

FOREWORD TO THE NDS COMMENTARY

The *National Design Specification* for Wood Construction (NDS®)* was first issued in 1944 as the *National Design Specification for Stress-Grade Lumber and Its Fastenings*. In 1977 the title of the Specification was changed to its present form. The 2015 edition is the sixteenth edition of the publication.

The Commentary presented herein is intended to respond to user needs for background information and interpretive discussion of the provisions of the Specification. The Commentary follows the same subject matter organization as the Specification itself. Discussion of a particular provision in the Specification is identified in the Commentary by the same section or subsection number assigned to that provision in the Specification. The Commentary on each provision addressed consists of one or more of the following: background, interpretation, and example. Information presented under background is intended to give the reader an understanding of the data and/or experience on which the provision is based. References containing more detailed information on the subject are

included. Interpretive discussion of how a provision should be applied is given where users have suggested the intent of a requirement is ambiguous.

It is intended that this NDS Commentary 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. Particular attention is directed to Section C2.1.2, relating to the designer's responsibility to make adjustments for particular end uses of structures.

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Inquiries, comments, and suggestions from the readers of this document are invited.

American Wood Council

C1 GENERAL REQUIREMENTS FOR STRUCTURAL DESIGN

C1.1 Scope

C1.1.1 Practice Defined

C1.1.1.1 This Specification defines a national standard of practice for the structural design of wood elements and their connections.

C1.1.1.2 Where the structural performance of assemblies utilizing panel products are dependent upon the capacity of the connections between the materials, such as in shear walls or diaphragms, the design provisions for mechanical connections in the Specification may be used for such assemblies when such application is based on accepted engineering practice or when experience has demonstrated such application provides for satisfactory performance in service.

C1.1.1.3 Requirements for the design of structural assemblies utilizing metal connector plates in accordance with principles of engineering mechanics were moved from the mechanical connections chapter to NDS 1.1.1.3 in the 2012 edition of the Specification in recognition of the scope of the referenced ANSI/TPI standard for design of metal plate-connected wood truss construction which includes more than design of the metal connector plates alone. This relocation was also coordinated with removal of metal connector plate requirements from the Specification, such as applicability of adjustment factors and the wet service factor, because they were redundant with those in the ANSI/TPI standard. It is the intent of this section

to require design of structural assemblies utilizing metal connector plates in accordance with principles of engineering mechanics such as those addressed in the ANSI/TPI standard for design of metal plate connected wood truss construction.

C1.1.1.5 The data and engineering judgments on which the Specification are founded are based on principles of engineering mechanics and satisfactory performance in service. However, they are not intended to preclude the use of other products or design procedures where it can be demonstrated that these products or design procedures provide for satisfactory performance in the intended application. Other criteria for demonstrating satisfactory performance may be proprietary or specialized design standards applicable to a particular component type. The appropriateness and acceptability of alternate criteria are determined by the designer and the code authority having jurisdiction.

C1.1.2 Competent Supervision

There are several areas in which competent supervision should be required such as joint details and placement of fasteners. Special attention should be given to end details of columns and beam-columns to assure that design assumptions related to load eccentricity are met.

C1.2 General Requirements

C1.2.1 Conformance with Standards

The provisions of this Specification assume conformance with the standards specified.

C1.2.2 Framing and Bracing

Unless otherwise specified in the Specification, all reference design values assume that members are adequately framed, anchored, tied and braced. Adequate bracing and anchorage of trusses and truss members to assure appropriate resistance to lateral loads is particularly important. Good practice recommendations (142) for installation between trusses of vertical sway (cross) bracing, continu-

ous horizontal bottom chord struts and bottom chord cross bracing are given in NDS Appendix A.10.

In addition to providing adequate permanent bracing and bridging in the structure to resist wind and other racking forces, sufficient temporary bracing of load-carrying

members should be used during construction to assure such members will withstand wind and temporary construction loads before adjacent members and cladding materials required by the design are installed.

C1.3 Standard as a Whole

The provisions of this Specification are intended to be used together. Unless otherwise noted, pertinent provisions from each chapter apply to every other chapter.

C1.4 Design Procedures

The Specification addresses both allowable stress design (ASD) and load and resistance factor design (LRFD) formats for design with wood structural members and their connections. In general, design of elements throughout a structure will utilize either the ASD or LRFD format; however, specific requirements to use a single design format for all elements within a structure are not included in this Specification. The suitability of mixing formats within a structure is the responsibility of the designer. Consideration should be given to building code limitations, where available. *ASCE 7 – Minimum Design Loads for Buildings and Other Structures* (3), referenced in building codes, limits mixing of design formats to cases where there are changes in materials.

C1.4.1 Loading Assumptions

The design provisions in the Specification assume adequacy of specified design loads.

C1.4.2 Governed by Codes

Design loads shall be based on the building code or other recognized minimum design loads such as *ASCE 7 - Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (3).

C1.4.3 Loads Included

This section identifies types of loads to consider in design but is not intended to provide a comprehensive list of required loading considerations.

C1.4.4 Load Combinations

The reduced probability of the simultaneous occurrence of combinations of various loads on a structure, such as dead, live, wind, snow and earthquake, is recognized for both ASD and LRFD in the model building codes and ASCE 7(3). Specific load reductions for ASD or load combinations for LRFD apply when multiple transient loads act simultaneously.

For ASD, some codes provide for a reduction in design load for wind or earthquake even when both are not considered to act simultaneously. This particular load reduction is accounted for in such codes by allowing all materials a 1/3 increase in allowable stress for these loading conditions. Because individual jurisdictions and code regions may account for load combinations differently, the building code governing the structural design should be consulted to determine the proper method.

All modifications for load combinations are entirely separate from adjustments for load duration, C_D , or time effect, λ , which are directly applicable to wood design values (see C2.3.2 and C2.3.7). It should be emphasized that reduction of design loads to account for the probability of simultaneous occurrence of loads and the adjustment of wood resistances to account for the effect of the duration of the applied loads are independent of each other and both adjustments are applicable in the design calculation.

C1.5 Specifications and Plans

C1.5.1 Sizes

To assure that the building is constructed of members with the capacity and stiffness intended by the designer, the basis of the sizes of wood products given in the plans and specifications should be clearly referenced in these documents. The use of nominal dimensions in the distribution

and sale of lumber and panel products has been a source of confusion to some designers, particularly those unfamiliar with wood structural design practices. The standard nominal sizes and the standard net sizes for sawn lumber are established for each product in national product standards (152). For proprietary or made-to-order products, special sizes should be specified.

C1.6 Notation

The system of notation used in the Specification helps to identify the meaning of certain frequently used symbols. Adjustment factors, identified by the symbol “C”, modify reference design values for conditions of use, geometry, or stability. The subscripts “D”, “F”, “L”, etc., are used to distinguish between different adjustment factors. In certain cases, upper and lower case subscripts of the same letter (“D” and “d”) are used to denote two different adjustments (load duration factor and penetration depth factor for split ring and shear plate connections, respectively). There is no particular significance to the use of the same letter

with different cases for different adjustment factors. The symbols “F” and “F’” denote reference and adjusted design values, respectively; where adjusted design values represent reference design values multiplied by all applicable adjustment factors. The symbol “f” indicates the actual or induced stress caused by the applied loads. The subscripts “b”, “t”, “c”, “v”, and “c_⊥” indicate bending, tension parallel to grain, compression parallel to grain, shear, and compression perpendicular to grain stress, respectively.

C2 DESIGN VALUES FOR STRUCTURAL MEMBERS

C2.1 General

C2.1.1 General Requirement

The Specification addresses both ASD and LRFD formats for structural design with wood products (see C1.4).

C2.1.2 Responsibility of Designer to Adjust for Conditions of Use

The Specification identifies adjustments to reference design values for service conditions generally encountered in wood construction. However, this Specification does not address all possible design applications or end use conditions.

The designer has the final responsibility for determining the proper adjustment of design values for specific loading and end use conditions. Particular attention is required to those uses where two or more extreme conditions of service converge. An example of such a use is one where it is known that the full design load will be applied continuously and the structural members will be consistently exposed to water at elevated temperatures. Assessment of the consequences of a failure of an individual member in the structure is an integral part of the designer's responsibility of relating design assumptions with design value adjustments appropriate to the end use conditions.

C2.2 Reference Design Values

Reference design values used in this Specification and tabulated in the NDS Supplement are ASD values based on normal load duration and moisture conditions specified.

C2.3 Adjustment of Reference Design Values

C2.3.1 Applicability of Adjustment Factors

The Specification requires adjustment of reference design values for specific conditions of use, geometry and stability. Such modifications are made through application of adjustment factors. The adjustment factors are cumulative except where indicated otherwise. In addition to the adjustment factors given in this section, other adjustments of reference design values for special conditions of use may be required. Such additional adjustments may include modifications for creep effects, variability in modulus of elasticity, and fire retardant treatment.

Each product chapter of the Specification provides a unique table to identify the applicability of adjustment factors for determining design values for sawn lumber, structural glued laminated timber, structural composite

lumber, prefabricated wood I-joists, and wood structural panels. The approach of adjusting a reference value in accordance with the applicable adjustment factors to arrive at either an ASD or LRFD adjusted design value is illustrated in the following example for sawn lumber bending stress, F_b :

$$\text{ASD: } F_b' = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_{fu})(C_i)(C_r)$$

$$\text{LRFD: } F_b' = F_b(C_M)(C_t)(C_L)(C_F)(C_{fu})(C_i)(C_r)(2.54)(0.85)(\lambda)$$

where:

F_b = Reference bending design value, psi

C_D = Load duration factor applicable for ASD only

$(C_M)(C_t)(C_L)(C_F)(C_{fu})(C_i)(C_r)$ = Adjustment factors from NDS 4.3 and Table 4.3.1

2.54 = Format conversion factor, K_F , from Table 4.3.1 applicable for LRFD only

0.85 = Resistance factor, ϕ , from Table 4.3.1 applicable for LRFD only

λ = Time effect factor, applicable for LRFD only

C2.3.2 Load Duration Factor, C_D (ASD Only)

C2.3.2.1 Load duration factors (C_D) are applicable to all reference design values except modulus of elasticity and compression perpendicular to grain. Exclusion of modulus of elasticity from load duration adjustment has been a provision of the Specification since the first edition. Load duration factors are based on the effect of time under load on ultimate load-carrying capacity. Increased deflection or deformation is a separate consideration, independent of ultimate strength. Compression perpendicular to grain design values were subject to adjustment for load duration when such values were based on proportional limit test values. For compression perpendicular to grain design values that are based on a deformation limit, the load duration factor does not apply.

Table 2.3.2 Frequently Used Load Duration Factors, C_D

Permanent Loads. In addition to construction dead loads due to materials, foundation soil loads and concentrated loads from equipment designed as part of the structure should be considered long-term loads that will be applied continuously or cumulatively for more than ten years. Special continuous loadings related to the particular purpose or use of the structure, such as water loads in cooling towers or heavy machinery in industrial buildings, also may be associated with durations exceeding ten years.

Ten Year or Normal Loading. Loads traditionally characterized as normal are code specified floor loads, either uniform live or concentrated, which include furniture, furnishings, movable appliances and equipment, all types of storage loads, and all people loads. Although maximum human traffic loads may be infrequent and of short duration, such as those occurring on balconies, exterior walkways and stairways, this type of loading is considered normal loading.

Two Month Loads. A two month load duration adjustment factor of 1.15 was used for all code specified snow loads prior to 1986. Maximum snow loads published in ASCE 7 (3) based on probability of occurrence are significantly greater in some high snow regions than the loads previously used in those areas. Evaluation of annual snow

load records available for some of these areas shows that the duration of the maximum snow load specified in ASCE 7 is much shorter than the two months duration previously assumed for all snow loads. The Specification provides for use of a larger snow load adjustment than 1.15 when information is available on the duration of the design snow load for a specific area.

Seven Day Loads. Where the minimum roof uniform load specified by the applicable building code exceeds the design snow load for the area and the specific building design, it is conventional practice to consider this load a construction type load for which a seven day or 1.25 load duration factor is applicable. If the roof snow load is less than 92 percent of the minimum roof load specified, the latter will be the limiting of the two load conditions.

One Day Loads. Prior to 1987, a one day or 1.33 factor was used as the load duration adjustment for wind and earthquake loads. In the current Specification, the load duration factor for these loads has been based on a 10 minute load duration.

Ten Minute Loads. The ten minute or 1.6 load duration factor is to be used with wind and earthquake loads in the current Specification. For allowable stress design (ASD), wind loads in the model building codes and ASCE 7 are maximum loads expected to occur less than once in 50 years and to have durations of from one to 10 seconds. Peak earthquake loads are known to have cumulative durations less than 5 minutes rather than the one day duration traditionally assigned. The ten minute load duration factor is conservatively estimated as the adjustment for the cumulative effect of these two load conditions.

Impact Loads. Loads in this category are considered to be those in which the load duration is one second or less. Such a duration is associated with an adjustment factor of 2.0 based on the general relationship between strength and load duration (see NDS Appendix B). Pressure treatment of wood with preservatives or fire retardant chemicals may reduce energy absorbing capacity as measured by work-to-maximum-load in bending; therefore, use of the 2.0 load duration factor in these applications is not permitted (173). Connections or wood structural panels are also not permitted to use the 2.0 load duration factor.

C2.3.2.2 Design of structural members is based on the critical combination of loads representing different durations and resistances adjusted for these different durations. Note that load duration adjustments are not applicable to modulus of elasticity (see C2.3.2.1), hence, a member subject to buckling should be analyzed for the critical load combination after the critical buckling design value has been calculated.

C2.3.2.3 Reduction of design loads to account for the probability of simultaneous occurrence of loads and the

adjustment of wood resistances to account for the effect of the duration of the applied loads are independent of each other and both adjustments are applicable in the design calculation (see C1.4.4).

C2.3.3 Temperature Factor, C_t

Temperature adjustments in the Specification apply when structural members are exposed to temperatures between 100°F and 150°F for extended periods of time, such as in industrial applications in which structural members are in close proximity to or in contact with heated fluids used in manufacturing processes. In general, adjustment of reference design values in the Specification for temperature should be considered for applications involving sustained heavy dead or equipment loads, or water immersion, or wet or high moisture content service conditions, when sustained or frequent extended exposure to elevated temperatures up to 150°F will occur.

Use of lumber or glued-laminated timber members in applications involving prolonged exposure to temperatures over 150°F should be avoided. Where such exposures do occur, adjustments for both immediate and permanent strength reductions should be made. Permanent effects should be based on the cumulative time the members will be exposed to temperature levels over 150°F during the life of the structure and the strength losses associated with these levels (183). Roof systems and other assemblies subject to diurnal temperature fluctuations from solar radiation are not applications that normally require adjustment of reference design values for temperature.

Reversible Effects at or Below 150°F. The increase in the strength properties of wood when cooled below normal temperatures and the decrease in these properties when it is heated up to 150°F are immediate and generally reversible. When the temperature of the wood returns to normal temperature levels, it recovers its original properties. In general, these reversible effects are linear with temperature for a given moisture content (51). The magnitude of the increase or decrease, however, varies with moisture content. The higher the moisture content, the larger the increase in wood strength properties with decreasing temperature and the larger the decrease in wood strength properties with increasing temperature.

Permanent Effects Over 150°F. Prolonged exposure to temperatures over 150°F can cause a permanent loss in strength when cooled and tested at normal temperatures. The permanent effect is in addition to the immediate or reversible effect that occurs at the exposure temperature. Permanent losses in strength resulting from exposures over 212°F are greater for heating in steam than in water

(183). For temperatures over 150°F, permanent decreases in strength are greater for heating in water than in dry air.

The use of 150°F as a nominal threshold for the beginning of permanent strength loss is substantiated by available test data showing an approximate 10 percent loss in bending strength (modulus of rupture) for material exposed for 300 days in water at 150°F and then tested at room temperature (183). Exposure in air at the same temperature would result in a smaller permanent strength loss.

Cold Temperatures. Adjustments for increasing reference design values for cooling below normal temperatures are difficult to establish in building design because of the variable nature of low temperature environments. Structural members that might be exposed to below freezing temperatures continuously for up to several months also are exposed to normal temperatures during periods of the year when the full design load might be resisted. For special applications such as arctic construction or transportation of cryogenic materials where the design load is always associated with low temperature environments, data from other sources may be used to make appropriate adjustments of design values (183, 51).

Elevated Temperatures Encountered in Normal Service. Temperatures higher than ambient can be reached in roof systems as a result of solar radiation. The temperatures reached in such systems are a function of many variables, including hour of day, season of year, cloud cover, wind speed, color of roofing, orientation, ventilation rate, presence of insulation and thickness of sheathing. Measurements of roof system temperatures in actual buildings (64) show that structural framing members in such roofs seldom if ever reach a temperature of 150°F, and when such levels are reached the duration is very short and is confined to the face of the member on which the sheathing is attached. Even in the severest of radiation and design conditions, the temperature of structural beams, rafters, and truss members in wood roofs generally do not reach 140°F. Normal temperature environments return as the sun sets.

The foregoing considerations and successful field experience are the basis for the long standing practice of applying the reference design values tabulated in the Specification without adjustment for temperature to structural wood roof members in systems designed to meet building code ventilation requirements. Reference design values also are appropriate for use with wood members directly exposed to solar radiation but otherwise surrounded by ambient air, such as members used in bridges, exterior balconies and stairways, and exterior vertical and horizontal structural framing.

C2.3.4 Fire Retardant Treatment

Fire retardant treatments are proprietary and chemical formulations vary between manufacturers. The fire retardant treatment manufacturers have established design values for wood products treated with their commercial formulations. It should be noted that use of individual company design value recommendations for fire retardant treated wood products is subject to approval of the authority having jurisdiction.

C2.3.5 Format Conversion Factor, K_F (LRFD Only)

Format conversion factors convert reference design values (allowable stress design values based on normal load duration) to *LRFD reference resistances* as described in ASTM D5457 (17). Specified format conversion factors, K_F , in NDS Table 2.3.5 and Applicability of Adjustment Factor Tables in Chapters 4, 5, 6, 7, 8, 9, and 10 and Appendix Table N1 are based on similar factors contained in ASTM D5457.

The *LRFD reference resistance* is a strength level design value for short term loading conditions. Consequently, the format conversion factor includes: 1) a conversion factor to adjust an allowable design value to a higher strength-level design value, 2) a conversion factor to adjust from a 10-year to a 10-minute (short-term) load basis, and 3) a conversion factor to adjust for a specified resistance factor, ϕ .

The term, *LRFD reference resistance*, is not specifically defined or calculated in the Specification but is included as part of the LRFD adjusted design value which includes all applicable adjustments to the reference design value. Because format conversion factors are based on calibrating ASD and LRFD formats for certain reference conditions, they apply only to reference design values in this Specification and should not apply where LRFD reference resistances are determined in accordance with the reliability normalization factor method in ASTM D5457.

To be consistent with ASD, which does not adjust compression perpendicular to grain stresses for duration of load and to parallel changes in ASTM D5457, the format conversion factor, K_F , for compression perpendicular to grain was changed from 2.08 to 1.67 in the 2012 edition of the Specification and the adjustment for time effect was removed. Also, beginning with the 2012 edition of the Specification, the numeric value of the format conversion

factor, K_F , for each material property is tabulated in lieu of the equation format of K_F (i.e., $K_F = \text{constant}/\phi$) used in prior editions of the Specification.

C2.3.6 Resistance Factor, ϕ (LRFD Only)

Specified resistance factors, ϕ , in NDS Table 2.3.6 and Appendix N, Table N2 are based on resistance factors defined in ASTM D5457 (17). Resistance factors are assigned to various wood properties with only one factor assigned to each stress mode (i.e., bending, shear, compression, tension, and stability). In general, the magnitude of the resistance factor is considered to, in part, reflect relative variability of wood product properties. Actual differences in product variability are accounted for in the derivation of reference design values.

C2.3.7 Time Effect Factor, λ (LRFD Only)

The time effect factor, λ (LRFD counterpart to the ASD load duration factor, C_D), varies by load combination and is intended to establish a consistent target reliability index for load scenarios represented by applicable load combinations. With the exception of the load combination for dead load only, each load combination can be viewed as addressing load scenarios involving peak values of one or more “primary” loads in combination with other transient loads. Specific time effect factors for various ASCE 7 (3) load combinations are largely dependent on the magnitude, duration, and variation of the primary load in each combination. For example, a time-effect factor of 0.8 is associated with the load combination $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$ to account for the duration and variation of the primary loads in that combination (roof live, snow, or rain water, or ice loads). The effect of transient loads in a particular load combination or even changes in the load factors within a given combination is considered to be small relative to the effect of the primary load on the load duration response of the wood. Consequently, specific time-effect factors need not change to address load factor or load combination changes over time. Footnote 2 of NDS Table N3 provides clarification that the specific load factors shown are for reference only and are intended to provide flexibility in assignment of the time effect factor in the event of changes to specified load factors.

C3 DESIGN PROVISIONS AND EQUATIONS

C3.1 General

C3.1.1 Scope

This Chapter provides general design provisions for structural wood members and connections. Product-specific adjustments to these provisions are included in product Chapters 4 through 9 of the Specification. Specific connection design provisions are addressed in NDS Chapters 11 through 14.

C3.1.2 Net Section Area

C3.1.2.1 These provisions direct the designer to take into account the effects of removing material from the cross-sectional area. Specific provisions pertaining to notches in bending members are given in NDS 3.2.3. Provisions for calculation of shear strength in notched bending members are given in NDS 3.4.3. For compression parallel to grain, NDS 3.6.3 provides for the use of gross section area when the reduced section of a column does not occur in the critical part of the length that is most subject to potential buckling.

C3.1.2.2 To avoid possible misapplication when non-uniform patterns are used, the provision requires staggered or offset fasteners in adjacent rows to be considered in the same critical section if the parallel to grain distance between them is less than 4 diameters.

C3.1.2.3 Where the parallel to grain distance between staggered split ring or shear plate connectors is less than or equal to one diameter, they should be considered to occur in the same critical section and used to determine net area. The limit should be applied to the parallel to grain offset or stagger of split rings or shear plates in adjacent rows.

C3.1.3 Connections

Particular attention should be given to the design of joints involving multiple fasteners and to those subject to moment forces. Only fastener types having the same general load-slip or stiffness characteristics should be employed in the same joint (see C11.1.4).

The provisions are intended to ensure each member in the joint carries its portion of the design load and that

symmetrical members and fasteners are used unless the induced moments are taken into account. A lapped joint is an example of an unsymmetrical connection where the induced bending moments need to be considered.

C3.1.4 Time Dependent Deformations

Consideration of time dependent deformations in built-up members should provide for equal inelastic deformation of the components. One application addressed by this section is the use of a flange member to strengthen or stiffen a single main member in a truss without increasing the size of other members in the same plane (142). Because component connections in these built-up systems do not provide full composite action, judgment must be used to establish the level of contribution of these components and the time dependent effects on these connections. Member creep effects should also be considered in making this assessment.

C3.1.5 Composite Construction

Structural composites of lumber and other materials utilize the characteristics of each to obtain desirable structural efficiencies and/or extended service life. Timber-concrete bridge decks, timber-steel flitch beams and plywood-lumber stress-skin panels and box beams are such composites. Proven design procedures for timber-concrete beams and timber-steel members are available in wood engineering handbooks and textbooks (58, 142). Detailed design and fabrication information for plywood-lumber structural components are available from APA-The Engineered Wood Association (106). The American Institute of Timber Construction provides design information for composites involving glued laminated timber (140).

C3.2 Bending Members - General

C3.2.1 Span of Bending Members

The design span length for simple, continuous, and cantilevered bending members is defined as the clear span plus one-half the required bearing length at each reaction to avoid unrealistic moment determinations where supports are wider than the required bearing.

C3.2.2 Lateral Distribution of Concentrated Load

Lateral distribution of concentrated loads to adjacent parallel bending members can be estimated using accepted engineering practice (see C15.1).

C3.2.3 Notches

C3.2.3.1 Notches are a special problem in bending members due to the stress concentrations occurring at the corners and the difficulty of calculating the effects of shear and perpendicular to grain stresses occurring at such locations. These stress concentrations can be reduced by using gradually tapered rather than square corner notches (181).

C3.2.3.2 A notch having a depth of up to 1/6 the bending member depth and a length up to 1/3 the bending member depth is assumed to have little practical effect on bending member stiffness (184, 181).

C3.3 Bending Members - Flexure

C3.3.3 Beam Stability Factor, C_L

The beam stability factor, C_L , adjusts the reference bending design value for the effects of lateral-torsional buckling. Lateral-torsional buckling is a limit state where beam deformation includes in-plane deformation, out-of-plane deformation and twisting. The load causing lateral instability is called the elastic lateral-torsional buckling load and is influenced by many factors such as loading and support conditions, member cross-section, and unbraced length. In the 2012 and prior editions of the Specification, the limit state of lateral torsional buckling is addressed using an effective length format whereby unbraced lengths are adjusted to account for load and support conditions that influence the lateral-torsional buckling load. Another common format uses an equivalent moment factor to account for these conditions. AWC Technical Report 14 (138) describes the basis of the current effective length approach used in the NDS and summarizes the equivalent moment factor approach and provides a comparison between the two approaches.

