolts})(550 \text{ lbs}) = 1{,}650 \text{ lbs}$

~~Adjusted~~ For side member, adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

$Z_{NT}' = F_t' \, t \, [w - n_{row} D_h]$

$Z_{NT}' = (788 \text{ psi})(1.5'')[3.5'' - 1(0.5625'')] = 3{,}470 \text{ lbs}$

Figure E2 Single Row of Bolts



~~Adjusted~~ For side member, adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$Z_{RTi}' = n_i F_v' \, t s_{critical}$

$Z_{RT1}' = 3(150 \text{ psi})(1.5'')(2'') = 1{,}350 \text{ lbs}$

In this sample calculation, the adjusted ASD connection capacity is limited to 1,350 pounds by row tear-out, Z_{RT}'.

March 2019

ERRATA
to the 2015 Edition of
the National Design Specification® (NDS®) for Wood Construction
(all versions)

Page	Revision
165	Revise the following calculations in Example E.8 Sample Solution of Row of Split Rings (remainder of example is unchanged):

E.8 Sample Solution of Row of Split Rings

Calculate the net section area tension and row tear-out adjusted ASD design capacities for the single-shear single- row split ring connection represented in Figure E3.

Main and Side Members:
#2 grade Southern Pine 2x4 lumber. See *NDS Supplement* Table 4B – Visually Graded Southern Pine Dimension Lumber for reference design values. Adjustment factors C_D, C_T, C_M, and C_i are assumed to equal 1.0 in this example for calculation of adjusted design values.

$F_t' = \cancel{825}\ \underline{675}$ psi

$F_v' = 175$ psi

Main member thickness, t_m: 1.5 in.
Side member thickness, t_s: 1.5 in.
Main and side member width, w: 3.5 in.

Connection Details:
Split ring diameter, D: 2.5 in. (see Appendix K for connector dimensions)
Adjusted ASD split ring design value, P′: 2,730 lbs (see Table 13.2A. For this trial design, the group action factor, C_g, is taken as 1.0).

Adjusted ASD Connection Capacity, nP′:

$nP' = $ (2 split rings)(2,730 lbs) = 5,460 lbs

Adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

$$Z_{NT}' = F_t' A_{net}$$

$$Z_{NT}' = F_t' \, [A_{2x4} - A_{bolt\text{-}hole} - A_{split\ ring\ projected\ area}]$$

$$Z_{NT}' = (\cancel{825}\ \underline{675}\text{ psi})[5.25\text{ in.}^2 - 1.5"(0.5625") - 1.1\text{ in.}^2]$$

$$= \cancel{2{,}728}\ \underline{2{,}232}\text{ lbs}$$

Adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$$Z_{RTi}' = n_i \frac{F_v' A_{critical}}{2}$$

$$Z_{RT1}' = [(2\text{ connectors})(175\text{ psi})/2](21.735\text{ in.}^2)$$

$$= 3{,}804\text{ lbs}$$

where:

$A_{critical} = 21.735$ in.^{2} (See Figures E4 and E5)

In this sample calculation, the adjusted ASD connection capacity is limited to ~~2,728~~ 2,232 pounds by net section area tension capacity, Z_{NT}'.

AMERICAN WOOD COUNCIL

May 2018

ERRATA
to the 2015 Edition of
the National Design Specification® (NDS®) for Wood Construction

Page	**Revision**
40	Revise K_{rs} as described in Equation (5.4-3) as follows (replace d_e with d_c):

K_{rs} = empirical radial stress factor

= $0.29(\cancel{d_e}\underline{d_c}/R_m) + 0.32 \tan^{1.2} \phi_T$

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AMERICAN WOOD COUNCIL

February 2018

ERRATA
to the 2015 Edition of
the National Design Specification® (NDS®) for Wood Construction

Page **Revision**

62 Revise Table 10.4.1.1 as follows:

Table 10.4.1.1 Shear Deformation Adjustment Factors, K_S

Loading	End Fixity	K_S
Uniformly Distributed	Pinned	11.5
	Fixed	57.6
Line Load at midspan	Pinned	14.4
	Fixed	57.6
Line Load at quarter points	Pinned	10.5
Constant Moment	Pinned	0
Uniformly Distributed	Cantilevered	4.8
Line Load at free-end	Cantilevered	3.6
Column Buckling	Pinned	11.8
	Fixed	~~23.7~~ 47.4

242 Revise Table C10.4.1.1 as follows:

Table C10.4.1.1 Shear Deformation Adjustment Factors

Loading	End Fixity	k_b	k_s	K_s
Uniformly Distributed	Pinned	5/384	1/8	11.5
	Fixed	1/384	1/8	57.6
Line Load at midspan	Pinned	1/48	1/4	14.4
	Fixed	1/192	1/4	57.6
Line Load at quarter points	Pinned	11/768	1/8	10.5
Constant Moment	-	1/12	0	0
Uniformly Distributed	Cantilevered	1/8	1/2	4.8
Line Load at free-end	Cantilevered	1/3	1	3.6
Column Buckling	Pinned	A	$A\pi^2$	11.8
	Fixed	~~2B~~ B	$4B\pi^2$	~~23.7~~ 47.4

February 2018

ADDENDUM
to the 2015 Edition of
the Commentary to the National Design Specification® (NDS®) for Wood Construction
(web and printed versions dated prior to 10-16)

A summary of those sections that have been updated are provided below with a brief description of the change. Corrected pages are also attached for convenience.

Chapter C3: Design Provisions and Equations

- C3.4.2 – clarification on reference design shear, V_r.

Chapter C10: Cross-Laminated Timber

- Editorial clarifications in sections C10.1.1 Application, C10.1.2 Definition, C10.1.4 Specification, and C10.2 Reference Design Values.

- The last paragraph in section C10.4.1 Deflection is expanded to address the shear deformation adjustment factor, K_S, to include the corresponding beam constants, k_b and k_s for each of the loading cases shown in NDS Table 10.4.1.1.

Chapter C12: Mechanical Connections

- In section C12.5.2 End Grain Factor, C_{eg}, new commentary (C12.5.2.3) is added.

- Editorial clarification in table headings (Tables C12.1.5.7 and C12.1.6.6).

- Editorial clarification in sections C12.2.2 Wood Screws (C12.2.1.5, C12.2.2.3, and C12.2.2.4), C12.2.3 Nails and Spikes (C12.2.3.5 and C12.2.3.6), C12.3.3 Dowel Bearing Strength (C12.3.3.4, C12.3.3.5, and C12.3.3.6), C12.3.5 Dowel Bearing Length (C12.3.5.2 and C12.3.5.3), C12.3.6 Dowel Bending Yield Strength, and C12.5.1 Geometry Factor, C_Δ (C12.5.1.4).

Chapter C16: Fire Design of Wood Members

- The term "fire resistance" replaces "fire endurance" throughout Chapter 16 Commentary.

- Editorial clarification in section C16.1 General (Beams and Columns, Flame Spread, and Fire Retardant Treatments sections).

- The definition for "t_{gi}" is revised to clarify that it is the time for the *char front* to reach the glued interface for each lamination.

- The last paragraph in section C16.2.4 Special Provisions for Structural Glued Laminated Timber Beams is expanded and includes two new figures, Figures C16A and C16B, which illustrate the core, inner and outer tension, and inner and outer compression laminations for unbalanced and balanced layups respectively.

Commentary References

- Several references are updated (2, 125, 126, 129, and 140) and a new reference is added (198).

lins that are adequately attached to the compression edge, the unsupported length, ℓ_u, of the bending member is the distance between purlins (61). The bending member must also be braced at points of bearing. Consistent with the intent of NDS 3.3.3.4, the nonmandatory phrase "and/or lateral displacement" was removed and does not appear in 2012 and later editions of the Specification because the requirement to "prevent rotation" at points of bearing is not optional and prevention of lateral displacement does not necessarily prevent rotation.

C3.3.3.5 Formulas are provided for determining the effective span length, ℓ_e, from the unsupported length, ℓ_u, for different loading and support conditions (138). The ℓ_e values for small span-to-depth ratios, $\ell_u/d < 7$ are limited to address unrealistically large ℓ_e values that otherwise would be calculated for these short, deep bending members (60).

The constants in the formulas for effective length in NDS Table 3.3.3 include a 15 percent increase in ℓ_u to account for the possibility of imperfect torsional restraint at lateral supports. The formulas given in the table are applicable where loads are applied to the compression edge of the bending member, the most conservative loading case. Formulas given in the footnote for load conditions not covered by the formulas in the body of the table represent the most limiting formula for the ℓ_u/d range from those given for specified load conditions. For more information on the derivation of these formulas, see *TR14* (138).

C3.3.3.6 The beam slenderness ratio, R_B, is comparable to the slenderness ratio for solid columns, ℓ_e/d, in terms of its effect on bending member design strength.

C3.3.3.7 Limiting the beam slenderness ratio, R_B, to a maximum value of 50 is a good practice recommendation intended to preclude design of bending members with high buckling potential. This limit parallels the limit on slenderness ratio for columns, ℓ_e/d (60).