It is common to assume buckling is not an issue in designing load-bearing beams used as headers over openings. However, long span header beams of slender cross-sections demand particular attention to stability issues. An example would be dropped garage door headers in which the load is transferred into the beam through a cripple wall that does not provide lateral support to the beam. In this instance, raising the beam in the wall and attaching it directly to the top plate which is braced by a horizontal floor or ceiling

diaphragm can be assumed to provide effective lateral support. Alternatively, the beam can be braced at points of bearing and designed as an unbraced member in accordance with NDS 3.3.3.

C3.3.3.1 For rectangular members, lateral-torsional buckling does not occur where the breadth of the bending member is equal to or greater than the depth and the load is applied in the plane of the member depth (184, 60). Note that lateral-torsional buckling does not occur in circular members.

C3.3.3.2 The rules for determining lateral support requirements based on depth to breadth ratios for sawn lumber bending members given in NDS 4.4.1 are alternate provisions to those of NDS 3.3.3. Specific span and loading conditions may be checked to compare the relative restrictiveness of the respective provisions.

C3.3.3.3 When the compression edge of a bending member is continuously supported along its length and bearing points are restrained against rotation and lateral displacement, lateral-torsional buckling under loads inducing compressive stresses in the supported edge are generally not a concern. However, the possibility of stress reversal, such as that associated with wind loading, should be considered to assure that the tension side of the bending member under the predominant loading case is adequately supported to carry any expected compressive forces. Also, bending members with large depth to breadth ratios should be braced on the tension edges.

C3.3.3.4 Where load is applied to the compression edge of a bending member using uniformly-spaced pur-

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. The volume factor, C_V , shall be included in the calculation of F_b^* when C_V is greater than 1.0 which can be the case for SCL per Section 8.3.6.

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:

$$f_v = 4V / 3A \quad (C3.4.2-1)$$

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 $F_s(Ib/Q)_{\text{eff}}$.

C3.4.3 Shear Design

C3.4.3.1 (a) For purposes of calculating shear forces, ignoring uniform loads within a distance equal to the bending member depth, “d”, of the support face assumes such loads are carried directly to the support by diagonal compression through the member depth. Concentrated loads within a distance “d” may be reduced proportionally to the distance from the face of the support. Where a member is loaded with a series of closely-spaced framing members (such as a girder loaded by floor joists), a uniform load condition may be assumed even though the framing members can be viewed as individual point loads.

C3.4.3.1 (b) Placement of the critical moving load is assumed to be one beam depth from the support. Other loads within a distance, d, of the support are permitted to be ignored similar to the provisions of NDS 3.4.3.1(a).

C3.4.3.1 (c) Placement of two or more moving loads should be evaluated to determine the location that provides the maximum shear stress. Other loads within a distance, d, of the support are permitted to be ignored similar to the provisions of NDS 3.4.3.1(a).

C3.4.3.2 (a) The equation for determining the adjusted design shear of a tension-side notched member reduces the effective shear capacity by the square of the ratio of the remaining member depth (d_n) to the unnotched member depth (d). This relationship has been verified by tests of bending members at various depths (115) and is related to the concentration of tension and shear stresses occurring at the reentrant corner of the notch. Both d and d_n are measured perpendicular to the length of the member where member length is represented by the longitudinal axis of the member. Therefore, for a sloped member such as a roof rafter, d and d_n are measured as the distance perpendicular to the rafter’s longitudinal axis, not the horizontal projected length of the rafter.

C3.4.3.2 (b) The equation for calculating the adjusted shear in members of circular cross section end-notched on the tension face parallels that for end-notched rectangular bending members. The area of the circular member (A_n) at the notch replaces the width (b) and depth at the notch (d_n) in the equation for the rectangular beam. It has been shown that maximum shear stresses near the neutral axis of an unnotched circular member calculated using $(VQ)/(Ib)$, or $(4V)/(3A)$, are within 5 percent of actual stresses (108). Therefore, the adjusted design shear of a tension-side notched circular member is conservatively estimated using the factor 2/3 rather than 3/4 in the equation.

C3.4.3.2 (c) Procedures used to calculate the adjusted shear in bending members of other than rectangular or circular cross section containing end notches on the tension face should account for any effects of stress concentrations that may occur at reentrant corners.

C3.4.3.2 (d) See C3.2.3.1.

C3.4.3.2 (e) Shear strength of bending members is less affected by end notches on the compression face than on the tension face (181).

C3.4.3.3 The shear force, V, is the shear in the beam determined as a result of structural analysis of the bending member for all loads including those imparted by connections (as shown in Figures 3E and 3I in the NDS). It is not the load applied at the connection which can be either less than or greater than the actual shear in the beam.

C3.4.3.3 (a) An equation for calculating the shear resistance at connections located less than five times the depth of the member from its end is similar to that for end-notched rectangular bending members where the ratio d_c/d is comparable to the factor d_n/d .

C3.4.3.3 (b) For connections that are at least five times the member depth from the end, net section is permitted to be used for calculating the shear resistance.

C3.4.3.3 (c) Bending members supported by concealed or partially hidden hangers whose installation involves kerfing or notching of the member are designed for shear using the notched bending member provisions of NDS 3.4.3.2.

C3.5 Bending Members - Deflection

C3.5.1 Deflection Calculations

Reference modulus of elasticity design values, E, in the Specification for wood bending members are average values. Individual pieces will have modulus of elasticity values higher or lower than the reference average value.

For solid rectangular and circular bending members, reference modulus of elasticity values are considered to contain a shear deflection component equivalent to that occurring in a rectangular bending member on a span-depth ratio of between 17 and 21 under uniformly distributed load. Assuming a modulus of elasticity to modulus of

rigidity ratio (E/G) of 16, shear-free modulus of elasticity may be taken as 1.03 and 1.05 times the reference value for sawn lumber and glued laminated timber, respectively. Standard methods for adjusting modulus of elasticity to other load and span-depth conditions are available (4).

Experience has shown that use of average modulus of elasticity values provide an adequate measure of the immediate deflection of bending members used in normal wood structural applications. It should be noted that the reduced modulus of elasticity value, E_{min} , is used in beam stability analyses and contains both a statistical and a safety level reduction.

C3.5.2 Long Term Loading

The reference modulus of elasticity values provide a measure of the immediate deflection of a member that occurs when a load is applied. If the load is sustained, the member will exhibit a slow but continual increase in deflection over time, otherwise known as creep. At moderate to low levels of sustained stress and under stable

environmental conditions, the rate of creep will decrease over time (52, 62).

Where creep is decreasing over time, total creep occurring in a specific period of time is approximately proportional to the stress level (123, 185). Total bending creep increases with increase in moisture content (34, 139) and temperature (112); and is greater under variable compared to constant relative humidity conditions (112). Creep deflection that is increasing at a constant rate should be considered a possible danger signal; and when creep deflection is increasing at an increasing rate, imminent failure is indicated (8, 139, 185).

Code specified maximum wind, snow, and live loads are pulse type loadings with low frequency of occurrence. Thus creep deflection is not a significant factor in most situations. Where dead loads or sustained live loads represent a relatively high percentage of the total design load, creep may be a design consideration. In such situations, total deflection from long-term loading, Δ_T , is estimated by increasing the immediate deflection, Δ_{LT} , associated with the long-term load component by the time dependent deformation factor, K_{cr} , provided in the Specification.

C3.6 Compression Members - General

C3.6.2 Column Classifications

C3.6.2.1 Simple solid columns are defined as single piece members or those made of pieces glued together to form a single member. Such glued members are considered to have the grain of all component pieces oriented in the same direction and to be made with a phenolic, resorcinol, or other rigid adhesive. The performance of columns made using elastomeric adhesives are not covered by the provisions of the Specification except where it has been established that the adhesive being used possesses strength and creep properties comparable to those of standard rigid adhesives.

C3.6.2.2 Design provisions for spaced columns are covered in NDS 15.2.

C3.6.2.3 Mechanically-laminated built-up columns are not designed as solid columns. Design provisions for these built-up columns are covered in NDS 15.3.

C3.6.3 Strength in Compression Parallel to Grain

In reduced section members, the actual compression stress parallel to grain, f_c , shall be checked as follows:

1. When the reduced section occurs in the critical buckling zone, the net section area shall be used to calculate $f_{c(net)}$, and $f_{c(net)} \leq F_c'$.
2. When the reduced section occurs outside the critical buckling zone, the gross section area shall be used to calculate $f_{c(gross)}$ and $f_{c(gross)} \leq F_c'$. In addition, the net section area shall be used to check for crushing, $f_{c(net)} \leq F_c^*$.

C3.6.4 Compression Members Bearing End to End

Compression design values parallel to grain (F_c^*) are applicable for bearing stresses occurring at the ends of compression members. See C3.10.1.

C3.6.5 Eccentric Loading or Combined Stresses

See C3.9 and C15.4.

C3.6.6 Column Bracing

Column bracing should be designed using accepted engineering practice. Design of bracing systems is beyond

the scope of the Specification; however, prescriptive recommendations are provided in Appendix A.

C3.6.7 Lateral Support of Arches, Studs, and Compression Chords of Trusses

Where roof joists or purlins are used between arches or compression chords, the column stability factor, C_p , should be calculated using the larger of:

- (i) the slenderness ratio, ℓ_{e1}/d_1 , based on distance between points of lateral support and the depth of the arch or chord (NDS Figure 3F).
- (ii) the slenderness ratio, ℓ_{e2}/d_2 , based on the distance between purlins or joists and the breadth of the arch or chord (NDS Figure 3F).

When continuous decking or sheathing is attached to the top of the arch or compression chord, it is common practice to assume that the slenderness ratio is the length between points of lateral support divided by the depth of the arch or chord, ℓ_{e1}/d_1 .

Use of the depth of the stud as the least dimension in calculating the slenderness ratio in determining the axial load-carrying capacity of sheathed or clad light-frame wall systems is a long standing practice. Experience has shown that wood structural panels, fiberboard, hardboard, gypsumboard, or other sheathing materials provide adequate lateral support of the stud across its thickness when properly fastened.

C3.7 Solid Columns

C3.7.1 Column Stability Factor, C_p

C3.7.1.2 In general, the effective length of a column is the distance between points of support that prevent lateral displacement of the member in the plane of buckling. It is common practice in wood construction to assume most column end conditions to be pin connected (translation fixed, rotation free) even though in many cases some partial rotational fixity is present. Where the end conditions in the plane of buckling are significantly different from the pinned assumption, recommended coefficients, K_e , for adjustment of column lengths are provided in Appendix G of the Specification.

As shown in Table G1 of NDS Appendix G, the recommended coefficients are larger than the theoretical values for all cases where rotational restraint of one or both ends of the column is assumed. This conservatism is introduced in recognition that full fixity is generally not realized in practice. The recommended values of K_e are the same as those used in steel design (125) except for the sixth case (rotation and translation fixed one end, rotation free and translation fixed other end) where a more conservative coefficient (20 percent larger than the theoretical value) is specified based on the ratio of theoretical/recommended value in the third case.

C3.7.1.4 The limitation on the slenderness ratio of solid columns to 50 precludes the use of column designs susceptible to potential buckling. The ℓ_e/d limit of 50 is comparable to the $K\ell/r$ limit of 200 (ℓ_e/d of 58) used in steel design (125).

Allowing a temporary ℓ_e/d ratio of 75 during construction is based on satisfactory experience with temporary bracing of trusses installed in accordance with truss industry standards (148); recognition that in most cases the assembly will carry only dead loads until load distributing and racking resisting sheathing elements are installed; and experience with a similar provision in steel design. In the latter regard, a $K\ell/r$ limit of 300 (ℓ_e/d of 87) is permitted during construction with cold-formed steel structural members (126). The critical buckling design load of a column with an ℓ_e/d ratio of 75 is approximately 45 percent that of a column with an equivalent cross-section and an ℓ_e/d ratio of 50.

C3.7.1.5 The column stability factor equation is applicable to all column slenderness ratios (ℓ_e/d). This equation provides a means of combining the compression design stress (F_c^*) with the critical buckling design stress (F_{cE}) to estimate an “effective” compression design value (30, 68, 81, 97, 191).

The parameter “c” was empirically established from the stress-strain relationship of very short columns (ℓ_e/d of 2.5). The column stability factor equation provides a good approximation of column strength if the short column tests adequately characterize the properties and non-uniformities of the longer columns (101). By empirically fitting the column stability factor equation to column strength data, estimates of “c” closely predicted test results at all ℓ_e/d ratios (189, 191, 190). A significant advantage of the methodology is that by selecting column test material representative of the non-uniform properties across the cross section and along the length that are associated with permitted grade characteristics such as knots, slope of grain and warp, the

combined effects of these variables on column behavior are included in the resultant value of “c” (190).

NDS equation 3.7-1 is formulated based on the assumption of a rectangular cross section and design property presentation in terms of adjusted stress (i.e. F_c^* and E_{min}'). An alternative format applicable for rectangular and non-rectangular sections where member properties are given in terms of adjusted stress times applicable section property (i.e. F_c^*A and $E_{min}'I$) is given as:

$$C_p = \frac{1 + \alpha_c}{2c} - \sqrt{\left[\frac{1 + \alpha_c}{2c} \right]^2 - \frac{\alpha_c}{c}} \quad (C3.7.1-1)$$

where:

$$\alpha_c = \frac{P_{cE}}{P_c^*} \quad (C3.7.1-2)$$

$$P_c^* = F_c^*A \quad (C3.7.1-3)$$

$$P_{cE} = \frac{\pi^2 E_{min}' I}{\ell_e^2} \quad (C3.7.1-4)$$

and

α_c = ratio of the critical column buckling resistance to reference compression resistance parallel to grain

P_{cE} = column buckling resistance, lbs

P_c^* = reference axial compression resistance parallel to grain multiplied by all applicable adjustment factors except C_p (i.e. F_c^*A), lbs

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_p , psi

A = area of cross section, in.²

E_{min}' = adjusted modulus of elasticity for beam stability and column stability calculations, psi

I = moment of inertia, in.⁴

ℓ_e = effective length of compression member between planes of lateral support, in.

For rectangular cross sections, column buckling resistance, P_{cE} , is based on the smaller of $E_{min}'I_1/\ell_{e1}$ and $E_{min}'I_2/\ell_{e2}$ where subscripts 1 and 2 denote principal axes between planes of lateral support.

C3.7.1.6 Continuous exposure to elevated temperature and moisture in combination with continuous application of full design loads is an example of a severe service condition. Particularly when such design environments are coupled with design uncertainties, such as end fixity

or stiffness of unsupported spliced connections, use of a reduced K_{cE} value should be considered. Included in such evaluations should be the possibility of eccentric application of the axial load and the need to design the member as a beam-column (see NDS 15.4).

C3.7.2 Tapered Columns

Analyses showed the general one-third rule (NDS Equation 3.7-3) was conservative for some end support conditions but unconservative for others (36). The use of a dimension taken at 1/3 the length from the smaller end underestimated the buckling load by 35 percent for a tapered column fixed at the large end and unsupported at the small end, and 16 percent for a tapered column simply supported (translation fixed) at both ends. Alternatively, the 1/3 rule was shown to overestimate the buckling load by 13 percent for a tapered column fixed at the small end and unsupported at the large end. These estimates were for a minimum to maximum diameter (dimension) ratio of 0.70. For these specific support conditions, NDS Equation 3.7-2 provides more realistic estimates of column strength. NDS Equation 3.7-3 remains applicable for other support conditions.

The one end fixed - one end unsupported or simply supported conditions referenced in NDS 3.7.2 correspond to the fifth and sixth buckling mode cases in NDS Appendix G. The condition of both ends simply supported corresponds to the fourth case. Values for the constant “a”, given under “Support Conditions” in NDS 3.7.2, are considered applicable when the ratio of minimum to maximum diameter equals or exceeds 1/3 (36).

The effective length factor, K_e , from NDS Appendix G is used in conjunction with the representative dimension (equivalent prism) when determining the stability factor, C_p , for tapered columns. It is to be noted that the actual compression stress parallel to grain, f_c , based on the minimum dimension of the column is not to exceed F_c^* .

C3.7.3 Round Columns

Round columns are designed as square columns of equivalent cross-sectional area and taper since the solid column provisions and equations in NDS 3.7.1 have been derived in terms of rectangular cross-sections. An alternative form of the equations in NDS 3.7.1, applicable to both rectangular and nonrectangular cross-sections, is provided in Section C3.7.1.5.

C3.8 Tension Members

C3.8.2 Tension Perpendicular to Grain

Average strength values for tension perpendicular to grain that are available in reference documents (181, 183) apply to small, clear specimens that are free of shakes, checks and other seasoning defects. Such information indicates that tension design values perpendicular to grain of clear, check- and shake-free wood may be considered to be about one-third the shear design value parallel to grain of comparable quality material of the same species (9). However, because of undetectable ring shakes, checking and splitting that can occur as a result of drying in service, very low strength values for the property can be encountered in commercial grades of lumber. For this reason, no sawn lumber tension design values perpendicular to grain have been published in the Specification. Cautionary provisions have been provided to alert the designer to avoid design configurations that induce tension perpendicular to grain stresses wherever possible. Connections where moderate to

heavy loads are acting through the tension side of a bending member (see NDS Table 12.5.1C, footnote 2) should be avoided. These connections should be designed to ensure that perpendicular to grain loads are applied through the compression side of the bending member, either through direct connections or top-bearing connectors.

If perpendicular to grain tension stresses are not avoidable, use of stitch bolts or other mechanical reinforcement to resist these loads should be considered. When such a solution is used, care should be taken to ensure that the reinforcement itself does not cause splitting of the member as a result of drying in service (140). Ultimately, the designer is responsible for avoiding tension perpendicular to grain stresses or for assuring that mechanical reinforcing methods are adequate.

Radial stresses are induced in curved, pitch tapered, and certain other shapes of glued laminated timber beams. Radial tension design values perpendicular to grain are given in NDS 5.2.2 and have been shown to be adequate by both test (23, 24, 113) and experience.

C3.9 Combined Bending and Axial Loading

C3.9.1 Bending and Axial Tension

Theoretical analyses and experimental results show the linear interaction equation for combined bending and tension stresses yields conservative results (189). It can be shown that the effect of moment magnification, which is not included in the equation, serves to reduce the effective bending ratio rather than increase it.

Where eccentric axial tension loading is involved, the moment associated with the axial load, $(6Pe)/(bd^2)$, should be added to the actual bending stress induced by the bending load when applying the interaction equation. The eccentricity, e , should carry the sign appropriate to the direction of eccentricity: positive when the moment associated with the axial load is increasing the moment due to the bending load and negative when it is reducing the moment.

Where biaxial bending occurs with axial tension, the interaction equation can be expanded to:

$$\frac{f_t}{F_t'} + \frac{f_{b1}}{F_{b1}^*} + \frac{f_{b2}}{F_{b2}^*} \leq 1.0 \quad (C3.9.2-1)$$

where the subscripts indicate the principle axes.

Reference bending design values, F_b , are not adjusted for slenderness, C_L , in NDS Equation 3.9-1 because the tension load acts to reduce the buckling stress and the combined stress is not the critical buckling condition. Critical buckling is checked separately using NDS Equation 3.9-2.

An alternative formulation of Equation C3.9.2-1 in terms of induced axial tension load and induced bending moment is given as:

$$\frac{T}{T'} + \frac{M_1}{M_1^*} + \frac{M_2}{M_2^*} \leq 1.0 \quad (C3.9.2-2)$$

T = induced axial tension load, lbs

T' = adjusted axial tension resistance (i.e. $F_t'A$), lbs

M_1 = induced bending moment in the strong axis, in.-lbs

M_1^* = reference strong axis moment resistance multiplied by all applicable adjustment factors except C_L (i.e. $F_{b1}^*S_1$), in.-lbs

M_2 = induced bending moment in the weak axis, in.-lbs

M_2^* = reference weak axis moment resistance multiplied by all applicable adjustment factors except C_L (i.e. $F_{b2}^*S_2$), in.-lbs

C3.9.2 Bending and Axial Compression

The interaction equation given in NDS 3.9.2 (NDS Equation 3.9-3) addresses effects of beam buckling and bending about both principal axes, and closely matches beam-column test data for in-grade lumber as well as similar earlier data for clear wood material (189, 187).

The ratio of actual (induced) to adjusted compression stress in NDS Equation 3.9-3 is squared based on tests of short beam-columns made from various species of 2×4 and 2×6 lumber (68, 193, 189). The squared relationship in Equation 3.9-3 is intended to be used for all wood member sizes.

The moment magnification adjustment for edgewise bending in NDS Equation 3.9-3 is $(1 - f_c/F_{cE1})$. This adjustment is consistent with similar adjustments for other structural materials, and is based on theoretical analysis, confirmed by tests of intermediate and long wood beam-columns (189, 187).

The moment magnification adjustment for flatwise bending in NDS Equation 3.9-3 is $[1 - (f_c/F_{cE2}) - (f_{b1}/F_{bE})^2]$. The first term, $(1 - f_c/F_{cE2})$, is consistent with the adjustment for edgewise bending discussed previously. The second term, $(f_{b1}/F_{bE})^2$, represents the amplification of f_{b2} from f_{b1} . This second term is based on theoretical analysis (187) and has been verified using beam-column tests made on clear Sitka spruce (99, 187). The biaxial bending calculations in NDS Equation 3.9-3 conservatively model cantilever and multi-span beam-columns subject to biaxial loads (192).

NDS Equation 3.9-4 checks an 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.

An alternative formulation of Equation C3.9-3 presented in terms of induced axial load and induced bending moment follows:

$$\left(\frac{P}{P'}\right)^2 + \frac{M_1}{M_1' \left(1 - \frac{P}{P_{cE1}}\right)} + \frac{M_2}{M_2' \left(1 - \frac{P}{P_{cE2}} - \frac{M_1}{M_{bE}}\right)} \leq 1.0 \quad (C3.9.2-3)$$

where:

P = induced axial compression load, lbs

P' = adjusted axial compression resistance (i.e. F_{cA}), lbs

M_1 = induced bending moment in the strong axis, in.-lbs

M_1' = adjusted strong axis moment resistance (i.e. $F_{b1}'S_1$), in.-lbs

M_2 = induced bending moment in the weak axis, in.-lbs

M_2' = adjusted weak axis moment resistance (i.e. $F_{b2}'S_2$), in.-lbs

P_{cE1} = critical column buckling resistance in the strong axis, lbs (see C3.7.1.5)

P_{cE2} = critical column buckling resistance in the weak axis, lbs (see C3.7.1.5)

M_{bE} = critical beam buckling moment (i.e. $2.4 E_{min}' I_2 / \ell_e$), in.-lbs where $E_{min}' I_2$ represents adjusted weak axis stiffness for beam stability and column stability calculations

and

$$\frac{P}{P_{cE2}} + \left(\frac{M_1}{M_{bE}}\right)^2 \leq 1.0 \quad (C3.9.2-4)$$

where:

$$P < P_{cE1} = \frac{\pi^2 E_{min}' I_1}{\ell_{e1}^2} \quad (C3.9.2-5)$$

$$P < P_{cE2} = \frac{\pi^2 E_{min}' I_2}{\ell_{e2}^2} \quad (C3.9.2-6)$$

$$M < M_{bE} = \frac{2.4 E_{min}' I_2}{\ell_e} \quad (C3.9.2-7)$$

For additional equations involving combined bending and axial compression including eccentric axial compression loading, see C15.4.

C3.10 Design for Bearing

C3.10.1 Bearing Parallel to Grain

Where end-grain bearing is a design consideration, the actual compression stress parallel to grain, f_c , shall not exceed the compression design value parallel to grain, F_c^* . For purposes of this section, the term “actual compressive bearing stress parallel to grain” and the term “compression stress parallel to grain,” f_c , are synonymous.

Examples of end-grain bearing configurations are end-to-end compression chord segments laterally supported by splice plates, butt end-bearing joints in individual laminations of mechanically laminated truss chords, roof-tied arch heel connections, notched chord truss heel joints, and columns supporting beams. Where the actual compression stress parallel to grain at the point of bearing is less than or equal to 75 percent of the compression design value parallel to grain ($f_c \leq 0.75 F_c^*$), direct end-to-end bearing of wood surfaces is permitted provided that abutting end surfaces are parallel and appropriate lateral support is provided. The required use of a metal bearing plate or equivalently durable, rigid, homogeneous material in highly loaded end-to-end bearing joints ($f_c > 0.75 F_c^*$) is to assure a uniform distribution of load from one member to another.

C3.10.2 Bearing Perpendicular to Grain

Ignoring any non-uniform distribution of bearing stress that may occur at the supports of a bending member as a result of the deflection or curvature of that member under load is long standing design practice.

C3.10.3 Bearing at an Angle to Grain

NDS Equation 3.10-1 for calculating the compressive stress at an angle to grain was developed from tests on Sitka spruce and verified for general applicability by tests on other species (184, 54, 59). The equation applies when the inclined or loaded surface is at right angles to the direction of load. The equation is limited to F_c' when the angle between direction of grain and direction of load, θ , is 0° and $F_{c\perp}'$ when this angle is 90° . Stresses on both inclined surfaces in a notched member should be checked if the limiting case is not apparent.

C3.10.4 Bearing Area Factor, C_b

Provisions for increasing reference compression perpendicular design values for length of bearing are based on the results of test procedures in ASTM D 143 (5) which involve loading a two-inch wide steel plate bearing on a two-inch wide by two-inch deep by six-inch long specimen. Research at the USDA Forest Products Laboratory on proportional limit stresses associated with bolt and washer loads showed that the smaller the width of the plate or bearing area relative to the length of the test specimen, the higher the proportional limit stress (146, 178). Early research conducted in Australia and Czechoslovakia confirmed the nature and magnitude of the bearing length effect (178).

The effect of length of bearing is attributed to the resisting bending and tension parallel to grain strengths in the fibers at the edges of the bearing plate (84, 178). Because of the localized nature of the edge effect, the contribution provided decreases as the length of the area under compressive load increases. When the bearing plate covers the entire surface of the supporting specimen (full bearing), test values will be lower than those obtained in the standard two-inch plate test. For the case of complete surface or full bearing (bearing length equals supporting member length), such as may occur in a pressing operation, compression perpendicular to grain is approximately 75% of the reference compression perpendicular to grain design value. Deformation will also exceed that associated with the standard test.

Note that potential buckling perpendicular to grain is a design consideration that is not evaluated as part of the ASTM D143 (5) test procedures. One method of checking for buckling perpendicular to grain would be to use the provisions for column buckling parallel to grain in NDS 3.7.1 with mechanical properties from approved sources.

Bearing adjustment factors are useful in special cases such as highly loaded washers, metal supporting straps or hangers on wood beams, highly loaded foundation studs bearing on wood plates and crossing wood members. See C4.2.6 for discussion of deformation occurring in this support condition relative to metal or end-grain bearing on side or face-grain.

C4 SAWN LUMBER

C4.1 General

C4.1.1 Scope

The design requirements given in Chapters 1 through 3 of the Specification are applicable to sawn lumber except where indicated otherwise. Chapter 4 of the Specification contains provisions which are particular to sawn lumber.

C4.1.2 Identification of Lumber

C4.1.2.1 The design provisions of the Specification applicable to sawn lumber are based on (i) use of lumber that displays the official grading mark of an agency that has been certified by the Board of Review of the American Lumber Standard Committee (ALSC), established under the U.S. Department of Commerce's *Voluntary Product Standard PS 20* (152); and (ii) use of design values tabulated in the *NDS Supplement—Design Values for Wood Construction (NDS Supplement)* are a compendium of reference design values published by grading rules-writing agencies approved by the ALSC Board of Review (152). Those agencies publishing approved grading rules and design values are given in the *Design Values Supplement* to the Specification under "List of Sawn Lumber Grading Agencies." It is the responsibility of the designer to assure that the design values given in the NDS Supplement are applicable to the material so identified. If design values other than those tabulated in the NDS Supplement are used, it is the designer's responsibility to assure that the reliability and adequacy of the assignments are such that they may be used safely with the design provisions of the Specification.

The requirement that glued lumber products bear a distinct grade mark indicating that the joint integrity is subject to qualification and quality control clarifies that the bond strength of the joint itself is to be monitored on a continuous basis under an inspection program.

C4.1.3 Definitions

C4.1.3.2 Categories and grades of "Dimension" lumber are standardized under the National Grading Rule for Softwood Dimension Lumber which was authorized by the American Softwood Lumber Standard PS20 (152). The rule provides standard use categories, grade names, and grade descriptions. The National Grading Rule includes

allowable knot sizes based on the strength ratio concept. Under this concept, the effect of a knot or other permitted strength reducing characteristic is expressed as the ratio of the assumed strength of the piece containing the characteristic to the strength of clear, straight-grain wood of the same species (8).

Grades established under the National Grading Rule are:

Structural Light Framing 2"–4" thick, 2"–4" wide

Select Structural

No. 1

No. 2

No. 3

Light Framing 2"–4" thick, 2"–4" wide

Construction

Standard

Utility

Studs 2"–4" thick, 2"–6" wide

Stud

Structural Joists & Planks 2"–4" thick, 5" and wider

Select Structural

No. 1

No. 2

No. 3

Design values for dimension lumber are based on in-grade tests of full-size pieces. Design values for Structural Light Framing and Structural Joists and Planks are consolidated under the common grade names (Select Structural, No. 1, No. 2, and No. 3) and separate width adjustments or values by width are provided (see NDS Supplement Tables 4A and 4B). There has been no change in the visual descriptions or maximum size of knots and other characteristics permitted in each width class of the grades established under the National Grading Rule.

C4.1.3.3 "Beams and Stringers" are uniformly defined in certified grading rules as lumber that is 5" (nominal) or more in thickness, and width 2" or more greater than the thickness. Such members, for example 6×10, 6×12, 8×12, 8×16, and 10×14, are designed for use on edge as bending members. Grades for which design

values are given in this Specification (NDS Supplement Table 4D) are:

- Select Structural
- No. 1
- No. 2

C4.1.3.4 “Posts and Timbers” are defined as lumber that is 5” (nominal) or more in thickness and width not more than 2” greater than thickness. These members, such as 6×6, 6×8, 8×10, and 12×12, are designed to support axial column loads. Grades of lumber in this classification are the same as those for “Beams and Stringers.” Posts and Timbers also may be used as beams; however, other grades and sections may be more efficient where strength in bending is a major consideration.

C4.1.4 Moisture Service Condition of Lumber

Design values tabulated in the Specification for sawn lumber apply to material surfaced in any condition and used in dry conditions of service. Such conditions are those in which the moisture content in use will not exceed a maximum of 19 percent. Adjustment factors, C_M , are provided in NDS Supplement Tables 4A through 4F for uses where this limit will be exceeded for a sustained period of time or for repeated periods.

Applications in which the structural members are regularly exposed directly to rain and other sources of moisture are typically considered wet conditions of service. Members that are protected from the weather by roofs or other means but are occasionally subjected to wind-blown moisture are generally considered dry (moisture content 19 percent or less) applications. The designer has final responsibility for determining the appropriate moisture content basis for the design.

Design values tabulated for Southern Pine timbers and Mixed Southern Pine timbers in NDS Supplement Table 4D have already been adjusted for use in wet service conditions. These values also apply when these species are used in dry service conditions.

C4.1.5 Lumber Sizes

C4.1.5.1 The minimum lumber sizes given in NDS Supplement Table 1A are minimum-dressed sizes established in the American Softwood Lumber Standard, PS 20 (152).

C4.1.5.2 Dry net sizes are used in engineering computations for dimension lumber surfaced in any condition. When lumber is surfaced in the Green condition, it is oversized to allow for shrinkage (152).

C4.1.5.3 Beams and Stringers and Posts and Timbers are manufactured in the Green condition to standard Green dimensions (152). The reference design values for such lumber, which are applicable to dry conditions of service, include adjustments for the effects of shrinkage. Standard Green sizes, therefore, are to be used in engineering computations with these grades.

C4.1.6 End-Jointed or Edge-Glued Lumber

Design values tabulated in the NDS Supplement apply to end-jointed lumber of the same species and grade as unjointed sawn lumber when such material is identified by the grademark or inspection certificate of an approved agency (see C4.1.2.1). This identification indicates the glued product is subject to ongoing quality monitoring, including joint strength evaluation, by the agency.

End-jointed, face-glued and edge-glued lumber may be used interchangeably with sawn lumber members of the same grade and species. The limitation on the use of finger-jointed lumber marked “STUD USE ONLY” or “VERTICAL USE ONLY” to those applications where any induced bending or tension stresses are of short duration is a provision to minimize possible joint creep associated with long-term loads. Bending and tension stresses associated with wind loads and seismic loads are examples of short duration stresses permitted in finger-jointed lumber marked for “STUD USE ONLY” or “VERTICAL USE ONLY.”

C4.1.7 Resawn or Remanufactured Lumber

Material that has been regraded after resawing qualifies for design values tabulated in the Supplement only when identified by the grademark or inspection certificate of an approved agency (see C4.1.2.1).

C4.2 Reference Design Values

C4.2.1 Reference Design Values

Design values tabulated in NDS Supplement Tables 4A through 4F have been taken from grading rules that have been certified by the Board of Review of the American Lumber Standard Committee as conforming to the provisions of the American Softwood Lumber Standard, PS 20 (152). Such grading rules may be obtained from the rules writing agencies listed in the NDS Supplement. Information on stress-rated board grades applicable to the various species is available from the respective grading rules agencies.

C4.2.2 Other Species and Grades

Where design values other than those tabulated in the Specification are to be used, it is the designer's responsibility to assure the technical adequacy of such assignments and the appropriateness of using them with the design provisions of the Specification (see C4.1.2.1).

C4.2.3 Basis for Reference Design Values

C4.2.3.2 Visually Graded Lumber

Dimension

In 1977, the softwood lumber industry in North America and the U.S. Forest Products Laboratory began a testing program to evaluate the strength properties of in-grade full-size pieces of visually-graded dimension lumber made from most commercially important species in North America (65). The testing program, conducted over an eight year period, involved the destructive testing of over 70,000 pieces of lumber from 33 species or species groups. The test method standard, ASTM D4761, covers the mechanical test methods used in the program (14). The standard practice, ASTM D1990, provides the procedures for establishing design values for visually graded dimension lumber from test results obtained from in-grade test programs (7).

Design values for bending, F_b , tension parallel to grain, F_t , compression parallel to grain, F_c , and modulus of elasticity, E , for 14 species or species combinations listed in Tables 4A and 4B of the NDS Supplement are based on in-grade test results. Further, the grade and size models developed under ASTM D1990 have been employed to establish grade and size relationships for those species whose index strengths are established by D245 methods. All design values for shear parallel to grain, F_v , and com-

pression perpendicular to the grain, $F_{c\perp}$, in these tables are based on ASTM D245 provisions (8).

Timbers

Design values and adjustment factors for size, wet service, and shear stress given in NDS Supplement Table 4D for Beams and Stringers and Posts and Timbers are based on the provisions of ASTM D245 (8).

Decking

Design values for Decking in NDS Supplement Table 4E are based on ASTM D245 provisions except for the wet service factor, C_M , for F_b which is based on ASTM D1990. Reference bending design values, F_b , in Table 4E for all species and species combinations except Redwood are based on a four-inch thickness. A 10 percent increase in these values applies when two-inch decking is used (see C_F adjustment factor in the Table).

C4.2.3.3 Machine Stress Rated (MSR) Lumber and Machine Evaluated Lumber (MEL)

Design values for F_b , F_t , F_c , and E given in NDS Supplement Table 4C for mechanically graded dimension lumber apply to material that meets the qualification and quality control requirements of the grading agency whose grademark appears on the piece. Stiffness-based stress-rating machines are set so that pieces passing through the machine will have the average E desired. For these machines, values of F_b are based on correlations established between minimum bending strength for lumber loaded on edge and E . Similarly, F_t and F_c values are based on test results for lumber in each F_b - E grade. Density-based grading machines operate under similar principles, using various density-based algorithms as the basis for grading decisions. For both machine types, machine settings are monitored and routinely verified through periodic stiffness and strength testing. Mechanically graded lumber also is required to meet certain visual grading requirements which include limitations on the size of edge knots and distorted grain on the wide face. Such limitations, expressed as a maximum proportion of the cross-section occupied by the characteristics, generally range from 1/2 to 1/6 depending on the level of F_b .

Machine Stress Rated (MSR) lumber is material that is categorized in classes of regularly increasing strength (F_b , F_t , and F_c) and E assignments. As F_b values increase, F_t values increase at a greater rate, starting from 0.39 of the F_b value for the 900f grade to 0.80 of the F_b value for the 2400f and higher grades. Alternatively, F_c values increase at a lower rate than F_b values, starting from 1.17 of the F_b value for the 900f grade to 0.70 of the F_b value

for the 3300f grade. F_b , F_t , and E values for MSR lumber in NDS Supplement Table 4C are essentially the same as those published in the 1986 edition. Previously, F_c values were taken as 80 percent of the corresponding F_b value. As noted, these assignments now vary depending on level of F_b .

Design values for Machine Evaluated Lumber (MEL) are characterized by several different levels of E , F_t , or F_c for each level of F_b rather than assignment of qualifying material to specific stress classes, each of which has a generally unique assignment for each property. The MEL approach allows a greater percentage of total lumber production from a mill to be mechanically rated than is possible under the MSR classification system.

C4.2.4 Modulus of Elasticity, E

Design values for Modulus of Elasticity, E , are estimates of the average values for the species and grade of material. Reference modulus of elasticity for beam and column stability, E_{min} , is based on the following equation:

$$E_{min} = \frac{E(1 - 1.64COV_E)(1.03)}{1.66} \quad (C4.2.4-1)$$

where:

E = reference modulus of elasticity

1.03 = adjustment factor to convert E values to a pure bending basis

1.66 = factor of safety

COV_E = coefficient of variation in modulus of elasticity (see NDS Appendix F)

E_{min} represents an approximate 5% lower exclusion value on pure bending modulus of elasticity, a 1.03 adjustment factor to convert from an apparent bending E at a span-to-depth ratio of 17:1 to a true bending E , and a 1.66 factor of safety. For more discussion, see NDS Appendix D.

C4.2.5 Bending, F_b

C4.2.5.1 When reference F_b values for dimension grades are applied to members with the load applied to the wide face, the flat use factor, C_{fu} , is to be used (see C4.3.7).

C4.2.5.4 Grade requirements for Beams and Stringers do not consider the effects of allowable knots and other permitted characteristics on the bending strength of the member under loads applied to the wide face. Therefore, reference bending design values, F_b , for Beams and Stringers in NDS Supplement Table 4D, used when checking

stresses where loads are applied on the wide face, should be adjusted by the applicable size factor in NDS Supplement Table 4D. Posts and Timbers are graded for bending in both directions and can be used in biaxial bending design situations.

C4.2.6 Compression Perpendicular to Grain, $F_{c\perp}$

Reference compression design values perpendicular to grain in the 1977 and earlier editions of the Specification were based on proportional limit stresses and were adjusted for load duration. This practice changed when ASTM D245 provisions were revised to recognize compression perpendicular to grain as a serviceability limit state where the property is used as a measure of bearing deformation (8). Since 1982, lumber $F_{c\perp}$ values referenced in the Specification have been based on a uniform 0.04-inch deformation level for the condition of a steel plate on wood bearing condition. Such values are not adjusted for load duration.

The change in the basis of compression design values perpendicular to grain was an outgrowth of the introduction of ASTM D2555 in 1966. This standard gave new clear wood property information for western species and prescribed strict criteria for assignment of properties to combinations of species (see C4.2.3.2). Implementation of this information and the grouping criteria through ASTM D245 in 1971 resulted in a significant reduction in the $F_{c\perp}$ design value for a commercially important species group. The reduction caused bearing stress to become the limiting design property for the group in truss and other structural applications even though lumber of the group in these uses had performed satisfactorily at the previous higher bearing stress level for over 25 years.

Subsequent evaluation indicated that bearing perpendicular to the grain loads are not associated with structural failure and that deformation levels at proportional limit stresses could vary 100 percent between species in the standard ASTM D143 test. This test consists of loading a 2-inch wide steel plate bearing on the middle of a 2-inch by 2-inch by 6-inch long wood specimen (5). It was concluded that a uniform deformation limit was the preferred basis for establishing design loads concerned with bearing perpendicular to the grain. New methodology was developed to enable the stress at any deformation level to be estimated for any species based on its proportional limit stress (26, 27). This methodology was coupled with field experience to establish a deformation limit of 0.04 inches in the standard 2-inch specimen as an appropriate design stress base for applied loads of any duration. Stresses at 0.04-inch deformation for individual species were sub-

sequently published in ASTM D2555 and provisions for basing compression design values perpendicular to grain on a deformation limit were introduced into ASTM D245.

In view of the outward load redistribution that occurs through the thickness of a member not subjected to a uniform bearing load along its length, and taking into account the effects of bearing deformation on the structure, establishment of a deformation limit state in terms of strain rate (deformation divided by member thickness) was not considered appropriate. On the basis of field experience, bearing stresses and deformations derived from the standard test of steel plate on 2 inch deep wood member are judged applicable to all lumber sizes. For the same stress, deformation of a joint consisting of two wood members both loaded perpendicular to grain will be approximately

2.5 times that of a metal to wood joint. The $F_{c\perp}$ values given in the 1982 edition of the Specification and continued in the present edition are about 60 percent greater than the proportional limit–normal load based values published in earlier editions but are applicable to wind, earthquake, snow and other load durations without adjustment.

The equation given in NDS 4.2.6 for adjusting reference $F_{c\perp}$ values to a 0.02-inch deformation limit is based on regression equations relating proportional limit mean stress to deformation at the 0.04 and the 0.02 levels (27). Use of this reduced compression design value perpendicular to grain may be appropriate where bearing deformations could affect load distribution or where total deflections of members must be closely controlled. Bearing deformation is not a significant factor in most lumber designs.

C4.3 Adjustment of Reference Design Values

C4.3.1 General

Applicable adjustment factors for sawn lumber are specified in Table 4.3.1 of the Specification.

C4.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2.

C4.3.3 Wet Service Factor, C_M

The wet service reduction value, C_M , for F_b , F_t , F_c , and E in NDS Supplement Tables 4A and 4B are based on provisions of ASTM D1990 (7). For F_v and $F_{c\perp}$, the values of C_M , are based on ASTM D245. The wet service factors account for the increase in cross-section dimensions associated with this exposure.

C4.3.4 Temperature Factor, C_t

See C2.3.3.

C4.3.5 Beam Stability Factor, C_L

See C3.3.3.

C4.3.6 Size Factor, C_F

C4.3.6.1 Design values for F_b , F_t , and F_c in NDS Supplement Table 4A for all species and species combinations are adjusted for size using the size factors, C_F , referenced at

the beginning of the table. These factors and those used to develop the size specific values given in NDS Supplement Table 4B for certain species combinations are based on the adjustment equation for geometry given in ASTM D1990 (7). This equation, based on in-grade test data, accounts for differences in F_b , F_t , and F_c related to width and for differences in F_b and F_t related to length (test span). Reference values in Tables 4A and 4B for F_b and F_t are based on the following standardized lengths:

Width, in.	Length, ft
2 to 6	12
8 to 10	16
12 and wider	20

For constant length, the ASTM D1990 size equation provides for significantly greater reductions in bending design values, F_b , as width increases than comparable previous adjustments for this property. Width adjustments for tension design values parallel to grain, F_t , and compression design values parallel to grain, F_c , in the equation are applicable. Additionally, the modification of F_b and F_t for length is presented in the D1990 equation. Based on the total conservatism of these combined adjustments relative to past practice, use of design values in NDS Supplement Tables 4A and 4B for any member span length is considered appropriate.

C4.3.6.2 Bending design values for Beams & Stringers and Posts & Timbers in NDS Supplement Table 4D apply to a 12 inch depth. The NDS size factor equation for adjusting these values to deeper members is based on the formula given in ASTM D245.

C4.3.6.3 Beams of circular cross section (see C4.3.6.2).

C4.3.6.4 Values of F_b referenced for decking in NDS Supplement Table 4E are for members 4 inches thick. The increases of 10 and 4 percent allowed for 2 inch and 3 inch decking are based on the NDS size equation in 4.3.6.2.

C4.3.7 Flat Use Factor, C_{fu}

Adjustment factors for flat use of bending members are based on the 1/9 power size equation discussed in C4.3.6.2 and C4.3.6.4. Relative to the test results that are available, the ASTM D245 equation gives conservative C_{fu} values. The flat use factor, C_{fu} , is to be used cumulatively with the size factor, C_F . Since C_{fu} is equal to or greater than 1.0 for dimension lumber 2" to 4" thick, its use is permitted but not required in accordance with Section 4.3.7.1. For Beams and Stringers, since C_{fu} is equal to or less than 1.0, its use is required. Posts and Timbers may use the design values in Table 4D without the C_{fu} adjustment factors ($C_{fu} = 1.0$).

C4.3.8 Incising Factor, C_i

Incising involves making shallow, slit-like holes parallel-to-grain in the surfaces of refractory wood species to obtain deeper and more uniform penetration of preservatives. Treating standards require incising to improve pressure treatment of thin sapwood species such as Douglas fir, Spruce and Hemlock, which may have heartwood surfaces or tend to be resistant to side penetration of preservative solutions.

The effect of the incising process has been found to reduce the strength properties of wood (73, 74, 105). More recent work on nominal 2x4 lumber of multiple species having either 660 or 880 incisions/ft² has shown that incising effects are dependent on: the number of incisions (density) per square foot of surface area; the depth, width and length of individual incisions; and the damage beneath the boundaries of the incisions (199, 200). The incising adjustment factors for E , F_b , F_t , F_v , and F_c given in Table 4.3.8 of the Specification are limited to dimension lumber using patterns in which the incisions are not deeper than 0.4 inches and no more than 1,100 per square foot in number.

To verify the current practices of incising across the Western U.S., a survey was recently conducted by Western Wood Preservers Institute and Oregon State University (206). It confirmed that the current incising machines used in Pacific Northwest treating plants produced average incising densities of 377 incisions/ft² with a standard deviation of 145 incisions/ft². The 16 plants responding to the survey used incision densities between 148-638 incisions/ft². Incision depth ranged from 0.04-0.24 inches and averaged 0.14 inch with a standard deviation of 0.06 inch.

The incising effect on larger member sizes is generally less than nominal 2x4 size lumber due to the larger cross-section and the common practice on timbers of using lower incision densities (<800/ft²).

Alternatively, incising factors for specific incising patterns, in conjunction with dimension lumber or timber sizes may be obtained from the company providing the incising and reported in their design documents. These incising factors may either be determined by test or by calculation using reduced section properties, and shall account for damage to wood around edges and below the bottom of incisions (199, 200).

For larger sizes, such as timbers, reviews of early testing of timbers and railway ties (105, 199) indicated that a slight decrease in strength properties for timbers was expected, but in some cases no strength reductions were reported.

For timber sizes with thicknesses larger than 5 inches, the effects of incising are small to minimal and C_i can be assumed to vary from 0.95 to 1.0 depending on timber species and size, incision depth and density, and incision damage to wood around edges and below the bottom of incisions. Thus, reductions provided in Table 4.3.8 are usually not applied to larger members such as solid sawn timbers.

C4.3.9 Repetitive Member Factor, C_r

The 15 percent repetitive member increase in reference bending design values, F_b , for lumber 2 to 4 inches thick is based on provisions in ASTM D245 (8) and D6555 (19). It is based on the increase in load-carrying capacity and stiffness obtained when multiple framing members are fastened together or appropriately joined by transverse load distributing elements. Such an increase has been demonstrated by both analysis and test (28, 107, 149, 194). It reflects two interactions: load-sharing or redistribution of load among framing members and partial composite action of the framing member and the covering materials (149). Application of the C_r adjustment requires no assumption as to which of the two types of interaction is involved or predominates. A C_r value of 15 percent is generally considered to be conservative for sawn lumber assemblies (111, 177, 179).

The criteria for use of the repetitive member increase are three or more members in contact or spaced not more than 24 inches and joined by transverse load distributing elements such that the group of members performs as a unit rather than as separate pieces. The members may be any piece of dimension lumber loaded in bending, including studs, rafters, truss chords, and decking, as well as joists.

The repetitive member increase also applies to an assembly of three or more essentially parallel members of equal size and of the same orientation which are in direct contact with each other (28). In this case the transverse elements may be mechanical fasteners such as nails, nail gluing, tongue-and-groove joints, or bearing plates. The required condition is that the three or more members act together to resist the applied moment.

C4.3.10 Column Stability Factor, C_p

See C3.7.1.

C4.3.11 Buckling Stiffness Factor, C_T

See C4.4.2.

C4.3.12 Bearing Area Factor, C_b

See C3.10.4.

C4.3.13 Pressure-Preservative Treatment

The provision in the NDS for use of reference design values with lumber that has been preservative treated (170, 169, 168, 171, 172, 175) is applicable to material that has been treated and redried in accordance with AWP Standards. In AWP Standards, the maximum temperature for kiln drying material after treatment is 165°F (22).

C4.3.14 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C4.3.15 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C4.3.16 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

C4.4 Special Design Considerations

C4.4.1 Stability of Bending Members

C4.4.1.1 Bending design values, F_b , given in NDS Supplement Tables 4A through 4F are based on a bending member having a compression edge supported throughout its length or having a depth to breadth ratio of one or less. When these conditions do not exist, F_b values are to be adjusted by the beam stability factor, C_L , calculated in accordance with the procedures of NDS 3.3.3. As an alternative method, bracing rules provided in NDS 4.4.1.2 are an acceptable method for providing restraint to prevent lateral displacement or rotation of lumber bending members (181).