C3.3.3.8 The beam stability factor equation is applicable to all beam slenderness ratios (R_B). This equation provides a means of combining the bending design stress (F_b^*) with the critical buckling design stress (F_{bE}) to estimate an "effective" bending design value.

C3.3.3.10 See C3.9.2 on biaxial bending.

C3.4 Bending Members – Shear

C3.4.1 Strength in Shear Parallel to Grain (Horizontal Shear)

C3.4.1.1 Shear strength perpendicular to the grain, also referred to as cross-grain or vertical shear, refers to shear stresses in the radial-tangential plane tending to cut the wood fibers perpendicular to their long axis. The strength of wood in this plane is very high relative to shear strength parallel to grain, or horizontal shear, which refers to shear stresses in the longitudinal-radial or longitudinal-tangential plane tending to slide one fiber past another along their long axes. As both parallel and perpendicular to grain shear occur simultaneously, parallel to grain shear strength is always the limiting case. Therefore, reference shear design values, F_v, are horizontal or parallel to grain shear stresses.

Shear in the tangential-longitudinal or radial-longitudinal plane tending to roll one fiber over another perpendicular to their long axes is termed rolling shear. Rolling shear, which occurs in structural plywood applications as shear in the plane of the plies, is not a design consideration in most lumber or timber product applications.

C3.4.1.2 Shear design provisions in NDS 3.4 are limited to solid flexural members such as sawn lumber, structural glued laminated timber, structural composite lumber, and mechanically laminated timber. Built-up components, such as trusses, are specifically excluded because of field experience that indicated the procedures might not be adequate for shear design of top-hung parallel chord trusses and similar components that contained load-bearing web and top chord connections near points of support. Shear design of built-up components is required to be based on testing, theoretical analysis, and/or documented experience due to the complexity of determining the effects of stress concentrations, the influence of embedded metal connectors, and questions regarding the applicability of the general practice of ignoring loads close to supports.

C3.4.2 Shear Design Equations

Actual shear stress parallel to grain, f_v, in a circular bending member may be determined as:

$$f_v = 4V / 3A \qquad \text{(C3.4.2-1)}$$

where:

V = shear force, lbs

A = cross-sectional area of circular member, in.

For cross-laminated timber, the reference design shear, V_r, is provided by the CLT manufacturer and is designated in NDS Chapter 10 as $F_s(Ib/Q)_{eff}$.

C10 CROSS-LAMINATED TIMBER

C10.1 General

C10.1.1 Application

The general requirements given in Chapters 1, 2, and 3 of the Specification are applicable to cross-laminated timber (CLT) except where indicated otherwise. Chapter 10 of the Specification contains provisions that specifically apply to CLT manufactured in accordance with APA PRG 320 (197). The provisions of NDS Chapter 10 contain only the basic requirements applicable to engineering design of CLT. Specific requirements, such as CLT design values and the wet service factor are available from the CLT manufacturer.

C10.1.2 Definitions

The definition for cross-laminated timber is based on the definition in APA PRG 320 (197).

C10.1.3 Standard Dimensions

10.1.3.1 Lamination thickness refers to the narrow face of a lamination perpendicular to the lamination length (face perpendicular to the glueline). Minimum and maximum thickness of 5/8 in. and 2 in., respectively, are based on APA PRG 320.

10.1.3.2 CLT panel thickness is measured perpendicular to the plane of the panel and is limited to 20 in. in accordance with APA PRG-320. CLT panel length is measured parallel to the major strength direction and CLT panel width is measured perpendicular to the major strength direction.

C10.1.4 Specification

The specific manufacturer's CLT product should be specified, including standard grade, where used, or the CLT configuration based on lamination grades, thicknesses, and layup (See C10.2). Standard grades of CLT consisting of specific lamination grades, thicknesses, and layups are provided in Annex A of APA PRG-320.

C10.1.5 Service Conditions

CLT design values are based on dry service conditions (moisture content in service less than 16%). For other conditions, the manufacturer should be consulted.

C10.2 Reference Design Values

Reference design values for specific grades and layups of CLT are provided in APA PRG-320. CLT design capacities are a function of the manufacturer's CLT layup and properties associated with the lamination grades. The user should contact the CLT manufacturer for design values and section properties for specific CLT products.

C10.4 Special Design Considerations

C10.4.1 Deflection

C10.4.1.1 When cross-laminated panels are loaded in out-of-plane bending, the shear deformation can be a significant portion of the total deformation. The provisions of NDS 10.4.1 provide a method of calculating the "apparent" stiffness, $(EI)_{app}$, from the properties provided in PRG 320 and from the CLT manufacturer.

Where effective bending stiffness values (EI_{eff}) and effective shear stiffness values (GA_{eff}) are provided by the CLT manufacturer, the apparent bending stiffness can be approximated as:

$$(EI)_{app} = \frac{EI_{eff}}{1+\dfrac{K_s EI_{eff}}{GA_{eff} L^2}} \qquad \text{(C10.4.1-1)}$$

Where only effective bending stiffness values (EI_{eff}) are provided, the apparent bending stiffness can be approximated using NDS Equation 10.4-1 where I_{eff} and A_{eff} are provided by the CLT manufacturer. Design values for strong axis and weak axis modulus of elasticity, E, are provided in PRG 320. PRG 320 assumes the modulus of rigidity, G, equals E/16. The value for I_{eff} and A_{eff} can be calculated as:

$$I_{eff} = EI_{eff} / E \qquad \text{(C10.4.1-2)}$$

$$A_{eff} = 16\ GA_{eff} / E \qquad \text{(C10.4.1-3)}$$

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

$$(EI)_{app\text{-}min} = (EI)_{app}\,(1\text{-}1.645(0.10))(1.03)/1.66 = 0.518(EI)_{app}$$

Shear deformation adjustment factors, K_s, provided in NDS Table 10.4.1.1 and Table C10.4.1.1 are based on relationships for the specified loading and end-fixity conditions from the Wood Handbook [183]. The value of K_s is derived when the beam deformation and shear deformation equations are combined for each load case to provide an adjustment that provides an "apparent" EI_{eff} value. The beam deflection, δ, for rectangular members is estimated as:

$$\delta = \frac{k_b W L^3}{EI_{eff}} + \frac{6 k_s W L}{5 GA_{eff}} \qquad \text{(C10.4.1-4)}$$

Where,

δ = beam deflection

k_b, k_s = beam constants based on beam loading, support conditions, and measurement location

W = total load on the beam

Setting the beam deformation equations equal to the bending deformation equation assuming an "apparent" stiffness, EI_{app} yields:

$$\frac{k_b W L^3}{EI_{app}} = \frac{k_b W L^3}{EI_{eff}} + \frac{6 k_s W L}{5 GA_{eff}} \qquad \text{(C10.4.1-5)}$$

Solving for EI_{app}:

$$EI_{app} = \frac{EI_{eff}}{1+\dfrac{6 k_s EI_{eff}}{5 k_b GA_{eff} L^2}} = \frac{EI_{eff}}{1+\dfrac{K_s EI_{eff}}{GA_{eff} L^2}} \qquad \text{(C10.4.1-6)}$$

From this derivation, it can be seen that $K_s = 6k_s/(5k_b)$.

For the case of Column Buckling Moment, the derivation is more complex. For a column pinned at each end, the bending deformation is estimated to be $\delta_b = A \sin(\pi x)$ and the shear deformation is estimated to be $\delta_s = A\pi^2 \sin(\pi x)$ and at midheight simplifies to $\delta_b = A$ and $\delta_s = A\pi^2$. For a column fixed at each end, the bending deformation is estimated to be $\delta_b = B$ and the shear deformation is estimated to be $\delta_s = 4B\pi^2$.

C

COMMENTARY: CROSS-LAMINATED TIMBER

Table C10.4.1.1 Shear Deformation Adjustment Factors

Loading	End Fixity	k_b	k_s	K_s
Uniformly Distributed	Pinned	5/384	1/8	11.5
	Fixed	1/384	1/8	57.6
Line Load at midspan	Pinned	1/48	1/4	14.4
	Fixed	1/192	1/4	57.6
Line Load at quarter points	Pinned	11/768	1/8	10.5
Constant Moment	-	1/12	0	0
Uniformly Distributed	Cantilevered	1/8	1/2	4.8
Line Load at free-end	Cantilevered	1/3	1	3.6
Column Buckling	Pinned	A	$A\pi^2$	11.8
	Fixed	2B	$4B\pi^2$	23.7

Cross-laminated timber panels are designed as one-way slabs using a beam analogy. Terminology in NDS Table 10.4.1.1 and Table C10.4.1.1 has been modified from typical beam terminology to address the panel width. For example, a point load at midspan of a beam is called a "line load at midspan" to indicate that the load is assumed to be applied across the panel (perpendicular to the span) at the midspan of the cross-laminated timber. Additional loading and end-fixity conditions are available in the literature.

C12.1.4.7 Edge distances, end distances, and fastener spacing requirements have been consolidated for dowel-type fasteners in NDS 12.5.

C12.1.5 Wood Screws

C12.1.5.1 ANSI/ASME Standard B18.6.1 is the quality reference standard for wood screws. It provides standard wood screw dimensions (see NDS Appendix L) but does not specify metal having specific strength properties. The designer is responsible for specifying the metal strength of the wood screws that are to be used. Bending yield strength of the wood screw (see NDS Appendix I) is a required input variable to the lateral design value yield limit equations of NDS 12.3.1. Additionally, the actual tensile stress in the wood screw at the root diameter must be checked when designing wood screw connections for withdrawal (see C11.2.3 of Specification).

C12.1.5.2 Lead hole requirements for wood screws are based on early research involving flat head wood screws up to 24 gage and 5 inches in length in seven species, including southern pine, cypress, and oak (43).

The provision allowing the insertion of wood screws without a lead hole in species with $G \leq 0.5$ when the screw is subject to withdrawal loads parallels the provision for 3/8 inch and smaller diameter lag screws (see C12.1.4.3).

C12.1.5.3 Wood screws resisting lateral loads are required to have shank and threaded portion lead holes based on early lateral load tests of wood screws (70, 181, 184). Lead holes are required for all wood screws subject to lateral loads regardless of wood specific gravity.

C12.1.5.4 Wood screws tests (43, 70, 181) are based on inserting the screw by turning rather than driving with a hammer.

C12.1.5.5 A lubricant is sometimes used to facilitate screw insertion and avoid screw damage. Tests have shown that the lubricant has no significant effect on reference design values (43, 70, 184).

C12.1.5.6 Minimum length of wood screw penetration requirements, including the length of the tapered tip, are provided to ensure that fasteners can achieve the reference design value calculated using the yield equations in NDS 12.3.1.

C12.1.5.7 Edge distances, end distances, and fastener spacing requirements have been consolidated across all diameters for dowel-type fasteners in NDS 12.5. For diameters less than ¼", specific requirements are not provided; however, Table C12.1.5.7 may be used to establish wood screw placement recommendations. Designers should note that wood species, moisture content, and grain orientation will impact spacing effects between fasteners in a row.

Table C12.1.5.7 Recommended Minimum Spacing for Wood Screws

	Wood Side Members	
	Not Prebored	Prebored
Edge distance	2.5D	2.5D
End distance		
- tension load parallel to grain	15D	10D
- compression load parallel to grain	10D	5D
Spacing between fasteners in a row		
- parallel to grain	15D	10D
- perpendicular to grain	10D	5D
Spacing between rows of fasteners		
- in-line	5D	3D
- staggered	2.5D	2.5D
	Steel Side Members	
	Not Prebored	Prebored
Edge distance	2.5D	2.5D
End distance		
- tension load parallel to grain	10D	5D
- compression load parallel to grain	5D	3D
Spacing between fasteners in a row		
- parallel to grain	10D	5D
- perpendicular to grain	5D	2.5D
Spacing between rows of fasteners		
- in line	3D	2.5D
- staggered	2.5D	2.5D

C12.1.6 Nails and Spikes

C12.1.6.1 ASTM F 1667 *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples* provides standard nail, spike, threaded hardened-steel nail, and post-frame ring shank nail dimensions (see NDS Appendix L) but does not specify metal of particular strength properties for these driven fasteners. The designer is responsible for specifying the metal strength of the driven fasteners that are to be used. Bending yield strength of the driven fastener (see NDS Appendix I) is a required input variable to the lateral design value yield limit equations of NDS 12.3.1. Additionally, the actual tensile stress in the driven fastener must be checked when designing driven fastener connections for withdrawal (see C11.2.3 of Specification).

C12.1.6.4 Toe-nailing procedures consisting of slant driving of nails at a 30° angle from the face of the attached member with an end distance (distance between end of side member and initial point of entry) of one-third the nail length are based on lateral and withdrawal tests of nailed joints in frame wall construction (181, 118). The toenail factors of NDS 12.5.4.1 and NDS 12.5.4.2 presume use of these driving procedures and the absence of excessive splitting. If such splitting does occur, predrilling or a smaller nail should be used. The vertically-projected length is used as the side member bearing length in yield limit equations when calculating lateral capacity of a toe-nailed connection.

C12.1.6.5 Minimum length of penetration requirements, including the length of the tapered tip, are provided to ensure that driven fasteners can achieve the design value calculated using the yield equations in NDS 12.3.1. The exception for clinching in double-shear connections is applicable to 0.148" (12d common, 20d box, or 16d sinker nails) or smaller diameter nails.

C12.1.6.6 Edge distances, end distances, and fastener spacing requirements have been consolidated across all diameters for dowel-type fasteners in NDS Table 12.5.1A through 12.5.1F. For diameters less than ¼", specific requirements are not provided; however, Table C12.1.6.6 may be used to establish nail placement recommendations. Designers should note that wood species, moisture content, and grain orientation will impact spacing effects between fasteners in a row.

C12.1.7 Drift Bolts and Drift Pins

C12.1.7.1 Drift bolts and drift pins are unthreaded rods used to join large structural members where a smooth surface without protruding metal parts is desired. The designer is responsible for specifying the metal strength of the drift bolt or pin that is to be used. Bending yield strength of the drift bolt or pin (see NDS Appendix I) is a required input variable to the reference lateral design value yield limit equations of NDS 12.3.1.

C12.1.7.2 Additional penetration into the members is required to resist withdrawal of the drift bolt or pin.

C12.1.7.3 Edge distances, end distances, and fastener spacing requirements have been consolidated across all diameters for dowel-type fasteners in NDS Table 12.5.1A through 12.5.1F.

Table C12.1.6.6 Recommended Minimum Spacing for Nails

	Wood Side Members	
	Not Prebored	Prebored
Edge distance	2.5D	2.5D
End distance		
- tension load parallel to grain	15D	10D
- compression load parallel to grain	10D	5D
Spacing between fasteners in a row		
- parallel to grain	15D	10D
- perpendicular to grain	10D	5D
Spacing between rows of fasteners		
- in-line	5D	3D
- staggered	2.5D	2.5D
	Steel Side Members	
	Not Prebored	Prebored
Edge distance	2.5D	2.5D
End distance		
- tension load parallel to grain	10D	5D
- compression load parallel to grain	5D	3D
Spacing between fasteners in a row		
- parallel to grain	10D	5D
- perpendicular to grain	5D	2.5D
Spacing between rows of fasteners		
- in line	3D	2.5D
- staggered	2.5D	2.5D

C12.1.8 Other Dowel-Type Fasteners

While specific installation instructions are not provided for all types of dowel-type fasteners, the provisions for withdrawal in NDS 12.2 and the generic yield equations in NDS 12.3.1 for lateral design apply. The designer is responsible for determining the proper installation requirements and for specifying the metal strength of these fasteners.

C12.2 Reference Withdrawal Design Values

C12.2.1 Lag Screws

C12.2.1.1 NDS Equation 12.2-1 was used to establish the lag screw reference withdrawal design values given in NDS Table 12.2A. This equation was derived from the following equation based on research (181, 100):

$$W = K_W G^{3/2} D^{3/4} \qquad (C12.2.1\text{-}1)$$

where:

W = reference withdrawal design value per inch of thread penetration into main member, lbs

K_W = 1800

G = specific gravity of main member based on oven dry weight and volume, where $0.31 \le G \le 0.73$

D = lag screw diameter (equivalent to unthreaded shank diameter for full body diameter lag screws), in., where 0.25" ≤ D ≤ 1.25"

The value of K_W represents approximately one-fourth (one-fifth increased by 20 percent) of the average constant at oven dry weight and volume obtained from ultimate load tests of joints made with five different species and seven sizes of lag screw (100), increased by 20 percent; or

$$K_W = 1.2\left(\frac{7500}{5}\right) \tag{C12.2.1-2}$$

The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2).

When the reference withdrawal capacity of a lag screw is determined by multiplying the reference unit design value by the length of penetration of the threaded portion into the side grain of the main member, the length of the tapered tip of the screw is not to be included. This tapered portion at the tip of the lag screw was not considered as part of the effective penetration depth in the original joint tests (100) and in the development of equation C12.2.1-1. In addition, the thickness of any washer used between the lag screw head and the cleat or side member should be taken into account when determining the length of penetration of the threaded portion in the main member. Standard lag screw dimensions, including minimum thread length and length of tapered tip, are given in Appendix L of the Specification.

C12.2.1.2 The unit reference withdrawal design value in lbs/in. is multiplied by the depth of thread penetration into a wood member to calculate the fastener reference withdrawal in pounds.

C12.2.1.3 Reference withdrawal design values for lag screws are reduced 25 percent when the screw is inserted in the end grain (radial-tangential plane) of the main member rather than the side grain (radial-longitudinal or tangential-longitudinal plane) based on lag screw joint tests (100). Because of the greater possibility of splitting when subject to lateral load, it has been recommended that insertion of lag screws in end grain surfaces be avoided (181, 96).

C12.2.1.4 (see C11.2.3).

C12.2.1.5 The required use of the end grain factor of 0.75 for lag screws installed into the narrow edge of CLT panels conservatively assumes the lag screw will be subject to strength reductions associated with installation in end grain. This assumption was judged practical to address varying grain orientations in the edge of cross-laminated timber panels (e.g. both end grain and side grain are present in the edge of CLT panels), and the ability to maintain minimum edge distances for larger diameter lag screws installed in the narrow face of a lamination. For cases where the narrow face of the laminations is large, such as 2 in., and the lag screw diameter is small such as ¼" and where installation is in side grain only with adequate edge distance, application of the 0.75 factor may not be warranted where strength reducing conditions associated with placement in end grain or with inadequate edge distances for side grain are not present.