C4.4.1.2 Sheathing, subflooring, or decking attached with two or more fasteners per piece provide acceptable edge restraint for a joist, rafter, or beam loaded through these load distributing elements. The requirement for bridging in the form of diagonal cross bracing or solid blocking in NDS 4.4.1.2(d) and the requirement for both edges to be supported in NDS 4.4.1.2(e) address: (i) redistribution of concentrated loads from long span members to adjacent members, and (ii) localized eccentricities due

to cupping or twisting of deep members as a result of drying in service. Intermittent bridging specified in NDS 4.4.1.2(d) is not required in combination with tension and compression edge bracing specified in NDS 4.4.1.2(e).

The approximate rules of NDS 4.4.1.2(c) are equivalent to the beam stability provisions of NDS 3.3.3.3. For larger depth to breadth ratios the NDS 4.4.1.2 bracing rules are more restrictive than provisions of NDS 3.3.3.3. For smaller ratios the NDS 4.4.1.2 bracing rules are less restrictive, with the difference between effective bending stress based on the two methods increasing as F_b increases and E decreases.

C4.4.1.3 Tests of heavily stressed biaxial beam-columns showed that the bracing members could buckle as a result of the combination of loads applied directly on the bracing member and the loads induced by the beam-column as it buckles (147). Bracing members providing lateral support to a beam-column will typically have only one edge braced (such as a sheathed purlin bracing a rafter). The bracing member should have sufficient capacity to carry the additional compression load produced by the beam-column as it tends to buckle.

C4.4.2 Wood Trusses

C4.4.2.1 These provisions recognize the contribution of wood structural panel (WSP) sheathing to the buckling resistance of compression truss chords (in the plane of the chord depth). Quantification of the increase in chord buckling resistance from plywood sheathing was based on research (53, 55) involving stiffness tests of sheathed 2×4 members, nail slip tests, use of existing methodology for estimating the nail slip modulus of combinations of materials (155, 159), and application of a finite element analysis program for layered wood systems (149). It was found that the sheathing contribution increases with decrease in modulus of elasticity of the chord, with increase in span, and with increase in fastener slip modulus. Effects of plywood thickness and chord specific gravity were found to be of lesser significance.

While research was conducted using plywood, the application applies to all WSP including OSB.

The difference between the two K_m factors reflects the effect of drying on the nail load-slip modulus. The equations apply to chord lengths up to 96 inches, 2×4 or smaller chords in trusses spaced 24 inches or less, and 3/8 inch or thicker WSP nailed to the narrow face of the chord using recommended schedules (38).

The analyses on which the equations are based assumed nails adjacent to joints between panel edges were located one inch from the panel edge, a chord specific gravity (oven dry volume basis) of 0.42 and an open joint without H clips between sheathing panels. Clips were estimated to increase the C_T factor by 5 percent (53).

Because the buckling stiffness factor decreases with increase in chord modulus of elasticity, the 1977 equations were based on the 5 percent exclusion value of E for the visually graded lumber species and grade having the highest reference design value. The 5 percent value was used because this is the basis for the E value used to establish the Euler column buckling load. It should be noted that the decrease in the relative contribution of sheathing that occurs as chord E increases above the 5 percent exclusion level is more than offset by the increase in the E of the chord itself.

C4.4.3 Notches

Prior to 1977, the Specification provided for the use of the net section at the notch for determining the bending strength of a notched bending member. This provision was based on early research which indicated that use of the net section at the notch was a sufficiently conservative design basis for commercial grades of sawn lumber (184, 181). It was recognized even at that time that stress concentra-

tions at the corners of the notch caused lower proportional limit loads and caused failure to begin at lower loads than those expected from an unnotched bending member having a depth equal to the net depth of the notched bending member (184, 181).

In the 1977 edition, as a result of field experience and new research related to crack propagation, the use of the net section procedure for determining induced bending moment in notched bending members was discontinued and specific notch limitations were established for different bending member sizes. These new provisions were continued in the 1986 and 1991 editions. The field performance history considered included: (i) large bending members end-notched to the quarter points of the span which exhibited splitting and tension perpendicular to grain separations at relatively low loads; and (ii) the long record of satisfactory performance of light-frame construction joists notched using good practice recommendations. Fracture mechanics research also confirmed and quantified the propensity of cracks to develop at square-cornered notches at relatively low bending loads (92, 91, 132). Narrow slit notches (3/32 inch long) were found to cause greater strength reductions than wide (greater than two inches long) notches of the same depth. The interaction of size and crack propagation has been characterized, with crack initiation increasing in proportion to the square root of the bending member depth for a given relative notch depth and constant induced bending and shear stress (183).

C4.4.3.1 Tension perpendicular to grain stresses occur with shear stresses at end notches to make a bending member more susceptible to splitting at the corner of such notches. The limitation on end notches in sawn lumber bending members to 1/4 or less the bending member depth is a good practice recommendation that also reflects experience and the effects of shrinkage stresses.

C4.4.3.2 The allowance of notches on both the tension and compression sides of two and three inch thick sawn lumber bending members up to 1/6 the depth of the member in the outer thirds of a single span is consistent with good practice recommendations for light-frame construction (180). The satisfactory field performance of notched joists meeting these limitations, without use of the net section at the notch to determine actual stress, is attributed in part to the fact that reference bending design values (F_b) for the dimension grades of lumber already include section reductions for edge knots ranging from 1/6 to 1/2 the depth of the member. The restriction on interior notches in the tension side of nominal four inch and thicker sawn lumber bending members is based on experience with larger bending members and fracture mechanics analyses, as well as consideration of the shrinkage stresses that occur in such members when seasoning in service. Such stresses

contribute to the perpendicular to grain stress conditions existing at the notch corners.

C4.4.3.3 The design provisions for shear in notched bending members given in NDS 3.4.3 include a magnification factor to account for tension perpendicular to grain stresses that occur with shear stresses making a bending member more susceptible to splitting at the corner of such notches.



C5 STRUCTURAL GLUED LAMINATED TIMBER

C5.1 General

Structural glued laminated timber, consisting of multiple layers of wood glued together with the grain of all layers approximately parallel, began its growth as a significant structural material in the United States in the 1930's. Technology developed in the formulation and use of casein glues to fabricate structural members in wood aircraft during and after World War I was extended to the construction of larger structural framing members used in buildings (181). The resistance of these glues to elevated relative humidities coupled with the use of pressing systems that could provide continuous pressure to all glue lines enabled the manufacture of large beams, arches and other curved shapes with assured durability. The subsequent development of resorcinol and other synthetic resin glues with high moisture resistance expanded the uses of structural glued laminated timber to bridges, marine construction and other applications involving direct exposure to the weather.

Glued laminated members are made of dry lumber laminations in which the location and frequency of knots and other strength reducing characteristics can be controlled. The result is a structural product in which splits, checks and loosening of fasteners associated with drying in service are greatly reduced and relatively high strength is achieved.

The early development of design values for structural glued laminated timber paralleled that for visually graded lumber. In 1934, methods published in the U.S. Department of Agriculture's Miscellaneous Publication 185 for the grading and determination of working stresses for structural timbers (167) were also applied to structural glued laminated timber. Under these procedures, strength values for small, clear, straight-grained wood were reduced for load duration, variability, size and factor of safety to basic stresses; and then these stresses were further reduced to account for the effects of knots, slope of grain and other characteristics permitted in the grade of lumber being used as laminations. These design values were assigned by the manufacturers to the species and grades of structural glued laminated timber being produced.

The earliest comprehensive procedures for establishing design values that were specifically developed for structural glued laminated timber were published in 1939

in U.S. Department of Agriculture Technical Bulletin 691 (166). These procedures provided for the use of lower grades of lumber in the inner laminations than in the outer laminations. A simplified method of establishing design values from basic stresses also was given which was based on use of only two grades of lumber: one allowing knots up to one-fourth the width of the piece and one allowing up to one-eighth the width of the piece.

Design procedures for structural glued laminated timber were codified as national standards of practice in 1943 as part of the War Production Board's Directive No. 29 (153) and then in 1944 as part of the first edition of the National Design Specification (96). Design values established in the first edition were the same as those for the grade of sawn lumber used (based on the procedures in Miscellaneous Publication 185) except that increases for seasoning were permitted in compression parallel to grain and for all properties except shear parallel to grain when lumber two inches or less in thickness was used. In addition, increases were permitted for constructions in which knot limitations were twice as restrictive as those applicable to inner laminations. The procedures published in 1939 in Technical Bulletin 691 also were allowed as alternative methods.

The regional lumber rules writing agencies used the new Forest Products Laboratory procedures (49) to establish specifications for the design and fabrication of structural glued laminated lumber which provided design values for various species and lamination grade combinations. Design values established by these regional agencies were published in the Specification from 1951 through the 1968 editions.

A national consensus product standard covering minimum requirements for the production of structural glued laminated timber was promulgated as Commercial Standard CS253-63 by the U.S. Department of Commerce in 1963 (133).

In 1970, the American Institute of Timber Construction (AITC) assumed responsibility for developing laminating combinations and related design values for structural glued laminated timber. Beginning with the 1971 edition of the Specification, the design values established by AITC (130, 131) have been those published in the Specification.

In 1973, the CS253 standard was revised and re-promulgated by the U.S. Department of Commerce as Voluntary Product Standard PS 56-73 (134). In 1983, the standard was adopted as an American National Standard through American National Standards Institute's (ANSI) consensus process, it is now published as ANSI/AITC A190.1 (2). This product standard includes requirements for sizes, grade combinations, adhesives, inspection, testing and certification of structural glued laminated timber products. Under A190.1, the grade combinations and related design values for structural glued laminated timber are required to be developed in accordance with ASTM D3737 or obtained by performance testing and analysis in accordance with recognized standards. Procedures embodied in this ASTM standard, first published in 1978, reflect the previously used methodology (49) as modified by data from a succession of more recent full-scale test programs (2).

C5.1.1 Scope

C5.1.1.1 The design requirements given in Chapters 1 through 3 of the Specification are applicable to structural glued laminated timber except where indicated otherwise. Chapter 5 of the Specification contains provisions which are particular to structural glued laminated timber.

The provisions of Chapter 5 contain only the basic requirements applicable to engineering design of structural glued laminated timber. Specific detailed requirements, such as those for curved and tapered members and connection details, are available from the American Institute of Timber Construction (140) and APA – The Engineered Wood Association.

C5.1.1.2 Where design values other than those given in NDS Supplement Tables 5A, 5B, 5C, and 5D, or as provided in the adjustments and footnotes of these tables are used, it shall be the designer's responsibility to assure that the values have been developed in accordance with all applicable provisions of ASTM D3737 and ANSI/AITC A190.1.

The design provisions in the Specification for structural glued laminated timber apply only to material certified by an approved agency as conforming to ANSI/AITC A190.1. The local building code body having jurisdiction over the structural design is the final authority as to the competency of the certifying agency and the acceptability of its grademarks.

C5.1.2 Definition

Laminations of structural glued laminated timber are usually made of sawn lumber. Laminated veneer lumber,

consisting of graded veneers bonded together with grain parallel longitudinally, and manufactured lumber, lumber of two or more pieces glued together, may be used for tension laminations where high tensile strength is required (2).

Adhesives and glued joints in structural glued laminated timber members are required to meet the testing and related requirements of ANSI/AITC A190.1.

C5.1.3 Standard Sizes

C5.1.3.1 The finished widths of structural glued laminated timber members are typically less than the dimensions of surfaced lumber from which it is made in order to allow for removal of excess adhesive from the edges of the laminations and preparation of a smooth surface. This is done by removing from 3/8 to 3/4 inch of the width from the original lumber width by planing or sanding.

For applications where appearance is not important, structural glued laminated timbers having a finished width matching the dimensions of standard framing lumber widths are available in a Framing appearance grade. This appearance grade is not generally suitable for members which will be exposed to view (128).

Widths other than standard sizes can be specified. These special widths require use of larger nominal lumber which may result in significant waste. For example, a 7 inch glued laminated beam would require the use 2x10 (9.25 inch) lumber laminations while a 6-3/4 inch beam would require 2x8 (7.25 inch) lumber laminations. While not specified as a standard size, other widths such as 3-1/2 inch and 5-1/2 inch may be available as stock items.

NDS Table 5.1.3 indicates standard widths for structural glued laminated timber members made from laminations of the indicated nominal widths. For example, structural glued laminated timber members with a net width of 3 inches or 3-1/8 inches are manufactured from nominal 4 inch wide lumber laminations and structural glued laminated timber members with a net width of 10-1/2 inches or 10-3/4 inches are made from nominal 12 inch wide lumber laminations. Dimension lumber is not typically available in nominal widths greater than 12 inches, so structural glued laminated timber members wider than 10-3/4 inches are typically manufactured from multiple-piece laminations (across the width). A 12 inch or 12-1/4 inch wide member is typically manufactured from a nominal 6 inch wide piece of lumber placed next to a nominal 8 inch wide piece of lumber (total nominal width of lamination is 14 inches). It could also be made from a nominal 10 inch wide piece of lumber placed next to a nominal 4 inch wide piece (total nominal width of

lamination is 14 inches). A 14 inch or 14-1/4 inch wide member, is made from a nominal 10 inch piece of lumber placed next to a nominal 6 inch piece of lumber (total nominal width of lamination is 16 inches). In members composed of multiple-piece laminations across the width, edge joints in the laminations are staggered so they don't align in adjacent laminations. Edge joints in members with multiple-piece laminations (across the width) are not typically bonded unless specifically required by the design.

C5.1.3.2 The sizes of structural glued laminated timber are designated by the actual size after manufacture. Depths are usually produced in increments of the thickness of the lamination used. For straight or slightly curved members, this is a multiple of 1-1/2 inches for western species and 1-3/8 inches for southern pine. The faces of southern pine lumber generally are resurfaced prior to gluing, thereby reducing the thickness of this material an additional 1/8 inch. For sharply curved members, nominal 1 inch rather than 2 inch thick lumber is used (140).

When members are tapered, the depth at the beginning and the end of the taper should be designated. In all cases, the length and net cross-section dimensions of all members should be specified.

C5.1.4 Service Conditions

C5.1.4.1 Where the equilibrium moisture content of members in service is less than 16 percent, the dry service design values tabulated in NDS Supplement Tables 5A, 5B, 5C, and 5D apply. A dry service condition for structural glued laminated timber prevails in most covered structures. However, members used in interior locations of high humidity, such as may occur in certain industrial operations or over unventilated swimming pools, may reach an equilibrium moisture content of 16 percent or more. In such conditions, wet service factors should be applied to reference design values.

C5.1.4.2 Glued laminated members used in exterior exposures that are not protected from the weather by a roof, overhang, or eave and are subject to water exposure for a sustained period of time are generally considered wet conditions of use. Adjustment factors, C_M , are provided in NDS Supplement Tables 5A through 5D for uses where this limit will be exceeded. Bridges, towers, and loading docks represent typical wet service applications. Uses in which the member is in contact with the ground should be considered wet use for those portions of the member that will attain a moisture content of 16 percent or more. Where wet service conditions apply, the susceptibility of the member to decay and the need for preservative treatment (see C5.3.11) should also be considered.

C5.2 Reference Design Values

C5.2.1 Reference Design Values

Reference design values in NDS Supplement Tables 5A (and Table 5A Expanded) and 5B are for members made with softwood species. Reference design values in NDS Supplement Tables 5C and 5D are for members made with hardwood species.

NDS Supplement Table 5A. Reference design values in this table are for softwood laminating combinations that have been optimized for members stressed in bending about the x-x axis (loads applied perpendicular to the wide face of the laminations). These values apply to members having 4 or more laminations. The stress class in the first column represents multiple laminating combinations, which have at least the indicated design properties. The stress class system was developed to simplify the design and specification of structural glued laminated timbers and to allow the manufacturer to supply laminated timbers which meet the stress class requirements, while making the most efficient use of available resources. Specification of a

particular laminating combination from NDS Supplement Table 5A Expanded is also permissible.

NDS Supplement Table 5A Expanded. Reference design values in this table are for softwood laminating combinations that have been optimized for members stressed in bending about the x-x axis (loads applied perpendicular to the wide face of the laminations). These values apply to members having 4 or more laminations and are divided into stress classes for which the combination qualifies. The combination symbol in the first column designates a specific combination and lay-up of grades of lumber. For example, 16F-V6 indicates a combination with a bending design value, F_{bx} , of 1600 psi (column 3—tension zone stressed in tension) made with visually graded lumber (V). In the same format, 24F-E1 indicates an F_{bx} of 2400 psi (column 3—tension zone stressed in tension) made with E-rated lumber. The second column of NDS Supplement Table 5A Expanded indicates the species used for the outer laminations and for the core laminations of the member. For example, DF/HF indicates Douglas Fir-Larch is used for the outer laminations and Hem-Fir is used for the core

laminations. Lay-up details for softwood combinations are available in AITC 117(131).

NDS Supplement Table 5B. Reference design values in this table are for softwood laminating combinations that have been optimized for stresses due to axial loading or to bending about the y-y axis (loads applied parallel to the wide face of the laminations). Each combination consists of a single grade of one species of lumber. The grade associated with each numbered combination can be obtained from AITC 117 (131).

NDS Supplement Table 5C. Reference design values in this table are for hardwood laminating combinations that have been optimized for members stressed in bending about the x-x axis (loads applied perpendicular to the wide face of the laminations). These values apply to members having 4 or more laminations. The combination symbol in the first column designates a specific combination and lay-up of grades of lumber. For example, 16F-V1 indicates a combination with a bending design value, F_{bx} , of 1600 psi (column 2—tension zone stressed in tension) made with visually graded lumber (V). In the same format, 24F-E2 indicates an F_{bx} of 2400 psi (column 2—tension zone stressed in tension) made with E-rated lumber. Lay-up details for hardwood combinations are available in AITC 119 (130).

NDS Supplement Table 5D. Reference design values in this table are for hardwood laminating combinations that have been optimized for stresses due to axial loading or to bending about the y-y axis (loads applied parallel to the wide face of the laminations). Each combination consists of a single grade of one species of lumber. The grade associated with each numbered combination can be obtained from the AITC 119 (130).

C5.2.2 Orientation of Member

Reference design values for structural glued laminated timber are primarily developed using provisions from ASTM D3737 (13). The various models contained in D3737 result in different design values based on the orientation of the member with respect to transverse loads. These different values reflect both the use of mixed-grade layups and differences in the placement of strength-reducing characteristics between laminations on edge and laminations used flatwise. The x-x and y-y axes are defined relative to the orientation of the laminations, not the overall dimensions of the laminated timber cross section.

C5.2.3 Balanced and Unbalanced Layups

Multiple-grade layups of structural glued laminated timber are optimized for bending by placing high grade

laminations near the top and bottom of the section and lower grade laminations in the core. Testing of glulam beams has demonstrated that the strength of tension laminations is critical and that laminations of a somewhat lower grade can be used in the compression zone of a beam without reducing the capacity of the beam. These *unbalanced* layups are more resource-efficient for applications such as simply-supported beams where the beam is bent in only one direction. *Balanced* layups are recommended for applications involving both positive and negative bending. The top faces of glulam beams are marked “TOP” to ensure proper installation.

C5.2.4 Bending, F_{bx}^+ , F_{bx}^- , F_{by}

Positive and negative bending design values are tabulated for bending about the x-x axis. The positive bending design value, F_{bx}^+ , is used for both flexural tension and flexural compression in sections subjected to bending causing compression at the top edge and tension at the bottom edge. The negative bending design value, F_{bx}^- , is used for both flexural tension and flexural compression in sections subject to bending causing tension at the top edge and compression at the bottom edge.

C5.2.5 Compression Perpendicular to Grain, $F_{c\perp x}$, $F_{c\perp y}$

The appropriate design value for compression perpendicular to grain is based on the density of the lamination(s) at the location of bearing stress. Design values for compression perpendicular to grain on the top and bottom of the beam are assigned based on the grade of the laminations at those locations. Design values for bearing on the side of glulam beams are assigned based on the lowest grade of lumber used in the layup. Because the laminations used on the top and bottom of glulam beams are commonly more dense than the core laminations, higher values of compression perpendicular to grain are commonly used for bearing on the top or bottom faces than on the side.

C5.2.6 Shear Parallel to Grain, F_{vx} , F_{vy}

Reference design values for shear have traditionally been derived from tests of small shear-block specimens. The current reference design values are based on tests of full-scale prismatic beams subjected to quasi-static loading conditions. Application of the shear reduction factor reduces the reference stress to approximately the value

derived from the shear-block tests for cases which were not tested as part of the full-scale test program.

The shear stress reduction of 0.84 and 0.95 for use with members of only two laminations or three laminations, respectively, shall be applied cumulatively with the Shear Reduction Factor, C_{vr} , in 5.3.10.

C5.2.7 Modulus of Elasticity, E_x , $E_{x \min}$, E_y , $E_{y \min}$

Reference design values for E_x and E_y represent the average bending moduli for bending about the x-x and y-y axes, respectively. These values include a 5% reduction to account for shear deformations and are customarily used for serviceability calculations. For calculations of beam and column stability, the appropriate reference design values ($E_{x \min}$ and $E_{y \min}$) are determined using the following equation:

$$E_{\min} = \frac{E[1 - 1.645(CoV_E)][1.05]}{1.66} = \frac{E[1 - 1.645(0.10)][1.05]}{1.66} = 0.528$$

where:

E = reference modulus of elasticity

1.05 = adjustment factor to convert E values to a pure bending basis

1.66 = factor of safety

CoV_E = coefficient of variation in modulus of elasticity (see NDS Appendix F)

C5.2.8 Radial Tension, F_{rt}

Radial tension stresses are induced in curved bending members when bending loads tend to flatten out the curve or increase the radius of curvature. In earlier editions, radial tension design values perpendicular to grain were established as 1/3 the corresponding shear design value parallel to grain for all species. This provision was based on strength data for small, clear specimens free of checks and other seasoning effects (9). It is important to note that the factor of 1/3 applies to the shear value for non-prismatic members ($F_{vx}C_{vr}$). As a result of field experience, the radial tension reference design value perpendicular to grain for Douglas Fir-Larch was limited to 15 psi except for conditions created by wind and earthquake loading. In 1991, this limit was expanded to all western species.

C5.2.9 Radial Compression, F_{rc}

Because radial stresses are maximum at the neutral axis of a curved beam, the appropriate design value to use for radial compression is the compression perpendicular to grain design value for the laminations used in the core of the beam. This value is equal to the design value for bearing on the side face of a beam, F_{cLy} .

C5.2.10 Other Species and Grades

See C5.1.1.2.

C5.3 Adjustment of Reference Design Values

C5.3.1 General

Applicable adjustment factors for structural glued laminated timbers are specified in Table 5.3.1 of the Specification.

C5.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2.

C5.3.3 Wet Service Factor, C_M

The wet service reduction value, C_M , for F_b , F_t , F_v , F_{cL} , F_c , and E in NDS Supplement Tables 5A, 5B, 5C, and 5D are based on provisions of ASTM D3737 (13). The wet service factors account for both the decrease in mechanical properties and the increase in cross-section dimensions

associated with this exposure. Since radial tension, F_{rt} , is based on shear parallel to grain, F_v , Footnote 2 in Table 5.3.1 is intended to clarify that C_M for F_v is also applicable to F_{rt} .

C5.3.4 Temperature Factor, C_t

See C2.3.3. Since radial tension, F_{rt} , is based on shear parallel to grain, F_v , Footnote 2 in Table 5.3.1 is intended to clarify that C_t for F_v is also applicable to F_{rt} .

C5.3.5 Beam Stability Factor, C_L

Reference bending design values, F_b , given in NDS Supplement Tables 5A, 5B, 5C, and 5D are based on members having a compression edge supported throughout its length or having a depth to breadth ratio of one or less. When these conditions do not exist, F_b values are to

be adjusted by the beam stability factor, C_L , calculated in accordance with the procedures of NDS 3.3.3. Because the tendency of the compression portion of the beam to buckle is a function of beam stiffness about the y-y axis for beams loaded in bending about the x-x axis, all glued laminated beam stability factor calculations are to be made with values of modulus of elasticity for bending about the y-y axis, E_{ymin} , modified by all applicable adjustment factors (see also C3.3.3).

C5.3.6 Volume Factor, C_V

The volume factor adjustment for structural glued laminated timber beams includes terms for the effects of width, length, and depth. The volume factor (C_V) equation (NDS Equation 5.3-1) is based on research involving tests of beams 5-1/8 and 8-3/4 inches wide, 6 to 48 inches deep, and 10 to 68 feet in length (90). This equation is based on the volume effect equation given in ASTM D3737 (13). The volume factor, C_V , applies when structural glued laminated timber bending members are loaded in bending about the x-x axis.

As indicated in Footnote 1 to NDS Table 5.3.1, the volume factor, C_V , is not applied simultaneously with the beam stability factor, C_L . The smaller of the two adjustment factors applies. This provision is a continuation of the practice of considering beam stability and bending size modifications separately. The practice is based on design experience and the position that beam buckling is associated with stresses on the compression side of the beam, whereas bending design values and the effect of volume on such values are related primarily to the properties of the laminations stressed in tension.