C12.2.2 Wood Screws

C12.2.2.1 NDS Equation 12.2-2 was used to establish the wood screw reference withdrawal design values given in NDS Table 12.2B. This equation was based on testing of cut thread wood screws in seven wood species (43):

$$W = K_W\, G^2\, D \tag{C12.2.2-1}$$

where:

W = reference withdrawal design value per inch of thread penetration in the main member, lbs

K_W = 2850

G = specific gravity of main member based on oven dry weight and volume, where 0.31 ≤ G ≤ 0.73

D = wood screw thread diameter, in., where 0.138" ≤ D ≤ 0.372"

The value of K_W represents one-fifth (one-sixth increased by 20 percent) of the average constant at oven dry weight and volume obtained from ultimate load tests of joints (43) made with seven different species and cut-thread wood screw; or

$$K_W = \frac{14250}{6} \tag{C12.2.2-2}$$

The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2).

Wood screw reference withdrawal design values are based on tests of cut thread wood screws. The shank or body diameter of a cut thread screw is the same as the outside diameter of the thread. The shank or body diameter of the rolled thread screw is the same as the root diameter. For the same nominal diameter of screw, both screw thread types have the same threads per inch, the same outside thread diameter, and the same thread depth. If the tensile strength of the screw is adequate and the lead hole provisions based on root diameter are used, the withdrawal resistance of rolled thread screws is considered equivalent to that of cut thread screws (182, 163).

The ANSI/ASME B18.6.1 standard states that the thread length is approximately two-thirds of the nominal screw length.

C12.2.2.2 The unit reference withdrawal design value in lbs/in. is multiplied by the depth of thread penetration into a wood member to calculate the fastener reference withdrawal in pounds.

C12.2.2.3 Early tests of wood screws in withdrawal from end grain surfaces of oak, southern pine, maple, and cypress gave somewhat erratic results relative to those for withdrawal from side grain (43). These irregular results were attributed to the tendency of the screw to split the wood in the end grain configuration. Average ratios of end grain withdrawal resistance to side grain withdrawal resistance ranged from 52 to 108 percent (43). Because of this variability, structural loading of wood screws in withdrawal from end grain has been prohibited. Wood screws installed in end grain are suitable for lateral resistance; however, to clarify that screws are permitted to be installed in end-grain but should not be assigned withdrawal design values, the end-grain adjustment factor, C_{eg}, is set to zero for withdrawal loading.

C12.2.2.4 Similar to the provisions of 12.2.2.3, wood screws installed in end-grain of cross-laminated timber laminations should not be assigned withdrawal design values (i.e. C_{eg}=0.0).). There is no reduction in withdrawal resistance for wood screws installed in the side grain of laminations at cross-laminated timber panel edges.

C12.2.2.5 See C11.2.3.

C12.2.3 Nails and Spikes

C12.2.3.1 NDS Equation 12.2-3 was used to establish the nail and spike reference withdrawal design values given in NDS Table 12.2C. This equation was based on research (94, 95):

$$W = K_W G^{5/2} D \tag{C12.2.3-1}$$

where:

W = nail or spike withdrawal design value per inch of penetration in main member, lbs

K_W = 1380

G = specific gravity of main member based on oven dry weight and volume, where $0.31 \leq G \leq 0.73$

D = shank diameter of the nail or spike, in., where $0.099" \leq D \leq 0.375"$

The value of K_W represents one-fifth (one-sixth increased by 20 percent) of the average constant at oven dry weight and volume obtained from ultimate load tests (184), increased by 20 percent; or

$$K_W = 1.2\left(\frac{6900}{6}\right) \tag{C12.2.3-2}$$

The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2).

For 8d, 10d, 16d, and 20d threaded hardened nails, reference withdrawal design values are the same as those for common wire nails of the same pennyweight class, although the wire diameters are slightly different (0.120, 0.135, 0.148, and 0.177 inches for threaded hardened nails versus 0.131, 0.148, 0.162, and 0.192 inches for common nails, respectively). Threaded hardened nail sizes of 20d, 30d, 40d, 50d, and 60d all have the same diameter (0.177 inches) and, therefore, use the same reference withdrawal design value. Threaded hardened nail sizes of 70d, 80d, and 90d all have the same diameter (0.207 inches) and use the same reference withdrawal design value as a 40d common nail.

Clinching. Withdrawal resistance of smooth-shank nails can be significantly increased by clinching (29).

C12.2.3.2 The unit reference withdrawal design value in lbs/in. for nails and spikes is multiplied by the depth of thread penetration into a wood member to calculate the fastener reference withdrawal in pounds.

C12.2.3.3 In the 2012 edition of the Specification, provisions were added for post-frame ring shank nails in accordance with ASTM F1667. The withdrawal design value equation (NDS Equation 12.2-4) is based on research conducted at the Forest Products Laboratory. The constant of 1800 incorporates a 20% reduction to account for effects of galvanized coatings for Southern pine rather than the average reduction of 15% from all species (basswood, SPF, Douglas fir, southern pine, and white oak) tested in the study (196).

C12.2.3.4 The unit reference withdrawal design value in lbs/in. for post-frame ring shank nails is multiplied by the depth of thread penetration into the side grain of a wood member to calculate the fastener reference withdrawal in pounds.

C12.2.3.5 Reduction of withdrawal design values up to 50 percent have been reported for nails driven in end grain surfaces (radial-tangential plane) as compared to

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side grain (radial-longitudinal or tangential-longitudinal planes) surfaces (184, 118). When coupled with the effects of seasoning in-service after fabrication, such reductions are considered too great for reliable design. On this basis, structural loading of nails in withdrawal from end grain has been prohibited. Nails installed in end grain are suitable for lateral resistance; however, to clarify that nails are permitted to be installed in end-grain but should not be assigned withdrawal design values, the end-grain adjustment factor, C_{eg}, is set to zero for withdrawal loading.

C12.2.3.6 Similar to the provisions of 12.2.3.5, nails installed in end-grain of cross-laminated timber laminations should not be assigned withdrawal design values (i.e. C_{eg}=0.0). There is no reduction in withdrawal resistance for nails installed in the side grain of laminations at cross-laminated timber panel edges.

C12.2.4 Drift Bolts and Drift Pins

C12.2.4.1 While specific provisions for determining withdrawal design values for round drift bolts or pins are not specifically included in the Specification, the following equation has been used where friction and workmanship can be maintained (184, 181):

$$W = 1200\ G^2\ D \qquad \text{(C12.2.4-1)}$$

where:

W = drift bolt or drift pin reference withdrawal design value per inch of penetration, lbs

G = specific gravity based on oven dry weight and volume

D = drift bolt or drift pin diameter, in.

Equation C12.2.4-1 assumes the fastener is driven into a prebored hole having a diameter 1/8 inch less than the fastener diameter (184). The reference withdrawal design values calculated with Equation C12.2.4-1 are approximately one-fifth average ultimate test values (184, 181).

C12.3 Reference Lateral Design Values

Reference lateral design values for dowel-type fasteners (bolts, lag screws, wood screws, nails and spikes) are based on a yield limit model which specifically accounts for the different ways these connections can behave under load. These behavior patterns or modes (see NDS Appendix I) are uniform bearing in the wood under the fastener, rotation of the fastener in the joint without bending, and development of one or more plastic hinges in the fastener (67, 122). Equations have been developed for each mode relating the joint load to the maximum stresses in the wood members and in the fastener (67, 121). The capacity of the connection under each yield mode is keyed to the bearing strength of the wood under the fastener and the bending strength of the fastener, with the lowest capacity calculated for the various modes being taken as the reference design value for the connection.

Although the yield limit model represents significantly different methodology than that used previously to establish fastener design values, the relative effects of various joint variables shown by both procedures are generally similar (85, 86, 89, 121). Short-term design values obtained from application of the yield limit equations have been reduced to the design value levels published in previous editions of the Specification for connections made with the same species and member sizes.

Bolts: Reference lateral design values for bolted connections are indexed to proportional limit estimates from bolted connection tests (44, 57, 146, 162) at reference conditions (seasoned dry, normal load duration).

Lag Screws: Reference lateral design values for lag screw connections are indexed to average short-term proportional limit test values (100) divided by 1.875. The 1.875 factor is based on an original reduction factor of 2.25, increased 20 percent for normal loading and experience. The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2).

Wood Screws: Reference lateral design values for wood screw connections are indexed to average short-term proportional limit test values (184, 70) divided by 1.33. The 1.33 factor is based on an original reduction factor of 1.6, increased 20 percent for normal loading and experience. The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2). Lateral design values for wood screw connections at reference conditions (seasoned dry, normal load duration) are about one-fifth of maximum tested capacities (184).

Nails & Spikes: Reference lateral design values for nailed connections are indexed to average short-term proportional limit test values (184, 50) divided by 1.33. The 1.33 factor is based on an original reduction factor of 1.6, increased 20 percent for normal loading and experience. The twenty percent increase was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see C2.3.2). Lateral design values for nailed connections at reference conditions (seasoned dry, normal load duration) are about one-fifth of maximum tested capacities for softwoods and one-ninth of maximum tested capacities for hardwoods (184, 50).

C12.3.1 Yield Limit Equations

The yield limit equations for single shear connections (NDS Equations 12.3-1 to 12.3-6) and for double shear connections (NDS Equations 12.3-7 to 12.3-10) were developed from European research (121, 78) and have been confirmed by tests of connections made with domestic species (21, 20, 88, 120, 121, 122). The limiting yield modes covered by these equations are bearing in the main or side members (Mode I), fastener rotation without bending (Mode II), development of a plastic hinge in the fastener in main or side member (Mode III) and development of plastic hinges in the fastener in both main and side members (Mode IV) (see NDS Appendix I).

The reduction term, R_d, in NDS Equations 12.3-1 through 12.3-10, reduces the values calculated using the yield limit equations to approximate estimates of the nominal proportional limit design values in previous editions of the Specification (157). For fasteners loaded perpendicular to grain with diameters equal to or greater than 0.25 inches, the reduction term is increased 25% (K_θ = 1.25) to match previous design values for connections loaded perpendicular to grain.

For detailed technical information on lateral design equations, see *AWC's Technical Report 12: General Dowel Equations for Calculating Lateral Connection Values* (137) available at www.awc.org.

C12.3.2 Common Connection Conditions

Reference lateral design values, Z, for connections with bolts, lag screws, wood screws, nails and spikes are calculated for common connection conditions and assumed fastener bending yield strengths using the yield limit equations in NDS 12.3.1. Assumptions used in the yield limit equations to develop the tables are provided in the table headings and footnotes.

C12.3.3 Dowel Bearing Strength

C12.3.3.1 The limiting wood stresses used in the yield limit equations are based on the load at which the load-deformation curve from a fastener embedment test intersects a line represented by the initial tangent modulus offset 5 percent of the fastener diameter (120). This nominal yield point is intermediate between the proportional limit and maximum loads for the material.

The effect of specific gravity on dowel bearing strength was established from 3/4-inch dowel embedment tests on Douglas fir, southern pine, spruce-pine-fir, sitka spruce, red oak, yellow poplar, and aspen. Diameter effects were evaluated from tests of 1/4, 1/2, 3/4, 1, and 1-1/2 inch dowels in southern pine using bolt holes 1/16-inch larger than the dowel diameter. Diameter was found to be a significant variable only in perpendicular to grain loading. Bearing specimens were 1/2-inch or thicker such that width and number of growth rings did not influence results (158).

The specific gravity values given in NDS Table 12.3.3A for each specie or species group are those used to establish dowel bearing strength values, F_e, tabulated in NDS Table 12.3.3. These specific gravity values represent average values from in-grade lumber test programs or are based on information from ASTM D 2555.

The equations provided in footnote 2 of NDS Table 12.3.3 were used to calculate tabulated values in NDS Table 12.3.3. These equations were derived from test data using methods described in ASTM D 5764 (158, 18).

C12.3.3.2 Dowel bearing strengths for wood structural panels using a dowel diameter of less than or equal to ¼ inch are provided in NDS Table 12.3.3B and are based on research conducted by APA-The Engineered Wood Association (25). Dowel bearing values for larger diameters in wood structural panels are available in APA 825E.

C12.3.3.3 Dowel bearing strengths for structural composite lumber are determined for each product using equivalency methods described in ASTM D5456 (16).

C12.3.3.4 For fasteners with D≥1/4" installed in end-grain, the dowel bearing strength is equal to the perpendicular to grain value, $F_{e\perp}$.

C12.3.3.5 For fasteners in the wide face of CLT, dowel bearing strength is based on the dowel bearing strength of the layer at the shear plane. The orientation of the layer at the shear plane may either be parallel or perpendicular to the major strength axis of the panel and should be part of the specification of the cross-laminated timber panel (see Commentary C10.1.4). For connections where the load-

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ing direction is parallel to grain for the layer at the shear plane, the dowel bearing strength is the parallel to grain dowel bearing strength, $F_{e\|}$. For connections where the loading direction is perpendicular to grain for the layer at the shear plane, the dowel bearing strength is the perpendicular to grain dowel bearing strength, $F_{e\perp}$. The influence of different dowel bearing strengths of crossing layers on cross-laminated connection design values is accounted for by adjustment of the bearing length in the crossing layers (See NDS 12.3.5.2).

C12.3.3.6 For fasteners with D≥1/4" that are installed into the edge of cross-laminated timber, the dowel bearing strength is assumed to be the same as for fasteners installed into end-grain (See NDS 12.3.3.4) which conservatively addresses varying grain orientations and the ability to maintain minimum edge distances within the narrow face of a cross-laminated timber lamination. For fasteners with D<1/4", the same dowel bearing strength, F_e, applies for either parallel or perpendicular to grain loading.

C12.3.4 Dowel Bearing Strength at an Angle to Grain

NDS Equation 12.3-11 (and Equation J-2 in NDS Appendix J) is used to calculate the dowel bearing strength for a main or side member loaded at an angle to grain. This equation is a form of the bearing angle to grain equation (NDS Equation J-1). The equation is entered with the parallel and perpendicular dowel bearing strengths for the member and the reference bolt design value is determined from the yield limit equations using $F_{e\theta}$ as the dowel bearing strength for the main or side member.

The reference design value obtained from the yield limit equations using dowel bearing strength at an angle to grain is similar to that obtained from using parallel to grain and perpendicular to grain *Z* values in NDS Equation J-3 to obtain a Z_θ design value for the connection (157). Determining a Z_θ design value using this latter approach can be used as an alternative to calculating $F_{e\theta}$ for use in each yield limit equation and allows the use of tabulated *Z* values from the Specification.

C12.3.5 Dowel Bearing Length

C12.3.5.2 For fasteners with D≥1/4", crossing layers in cross-laminated timbers will have different dowel bearing strengths than the layer at the shear plane due to the difference in grain orientation. The influence of varying dowel bearing strengths in crossing layers on connection design values is addressed by use of an “effective” bearing length. For connections where the loading direction is parallel to grain for the layer at the shear plane, the dowel bearing length should be reduced by multiplying the bearing length in each crossing layer (perpendicular to grain) by the ratio of $F_{e\perp}/F_{e\|}$. For connections where the loading direction is perpendicular to grain for the layer at the shear plane, the dowel bearing length can conservatively remain unadjusted or it can be increased in the crossing layers (parallel to grain) by the ratio of $F_{e\|}/F_{e\perp}$. Actual penetration lengths should be used for checking minimum penetration requirements. For connections loaded at an angle to grain, the procedures in NDS Appendix J for developing design values based on parallel and perpendicular to grain design values should be used with these “effective” bearing lengths. Methods of installation should avoid placing fasteners in gaps between adjacent boards in a lamination, especially where they might occur in the lamination at the shear plane.

C12.3.5.3 An analysis provided in Technical Report 12 (137) shows that the NDS requirement closely approximates results from the more detailed evaluation of the influence of a tapered tip on bearing resistance. For wood screws, nails and spikes, the length of the tapered tip is not generally standardized, but for purposes of accounting for the tip length in the bearing length calculation, E, is permitted to be taken as 2 diameters (2D). For lag screws, E is permitted to be taken from NDS Appendix L, Table L2.

C12.3.6 Dowel Bending Yield Strength

The dowel bending yield strength, F_{yb}, of fasteners such as nails (79), wood screws, lag screws, and bolts are given in NDS Appendix I. For most steel fasteners, F_{yb} equal to 45,000 psi is a conservative value and is equivalent to the bolt strength reported in the original bolt test research (146).

C12.3.7 Dowel Diameter

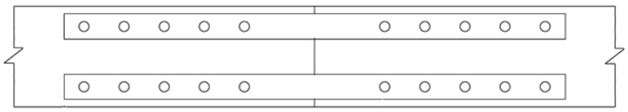
The reduced moment resistance in the threaded portion of dowel-type fasteners can be accounted for by use of root diameter, D_r, in calculation of reference lateral design values. Use of diameter, D, is permitted when the threaded portion of the fastener is sufficiently far away from the connection shear plane(s). For more information, see NDS Appendix I.5.

Reference lateral design values for reduced body diameter lag screw and rolled thread wood screw connections are based on root diameter, D_r, to account for the reduced diameter of these fasteners. These values, while conservative, can also be used for full-body diameter lag screws and cut thread wood screws. For bolted connections, reference lateral design values are based on diameter, D.

recognition of the combined effect of bolt hole oversizing and alignment resulting in 1/32" movement.

Special detailing can be utilized in cases where distances between outer rows of bolts exceed the limits in Table 12.5.1F, such as use of multiple splice plates or a single splice plate with slotted holes to allow shrinkage. Such an example of multiple splice plates is shown in Figure C12.5.1.3.