C5.3.7 Flat Use Factor, C_{fu}

The flat use factor, C_{fu} , applies where glued laminated timber bending members are loaded in bending about the y-y axis. The C_{fu} factors given in NDS Supplement Tables 5A, 5B, 5C, and 5D are applied only to the tabulated F_{byy} design values in these tables and cover only those members which are less than 12" in dimension parallel to the wide face of the laminations, d_y . For bending members loaded parallel to the wide face of the laminations with the dimension of the member in this direction greater than 12", a flat use factor based on Equation 4.3-1 of the Specification should be used.

Provisions reflect standard nomenclature used by the glued-laminated timber industry. The term " d_y " refers to the member dimension parallel to the wide faces of the laminations. NDS Figure 5B clarifies the dimension " d_y ",

member orientation, and direction of loading, for applicability of the flat use factor.

C5.3.8 Curvature Factor, C_c

When the individual laminations of structural glued laminated timber members are bent to shape in curved forms, bending stresses are induced in each lamination that remain after gluing. In addition, the distribution of stresses about the neutral axis of curved members is not linear. The curvature factor, C_c , is an adjustment of reference bending design values, F_b , to account for the effects of these two conditions.

The curvature factor equation given in NDS 5.3.8 is based on early tests (166). The limits on the ratio of lamination thickness to radius of curvature of 1/100 for southern pine and hardwoods and 1/125 for other softwood species are imposed to avoid overstressing or possible breaking of the laminations.

Radii of curvature used in practice generally are larger than those allowed by the specified minimum thickness/radius of curvature ratios. For nominal 1 inch thick laminations (3/4 inch net), radii of curvature of 7 feet and 9.3 feet are typically used with southern pine and other softwood species, respectively. For nominal 2 inch laminations (1.5 inches net), a radius of curvature of 27.5 feet is commonly used for all species.

C5.3.9 Stress Interaction Factor, C_i

Prismatic beam design neglects the potential interaction of shear and bending stresses, because the maximum values of these stresses occur at different locations in the cross section. For beams with a tapered edge, the shear stress distribution is altered. In addition, perpendicular to grain stresses are developed along the tapered edge. Taper cuts are, therefore, not recommended on the tension faces of beams since they cause tension stresses to develop perpendicular to grain. The stress interaction factor, C_i , accounts for the interaction of flexure, shear, and perpendicular-to-grain stresses along the tapered edge.

The stress interaction factor, C_i , does not apply simultaneously with the volume effect factor, C_V , for members tapered on the compression face, because, in this case, the volume factor accounts for tension-side effects, and the combined beam stability factor and stress interaction factor account for compression-side effects. Therefore, in the case of a tapered compression face, the lesser of C_V or $C_L C_i$ applies. Similarly, the stress interaction factor, C_i , does not apply simultaneously with the lateral stability factor, C_L , for members tapered on the tension face, because in this case the beam stability factor accounts for

compression side effects and the combined volume factor and stress interaction factor account for tension side effects. Therefore, in the case of a tapered tension face, the lesser of $C_V C_1$ or C_L applies.

C5.3.10 Shear Reduction Factor, C_{vr}

The shear reduction factor, C_{vr} , previously appearing as a footnote to glued-laminated timber design value tables in the NDS Supplement, was added to the 2012 edition of the Specification as an adjustment factor applicable for shear design of other than prismatic beams such as for the design of notched members, curved members, tapered members, design for radial tension, and shear design at connections.

The term “non-prismatic” is used to designate a broad range of products including curved beams and arches, tapered beams, pitched and tapered curved beams, and generally members with any geometry other than straight or mildly cambered with constant depth.

C5.3.11 Column Stability Factor, C_p

See C3.7.1.

C5.3.12 Bearing Area Factor, C_b

See C3.10.4.

C5.3.13 Pressure-Preservative Treatment

The provision in the NDS for use of reference design values with structural glued laminated timber that has been preservative treated is applicable to material that has been treated and redried in accordance with AWP Standards. In AWP Standards, the maximum temperature for kiln drying material after treatment is 165°F (22).

C5.3.14 Format Conversion Factor, K_f (LRFD Only)

See C2.3.5.

C5.3.15 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C5.3.16 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

C5.4 Special Design Considerations

C5.4.1 Curved Bending Members with Constant Cross Section

C5.4.1.3 The equation for determining radial stress in a curved member of constant rectangular cross section is based on research published in 1939 (166). Radial stresses in curved members having variable cross section are determined by different procedures (46, 56). Complete design procedures for such members are available from other recognized sources (140).

Where the bending moment acts to reduce curvature, the radial stress is to be checked against the adjusted radial tension design value perpendicular to grain, F_{rt}' (see C5.2.2). When mechanical reinforcing is provided which is sufficient to resist all induced radial stresses, the actual radial stress is still limited to no more than $(1/3) F_{vx}'$.

Where the bending moment acts to increase curvature, the radial stress is to be checked against the adjusted compression design value perpendicular to grain. The appropriate compression perpendicular-to-grain design value for use is the value corresponding to the lamination grades used in the core of the beam, $F_{c\perp}$.

C5.4.1.4 The downward deflection of curved beams results in lateral displacement of the unrestrained beam ends. This lateral displacement must be considered in the design of beams and in the connection between beams and walls.

C5.4.2 Double-Tapered Curved Bending Members

C5.4.2.1 The bending stress, f_b , for curved beams of variable cross section, such as pitched tapered curved beams, is greater than for straight prismatic members and is increased by the bending stress factor, K_ϕ , (NDS Equation 5.4-2) due to the shape of the member.

C5.4.2.2 Design of double-tapered curved members for shear is identical to the design of prismatic members, except loads within a distance d from the support are not permitted to be neglected and the shear stress reduction factor is applied.

C5.4.2.3 The shape of double-tapered curved members alters the stress distribution for radial stresses in the curved segment of the beam, rendering Equation 5.4-1 inapplicable. The empirical radial stress factor, K_{rs} , is used with the flexure

formula at the section through the beam apex to estimate the value of the radial stress at that section for the case of a beam subject to pure bending moment. The empirical load-shape radial stress reduction factor, C_{rs} , accounts for reduced radial stresses that exist in a uniformly loaded beam.

C5.4.2.4 Due to the variable cross section, strict evaluation of the deflection of a double-tapered curved beam requires more advanced analysis techniques such as virtual work or finite element modeling. A simplified approach has been developed that uses a prismatic rectangular beam of *equivalent depth* to estimate the deflection. This approach is permitted to be used in lieu of more complex methods.

Vertical deflection of double-tapered curved beams results in lateral displacement of the beam ends. NDS Equation 5.4-5 provides an estimate of the displacement at each support. The total horizontal displacement is equal to $2\Delta_H$.

C5.4.3 Lateral Stability for Tudor Arches

Prescriptive limitations on the ratio of tangent point depth to breadth (d/b) for glulam arches are based on long-standing industry recommendations and field experience over many years.

In determining the adequacy of lateral support, decking or subflooring applied directly to a beam with two or more fasteners per piece is acceptable edge restraint for a beam loaded through such decking or subflooring. Rafters, joists, or purlins attached two feet or less on center to the side of a beam and stabilized through the attachment of sheathing or subflooring are acceptable edge restraint for a beam that is loaded through such rafters, joists, or purlins. Recent research has shown that the bottom edges of rafters, joists, or purlins attached to the sides of beams by strap hangers or similar means do not have to be fixed to provide adequate lateral support to the beam if their top edges are restrained (164, 165).

C5.4.4 Tapered Straight Bending Members

C5.4.4.1 Because glulam beams are typically manufactured with high grades of lumber in the top and bottom laminations and lower grades in the core (see C5.2.2 and C5.2.3), field-tapering a member will result in reductions in the reference design values for the section. For beams manufactured with taper in the laminating plant, the high grades are maintained along the full length of the taper cut to ensure that the full design value is maintained. Manufacturers or trade associations representing the manufacturers

should be contacted to determine the appropriate reference design values for field-tapered members (see also C5.3.9).

C5.4.4.2 Design of tapered straight members for shear is identical to the design of prismatic members, except loads within a distance d from the support are not permitted to be neglected and the shear stress reduction factor is applied.

C5.4.4.3 Due to the variable cross section, strict evaluation of the deflection of a tapered beam requires more advanced analysis techniques such as virtual work or finite element. A simplified approach has been developed that uses a prismatic rectangular beam of *equivalent depth* to estimate the deflection. This approach is permitted to be used in lieu of more complex methods.

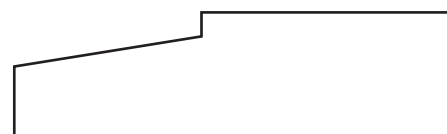
C5.4.5 Notches

The designer has the responsibility of determining if structural glued laminated timber bending members should be notched and how load-carrying capacity should be calculated. Current good engineering practice is to avoid all notching of such bending members on the tension side except where end notching at the supports is necessary. End notches on the tension face are limited to the lesser of $1/10$ of the bending member depth or 3 inches (140). Notches on the compression face of the beam are less severe and consequently less restrictive, particularly for taper cuts.

The effect of notches located at the ends of beams is generally limited to a reduction of shear capacity for the beam. However, notches that extend a long distance from the end of the member may also reduce the bending load-carrying capacity and increase the deflection. For notches extending more than $1/3$ of the length of the member, the effects of the notch on bending and deflection must also be considered based on 5.4.4. The methods of NDS 3.4.3 are used to calculate the design shear of glued laminated timber members with end notches (140). The shear reduction factor of 5.3.10 applies for the analysis of members with notches.

The provisions for compression-side notches do not consider the case of a taper cut combined with a square-cornered notch such as illustrated in Figure C5.4.5. At a minimum, for such a case, each type of modification must comply with the applicable limitation. No further guidance is available for the evaluation of this case.

Figure C5.4.5 Taper Cut Combined with a Square-Cornered Notch



C6 ROUND TIMBER POLES AND PILES

C6.1 General

Round timber piles have been widely used in the United States in the construction of railroads, highways, harbors and dams, as well as for building foundations, since the middle of the 18th century. In addition to availability and cost, the natural taper of round timber piles makes them relatively easy to drive, compacts the soil around the pile during driving, and provides a larger diameter butt end capable of withstanding driving forces and supporting loads from other structural members (176).

The earliest standardization effort involving timber piles was the establishment of uniform size and grade characteristics in ASTM D25, *Standard Specification for Round Timber Piles* (10). First developed in 1915, the current edition of this standard includes specifications for minimum butt and tip sizes for various pile lengths, establishes limits on crook and knot sizes, and sets minimum rate of growth and percent summerwood quality requirements.

The establishment of standard physical characteristics for timber piles in ASTM D25 was subsequently followed by the development of standard requirements for preservative treatment. Such specifications were available from the American Wood Preservers' Association (AWPA) since well before World War II (184). This Association's Standard C3, *Piles-Preservative Treatment by Pressure Processes*, establishes conditioning, pressure, temperature, retention, and penetration limitations and requirements for various preservative treatments by species and pile use (22). Because of the effect treatment processes can have on strength properties, standardization of the processes used are an important element in the specification and use of timber piles.

Engineering design with timber piles in the early years was largely based on experience, observation of the performance of piles under similar loading conditions, and the results of static loading tests. Piles were considered to fall into two groups: those in which the pile tip bears on a solid layer and were designed as columns, and those in which the pile receives most of its support from soil friction on the sides and were designed from driving records or empirical formulas (184). Standard design procedures were not available.

To meet the growing need for uniform design recommendations, the American Association of State Highway Officials (AASHTO) began to specify allowable pile compression design values of 1200 psi for Douglas fir and slightly lower values for other species in the 1940's (176). However, maximum pile loads in the order of 36,000 to 50,000 pounds per pile also were specified which generally was the limiting criterion.

In the 1950's, AASHTO, the American Railway Engineering Association, and other user groups began to establish pile design values using the procedures of ASTM D245, *Standard Methods for Establishing Structural Grades of Lumber* (176) (see C4.2.3.2). Building codes also began to establish allowable pile stresses using basic stresses and other information given in ASTM D245 (161).

Uniform national standards for development of strength values for timber piles became available in 1970 with the publication of ASTM D2899, *Standard Method for Establishing Design Stresses for Round Timber Piles* (11). This consensus standard provided for the establishment of stresses for piles of any species meeting the size and quality requirements of ASTM D25. Under D2899, clear wood property information from ASTM D2555 (9) were adjusted for grade, relation of pile tip strength to clear wood strength, variability of pile strength to that of small clear specimens, load duration, and treatment conditioning effects.

In 1997, reference design values were added for construction poles based on D2899 per reference in ASTM D3200.

In 2003, the provisions of D2899 were revised to derive single pile design values rather than pile cluster design values derived in previous versions of D2899. As a result, beginning with the 2012 edition of the Specification the single pile basis became the reference condition for pile design values and adjustments to pile design values.

C6.1.1 Scope

C6.1.1.2 The provisions of Chapter 6 of the Specification relate solely to the properties of round timber poles and piles. It is the responsibility of the designer to determine

soil loads, such as frictional forces from subsiding soils and fills, the adequacy of the surrounding soil or water to provide sufficient lateral bracing, the method of pole or pile placement that will preclude damage to the wood member, the bearing capacity of the strata at the pile tip, and the effects of any other surrounding environmental factors on the supporting or loading of poles or piles.

C6.1.2 Specifications

C6.1.2.1 In addition to setting standard pile sizes, ASTM D25 (10) establishes minimum quality requirements, straightness criteria, and knot limitations. All pile tips are required to have an average rate of growth of 6 or more rings per inch and percent summerwood of 33 percent or more in the outer 50 percent of the radius; except less than 6 rings per inch growth rate is acceptable if the summerwood percentage is 50 percent or more in the outer 50 percent of the tip radius. Thus, 75 percent of the cross-sectional area of pile tips conforming to ASTM D25 essentially meets lumber requirements for dense material (8).

Knots in piles are limited by ASTM D25 to a diameter of not more than one-sixth of the circumference of the pile at the point where they occur. The sum of knot diameters in any one-foot length of pile is limited to one-third or less of the circumference.

ASTM D3200 establishes standard sizes and minimum grades for construction poles based on ASTM D25 for piles.

C6.1.2.2 Preservative treatment requirements and limitations differ depending upon where the piles are to be used. Designation of the applicable treatment standard and use condition defines the treatment desired by the specifier.

C6.1.3 Standard Sizes

Standard sizes (10) for round timber piles range from 7 to 18 inches in diameter measured 3 feet from the butt.

Pile lengths range from 20 to 85 feet for southern pine and to 120 feet for Douglas fir and other species. Pile taper is controlled by establishing a minimum tip circumference associated with a minimum circumference 3 feet from the butt for each length class; or by establishing a minimum circumference 3 feet from the butt associated with a minimum tip circumference for each length class. This provides a known tip area for use in engineering design as well as a conservative estimate of the area at any point along the length of the pile.

Standard sizes (12) for round timber construction poles range from 5 to 12 inches in diameter measured at the tip. Pole lengths range from 10 to 40 feet.

C6.1.4 Preservative Treatment

C6.1.4.1 Green timber piles are generally conditioned prior to pressure treatment (22). For southern pine the conditioning usually involves steaming under pressure to obtain a temperature of 245°F and then applying a vacuum. The process results in water being forced out of the outer part of the pile, but does not dry it to a seasoned condition (181, 63). Conditioning of Douglas fir is usually done by the Boulton or boiling-under-a-vacuum-process. This method of conditioning, which partially seasons the sapwood portion of the pile, involves heating the material in the preservative oil under a vacuum at temperatures up to 220°F (181, 63). The Boulton process also is used with hardwood species.

Both the steaming and Boulton conditioning processes affect pile strength properties (11, 176). The effects of conditioning (steaming, Boultonizing, and drying) are not included in pile design values given in Table 6A of the NDS Supplement and must be addressed with the reduction factors in 6.3.5.

C6.1.4.2 Decay does not occur in softwood species and in most hardwoods that are completely saturated and an air supply is not available (63, 127). Permanently submerged piles meet these conditions.

C6.2 Reference Design Values

C6.2.1 Reference Design Values

C6.2.1.1 Reference design values for round timber piles are specified in NDS Supplement Table 6A and are based on ASTM D2899 (11). All values are derived from the properties of small clear specimens of the applicable species as given in ASTM D2555 (9) adjusted as appropriate for the specific property for variability, load duration,

grade, lower strength of pile tip, and lower variability of piles compared to small clear specimens (160).

C6.2.1.2 Design values for round timber poles are specified in NDS Supplement Table 6B and are based on ASTM D3200 which uses provisions from ASTM D2899 (11) with similar adjustments used for round timber piles.

C6.2.2 Other Species or Grades

Where piles of species other than those listed in NDS Supplement Table 6A are used, it is the designer's respon-

sibility to assure that the methods of ASTM D2899 for establishing design values are properly applied, including appropriate adjustments for conditioning process.

C6.3 Adjustment of Reference Design Values

C6.3.1 General

Adjustment factors for round timber poles and piles are specified in Table 6.3.1 of the Specification.

C6.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2. As shown in NDS Table 6.3.1, the load duration factor, C_D , is not applicable to compression design values perpendicular to grain, $F_{c\perp}$ for round timber poles and piles. Pressure impregnation of water-borne preservatives or fire retardant chemicals to retentions of 2.0 pcf or more may significantly reduce energy absorbing ability as measured by work-to-maximum-load in bending. For this reason, the impact load duration adjustment is not to be applied to members pressure-treated with preservative oxides for salt water exposure or those pressure-treated with fire retardant chemicals.

C6.3.4 Temperature Factor, C_t

See C2.3.3.

C6.3.5 Condition Treatment Factor, C_{ct}

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.

C6.3.6 Beam Stability Factor, C_L

A round member can be considered to have a d/b ratio of 1 and therefore, in accordance with NDS 3.3.3.1, C_L equals 1.0.

C6.3.7 Size Factor, C_F

Bending design values, F_b , for round timber poles and piles that are larger than 13.5 inches in diameter at the critical section in bending are adjusted for size using the

same equation used to make size adjustments with sawn lumber Beams & Stringers and Posts & Timbers (see NDS 4.3.6.3 and C4.3.6.2). When applied to round timbers, Equation 4.3-1 is entered with a d equal to the depth of a square beam having the same cross-sectional area as that of the round member. The equivalency of the load-carrying capacity of a circular member and a conventionally loaded square member of the cross-sectional area has long been recognized (98).

C6.3.8 Column Stability Factor, C_P

See C3.7.1. Column stability provisions from NDS 3.7.1 can be used for round timber poles and piles by substituting $r\sqrt{12}$ for the depth, d , in the equations, where r is the applicable radius of gyration of the column cross section.

C6.3.9 Critical Section Factor, C_{cs}

The critical section factor, C_{cs} , accounts for the effect of tree height on compression design values parallel to grain. The specific adjustment, based on D2899, provides for an increase in the design value as the critical section moves from the pile tip toward the pile butt and is limited to a maximum increase of 10%.

C6.3.10 Bearing Area Factor, C_b

See C3.10.4.

C6.3.11 Load Sharing Factor (Pile Group Factor), C_{ls}

Reference design values in NDS Supplement Table 6A are based on a single pile. Where piles are used in clusters such that the pile group deforms as a single element, the single pile reference compression design value parallel to grain, F_c , and the single pile reference bending design value, F_b , are permitted to be multiplied by C_{ls} in NDS Table 6.3.11.

The load sharing factor (pile group factor) is applicable to F_b and F_c and is not applicable to soil-related properties such as pile tip bearing or skin friction.

C6.3.12 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C6.3.13 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C6.3.14 Time Effect Factor, λ (LRFD Only)

See C2.3.7.



C7 PREFABRICATED WOOD I-JOISTS

C7.1 General

Prefabricated wood I-joists utilize 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. Wood I-joists are generally produced as proprietary products. Acceptance reports and product literature should be consulted for current design information.

C7.1.1 Scope

The general requirements given in Chapters 1, 2, and 3 of the Specification are applicable to prefabricated wood I-joists except where indicated otherwise. Chapter 7 of the Specification contains provisions which specifically apply to prefabricated wood I-joists manufactured and evaluated in accordance with ASTM D 5055 (15). The provisions of NDS Chapter 7 contain only the basic requirements applicable to engineering design of prefabricated wood I-joists. Specific detailed requirements, such as those for bearing, web stiffeners, web holes, and notches, are available in the prefabricated wood I-joist manufacturer's literature and code evaluation reports. Code evaluation reports, where available for a specific product and application, contain information to assist in the approval and use of the specific product in accordance with requirements of the applicable building code.

C7.1.2 Definition

Prefabricated wood I-joists are specialized products, manufactured with specially designed equipment. Expertise in adhesives, wood products, manufacturing, and

quality assurance are necessary ingredients for the fabrication of high-quality prefabricated wood I-joists.

Standard Sizes

Prefabricated wood I-joists are available in a range of sizes to handle a variety of applications. Common I-joist depths for residential flooring applications are 9.5", 11.875", 14", and 16". These sizes do not match standard sawn lumber depths to minimize the combined use of sawn lumber with wood I-joists in the same floor system. Mixing I-joists and sawn lumber in the same system is not recommended because differences in dimensional change between sawn lumber and wood I-joists can affect load distribution as the products reach equilibrium moisture content.

C7.1.3 Identification

Prefabricated wood I-joists are typically identified by product series and company name, plant location or number, qualified agency name or logo, code evaluation report numbers, and a means for establishing the date of manufacture.

C7.1.4 Service Conditions

Prefabricated wood I-joists are typically used in dry service conditions (where moisture content of sawn lumber is less than 16%). I-joist manufacturers state dry service conditions based on the moisture content of lumber since the equilibrium moisture content (EMC) of I-joists is lower under the same environmental conditions and varies based on the manufacturing process. For other conditions, the I-joist manufacturer should be consulted.

C7.2 Reference Design Values

Prefabricated wood I-joists are proprietary products and reference design values vary among manufacturers and product lines. Reference design values are obtained from

the manufacturer through the manufacturer's literature or code evaluation report.

C7.3 Adjustment of Reference Design Values

C7.3.1 General

Applicable adjustment factors for prefabricated wood I-joists are specified in Table 7.3.1 of the Specification. Volume effects are accounted for either directly in testing or indirectly in analysis as detailed in ASTM D 5055 (15) and need not be considered in design.

C7.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2. Duration of load effects in NDS 2.3.2 apply to all prefabricated wood I-joist design values except for those relating to stiffness, EI , EI_{min} , and K .

C7.3.3 Wet Service Factor, C_M

Prefabricated wood I-joists are limited to use in dry service conditions unless specifically allowed by the manufacturer (see NDS 7.1.4). I-joists are assembled with exterior adhesives and can tolerate the environmental conditions of typical jobsites. Care should be taken, however, to follow the manufacturer's recommendations for proper jobsite storage to minimize dimensional changes associated with changes in moisture content.

C7.3.4 Temperature Factor, C_t

See C2.3.3. Prefabricated wood I-joist reference design values are adjusted by the same temperature adjustment factors as other wood products (see Table C7.3-1).

C7.3.5 Beam Stability Factor, C_L

Bending design values provided in manufacturers' code evaluation reports are based on the I-joist having the compression flange supported throughout its entire length to prevent lateral displacement. This condition is ensured

by direct attachment of sheathing to the I-joist. In addition, wood I-joists should be provided with lateral support at points of bearing to prevent cross-sectional rotation (twisting). Provisions applicable to wood I-joists are consistent with provisions in NDS 3.3.3 as follows:

- NDS 7.3.5.1 permits $C_L=1.0$ when the compression flange of an I-joist is supported throughout its length to prevent lateral displacement, consistent with NDS 3.3.3.3.
- NDS 7.3.5.2 provides a method for designing the compression flange of an I-joist as an unbraced or partially braced column when the compression flange is not braced throughout its length, consistent with NDS 3.3.3.4.

Derivation of the beam stability equations in the NDS are based on the assumption that the beam is braced against cross-sectional rotation (twisting) and lateral displacement at points of bearing (i.e., the tension edge and the compression edge of the beam are held in line see Figure C7.3.5). As an example, bracing against cross-sectional rotation (twisting) will prevent lateral-torsional buckling, but bracing against lateral displacement will not necessarily prevent rotation at the point of bearing (e.g., set of floor joists "turning over").

C

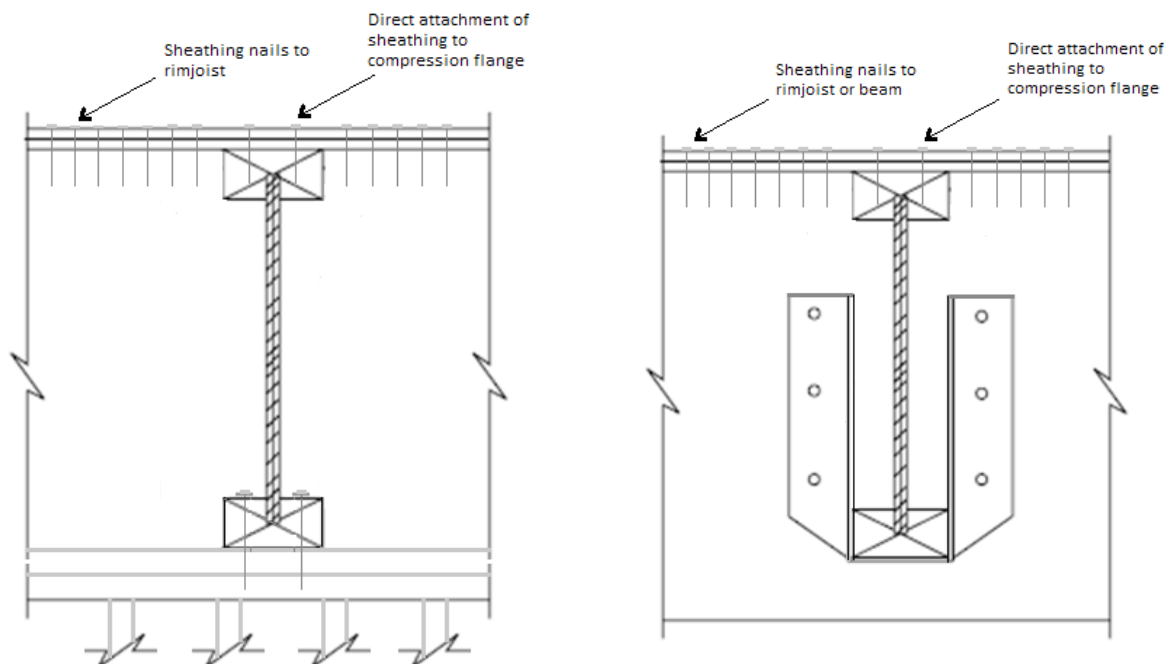
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Table C7.3-1 Temperature Factor, C_t , for Prefabricated Wood I-Joists

Reference Design Values	In-Service Moisture Conditions ¹	C_t		
		$T \leq 100^\circ\text{F}$	$100^\circ\text{F} < T \leq 125^\circ\text{F}$	$125^\circ\text{F} < T \leq 150^\circ\text{F}$
EI , EI_{min}	Wet or Dry	1.0	0.9	0.9
M_r , V_r , R_r , and K	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

1. Wet and dry service conditions for wood I-joists are specified in NDS 7.1.4.

Figure C7.3.5 Example of Direct Attachment of Sheathing to Support Compression Flange and Lateral Support at Points of Bearing



C7.3.6 Repetitive Member Factor, C_r

The repetitive member factor varies with composite action across a range of I-joist depths and series, I-joist stiffness variability, sheathing types, sheathing stiffnesses, and sheathing attachment. For several technical reasons, the magnitude of the repetitive member factor is typically much smaller than for sawn lumber. To provide a factor that could be applied across all applications, this factor was set at 1.0 in ASTM D 5055 (15) and D 6555 (19).

C7.3.7 Pressure-Preservative Treatment

Common treatments associated with I-joists include light solvent-based preservatives applied under low pressure offering protection against wood destroying fungi or insects. Any treatment of I-joists that require high pressure

or harsh drying cycles should be avoided. Manufacturers should be consulted for any applications that require preservative treatment.

C7.3.8 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C7.3.9 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C7.3.10 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

C7.4 Special Design Considerations

C7.4.1 Bearing

The end conditions of an I-joist require specific attention by the designer when considering the differences of designing with an "I" shape versus rectangular sections.

The limit states at the bearing of an I-joist include the I-joist reaction (integrity of the web/flange rout), flange compression, compression of the support, reaction hardware (hangers), and shear.

The manufacturer's literature or code evaluation reports should be consulted for design assumptions at end conditions. Reaction capacity, R_e , and shear capacity, V_e , are typically published separately and should be checked independently. Published reaction capacity is based on testing conducted at one or more bearing lengths. Extrapolation beyond tested conditions is not appropriate. For end bearing, the minimum bearing length is typically 1.75", but never less than 1.5". The reaction capacity may or may not include the compression of the flange or bearing plate. The published capacities of joist hangers only include the capacity of the hanger. A complete design would include checking the I-joist capacity for the bearing length of the particular joist hanger.

C7.4.2 Load Application

The manufacturer's literature or code evaluation reports should be consulted for design assumptions where loads are not applied to the top flange or where concentrated loads or other non-uniform loads are applied to the I-joist.

C7.4.3 Web Holes

The manufacturer's literature or code evaluation reports should be consulted for the effect of web holes on strength and stiffness.

C7.4.4 Notches

The manufacturer's literature or code evaluation reports should be consulted when notching of the flange is being considered. However, as a general rule, flange notching is not permitted.

C7.4.5 Deflection

I-joist stiffness is presented as the product of the material modulus of elasticity and the effective moment of inertia (EI). I-Joist floor systems are typically designed to $L/480$ deflection limits rather than the code minimum of $L/360$. Consideration of creep deflection for unique applications, such as those with heavy dead loads, may be done in accordance with NDS 3.5.2.

C7.4.6 Vertical Load Transfer

See C7.4.2.

C7.4.7 Shear

See C7.4.1 and C7.4.2.

C8 STRUCTURAL COMPOSITE LUMBER

C8.1 General

Structural composite lumber (SCL) is manufactured from wood strands 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 growth characteristics and monitoring manufacturing through quality control procedures, the resulting material has consistent quality, and maximizes the strength and stiffness of the wood fiber.

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

C8.1.1 Scope

The general requirements given in Chapters 1, 2, and 3 of the Specification are applicable to structural composite lumber except where indicated otherwise. Chapter 8 of the Specification contains provisions which are particular to structural composite lumber. The provisions of NDS Chapter 8 contain only the basic requirements applicable to engineering design of structural composite lumber manufactured in accordance with ASTM D 5456 (16). Specific detailing requirements, such as those for notches, are available from structural composite lumber manufacturers' literature or code evaluation reports. Code evaluation reports, where available for a specific product

and application, contain information to assist in the approval and use of the specific product in accordance with requirements of the applicable building code.

C8.1.2 Definitions

Definitions for structural composite lumber, including laminated veneer lumber and parallel strand lumber, are based on definitions in ASTM D 5456 (16).

C8.1.3 Identification

Structural composite lumber is typically identified by product grade and company name, plant location or number, quality assurance agency name or logo, code evaluation report numbers, and a means for establishing the date of manufacture.

C8.1.4 Service Conditions

Structural composite lumber (SCL) is typically used in dry service conditions (where moisture content of sawn lumber is less than 16%). SCL manufacturers state dry service conditions based on the moisture content of lumber since the equilibrium moisture content (EMC) of SCL is lower under the same environmental conditions and varies based on the manufacturing process. For other conditions, the SCL manufacturer should be consulted.

C8.2 Reference Design Values

Structural composite lumber is a proprietary product and design values vary among manufacturers and product lines. Reference design values should be obtained from the manufacturer through the manufacturer's literature or code evaluation report.

C8.3 Adjustment of Reference Design Values

C8.3.1 General

Applicable adjustment factors for structural composite lumber are specified in Table 8.3.1 of the Specification.

C8.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2.

C8.3.3 Wet Service Factor, C_M

Structural composite lumber is limited to use in dry service conditions unless specifically allowed by the manufacturer (see NDS 8.1.4).

C8.3.4 Temperature Factor, C_t

See C2.3.3.

C8.3.5 Beam Stability, C_L

See C3.3.3.

C8.3.6 Volume Factor, C_V

C8.3.6.1 Volume effects of SCL beams are two dimensional in that increasing the width does not result in a strength reduction. Further, since SCL bending properties are established based on testing at span-to-depth ratios between 17 and 21 (representative of most applications where bending strength controls), the adjustment is based only on member depth. The volume factor is determined in accordance with ASTM D5456 (16) and uses an exponent unique to each manufacturer (based on variability of the product) to adjust the reference bending design value from a reference depth to the end-use depth in application. C_V is permitted to be greater than 1 when the end-use member depth is shallower than the reference depth.

C8.3.6.2 Similar to the volume factor for bending in SCL, ASTM D5456 requires development of a volume factor for tension which can be considered a length factor, adjusting from the length used in testing to the end-use length in application. As with bending, the exponent for the volume factor in tension is unique to each manufacturer.

The SCL manufacturer may also choose to publish reference tension design values based on a reference length longer than tested, in which case C_V is permitted to be greater than 1 when the end-use member length is shorter than the reference length.

C8.3.7 Repetitive Member Factor, C_r

The repetitive member factor for SCL is based on assumptions used to develop the repetitive member factor for sawn lumber (see C4.3.9), except that lower strength and stiffness variability limits the magnitude of the factor.

C8.3.8 Column Stability Factor, C_P

See C3.7.1.

C8.3.9 Bearing Area Factor, C_b

See C3.10.4.

C8.3.10 Pressure-Preservative Treatment

Per NDS 8.1.4, structural composite lumber is limited to use in dry service conditions unless specifically allowed by the manufacturer. Manufacturers should be consulted for any applications that require preservative treatment.

C8.3.11 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C8.3.12 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C8.3.13 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

C8.4 Special Design Considerations

C8.4.1 Notches

The designer has the responsibility of determining if structural composite lumber bending members should be notched and how load-carrying capacity should be calculated. Current good engineering practice is to avoid all notching of such bending members on the tension side except where end notching at the supports is necessary. This end notching is limited to 1/10 of the bending member depth, similar to structural glued laminated timber (140). The methods of NDS 3.4.3, used to calculate shear force at end notches in sawn lumber and structural glued laminated timber members, are permitted for structural composite lumber under the same design assumptions. Where different assumptions are made, the manufacturer should be consulted.

C9 WOOD STRUCTURAL PANELS

C9.1 General

C9.1.1 Scope

The general requirements given in Chapters 1, 2, and 3 of the Specification are applicable to wood structural panels except where indicated otherwise. Chapter 9 of the Specification contains provisions that specifically apply to wood structural panels manufactured in accordance with USDOC PS 1 (150) or PS 2 (151). The provisions of NDS Chapter 9 contain only the basic requirements applicable to engineering design of wood structural panels. Specific requirements, such as the wet service factor, the Grade and Construction factor, and the panel size factor are available from the wood structural panel manufacturer or the qualified agency.

C9.1.2 Identification

C9.1.2.1 Panel grades for plywood manufactured in conformance with USDOC PS 1 (150), *Construction & Industrial Plywood*, are designated by the grade of the face and back veneers (e.g., C-D, C-C, A-C, etc.) or by intended end-use (e.g., Underlayment, Marine, Concrete Form, etc.). Corresponding grade names in PS 1 for Sheathing, Structural I Sheathing, and Single Floor are C-D, Structural I C-D, and Underlayment, respectively.

Panel grades for products manufactured in conformance with USDOC PS 2 *Performance Standard for Wood-Based Structural-Use Panels* (151), are identified by intended end-use and include: Sheathing, Structural I Sheathing, and Single Floor.

Sheathing grade panels are intended for use as structural covering material for roofs, subfloors, and walls. Structural I sheathing panels meet increased requirements for cross-panel strength and stiffness and are typically used in panelized roof systems, diaphragms, and shear walls. Single Floor grade panels are used as a combination subfloor and underlayment and may be used under several different types of finish flooring as well as subflooring in a two-layer floor system with underlayment.

Bond classification is related to the moisture resistance of the glue bond under intended end-use conditions and does not relate to the physical (i.e., erosion, ultraviolet) or biological

(i.e., mold, fungal decay, insect) resistance of the panel. Structural-use panels manufactured in conformance with PS 1 or PS 2 must meet the bond classification requirements for *Exterior* or *Exposure 1*.

Exterior is defined in PS 1 and PS 2 as a bond classification for panels that are suitable for repeated wetting and redrying or long-term exposure to weather or other conditions of similar severity. *Exterior* plywood is manufactured with a minimum C-grade veneer.

Exposure 1 is defined in PS 1 and PS 2 as a bond classification for panels that are suitable for uses not permanently exposed to the weather. Panels classified as *Exposure 1* are intended to resist the effects of moisture on structural performance due to construction delays or other conditions of similar severity.

C9.1.2.2 Span ratings indicate the maximum on center spacing of supports, in inches, over which the panels should be placed for specific applications. The span rating system is intended for panels that are applied with the strength axis across two or more spans. The strength axis is typically the axis parallel to the orientation of oriented strand board (OSB) face strands or plywood face veneer grain and is the long dimension of the panel unless indicated otherwise by the manufacturer.

The span rating for Sheathing grade panels is provided as two numbers separated by a slash (e.g., 32/16 or 48/24). The first number is the maximum recommended on center support spacing in inches for roof applications. The second number is the maximum recommended on center support spacing when the panel is used for subflooring in residential and many light commercial applications. For example, a panel with a span rating of 32/16 may be used for roof sheathing over supports spaced up to 32 inches on center or as a subfloor over supports spaced up to 16 inches on center.

Recommendations for use of Sheathing grade panels also include wall applications. Panels with roof span ratings of 16 or 20 may be installed with their strength axis either parallel or perpendicular to the wall studs space at 16 inches or less on center. Similarly, panels with roof

span ratings of 24 or maximum may be installed with their strength axis either parallel or perpendicular to the wall studs spaced at 24 inches or less on center.

Sheathing grade panels may also be used in wall applications, according to manufacturers' recommendations, both parallel and perpendicular to studs. Sheathing panels with span ratings of Wall-16 or Wall-24 are for use only as wall sheathing. The numerical index (16 or 24) corresponds to the maximum on center spacing of the studs. Wall sheathing panels are typically performance tested with the strength axis parallel to the studs. For this reason, wall sheathing panels may be applied with either the strength axis parallel to the supports or perpendicular to the supports.

The span rating for Single Floor grade panels appears as a single number and represents the maximum recommended on center (oc) support spacing in inches. Typical span ratings for Single Floor products are 20 oc and 24 oc, although 16 oc, 32 oc, and 48 oc panels are also available.

C9.1.3 Definitions

C9.1.3.3 Oriented strand board (OSB) was first commercially introduced in the early 1980s succeeding "waferboard." Waferboard is a mat-formed panel product that utilizes random distribution of rectangular wafers, whereas OSB is a mat-formed panel product with oriented layers resulting in directional properties.

C9.1.3.4 The term "ply" refers to the individual sheets of veneer used to construct plywood. A "layer" is defined as a single ply of veneer or two or more adjacent plies with grain oriented in the same direction. Veneer is classified into the following six grades:

- N:** Highest grade level. No knots, restricted patches.
- A:** Higher grade level. No knots, allows more patches than N-grade but quantity of patches is also restricted.

B: Solid surface-Small round knots. Patches and round plugs are allowed.

C Plugged: Special improved C grade.

C: Small knots, knotholes, patches. Lowest grade allowed in Exterior plywood.

D: Larger knots, knotholes, some limited white pocket in sheathing grades.

C9.1.4 Service Conditions

C9.1.4.1. When the equilibrium moisture content of wood structural panels in service is less than 16 percent (including either Exposure 1 or Exterior bond classification), the dry service design values apply. A dry service condition prevails in most covered structures. However, members used in interior locations subject to high humidity, such as may occur in certain industrial operations or over unventilated swimming pools, may reach an equilibrium moisture content of 16 percent or more. In such conditions, wet service factors (see C9.3.3) should be applied to reference design values. However, preservative treated panels should be used where there is a potential for wood decay such as when panels maintain an in-service MC of 19% or more, either from sustained high humidity levels or prolonged exposure to moisture.

Exterior exposures that are not protected from the weather are generally considered wet conditions of use. Wet service adjustment factors, C_M , are provided in Commentary Table C9.3.3 for uses where this limit will be exceeded for a sustained period of time or for repeated periods. Uses in which the panel is in contact with the ground should be considered wet use for those portions of the panel that will attain a moisture content of 16 percent or more. Where wet service conditions apply, the need for preservative treatment should be considered, as untreated panels used in these conditions are susceptible to degradation from fungal decay.

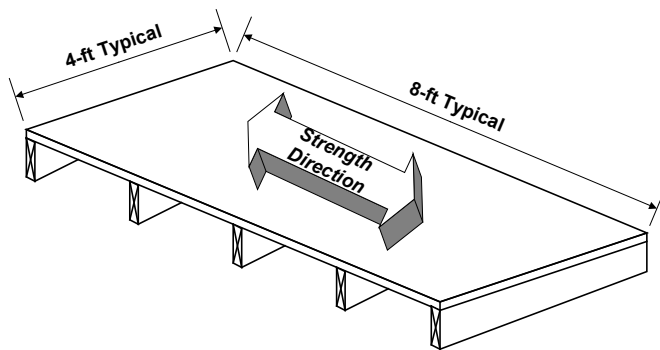
C9.2 Reference Design Values

C9.2.1 Panel Stiffness and Strength

C9.2.1.1 Minimum design stress values for wood structural panels are available from the panel manufacturer or the qualified agency for the panel grade and span rating. These unit design stress values, where provided, can be combined with the design section properties (see C9.2.4) to calculate panel stiffness and strength design capacities.

Panel stiffness and strength design capacities for specific panels may be available from the panel manufacturer.

C9.2.1.2 Structural panels have a strength axis direction and a cross panel direction (see Figure C9.2.1). 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 the long dimension of the panel unless indicated otherwise by the manufacturer.

Figure C9.2.1 Structural Panel with Strength Direction Across Supports

C9.2.2 Strength and Elastic Properties

Reference strength and stiffness design values are available from the panel manufacturer (see C9.2.1.1).

C9.2.3 Design Thickness

Section properties associated with the nominal panel thickness should be used in design calculations (see C9.2.4), unless otherwise indicated. The relationship between the nominal panel thickness and the span rating is provided in Table C9.2.3.

Table C9.2.3 Relationship Between Span Rating and Nominal Thickness

Span Rating	Nominal Thickness (in.)										
	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	7/8	1	1-1/8
Sheathing											
24/0	P	A	A	A							
24/16		P	A	A							
32/16			P	A	A	A					
40/20					P	A	A	A			
48/24							P	A	A		
Single Floor											
16 oc					P	A					
20 oc					P	A					
24 oc							P	A			
32 oc									P	A	
48 oc											P

P = Predominant nominal thickness for each span rating.

A = Alternative nominal thickness that may be available for each span rating. Check with suppliers regarding availability.

C9.2.4 Design Section Properties

The section properties associated with the nominal panel thickness and span rating are provided in Table C9.2.4. These values should be used with the panel stiffness and strength design stress values. Alternatively, these values can be combined with the panel stiffness and strength design stress values to provide panel stiffness and strength design capacities (see C9.2.1.1).

Table C9.2.4 Panel Section Properties¹

Nominal Thickness (in.)	Approximate Weight ² (psf)	Thickness <i>t</i> (in.)	Area <i>A</i> (in. ² /ft.)	Moment of Inertia <i>I</i> (in. ⁴ /ft.)	Section Modulus <i>S</i> (in. ³ /ft.)	Statical Moment <i>Q</i> (in. ³ /ft.)	Shear Constant <i>I_b/Q</i> (in. ² /ft.)
3/8	1.1	0.375	4.500	0.053	0.281	0.211	3.000
7/16	1.3	0.437	5.250	0.084	0.383	0.287	3.500
15/32	1.4	0.469	5.625	0.103	0.440	0.330	3.750
1/2	1.5	0.500	6.000	0.125	0.500	0.375	4.000
19/32	1.8	0.594	7.125	0.209	0.705	0.529	4.750
5/8	1.9	0.625	7.500	0.244	0.781	0.586	5.000
23/32	2.2	0.719	8.625	0.371	1.033	0.775	5.750
3/4	2.3	0.750	9.000	0.422	1.125	0.844	6.000
7/8	2.6	0.875	10.500	0.670	1.531	1.148	7.000
1	3.0	1.000	12.000	1.000	2.000	1.500	8.000
1-1/8	3.3	1.125	13.500	1.424	2.531	1.898	9.000

1. Properties based on rectangular cross section of 1-ft. width.

2. Approximate plywood weight for calculating actual dead loads. For OSB and composite panels, increase tabulated weights by 10%.

C9.3 Adjustment of Reference Design Values

C9.3.1 General

Applicable adjustment factors for wood structural panels are specified in Table 9.3.1 of the Specification.

C9.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2.

C9.3.3 Wet Service Factor, C_M , and Temperature Factor, C_t

Wet Service Factor

Design capacities for panels can be used without adjustment for moisture effects where the panel moisture content in service is expected to be less than 16% (see C9.1.4). Adjustment factors for conditions where the panel moisture content in service is expected to be 16% or greater should be obtained from the manufacturer, industry associations, or third-party inspection agencies. Wet service adjustment factors traditionally used include:

Table C9.3.3 Wet Service Factor, C_M

Reference Design Capacity	C_M
Strength ($F_b S$, $F_t A$, $F_c A$, $F_s (I_b/Q)$, $F_v t_v$)	0.75
Stiffness (EI , EA , $G_v t_v$)	0.85

Wood structural panels used in structural applications such as roof and wall sheathing, subfloors, diaphragms, and built-up members must be manufactured with either an “Exposure 1” or “Exterior” bond classification (see C9.1.2).

Temperature Factor

The temperature factor, C_t , shall be applied when wood structural panels are exposed to in-service sustained temperatures in excess of 100°F (see C2.3.3). In the range of 100°F to 200°F, the temperature factor is applicable only when the moisture content of the wood structural panels can be expected to remain at or above 12%. The rationale behind the latter recommendation is that the strength increases due to panel drying under the higher temperature is sufficient to offset the strength decreases due to the temperature itself. The temperature factor can be estimated using the following equation:

$$C_t = 1.0 - 0.005 (T - 100) \quad (C9.3-1)$$

where:

T = Temperature (°F)

C9.3.4 Panel Size Factor, C_s

Design capacities for bending strength and axial tension strength can be used without a panel size adjustment for panels that are 24 inches or greater in width (i.e., dimension perpendicular to the strength axis). For panels

less than 24 inches in width, the capacities should be reduced by applying the appropriate panel size adjustment factor, C_s , as shown in NDS Table 9.3.4 or obtained from the manufacturer, industry associations, or third-party inspection agency.

C9.3.5 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C9.3.6 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C9.3.7 Time Effect Factor, λ (LRFD Only)

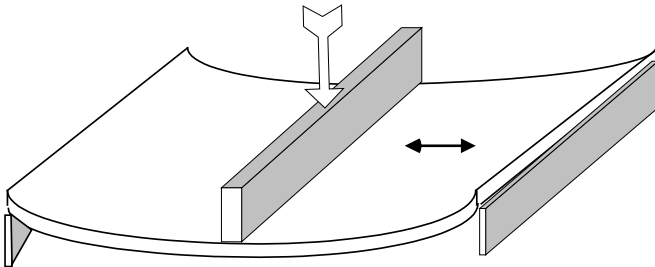
See C2.3.7.

C9.4 Design Considerations

C9.4.1 Flatwise Bending

Special care should be taken to ensure that the section properties associated with the proper strength axis are used to calculate the bending capacity of the panel (see Figure C9.4.1).

Figure C9.4.1 Example of Structural Panel in Bending



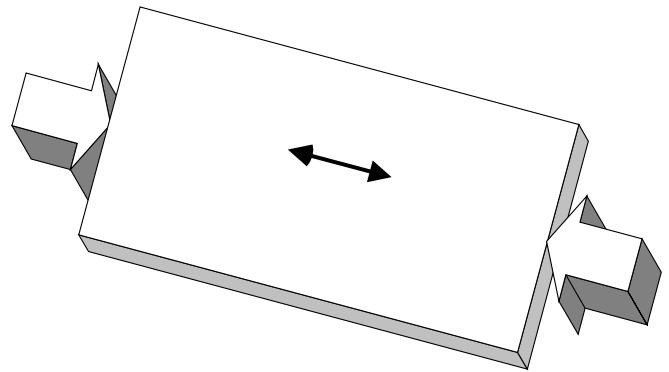
C9.4.2 Tension in the Plane of the Panel

Special care should be taken to ensure that the section properties associated with the proper strength axis are used to calculate the tensile capacity of the panel.

C9.4.3 Compression in the Plane of the Panel

Special care should be taken to ensure that the section properties associated with the proper strength axis are used to calculate the compression capacity of the panel (see Figure C9.4.3).

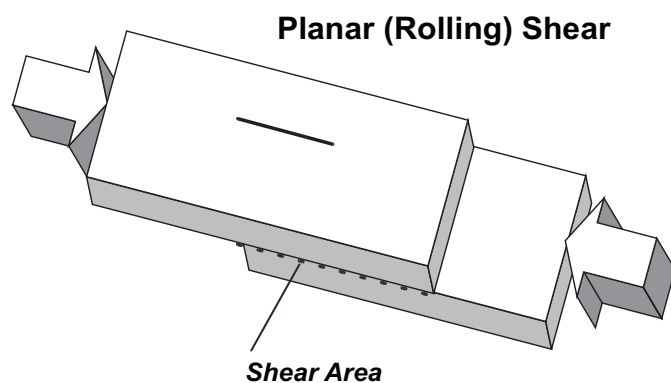
Figure C9.4.3 Structural Panel with Axial Compression Load in Plane of the Panel



C9.4.4 Planar (Rolling) Shear

Special care should be taken to ensure that the section properties associated with the proper strength axis is used to calculate the planar shear (also called shear-in-the-plane or rolling shear) capacity of the panel (see Figure C9.4.4).