Figure C12.5.1.3 Connection Illustrating Use of Multiple Splice Plates



C12.5.1.4 For fasteners installed in the edge of cross-laminated timber panels, special end distance, edge distance and fastener spacing conditions are provided in NDS Table 12.5.1G, while all other requirements follow the general provisions of NDS 12.5.1. For fasteners installed in the wide face of cross-laminated panels, end distances, edge distances, and fastener spacing requirements should follow the requirements for other wood products in NDS 12.5.1. Placement of fasteners in gaps should be avoided.

C12.5.2 End Grain Factor, C_{eg}

C12.5.2.1 Reducing reference withdrawal design values for lag screws 25 percent when the screw is inserted in the end grain (radial-tangential plane) of the main member rather than the side grain (radial-longitudinal or tangential-longitudinal plane) is based on lag screw joint tests (100).

Early tests of wood screws in withdrawal from end grain surfaces of oak, southern pine, maple, and cypress gave somewhat erratic results relative to those for withdrawal from side grain (43). These irregular results were attributed to the tendency of the screw to split the wood in the end grain configuration. Average ratios of end grain withdrawal resistance to side grain withdrawal resistance ranged from 52 to 108 percent (43). Because of this variability, structural loading of wood screws in withdrawal from end grain has been prohibited. Where splitting is avoided, use of an end grain to side grain withdrawal design value ratio of 75 percent has been suggested (184, 183).

Reduction of withdrawal design values up to 50 percent have been reported for nails driven in end grain surfaces (radial-tangential plane) as compared to side grain (radial-longitudinal or tangential-longitudinal planes) surfaces (184, 118). When coupled with the effects of seasoning in service after fabrication, such reductions are considered too great for reliable design. It is considered to be on this basis that loading of nails and spikes in withdrawal from end grain has been prohibited.

C12.5.2.2 The use of a 0.67 adjustment factor on reference lateral design values for lag screws, wood screws, nails, or spikes driven in the end grain is based on early research on joints made with softwood species (181, 184).

C12.5.2.3 For fasteners with $D \geq 1/4"$, the use of a 0.67 adjustment factor for fasteners installed in the edge of a cross-laminated timber panel is based on the assumption that fasteners will be installed into end grain of the cross-laminated timber lamination (see Commentary C12.3.3.6) regardless of whether installation is actually into end grain. Testing of large fasteners installed into end-grain or between laminations with end-grain and side-grain indicated that the 0.67 adjustment factor was sufficiently conservative, even when gaps were present (198). For smaller diameter fasteners with $D < 1/4"$, the end grain factor in 12.5.2.2 applies where installation is into end grain (see Commentary C12.5.2.2) of a cross-laminated timber lamination.

C12.5.3 Diaphragm Factor, C_{di}

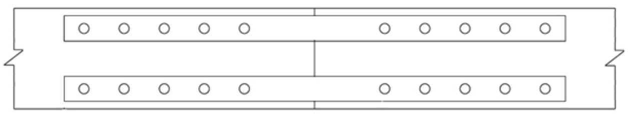
Diaphragms are large, flat structural units acting like a deep relatively thin beam or girder. Horizontal wood diaphragms consist of floor or roof decks acting as webs, and lumber, structural glued laminated timber, SCL, or I-joist members acting as the flanges. Such assemblies distribute horizontal forces acting on the flanges to vertical resisting elements (103). Shear walls consisting of wall sheathing materials attached to top and bottom plates and vertical framing members also are diaphragms. Such shear walls or vertical diaphragms act to transfer loads from horizontal diaphragms down to the supporting foundation. The diaphragm factor, C_{di}, applies to both horizontal and vertical diaphragms (144, 145).

C12.5.4 Toe-Nail Factor, C_{tn}

C12.5.4.1 The 0.67 adjustment of reference withdrawal design values for toenailing is based on the results of joint tests comparing slant driving and straight driving (184) and of typical toenailed and end nailed joints used in frame wall construction (118) where the attached member is pulled directly away from the main member. It is applicable to joints fabricated at all levels of seasoning. This includes multiple nail joints fabricated of unseasoned wood and then loaded after seasoning (184, 183, 118). Toenailing with cross slant driving can produce stronger joints than end or face nailing. For example, a stud to plate joint recognition of the combined effect of bolt hole oversizing and alignment resulting in 1/32" movement.

Special detailing can be utilized in cases where distances between outer rows of bolts exceed the limits in Table 12.5.1F, such as use of multiple splice plates or a single splice plate with slotted holes to allow shrinkage. Such an example of multiple splice plates is shown in Figure C12.5.1.3.

Figure C12.5.1.3 Connection Illustrating Use of Multiple Splice Plates



C12.5.1.4 For fasteners installed in the edge of cross-laminated timber panels, special end distance, edge distance and fastener spacing conditions are provided in NDS Table 12.5.1G, while all other requirements follow the general provisions of NDS 12.5.1. For fasteners installed in the wide face of cross-laminated panels, end distances, edge distances, and fastener spacing requirements should follow the requirements for other wood products in NDS 12.5.1. Placement of fasteners in gaps should be avoided.

C12.5.2 End Grain Factor, C_{eg}

C12.5.2.1 Reducing reference withdrawal design values for lag screws 25 percent when the screw is inserted in the end grain (radial-tangential plane) of the main member rather than the side grain (radial-longitudinal or tangential-longitudinal plane) is based on lag screw joint tests (100).

Early tests of wood screws in withdrawal from end grain surfaces of oak, southern pine, maple, and cypress gave somewhat erratic results relative to those for withdrawal from side grain (43). These irregular results were attributed to the tendency of the screw to split the wood in the end grain configuration. Average ratios of end grain withdrawal resistance to side grain withdrawal resistance ranged from 52 to 108 percent (43). Because of this variability, structural loading of wood screws in withdrawal from end grain has been prohibited. Where splitting is avoided, use of an end grain to side grain withdrawal design value ratio of 75 percent has been suggested (184, 183).

Reduction of withdrawal design values up to 50 percent have been reported for nails driven in end grain surfaces (radial-tangential plane) as compared to side grain (radial-longitudinal or tangential-longitudinal planes) surfaces (184, 118). When coupled with the effects of seasoning in service after fabrication, such reductions are considered too great for reliable design. It is considered to be on this basis that loading of nails and spikes in withdrawal from end grain has been prohibited.

C12.5.2.2 The use of a 0.67 adjustment factor on reference lateral design values for lag screws, wood screws, nails, or spikes driven in the end grain is based on early research on joints made with softwood species (181, 184).

C12.5.2.3 For fasteners with D≥1/4", the use of a 0.67 adjustment factor for fasteners installed in the edge of a cross-laminated timber panel is based on the assumption that fasteners will be installed into end grain of the cross-laminated timber lamination (see Commentary C12.3.3.6) regardless of whether installation is actually into end grain. Testing of large fasteners installed into end-grain or between laminations with end-grain and side-grain indicated that the 0.67 adjustment factor was sufficiently conservative, even when gaps were present (198). For smaller diameter fasteners with D<1/4", the end grain factor in 12.5.2.2 applies where installation is into end grain (see Commentary C12.5.2.2) of a cross-laminated timber lamination.

C12.5.3 Diaphragm Factor, C_{di}

Diaphragms are large, flat structural units acting like a deep relatively thin beam or girder. Horizontal wood diaphragms consist of floor or roof decks acting as webs, and lumber, structural glued laminated timber, SCL, or I-joist members acting as the flanges. Such assemblies distribute horizontal forces acting on the flanges to vertical resisting elements (103). Shear walls consisting of wall sheathing materials attached to top and bottom plates and vertical framing members also are diaphragms. Such shear walls or vertical diaphragms act to transfer loads from horizontal diaphragms down to the supporting foundation. The diaphragm factor, C_{di}, applies to both horizontal and vertical diaphragms (144, 145).

C12.5.4 Toe-Nail Factor, C_{tn}

C12.5.4.1 The 0.67 adjustment of reference withdrawal design values for toenailing is based on the results of joint tests comparing slant driving and straight driving (184) and of typical toenailed and end nailed joints used in frame wall construction (118) where the attached member is pulled directly away from the main member. It is applicable to joints fabricated at all levels of seasoning. This includes multiple nail joints fabricated of unseasoned wood and then loaded after seasoning (184, 183, 118). Toenailing with cross slant driving can produce stronger joints than end or face nailing. For example, a stud to plate joint

C16 FIRE DESIGN OF WOOD MEMBERS

C16.1 General

The design provisions in the Specification are intended for use in allowable stress design (ASD). These provisions do not address procedures for rehabilitating a structure or an assembly following fire damage.

Introduction to Fire Design

The model building codes in the U.S. cover virtually every safety-related topic related to the construction of buildings, and fire-related issues comprise a surprisingly large portion of the model codes. Designing for fire safety is a complex and multifaceted issue. The following information provides an overview of the subject.

To provide fire safety in any structure, many approaches are considered. This involves a combination of (1) preventing fire occurrence by reducing potential ignition sources, (2) controlling fire growth, and (3) providing protection to life and property. All need systematic attention to provide a high degree of economical fire safety. The building design professional can control fire growth within the structure by generating plans that include features such as protecting occupants, confining fire in compartment areas, and incorporating fire suppression and smoke or heat venting devices at critical locations.