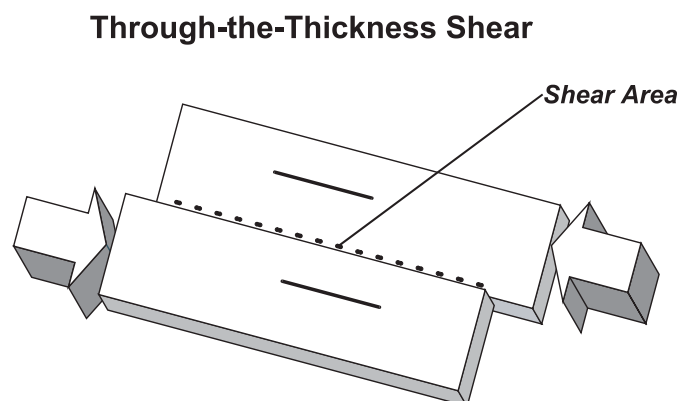
Figure C9.4.4 Shear-in-the-Plane for Wood Structural Panels



C9.4.5 Through-the-Thickness Shear

The section property for shear-through-the-thickness is the same both along the panel axis and across the panel axis (see Figure C9.4.5).

Figure C9.4.5 Through-the-Thickness Shear for Wood Structural Panels



C9.4.6 Bearing

The design bearing stress on the panel face is independent of panel axis orientation.

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's literature or code evaluation reports. Code evaluation reports, where available for a specific product and application, contain information to assist in the approval and use of the specific product in accordance with requirements of the applicable building code.

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.

C10.3 Adjustment of Reference Design Values

C10.3.1 General

Applicable adjustment factors for cross-laminated timber are specified in Table 10.3.1 of the Specification.

C10.3.2 Load Duration Factor, C_D (ASD Only)

Load duration factors for CLT are the same as for other wood products (see C2.3.2) with the exception of rolling shear, $F_s(Ib/Q)_{eff}$, which is not adjusted in CLT since the value is limited by the tendency of the crossing laminations to “roll over” rather than exhibit a strength limit state associated with shear stress in the laminations.

C10.3.3 Wet Service Factor, C_M

See C10.1.5

C10.3.4 Temperature Factor, C_t

See C2.3.3.

C10.3.5 Curvature Factor, C_c

Design of curved CLT is beyond the scope of the NDS. The CLT manufacturer should be contacted to determine whether curved product is available and, if so, the proper design.

C10.3.6 Beam Stability Factor, C_L

Cross-laminated timber loaded in out-of-plane bending, such as vertical loads on floor slabs or lateral loads on walls, are not susceptible to lateral-torsional buckling and $C_L=1.0$. Cross-laminated timber panels loaded by in-plane bending are susceptible to lateral-torsional buckling and should be adjusted by the beam stability factor. Design capacities for CLT used as beams or headers over door and window openings have not been standardized and are not available in PRG 320. For beam applications, consult the CLT manufacturer for design capacities.

C10.3.7 Column Stability Factor, C_P

Cross-laminated timber panels loaded by in-plane compression are susceptible to buckling and should be adjusted by the column stability factor. Due to the signifi-

cant shear deformation that can occur between the parallel and perpendicular CLT laminations, the effective bending stiffness, $(EI)_{eff}$, should be adjusted for shear stiffness per NDS 10.4.1 using a Shear Deformation Adjustment Factor, K_s , of 11.8 associated with the loading case of a constant moment.

C10.3.8 Bearing Area Factor, C_b

See C3.10.4.

C10.3.9 Pressure-Preservative Treatment

Per NDS 10.1.5, cross-laminated timber is limited to use in dry service conditions unless specifically allowed by the manufacturer. Manufacturers should be consulted for applications that require preservative treatment.

C10.3.10 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C10.3.11 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C10.3.12 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

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.

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

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

To estimate $(EI)_{app-min}$ 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:

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

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:

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

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:

$$\frac{k_b WL^3}{EI_{app}} = \frac{k_b WL^3}{EI_{eff}} + \frac{6k_s WL}{5GA_{eff}} \quad (C10.4.1-5)$$

Solving for EI_{app} :

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

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 $\delta_s = 4B\pi^2$.

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	B	$4B\pi^2$	47.4

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.

C11 MECHANICAL CONNECTIONS

C11.1 General

C11.1.1 Scope

C11.1.1.2 See C3.1.3, C3.1.4, and C3.1.5.

C11.1.1.3 The adequacy of alternate methods or procedures for designing and verifying the reference design values of connections that differ from those in the Specification is the responsibility of the designer or the authority accepting or approving such alternate methods or procedures. This responsibility includes providing for appropriate margins of safety; assuring the applicability of load duration, wet service and other adjustment factors in the Specification; and confirming the applicability of test results to field fabrication and service conditions (see C1.1.1.3).

C11.1.2 Stresses in Members at Connections

All connection designs should be checked for conformance of structural members to the net section area requirements of NDS 3.1.2 and the shear design provisions of NDS 3.4.3 (see C3.1.2 and C3.4.3). All single shear or lapped joints also should be checked to determine the adequacy of the member to resist the additional stresses induced by the eccentric transfer of load at the joint (see NDS 3.1.3). This often will involve bending and compression or bending and tension interaction where the bending moment induced by the eccentric load at the joint results in bending about the weak axis of the member.

Where multiple fasteners are used, the capacity of the fastener group may be limited by wood failure at the net section or by tear-out around the fasteners caused by local stresses. One method for evaluating member strength for local stresses around fastener groups is outlined in NDS Appendix E.

C11.1.3 Eccentric Connections

Fastener eccentricity that induces tension perpendicular to grain stresses in the main wood member at the connection should be avoided. Where multiple fasteners

occur with eccentricity, fasteners are to be placed, insofar as possible, such that the wood between them is placed in compression rather than in tension (see NDS Figure 11A).

In 1948, provisions for shear design of bending members at connections were introduced in an attempt to limit tension perpendicular to grain stresses at eccentric connections. In 1982, a provision was added prohibiting eccentric connections that induce tension perpendicular to grain stresses unless it has been shown by analysis or testing that such joints can safely carry all applied loads. The determination of the type and extent of the analysis and/or testing required to demonstrate the adequacy of eccentric connections that induce tension perpendicular to grain stresses in the wood members is the responsibility of the designer. Use of stitch bolts or plates to resist such stresses when they cannot be avoided is a common practice.

It is to be emphasized that tension design values perpendicular to grain are not given in the Specification (see C3.8.2).

C11.1.4 Mixed Fastener Connections

The individual fasteners in a connection should generally be of the same size to assure comparable load-slip or stiffness characteristics. Such equivalency is required to obtain appropriate distribution of load among fasteners in the connection and is a condition for use of the group action factor, C_g , of NDS 11.3.6.

It is recognized that some designers have used different fastener types in the same connection where the addition of one or more fasteners of the type being used is precluded by area restrictions or is considered uneconomical. Such mixed-type connections, for example, the use of a single 1/2-inch bolt with three split-ring connectors or the use of a 16d nail with two 1/2-inch bolts, are not covered by the design provisions of the Specification. Because of the different load-slip behavior of different fastener types, the allowable load on such connections cannot be assumed to

be the sum of the allowable loads for each fastener type, even when the different types are in different rows.

Allowable loads for connections employing more than one type or size of fastener shall be based on analyses that account for different connection stiffnesses, on test results, or on field experience (see C1.1.1.3). It is the designer's responsibility to assure that load capacities assigned to such connections contain adequate margins of safety and are achievable under field conditions.

C11.1.5 Connection Fabrication

Design values for connection joints have been applied to connections having both tight and loose nuts. This pro-

vision is based on the original bolted joint tests used to establish design values in which the nuts were intentionally not tightened in order to simulate the additional shrinkage that can occur during service (146). It is to be noted that these provisions only apply to the loosening of nuts that may occur from shrinkage and not the effects of moisture on bearing strength or the effects of checks and cracks that may occur from seasoning after fabrication. Reduction of connection design values for these factors is required when connections are assembled with wet or partially seasoned wood (see NDS 11.3.3).

C11.2 Reference Design Values

C11.2.1 Single Fastener Connections

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 dowel 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 design value for the connection.

The yield limit model provides a consistent basis for establishing the relative effects of side and main member thickness and bearing strength, and fastener bending strength on the load-carrying capacity of connections involving dowel-type fasteners. Because the yield strength of a wood connection is not well defined on the load-deformation curve for a connection, the limiting wood stresses used in the yield model 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 load for the material and for the connection.

Reference lateral design values for connections in previous editions of the Specification represented nominal proportional limit values. For purposes of transition and to build on the long record of satisfactory performance obtained with these previous values, short-term design values based on direct application of the yield limit equations have been reduced to design levels published in previous editions for connections made with equivalent species and member sizes. This calibration was accomplished by establishing average ratios of previous Specification design values to yield limit model design values for each yield mode and direction of loading (parallel and perpendicular to grain). This soft conversion procedure retained historical safety levels while resulting in some design values for each fastener type being somewhat higher and some lower than previous values depending upon the fastener diameter and the thickness of main and side member.

C11.2.2 Multiple Fastener Connections

The reference design value for a connection containing two or more fasteners is obtained by summing the reference design values for each individual fastener. It is to be understood that this provision requires application of the group action factor of NDS 11.3.6 to the individual fastener reference design value wherever a row of two or more split ring connectors, shear plate connectors, or dowel-type fasteners are involved.

Summation of individual fastener reference design values to obtain a total reference design value for a connection containing two or more fasteners is limited to designs involving the same type and the same size of fas-

tener (see C11.1.4). Fasteners of the same type, diameter, and length joining the same members and resisting load in the same shear plane may be assumed to exhibit the same yield mode.

C11.2.3 Design of Metal Parts

Metal parts, including fasteners, are to be designed in accordance with national standards of practice and specifications applicable to the material. Tension stresses in fasteners as a result of withdrawal loads, shear in cross-sections of fasteners, bearing of fasteners on metal side plates, tension and shear of plates, and buckling of plates and rods are included under this provision.

Standard metal design practices are not to be used to account for bending stresses occurring in dowel-type fas-

teners in wood connections subject to lateral loads. These stresses are accounted for in this Specification under the provisions for the particular fastener type involved. Where the design value for a connection involving metal fasteners is limited by the provisions of this Specification, the adjustment factors of NDS 11.3 are to be applied. Where the design value of the connection is limited by the strength of the metal fastener or part, the adjustment factors of NDS 11.3 are not to be applied.

C11.2.4 Design of Concrete or Masonry Parts

Concrete or masonry parts are to be designed in accordance with national standards of practice and specifications applicable to the material.

C11.3 Adjustment of Reference Design Values

C11.3.1 Applicability of Adjustment Factors

Applicable adjustment factors for connections subject to lateral loads, withdrawal loads, and fastener head pull-through loads are specified in Table 11.3.1 of the Specification.

C11.3.2 Load Duration Factor, C_D (ASD Only)

See C2.3.2. Reference design values for wood connections derived from the results of standard short-term tests (5-10 minute duration) and/or calculated using properties derived from short-term tests include a 1.6 reduction to account for the potential effects of long-term loading. When wood connections are used to resist short-term loads, the reference design values can be increased by a factor of up to 1.6 based on the provisions of NDS 2.3.2. Load duration factors greater than 1.6, including the impact load duration factor of 2.0, are not to be applied to design loads for connections.

C11.3.3 Wet Service Factor, C_M

The wet service factors in NDS Table 11.3.3 for bolts and lag screws, split ring and shear plate connectors, wood screws, and nails were recommended as part of early research on wood connections (184, 181).

The 0.80 factor for metal plate connectors installed in partially seasoned or wet lumber is based on the results of both truss and tension in-line joint tests (1,109,195).

The factor of 0.40 in NDS Table 11.3.3 for multiple rows of dowel fasteners installed in partially seasoned wood used in dry conditions of service is based on limited tests of connections fabricated with unseasoned members joined at right angles to each other and tested after drying (181).

C11.3.4 Temperature Factor, C_t

The temperature adjustment factors for connections in NDS Table 11.3.4 are equivalent to those for bending, compression, and shear design values in NDS 2.3.3 (see C2.3.3). Bearing under metal fasteners is closely correlated with compression parallel to grain or compression perpendicular to grain properties.

C11.3.5 Fire Retardant Treatment

See C2.3.4.

C11.3.6 Group Action Factors, C_g

Modification factors for two or more split ring connectors, shear plate connectors, or dowel-type fasteners in a row were added to the Specification in the 1973 edition. Earlier tests of bolted and shear plate connector joints had shown that the load capacity of connections containing multiple fasteners in a row was not directly proportional to

the number of fasteners, with those located near the ends of the row carrying a greater proportion of the applied load than those located in the interior of the row (35, 39, 40, 66, 72).

The tables of factors included in the 1973 edition to account for the non-uniform loads on a row of fasteners was based on a linear analysis wherein the direct stresses in the main and side members of the connection were assumed to be uniformly distributed across their cross section, and the relationship between fastener slip and fastener load was assumed to be linear (77). This analytical procedure showed that the transfer of load from side to main members and the proportion of the total load carried by each fastener were determined by the moduli of elasticity (E) and cross-sectional areas of the side and main members, the number of fasteners in a row, the spacing between fasteners, and the joint load/slip modulus.

Two tables of modification factors for joints containing two or more fasteners in a row were developed using the linear analysis: one for connections with wood side plates and one for connections with metal side plates. For purposes of simplicity, factors were tabulated only in terms of the number of fasteners in the row and the cross-sectional areas of the members being joined. Other variables were assumed to have the following values (156):

Wood to wood connections:

E of side and main members	1,800,000 psi
Load-slip fastener modulus	220,000 lbs/in.
Spacing between fasteners	6.5 inches

Wood to metal connections:

E of main member	1,400,000 psi
Load-slip fastener modulus	330,000 lbs/in.
Spacing between fasteners	5.75 inches

With the foregoing constant values, the analytical procedure was used to calculate modification factors for 3 to 8 fasteners in a row and then results were extrapolated up to 12 fasteners and down to 2 fasteners in a row (156). The resulting tables of factors, ranging from 1.00 for two fasteners in a row to as low as 0.34 and 0.15 for 12 fasteners in a row in joints made with wood and metal side plates, respectively, were continued essentially unchanged through the 1986 edition. The group action factor equation given in NDS 11.3.6 consolidated the analytical procedure used to establish the modification factors given in previous editions (188). Concurrent with the development of the compact single equation for accounting for group action, more recent load-slip data for bolted joints and split ring and shear plate connectors have been used to establish

new representative load-slip moduli for different types of connections (188).

It is to be noted that the variable A_s in the group action equation (NDS Equation 11.3-1) represents the sum of the cross-sectional area of the side members. Thus the equation accounts for single shear as well as double shear connections. For a connection with four or more members, each shear plane is evaluated as a single shear connection (see NDS 12.3.8). Where such a connection contains two or more fasteners in a row, a group action factor is calculated for each shear plane using an A_s based on the thinnest member adjacent to the plane being considered.

Perpendicular to Grain Loading. The number of fasteners in a row perpendicular to grain are generally limited in order to avoid splitting that can occur as a result of drying (see C11.3.3). When a row of multiple fasteners are used perpendicular to grain, it is standard practice to use the same group action factor as that for fasteners aligned parallel to grain. This practice is based on the assumption that use of the member and connection stiffnesses perpendicular to grain ($E_{\perp}$ and $\gamma_{\perp}$) in NDS Equation 11.3-1 would result in similar group action factors.

C11.3.7 Format Conversion Factor, K_F (LRFD Only)

See C2.3.5.

C11.3.8 Resistance Factor, ϕ (LRFD Only)

See C2.3.6.

C11.3.9 Time Effect Factor, λ (LRFD Only)

See C2.3.7.

C12 DOWEL-TYPE FASTENERS

C12.1 General

C12.1.1 Scope

Chapter 12 groups the design for a range of fasteners known as “dowel-type” fasteners. These include fasteners such as bolts, lag screws, wood screws, nails, spikes, drift bolts, and drift pins.

C12.1.3 Bolts

C12.1.3.1 ANSI/ASME Standard B18.2.1 *Square and Hex Bolts and Screws (Inch Series)*, is the quality reference standard for bolts. Bolt design provisions and tabulated bolt design values apply only to bolts having diameters of 1 inch or less. This limit was in response to reported field problems with connections involving large diameter bolts in structural glued laminated timber members and the results of research (31, 135). The latter showed drying in service, workmanship variables, and perpendicular to grain load components could interact to affect the capacity of connections made with multiple large diameter, relatively stiff bolts. Use of these procedures to establish reference design values for large diameter bolted connections is the sole responsibility of the designer.

C12.1.3.2 Generally, smaller diameter bolts will use the smaller oversize hole value and larger bolts the larger oversize value. The same target oversize is to be used for all holes in the same connection. Proper alignment, especially in groups of fasteners, is required to properly distribute the load into each fastener. Forcible driving of the fastener can damage the wood bearing surface and reduce the capacity of the connection.

C12.1.3.3 Use of standard cut washers or equivalent metal parts under the head and nut prevent localized crushing of the wood at bolt holes.

C12.1.3.4 Edge distance, end distance, and fastener spacing requirements have been consolidated for dowel-type fasteners in NDS 12.5.

C12.1.4 Lag Screws

C12.1.4.1 ANSI/ASME Standard B18.2.1 *Square and Hex Bolts and Screws (Inch Series)*, is the quality reference standard for lag screws. It provides standard hex lag 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 lag screws that are to be used. Bending yield strength of the lag screw (see NDS Appendix I) is a required input variable to the yield equations of NDS 12.3.1. Additionally, the actual tensile stress in the lag screw at the root diameter must be checked when designing lag screw connections for withdrawal (see NDS 11.2.3).

C12.1.4.2 Lead hole requirements for three specific gravity classes are based on early lag screw research involving tests of Douglas fir, southern pine, white oak, redwood, and northern white pine (100).

C12.1.4.3 Provision for allowing 3/8 inch and smaller diameter lag screws loaded primarily in withdrawal to be inserted without a lead hole in wood of medium to low specific gravity was added to address the use of small lag screws. On the basis of field experience, early lag screw research (100), and information on the withdrawal resistance of tapping screws inserted with different size lead holes (163), use of small lag screws without lead holes is deemed acceptable when the following conditions are met:

1. The lag screws are being loaded primarily in withdrawal.
2. The lag screws are inserted in wood with specific gravity (G) ≤ 0.5 .
3. Placement of lag screws avoids excessive splitting.

A lag screw subjected to combined withdrawal and lateral loading may be considered loaded primarily in withdrawal when the axis of the screw is at an angle of 75 degrees or more to the grain of the wood member holding the threaded portion of the screw. The requirement that unusual splitting be avoided when lead holes are not used is to be considered a performance requirement that (i) is related to the ability of the screw to hold the cleat or side member to the main or foundation member, and (ii) is applicable to both members being joined.

C12.1.4.5 A lubricant is sometimes used to facilitate lag screw insertion even when small diameter lag screws are inserted without the use of lead holes.

C12.1.4.6 Minimum length of lag screw penetration requirements, not including the length of the tapered tip, are provided to ensure that fasteners can achieve the design value calculated using the yield equations in NDS 12.3.1.

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 1/4", 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* (202) provides standard dimensions for common, box, and sinker steel wire nails, spikes, Roof Sheathing Ring Shank (RSRS) nails, and Post Frame Ring Shank (PF) nails (See NDS Appendix L). The Supplementary Requirements of ASTM F1667 regarding Nail Bending Yield Strength are required to be used for engineered construction. The method in which F_{yb} is determined for nails is incorporated in ASTM F1575. F_{yb} requirements are referenced in the Supplementary Requirements of ASTM F1667 or section 12.1.6 of the NDS. For any deformed shank nails (e.g. RSRS, PF), moment capacity is evaluated at the transition zone (between the smooth shank and deformations) and diameter, D , of the smooth shank is used in the calculation of F_{yb} . The resulting use of F_{yb} and D produces the appropriate yield moment of the nail when D is used in yield theory calculations. Additionally, the actual tensile stress in the driven fastener must be checked when designing driven fastener connections for withdrawal (see C11.2.3 of the Specification).

C12.1.6.3 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.

C12.1.6.4 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.

C12.1.6.5 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 1/4", 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):

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

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 $0.31 \leq G \leq 0.73$

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

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

$$K_w = 1.2 \left(\frac{7500}{5} \right) \quad (\text{C12.2.1-2})$$

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 $\frac{1}{4}"$ 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):

$$W = K_w G^2 D \quad (\text{C12.2.2-1})$$

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 $0.31 \leq G \leq 0.73$

D = wood screw thread diameter, in., where $0.138" \leq D \leq 0.372"$

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

$$K_w = \frac{14250}{6} \quad (\text{C12.2.2-2})$$

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 Smooth shank nails or spikes

(a) NDS Equation 12.2-3 is used to establish the smooth shank (bright or galvanized) nail and spike reference withdrawal design values given in NDS Table 12.2C. This equation is based on research (94, 95):

$$W = K_w G^{5/2} D \quad (C12.2.3-1)$$

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 $0.092" \leq D \leq 0.375"$

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

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

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).

(b) NDS Equation 12.2-4 is used to establish smooth shank stainless steel nail and spike reference withdrawal design values given in NDS Table 12.2D. Testing of A304 and A316 stainless steel nails has shown withdrawal capacity of smooth-shank stainless steel nails to be less than that of bright or galvanized carbon steel nails (203). This strength reduction is attributed to the reduced friction between nail and wood fiber. Withdrawal design values from Eq. 12.2-4 are one-fifth of the average ultimate values. Eq. 12.2-4 is slightly different than and produces smaller values than the equation proposed in the research to improve fit to data.

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

(c) The unit reference withdrawal design value in lbs/in. for nails and spikes is multiplied by the depth of penetration into a wood member, including the tip, to calculate the fastener reference withdrawal in pounds. The equations were developed assuming the tip was part of the penetration length.

C12.2.3.2 Deformed shank nails.

(a) In the 2012 edition of the Specification, provisions were added for Post-Frame Ring Shank nails and in the 2018 edition of the Specification, provisions were added for Roof Sheathing Ring Shank nails in accordance with ASTM F1667. The withdrawal design value equation (NDS

Equation 12.2-5) is based on research conducted at the Forest Products Laboratory.

(b) The constant of 1800 in NDS Equation 12.2-5 incorporates a 20% reduction to account for effects of galvanized coatings from testing in Southern pine rather than the average reduction of 15% from testing in all species (basswood, SPF, Douglas fir, Southern pine, and white oak) tested in the study (196). Therefore, when using uncoated carbon steel Post-Frame Ring Shank or Roof Sheathing Ring Shank nails, a 1.25 increase factor is permitted (e.g. $0.80 \times 1.25 = 1.0$). Effects of other coatings have not been quantified.

(c) The unit reference withdrawal design value in lbs/in. for Roof Sheathing Ring Shank nails and Post-Frame Ring Shank nails is multiplied by the depth of ring shank penetration into the side grain of a wood member, including the tip, to calculate the fastener reference withdrawal in pounds. The equations were developed assuming that the tip was part of the penetration length.

(d) Reference withdrawal design values for smooth shank nails are permitted to be used for other deformed shank nails of the same shank diameter, D , recognizing that tests of deformed shank nails indicate that shank deformations do not reduce the withdrawal value below the values for smooth shank nails. The permissible use of the smooth shank nail withdrawal value replaces criteria in the 2015 NDS and prior editions that recognized slightly larger withdrawal strengths for “threaded nails” with unspecified geometry of deformations.

C12.2.3.3 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. 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.

C12.2.3.4 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):

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

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).

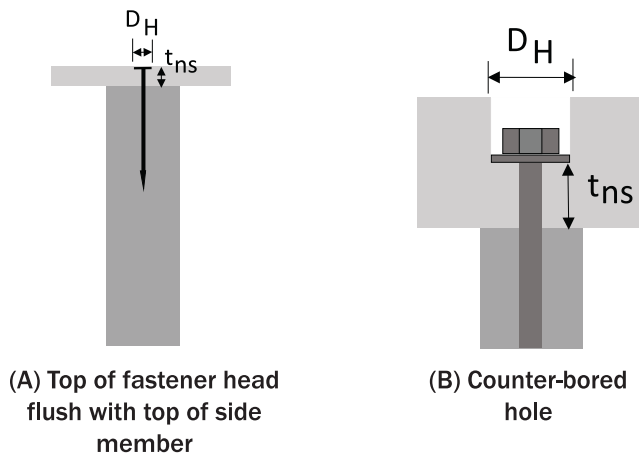
C12.2.5 Fastener Head Pull-Through

C12.2.5.1 Fastener head pull-through data used to set industry recommendations for wood structural panels, combined with historical results of pull-through data from tests of various fastener head shapes and washer diameters pulled through lumber and wood structural panels, were analyzed to develop fastener head pull-through provisions (204). NDS Equation 12.2-6a is based on the fastener head diameter model and was used to establish the reference pull-through design values, W_H , given in NDS Table 12.2F. NDS Equation 12.2-6b was added to limit the head pull-through capacity for net side member thicknesses greater than 2.5 times the fastener head diameter based on analysis of available data which indicated that the increased head pull-through capacity reached a limit where no further increase for side member thickness was appropriate.