Controlling construction features to facilitate rapid egress, protection of occupants in given areas, and preventing fire growth or spread are regulated by codes as a function of building occupancy. If the design professional rationally blends protection solutions for these items with the potential use of a fire-suppression system (sprinklers, for example), economical fire protection can be achieved.

Although attention could be given to all protection techniques available to the building design professional, this discussion is limited to the provisions that limit fire growth and limit spread of fire to neighboring compartments or buildings.

Fire-Rated Assemblies

The previous section explained that some occupancies require the use of fire-rated assemblies or members to prevent collapse or fire spread from one compartment of a building to another or from one building to another.

Members and assemblies are rated for their ability either to continue to carry design loads during fire exposure or to prevent the spread of fire through them. Such ratings are arrived at either by calculation or experiment for both members and assemblies. The standard fire exposure is defined in ASTM E119. A 1-hour fire-resistance rating for wall, floor, and floor-ceiling assemblies incorporating nominal 2" structural lumber can be accomplished through the use of fire-resistive membranes such as gypsum wallboard. However, fastening of these surface materials is critical for assembly performance and is carefully specified. For some wood assemblies, 2-hour ratings have been achieved.

Experimental ratings are available for several generic assemblies. Ratings for proprietary assemblies are typically supplied by the producers. Typically rated floor-ceiling assemblies for various products are provided in AWC's *DCA No. 3–Fire Rated Wood Floor and Wall Assemblies,* available at www.awc.org.

Analytically Rated

In lieu of experimentally rating the fire resistance of members and assemblies, major building codes will accept engineering calculations of the expected fire resistance, based upon engineering principles and material properties. This applies to the rating of previously untested members or assemblies, or in cases where it is desired to substitute one material or component for another. Although calculation procedures may be conservative, they have the advantage of quickly rating an assembly or member and allowing interpolation or some extrapolation of expected performance. Additional details regarding the analytical approach are provided in AWC's *DCA No. 4 - Component Additive Method (CAM) for Calculating and Demonstrating Assembly Fire Resistance*, available at www.awc.org.

Beams and Columns

Heavy timber construction has traditionally been recognized to provide a fire-resistant building. This is primarily due to the large size of the members, the connection details, and the lack of concealed spaces. Such a construction type has often satisfied the fire-resistive requirement in all building codes by simple prescription. Although heavy timber construction has not been "rated" in the United States, Canada has assigned it a 45-minute fire-resistance rating.

Using calculations, exposed columns and beams of sawn lumber, glued-laminated timber, laminated veneer lumber, parallel strand lumber, and laminated strand lumber can be designed for desired fire-resistance ratings. Additional details regarding the analytical approach are provided in NDS Chapter 16 and AWC's *DCA No. 2–Design of Fire-Resistive Exposed Wood Members*, available at www.awc.org.

Fireblocking and Draftstopping

In all construction types, no greater emphasis can be placed on the control of construction to reduce the fire growth hazard than the placement of fire and draft stops in concealed spaces. The spread of fire and smoke through these concealed openings within large rooms or between rooms is a continuous cause of major life and property loss. As a result, most building codes contain requirements for fireblocking and draftstopping within building plans. Fire-blocking considered acceptable are (1) two-inch nominal lumber, (2) two thicknesses of one-inch nominal lumber, and (3) two thicknesses of 3/4-inch plywood, with staggered joints.

Draftstopping does not require fire resistance of fireblocking. Therefore, draftstopping material is not required to be as thick. Typical draftstop materials and their minimum thicknesses are (1) l/2-inch gypsum wallboard and (2) 3/8-inch plywood. Building codes consider an area between draftstops of 1,000 square feet as reasonable. Concealed spaces consisting of open-web floor truss components in protected floor-ceiling assemblies are an important location to draftstop parallel to the component. Areas of 500 square feet in single-family dwellings and 1,000 square feet in other buildings are recommended, and separation of areas between family compartments is absolutely necessary. Critical draftstop locations are in the concealed spaces in floor-ceiling assemblies and in attics of multifamily dwellings when separation walls do not extend to the roof sheathing above.

Other important locations to fireblock in wood-frame construction are in the following concealed spaces:

1. Stud walls and partitions at ceiling and floor levels.
2. Intersections between concealed horizontal and vertical spaces such as soffits.
3. Top and bottom of stairs between stair stringers.
4. Openings around vents, pipes, ducts, chimneys (and fireplaces at ceiling and floor levels) with noncombustible fire stops.

Flame Spread

Regulation of materials used on interior building surfaces (and sometimes exterior surfaces) of other than one- and two-family structures is provided to minimize the danger of rapid flame spread. ASTM E84 gives the method used to obtain the flame-spread property for regulatory purposes of paneling materials. The flame spread test apparatus is calibrated such that red oak flooring samples that have been equilibrated to specified conditions will yield a flame spread index of 100, and noncombustible fiber-cement board samples will yield a flame spread index of 0. A list of accredited flame-spread ratings for various commercial woods and wood products is given in AWC's *DCA No. 1–Flame Spread Performance of Wood Products*, available at www.awc.org.

Fire Retardant Treatments

It is possible to make wood highly resistant to the spread of fire by pressure impregnating it with an approved chemical formulation. Wood including wood treated with a fire-retardant chemical, will char if exposed to fire; however, the most significant contribution of chemicals is a reduction in the spread of fire. Wood that has absorbed adequate amounts of a fire-retardant chemical will cease to burn when the source of ignition is removed.

Two general methods of improving resistance of wood to fire are (1) impregnation with an effective chemical, and (2) coating the surface with a layer of intumescent paint. For interiors or locations protected from weather, impregnation treatments can be considered permanent and have considerable value in preventing ignition. Surface applications offer the principal means of increasing fire-retardant properties of existing structures. However, surface applications may require periodic renewal if their effectiveness is to be maintained. In the past, the only effective chemicals were water soluble, making fire-retardant treatments inadaptable to weather exposure. Impregnated fire retardants that are resistant to both high humidity and exterior exposures are becoming increasingly available on the market for treated lumber and plywood products. In addition, factory-applied, fire-retardant coatings may be available for other products recognized within this standard. See product-specific recommendations regarding proper treatments and specifications.

C16.2 Design Procedures for Exposed Wood Members

The mechanics-based design procedures in the Specification for exposed wood members are based on research described in AWC's *Technical Report 10: Calculating the Fire Resistance of Exposed Wood Members* (136). The design procedure calculates the capacity of exposed wood members using basic wood engineering mechanics. Actual mechanical and physical properties of the wood are used and member capacity is directly calculated for a given period of time. Section properties are computed assuming an effective char rate, β_{eff}, at a given time, t. Reductions of strength and stiffness of wood in the heated zone adjacent to the char layer are addressed by accelerating the char rate 20%. Average member strength properties are approximated from existing accepted procedures used to calculate design properties. Finally, wood members are designed using accepted engineering procedures found in NDS for allowable stress design. The design procedures presented in Chapter 16 are not intended to be used for design and retrofit of a structure after a fire event.

C16.2.1 Char Rate

C16.2.1.1 To estimate the reduced cross-sectional dimensions, the location of the char base must be determined as a function of time on the basis of empirical charring rate data. The char layer can be assumed to have zero strength and stiffness. The physical shape of the remaining section and its load-carrying capacity should be adjusted to account for rounding at the corners, and for loss of strength and stiffness in the heated zone. In design there are various documented approaches to account for these results:

- additional reduction of the remaining section;
- uniform reduction of the maximum strength and stiffness; or
- more detailed analysis with subdivision of the remaining section into several zones at different temperatures.

Extensive char rate data is available for one-dimensional wood slabs. Data is also available for two-dimensional timbers, but most of this data is limited to larger cross-sections. Evaluation of linear char rate models using one-dimensional char rate data suggests that charring of wood is slightly nonlinear, and estimates using linear models tend to underestimate char depth for short time periods (< 60 minutes) and overestimate char depth for longer time periods (> 60 minutes). To account for char rate nonlinearity, a nonlinear, one-dimensional char rate model based on the results of 40 one-dimensional wood slab charring tests of various species was developed (154). This non-linear model addressed accelerated charring which occurs early in the fire exposure by applying a power factor to the char depth, x_{char}, to adjust for char rate nonlinearity:

$$t = m(x_{char})^{1.23} \qquad \text{(C16.2-1)}$$

where:

t = exposure time (min.)

m = char slope (min./in.$^{1.23}$)

x_{char} = char depth (in.)

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

To develop a two-dimensional, nonlinear char rate model, one-dimensional non-linear char rate model was modified to enable values for the slope factor, m, to be estimated using nominal char rate values (in./hr.), β_n. The nominal char rate values, β_n, are calculated using measured char depth at approximately one hour. Substituting and solving for the char depth, x_{char}, in terms of time, t:

$$x_{char} = \beta_n t^{0.813} \qquad \text{(C16.2-2)}$$

To account for rounding at the corners and reduction of strength and stiffness of the heated zone, the nominal char rate value, β_n, is increased 20% in NDS Equation 16.2-1. For sawn timbers and structural glued laminated timbers, the nominal char rate, β_n, has been assumed to be approximately 1.5 inches/hr. This assumption has been shown to provide good, repeatable results as discussed in *Technical Report 10: Calculating the Fire Resistance of Exposed Wood Members* (136). Recent analysis and testing also reported in *Technical Report 10* have verified that the char model provides reliable estimates for sawn lumber, structural composite lumber identified as laminated veneer lumber, parallel strand lumber, laminated strand lumber, and cross-laminated timber.

C16.2.1.2 For sawn lumber and timbers, structural glued-laminated timbers, laminated veneer lumber, parallel strand lumber, and laminated strand lumber, the depth of char can be directly estimated using NDS Equation 16.2-1 and assuming a nominal char rate, β_n, of 1.5 inches/hr. The effective depth of char, a_{char}, can be calculated as:

$$a_{char} = 1.8t^{0.813}$$

C16.2.1.3 For cross-laminated timber, falloff of laminations has been noted in some full-scale tests. The falloff appears to occur as the char front reaches the glueline. To model this effect, the time required for the char front to reach the glueline of each lamination, starting from the time the char front reaches the prior lamination, can be calculated as:

$$t_{gi,i} = \left(\frac{h_{lam,i}}{\beta_n}\right)^{1.23}$$

where:

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

h_{lam} = lamination thickness (in.)

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

$$t - \sum_{i=1}^{n_{lam}} t_{gi,i} \geq 0$$

where:

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

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

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

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

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

where:

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

and

$$n_{lam} = \frac{t}{t_{gi}}$$

Effective char depths, a_{char}, for cross-laminated timber with equal lamination depths have been calculated in NDS Table 16.2.1B.

C16.2.1.4 For sawn lumber and timbers, structural glued-laminated timbers, laminated veneer lumber, parallel strand lumber, and laminated strand lumber, section properties can be calculated using standard equations for area, section modulus, and moment of inertia using reduced cross-sectional dimensions. The dimensions are reduced by a_{char} calculated per C16.2.1.2 for each surface exposed to fire. Cross-sectional properties for a member exposed on all four sides are shown in Table C16.2-1. Other exposures can be calculated using this method.

C16.2.1.5 For cross-laminated timbers, reduced section properties are calculated using the effective char depth, a_{char}, calculated per C16.2.1.3; however, due to the proprietary nature of cross-laminated timber layups, the impact of charring should be checked with the manufacturer. Alternatively, the capacity of the cross-laminated timber can be conservatively determined by limiting the design to a panel with the same number of full-depth laminations remaining after a given fire resistance time.

C16.2.2 Member Strength

Generally, average unheated member strength can be approximated from tests or by using design stresses derived from actual member strength data. To approximate an average member strength using a reference design value, the reference design value can be multiplied by an adjustment factor, K, to adjust from a 5% exclusion value allowable design value to an average ultimate value. In the 2012 edition of the Specification, the sequence of required adjustments was clarified to ensure that F_b^* is multiplied by

Table C16.2-1 Cross-Sectional Properties for Four-Sided Exposure

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

the adjustment factor, K, prior to calculation of the beam stability factor, C_L (Equation 3.3-6). Similarly, that F_c^* is multiplied by the adjustment factor, K, prior to calculation of the column stability factor, C_P (Equation 3.7-1).

The adjustment factor, K, has two components, the inverse of the applicable design value adjustment factor, 1/k, and the inverse of the variability adjustment factor, c. To develop general design procedures for solid-sawn lumber, structural glued laminated timber, and structural composite lumber, the following design value adjustment factors and estimates of COV were used to conservatively develop an allowable design stress to average ultimate strength adjustment factor, K, shown in Table C16.2-2.

C16.2.3 Design of Members

The induced stress cannot exceed the average member capacity of a wood member exposed to fire for a given time, t. The average member capacity can be estimated using cross-sectional properties reduced for fire exposure and average ultimate strength properties derived from reference design values.

C16.2.4 Special Provisions for Structural Glued Laminated Timber Beams

The outer laminations of glued laminated timber bending members in Table 5A of the NDS Supplement are typically higher strength laminations. When the beam is exposed to fire, these laminations are the first to be charred. In order to maintain the ultimate capacity of the beam when these laminations are completely charred, core laminations should be replaced with the higher strength laminations in the beam layup. For unbalanced beams, only the core laminations adjacent to the tension side lamination need to be replaced as shown in Figure C16A(b) and C16A(c), respectively. For balanced beams, the core laminations adjacent to the outer laminations on both sides need to be replaced as shown in Figure C16B(b) and C16B(c), respectively.

Figure C16A Typical Unbalanced Beam Layup

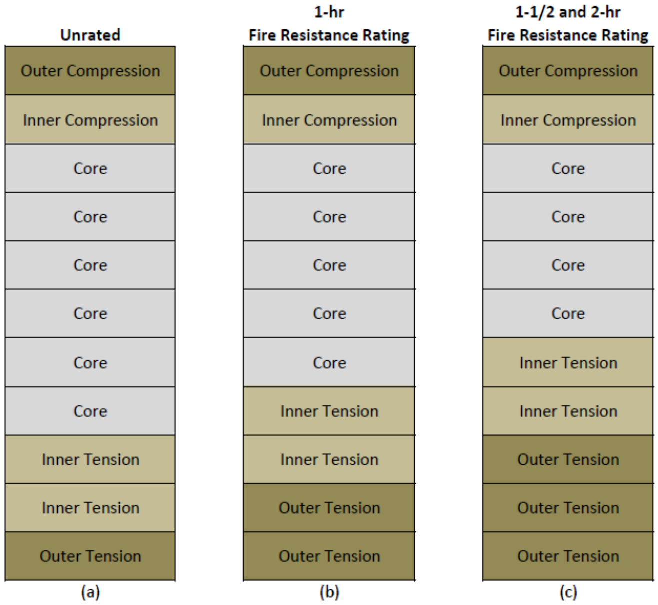
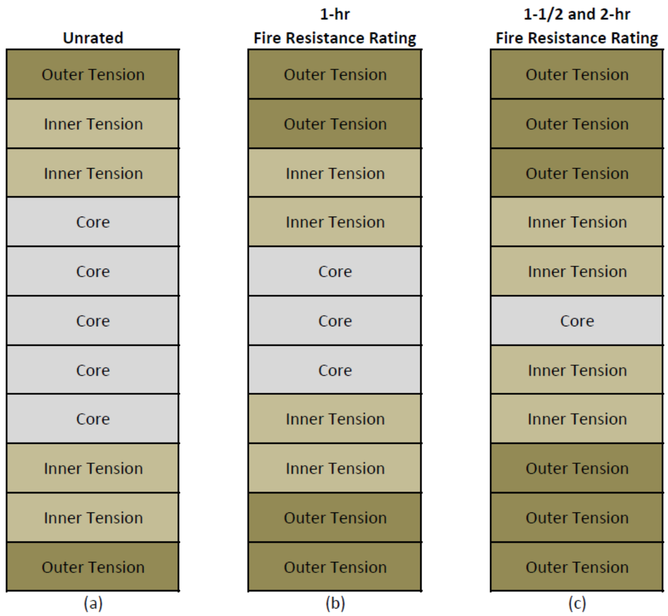


Figure C16B Typical Balanced Beam Layup



C16.2.5 Provisions for Timber Decks

Sides of individual timber decking members are shielded from full fire exposure by adjacent members collectively acting as a joint. Partial exposure can occur as members shrink and joints between members open. The degree of exposure is a function of the view angle of the

Table C16.2-2 Allowable Design Stress to Average Ultimate Strength Adjustment Factors

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

1. Taken from Table 10 of ASTM D245 *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber*.
2. Taken from Table 4-6 of *1999 Wood Handbook*.
3. Taken from Appendices D and H of *National Design Specification for Wood Construction*.

radiant flame and the ability of hot volatile gases to pass through the joints. When the joint is completely open, such as can occur with butt-jointed timber decking, hot gases will carry into the joint and the sides of the decking members will char. This charring can be conservatively approximated assuming the sides of a member along the joint char at the effective char rate. When the joint is open but covered by sheathing, as with butt-jointed timber decking covered with wood structural panels, passage of hot gases is limited, and tests have shown that charring can be approximated assuming a partial exposure char rate along the joint equal to one-third of the effective char rate. For joints which are not open, as with tongue-and-groove timber decking, tests have shown that charring of the sides of members is negligible and can be ignored.

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AMERICAN WOOD COUNCIL

August 2015

ERRATA
to the 2015 Edition of
the National Design Specification® (NDS®) for Wood Construction
(web version dated 11-14)

Page	**Revision**
62	Revise Table 10.4.1.1 as follows:

Table 10.4.1.1 Shear Deformation Adjustment Factors, K_S

Loading	End Fixity	K_S
Uniformly Distributed	Pinned	11.5
	Fixed	57.6
Line Load at midspan	Pinned	14.4
	Fixed	57.6
Line Load at quarter points	Pinned	10.5
Constant Moment	Pinned	~~11.8~~ 0
Uniformly Distributed	Cantilevered	4.8
Line Load at free-end	Cantilevered	3.6
Column Buckling	Pinned	11.8
	Fixed	23.7

152 Revise the first sentence of Section 16.2.1.3 as follows:

16.2.1.3 For cross-laminated timber manufactured with laminations of equal thickness, the effective char depth, a_{char}, shall be calculated as follows:

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