For fasteners where the head is driven flush with the side member surface, the net side member thickness, t_{ns} , is the thickness of the side member (see Figure C12.2.5A). For fasteners that are installed in counter-bored holes, the net side member thickness is measured from the bottom of the hole (see Figure C12.2.5B).

Head pull-through equations in NDS Section 12.2.5 (see Equation C12.2.5-1) are for round-head fasteners and washers. For fasteners with other than round heads, such as clipped or oval head nails and hex head bolts and lag screws, analysis of underlying data is considered to support the use of a more general fastener head perimeter model (see Equation C12.2.5-2).

Figure C12.2.5 Net Side Member Thickness, t_{ns}



Fastener head diameter equation for establishing fastener head pull-through reference design values:

$$W_H = 690 \pi D_H G^2 t_{ns} \text{ for } t_{ns} \leq 2.5 D_H \quad (\text{C12.2.5-1})$$

Fastener head perimeter equation for establishing fastener head pull-through reference design values:

$$W_H = 690 P_H G^2 t_{ns} \text{ for } t_{ns} \leq 0.8 P_H \quad (\text{C12.2.5-2})$$

where:

πD_H = perimeter for fasteners with round heads, in.

D_H = fastener head diameter, in.

G = specific gravity of side member

t_{ns} = net side member thickness, in.

P_H = fastener head perimeter, P_H , in.

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% ($K_0 = 1.25$) 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 nomi-

nal 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 1/4 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 $D \geq 1/4$ " 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 loading direction is parallel to grain for the layer at the shear plane, the dowel bearing strength is the parallel to grain dowel bearing strength, $F_{e\parallel}$. 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 $D \geq 1/4"$ 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_{e0} 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_0 design value for the connection (157). Determining a Z_0 design value using this latter approach can be used as an alternative to calculating F_{e0} 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 $D \geq 1/4"$, 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 $F_{e\perp}/F_{e\parallel}$. 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 $F_{e\parallel}/F_{e\perp}$. 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

Reference lateral design values for both smooth shank and deformed shank nails are based on the shank diameter, D . In prior editions of the Specification, lateral design values for deformed shank nails used the root diameter, D_r , and a F_{yb} value associated with hardened medium carbon steel nails.

A change in ASTM F 1575 (205) clarified that nail bending yield strength, F_{yb} , is based on the shank diameter, D , not on the root diameter, D_r . This change in ASTM 1575 facilitated simplification of nail moment resistance in the NDS yield equations for nails specified in ASTM F1667 since D is provided for all nail types.

The reduced moment resistance in the threaded portion of dowel-type fasteners, other than nails, 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 .

One alternate method of accounting for the moment and bearing resistance of the threaded portion of the fastener and moment acting along the length of the fastener is provided in AWC's *Technical Report 12—General Dowel Equations for Calculating Lateral Connection Values* (137). A general set of equations permits use of different fastener diameters for bearing resistance and moment resistance in each member.

C12.3.8 Asymmetric Three Member Connections, Double Shear

Conservatively, the Specification requires the use of minimum side member bearing length and minimum dowel diameter in the calculation of design values for asymmetric three member connections. Inherent in this calculation is the assumption that the load to each side member is equivalent. Where other load distributions occur, more complex analysis may be needed.

C12.3.9 Multiple Shear Connections

The Specification requires evaluation of each individual shear plane using the yield limit equations of NDS 12.3.1 and then assigning the lowest value to the other shear planes. Interior members should be checked for the combined loading from the adjacent shear planes to ensure that sufficient bearing capacity exists (such as would exist in a double shear connection limited by Mode I_m).

C12.4 Combined Lateral and Withdrawal Loads

C12.4.1 Lag Screws and Wood Screws

Results of lag screw tests indicated that loading at an angle to the fastener axis to induce lateral and withdrawal components did not reduce the maximum connection capacity. However, when joint resistance was evaluated at the design load level, an interaction of the load components was observed with larger diameter screws at load angles less than 45° (87). Analysis at design load level was performed due to the differences in design level to maximum capacity ratios for lateral and withdrawal. NDS Equation 12.4-1 can also be used to determine the reference design value of lag screws or wood screws embedded at an angle to grain in the wood member and loaded in a direction

C12.3.10 Load at an Angle to Fastener Axis

Two member connections in which the load acts at an angle to the axis of the fastener are checked using the component of the load acting at 90° to the axis and member thicknesses equal to the length of the fastener in each member measured at the centerline of the fastener (see NDS Figure 12E). Reference design values for connections in which the load acts at an angle to the fastener axis are based on the yield limit equations of NDS 12.3.1. The lowest value of Z obtained, using t_m and t_s equal to the length of the fastener in each member, divided by the cosine of the angle of intersection of the two members is the maximum reference design value for the connection.

The adequacy of the bearing area under washers and plates to resist the component of force acting parallel to the fastener axis can be checked using adjusted compression design values perpendicular to grain, $F_{c\perp}'$.

C12.3.11 Drift Bolts and Drift Pins

Reference lateral design values for drift bolts or pins (181) are 75 percent of the reference design value for common bolts of the same diameter to compensate for the absence of head, nut, and washer. End distance, edge distance, and spacing requirements, and group action adjustments that are applicable to bolts, are also applicable to drift bolts and drift pins.

normal to the wood member. For this condition, α , would be defined as the angle perpendicular to the fastener axis.

C12.4.2 Nails and Spikes

It is assumed that current adjustments for toe-nailed connections address the effects of combined lateral and withdrawal loading and do not require further modification.

Research on the effects of combined lateral and withdrawal loading on nailed connections (37) involved tests of Engelmann spruce, Douglas fir, and red oak single shear connections made with 8d common nails. Nail penetration depths of 6, 10, and 14 diameters into the main member and load angles of 0 degrees, 90 degrees, and six intermediate directions were investigated. Two tests were conducted

at each load angle. The interaction equation found to best describe maximum connection load results for each species and penetration depth was of the form:

$$P = \frac{(1 + K \sin 2\alpha)(W'pZ')}{(W'p) \cos \alpha + (Z') \sin \alpha} \quad (\text{C12.4.2-1})$$

where:

P = maximum load at angle to grain

W' = maximum load at 90° (withdrawal load perpendicular to grain per inch of penetration in the main member)

p = depth of penetration

Z' = maximum load at 0° (lateral load)

α = angle between wood surface and direction of applied load

K = factor based on least squares analysis of test data for each species-penetration group

The average value of K for the six species and penetration groups evaluated was 0.535, and ranged from 0.151 to 1.406. Average K values by species were 0.432, 0.864, and 0.309 for Douglas fir, Engelmann spruce, and red oak, respectively. When K is conservatively assumed to equal 0, Equation C12.4.2-1 reduces to NDS Equation 12.4-2 or, in another format the following:

$$\frac{R_w}{(W'p)} + \frac{R_z}{Z'} \leq 1 \quad (\text{C12.4.2-2})$$

where:

R_w = connection withdrawal force

R_z = connection lateral force

C12.5 Adjustment of Reference Design Values

C12.5.1 Geometry Factor, C_Δ

C12.5.1.1 For dowel-type fasteners with diameters less than 1/4", no reduction for geometry is specified.

C12.5.1.2 For dowel-type fasteners with diameters equal to or greater than 1/4", the geometry factor provides a proportionate reduction of reference lateral design values for less than full end distance or less than full spacing distance. The lowest geometry factor for any fastener applies to all other fasteners in that same connection, not just to the end fastener or a pair of fasteners in a row. It should be noted that further reductions may be necessary when checking stresses in members at connections (see NDS 11.1.2).

The requirement that fastener design values for multiple shear plane connections or asymmetric three member connections be based on the application of the lowest geometry factor for any shear plane to all fasteners in the joint assumes that the total joint capacity is proportional to the number of shear planes.

End Distance: Requirements in NDS 12.5.1.2 (a) and NDS Table 12.5.1A for parallel to grain loading are based on early recommendations (146). For tension loads (fasteners bearing toward the member end), the minimum end distances of 7D for softwoods and 5D for hardwoods for $C_\Delta = 1.0$ were established by test. For compression loads (fasteners bearing away from the member end), the minimum end distance of 4D for $C_\Delta = 1.0$ was based on the minimum spacing of fasteners in a row for $C_\Delta = 1.0$

(146). End distances for angle to grain tension loadings may be linearly interpolated from those for perpendicular to grain and tension parallel to grain design values.

The provisions for use of reduced end distances for connections when proportionate reductions ($0.5 \leq C_\Delta \leq 1.0$) are made in design values are supported by early research (184, 181, 146) which showed a linear relationship between end distance and joint proportional limit strength. A subsequent study showed that a minimum end distance of only 5D was sufficient to develop the full proportional limit load of Douglas fir joints made with metal side plates and loaded in tension parallel to grain (119). Other research further substantiates the adequacy of the end distance requirements for connections loaded in both compression and tension parallel to grain (102, 110). End distances less than 50 percent of those required for $C_\Delta = 1.0$ are not allowed.

Shear Area: Requirements in NDS 12.5.1(b) are for members loaded at an angle to the fastener axis. End distance requirements are expressed in terms of equivalent shear areas. Shear area for such a joint is defined as the triangular area in the thickness plane of the member which is enclosed between the tip of the member and the centerline of the fastener (NDS Figure 12E). This shear area for the angled member is compared to the shear area of a joint in which both members are loaded perpendicular to the fastener axis (members parallel to each other) and which meet end distance requirements. The equivalent shear area

for the parallel member joint is the product of the required end distance and the length of the fastener in the member.

As with end distance requirements for parallel member connections, reduced shear areas less than 50 percent of those required for $C_A = 1.0$ are not allowed. It is recommended as good practice that the distance between the fastener axis and the inside juncture of the angled side member and the main member (see NDS Figure 12E) be at least $1.5D$.

Spacing Requirements for Fasteners in a Row: For fasteners in a row, the spacing requirements contained in NDS 12.5.1(c) and NDS Table 12.5.1B are assumed to be sufficient to cover the effects of non-uniform distribution of shear stresses through the thickness of the member (concentrated at the edges) that occur as the fastener bends (146). Reduced spacings less than 75 percent of those required for $C_A = 1.0$ are not allowed.

If the direction of loading is perpendicular to grain, the minimum spacing for $C_A = 1.0$ is based on the attached member. If the attached member is steel, then steel spacing controls from the appropriate steel standards (125). If the attached member is a wood member loaded parallel to grain, then parallel to grain spacing controls. If the attached member is wood loaded perpendicular to grain, then $4D$ should be adequate. Evaluating the wood members for shear per NDS 3.4.3.3 would also be advisable.

C12.5.1.3 For dowel-type fasteners with diameters equal to or greater than $\frac{1}{4}$ ", the connections need to be designed to comply with several related limitations, including edge distance and spacing between the rows of fasteners and the maximum perpendicular to grain distance between outermost fasteners.

Edge Distance: Requirements in NDS Table 12.5.1C for parallel to grain loading of $1.5D$ or the greater of $1.5D$ or $1/2$ the spacing between rows for ℓ/D greater than 6, and for loaded edge-perpendicular to grain loading of $4D$ are based on early research (146). The unloaded edge-perpendicular to grain minimum of $1.5D$ is a good practice recommendation.

NDS 12.5.1 does not provide specific guidance on edge distance requirements for loads applied at angles other than 0° and 90° , nor does it provide specific geometry factors for reduced edge distances.

The ratio of the fastener length in side member to fastener diameter, ℓ/D , in NDS Table 12.5.1A is based on the total thickness of both wood side members when connections of three or more wood members are involved. For connections involving metal main or side members, only the ℓ/D ratio for the wood members are considered for determination of edge distance requirements in this section. Metal parts must still be designed per NDS 11.2.3.

Avoidance of heavy or medium suspended loads below the neutral axis of a beam was added as a result of several reported field problems involving glued laminated timber beams subject to a line of concentrated loads applied through bolted hangers or ledger strips attached in the tension zone or at the bottom edge of the beam. Concentrated loads less than 100 pounds and spaced more than 24 inches apart may be considered a light load condition.

For perpendicular to grain connections, the member is required to be checked for shear in accordance with 3.4.3.3 of the Specification using a reduced depth, d_c , equivalent to the beam depth (d) less the distance from the unloaded edge of the beam to the center of the nearest fastener.

Spacing Requirements Between Rows: For perpendicular to grain loading, NDS Table 12.5.1D provisions are based on early research (146). These requirements relate the tendency of the fasteners to bend and cause non-uniform bearing stresses and the resistance of the wood between rows to resist splitting. It is for this reason that staggering of fasteners loaded perpendicular to grain is desirable (see NDS 12.6.1). In computing the ℓ/D ratio for determining the appropriate minimum spacing between rows for perpendicular to grain loading, the ratio for side members is based on the sum of the bearing length in each side member where three or more wood member joints are involved.

For parallel to grain loading, NDS Table 12.5.1D permits rows of fasteners to be spaced $1.5D$; however, additional spacing may be required when installing bolts and lag screws to accommodate larger head and washer dimensions and clearance requirements for wrench sockets. Note that the steel industry recommends a minimum center-to-center spacing between holes of $2.67D$, with a preferred distance of $3D$ (125).

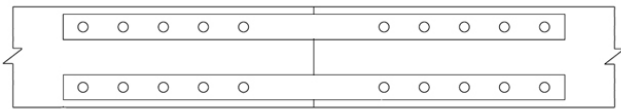
The limitation on row spacing applies to metal as well as wood side plates, to members loaded perpendicular as well as parallel to grain, and to three or more member connections occurring at truss panel points.

Maximum Perpendicular to Grain Distance Between Outermost Fasteners: For parallel or perpendicular to grain loading, limiting the maximum distance between outer rows of fasteners on the same splice plate to 5" avoids splitting that could occur in members at connections as a result of restraint of shrinkage associated with drying in service. Table 12.5.1F was added in the 2012 edition of the Specification in recognition that structural glued laminated timber is manufactured at lower moisture contents which reduces the amount of shrinkage during drying in service. The 10" spacing was based on judgment following review of several analyses that accounted for effects of moisture content change, shrinkage rate, hole oversize and hole alignment on potential restraint of shrinkage between

connected members. The proposed 10" was based on an analysis and the following assumptions: 1) a maximum moisture content at time of fabrication of 16% and an in-service moisture content of 6%; 2) a reduced shrinkage rate applicable to structural glued laminated timbers versus sawn lumber to account for a mixture of tangential and radial shrinkage in the varying layers of glulam; and 3) 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}

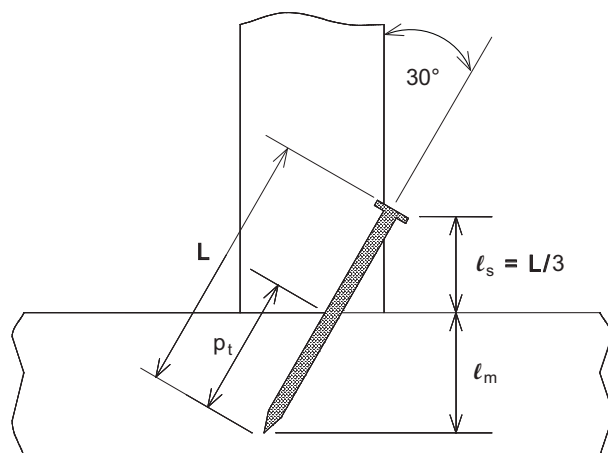
Diaphragms are large, flat structural units acting like a deep relatively thin beam or girder. Horizontal wood diaphragms consist of floor or roof decks acting as webs, and lumber, structural glued laminated timber, SCL, or I-joist members acting as the flanges. Such assemblies distribute horizontal 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 made of four 8d toenails was reported to be stronger than the same joint made with two 16d end nails (181, 118). Where toenailed connections are resisting withdrawal, the depth of penetration, p_t , of the nail in the member holding the point may be taken as the actual length of nail in the member as shown in Figure C12.5.4-1.

Figure C12.5.4-1 Effective Penetration and Side Member Thickness for Toe-Nails Subject to Lateral Loads



C12.5.4.2 The toe-nail factor of 0.83 is an adjustment based on the intermediate condition between the full lateral design value for side-grain connections and the full lateral design value for end-grain conditions where $C_{eg} = 0.67$. Where toe-nailed connections are resisting lateral loads, the bearing length, ℓ_m , of the nail in the member holding the point may be taken as the vertically projected length of nail in the member (see Figure C12.5.4-1) calculated as:

$$\ell_m = L \cos 30^\circ - L/3 \quad (\text{C12.5.4-1})$$

where:

L = length of nail, in.

For purposes of establishing the single shear reference lateral design value applicable to a toe-nailed connection, the side member bearing length, L_s , of the nail (see Figure C12.5.4-1) shall be taken as:

$$\ell_s = L/3 \quad (\text{C12.5.4-2})$$

Equation C12.5.4-2 only applies to nails driven at an angle of approximately 30° to the face of the member being attached and one-third the nail length from the end of that member.

C12.6 Multiple Fasteners

C12.6.1 Symmetrically Staggered Fasteners

See C12.5.1.3 Spacing Requirements Between Rows.

C12.6.2 Fasteners Loaded at an Angle to Grain

General provisions for the placement and spacing of fasteners to cover all directions of loading and any number of members in a connection are beyond the scope of the Specification. For this reason, the gravity axis of all members must pass through the center of fastener resistance to maintain uniform stress in main members and uniform distribution of load to all fasteners. If it is not possible

to achieve intersection of member gravity axes with the center of resistance of the fastener group, the designer has the responsibility to fully evaluate and account for the effects of the resultant eccentric loading on both the load-carrying capacity of the members and the capacity of the connection (see C11.1.3).

C12.6.3 Local Stresses in Connections

See C11.1.2.

C13 SPLIT RING AND SHEAR PLATE CONNECTORS

C13.1 General

C13.1.1 Scope

Split ring and shear plate connectors act like dowels or keys in distributing loads from one member to another in a joint (184). The large diameters of the rings or plates, relative to the diameters of bolts, and the relatively shallow depth of the connectors in the members provide for increased bearing areas without penalizing reductions in net section areas. As a result, these connectors can develop significantly higher design values than those obtainable from bolts alone.

Split ring connectors are installed in precut grooves made with a special power-driven drill and cutting tool. They are used in wood-to-wood joints where high lateral joint loads are involved; such as in bowstring trusses, arches, and bridges. The bolt or lag screw passing through the center of the ring holds the faces of the joint members in contact.

Similar to split rings, shear plates are installed in precut grooves but are flush to the surface when fully seated. Two shear plates are the equivalent of one split ring, with the load being transferred from one plate to the other in the joint through shear in the bolt or lag screw. Shear plates are primarily used in wood-to-steel connections; such as steel gusset plate joints or column-foundation connections where the metal replaces one of the plates, and in demountable wood-to-wood connections, such as stadium bleachers (142).

The design provisions for split ring and shear plate connectors in the Specification are based on early research (104, 117).

C13.1.2 Terminology

A connector unit is described in terms of the metal parts required for a single shear plane. For a split ring connection, one ring is used in matching grooves in the members adjacent to one plane. For shear plate connections, two matching shear plates, one in appropriate grooves in each member, are used in wood-to-wood joints. In a wood-to-metal joint, the steel strap or plate replaces one of the shear plates. In all three cases, the bolt or lag

screw tying the joint together is considered loaded in single shear. Where more than one connector unit is on the same bolt, as in the case of a three member joint where the main member has connectors on the same bolt on both faces, an adjusted single shear design value for each shear plane is provided in the design value tables (see NDS Tables 13.2A and 13.2B).

C13.1.3 Quality of Split Ring and Shear Plate Connectors

C13.1.3.1 The split ring is wedge shaped (beveled toward the edges) to facilitate installation and assure a tight fit when fully seated. The diameter of the inside groove for the split ring is 2 percent larger than the inside diameter of the ring, thus requiring the ring to be sprung slightly when inserted. This provides for any subsequent shrinkage of the members and for simultaneous bearing of the inner surface of the connector against the inner core of wood created by the grooving operation and bearing of the outer surface of the connector on the opposite side against the outside wall of the groove (117, 142). The position of the tongue-slot joint in the ring relative to the direction of loading is not significant (117).

The two small perforations in the central portion of pressed steel shear plates serve to facilitate temporary attachment of the connector to the joint member when off-site fabrication is employed and in the erection and dismantling of temporary structures in the field. The perforations do not affect plate load-carrying performance.

C13.1.3.2 Design values in NDS Tables 13.2A and 13.2B correspond to the dimensions for split rings and shear plates, respectively, in Appendix K of the Specification. In addition to connector diameter, the depth of the connector in the member and its thickness affect joint load-carrying capacity. Only those split rings that have equivalent or larger inside diameter, metal depth, and metal thickness than those given in NDS Appendix K qualify for the connector design values provided in NDS Table 13.2A. Similarly, only those shear plates that have equivalent or larger plate diameter, plate depth, and plate thickness than

those given in NDS Appendix K qualify for the connector design values provided in NDS Table 13.2B.

The projected areas given in NDS Appendix K for split rings are calculated as the sum of the inside groove diameter and twice the groove width times the groove depth. The projected areas for shear plates given in NDS Appendix K are based on the groove diameter times the groove depth for the nominal shear plate dimensions shown. Tabulated projected areas for split ring and shear plate connectors given in NDS Appendix K are to be used in checking localized wood stresses in accordance with NDS 11.1.2 and 13.3.4.3.

C13.1.3.3 Bolts used with split rings or shear plates are required to meet the quality provisions of NDS 12.1.2 for full body diameter bolts to prevent use of undersized fasteners that do not provide full bearing with the connectors.

C13.1.3.4 Lag screws used with split rings or shear plates are required to meet the quality provisions of NDS 12.1.3 for full body diameter lag screws to prevent use of undersized fasteners that do not provide full bearing with the connectors.

C13.1.4 Fabrication and Assembly

C13.1.4.1 Cutterheads should be designed specifically for the dimensions provided by the particular connector manufacturer.

C13.1.4.3 Washers may be used in shear plate connections involving steel straps and plates when use of a longer bolt or lag screw is necessary to avoid bearing of the threaded portion of the bolt or screw on the strap or plate.

C13.1.4.4 Reference design values for split ring and shear plate connectors apply to joints in which the members are in contact, are fabricated of wood having a moisture content of 15 percent or lower to a depth of at least 3/4 inch from the surface, and will remain dry in service. Effects of normal variations in moisture content that occur in dry conditions of service are accounted for in the reference values.

When connectors are installed in unseasoned or partially seasoned wood intended for use in dry conditions of service, reference design values are to be adjusted in accordance with the factors in NDS 11.3.3. Such joints will need to be tightened as the members season in service by periodically turning down the nuts on the bolts until service equilibrium moisture content is reached.

It is good practice to exclude visible face knots within a distance of one-half the connector diameter along the grain from the edge of the connector unit (181, 117). Where visible knots are included within a one-half connector diameter distance of the critical section, the net section based on the projected area of the connector unit and bolt or screw should be further reduced for the cross-sectional area of such included knots (see NDS 3.1.2.3).

C13.2 Reference Design Values

C13.2.1 Reference Design Values

Early connector tests of joints made with Douglas fir, southern pine, white oak, and other representative species showed that joint load-carrying capacity was directly related to the specific gravity of the wood members (184, 181, 104, 117).

Reference design values in NDS Tables 13.2A and 13.2B represent maximum joint test loads reduced by a factor of 3.6 that includes adjustments for variability and load duration (184, 181, 117). These reference design values, applicable to normal loading conditions, are considered to be less than 70 percent of proportional limit test loads (181, 117). Reference design values apply only to those joint designs which meet the minimum thickness requirements in NDS Tables 13.2A or 13.2B and the end distance, edge distance, and spacing requirements corresponding to $C_A=1.0$ in NDS Table 13.3. Net thickness requirements refer to the actual thickness of the member before grooving.

C13.2.1.1 Reference design values for split ring connections in NDS Table 13.2A and for shear plate connections in NDS Table 13.2B are given in terms of the number of faces a member has with a connector on the same bolt and on the thickness of that member. The lowest reference design value for the two members being joined is the reference design value for the shear plane.

C13.2.1.2 The 2900 pound limit for the 2-5/8 inch shear plate is the maximum reference bearing load for a pressed steel plate without a reinforcing hub about the bolt hole. The 4400 and 6000 pound limit for the 4 inch plates used with 3/4 and 7/8 inch bolts, respectively, are the maximum reference shear design values for A307 bolts of these diameters. The 4 inch plates have integral re-enforcing hubs about the central bolt hole. The limiting values specified in footnote 2 of NDS Table 13.2B are based on metal strength; therefore, these metal parts should be designed per NDS 11.2.3. The strength of metal parts should not be adjusted by factors given in NDS 11.3.1 (e.g., ASD Load Duration Factor, C_D).

C13.2.2 Thickness of Wood Members

C13.2.2.1 The minimum member thicknesses required for use of the split ring and shear plate connector values in NDS Tables 13.2A and 13.2B have been established from the results of joint tests (117).

C13.2.2.2 The provision for use of linear interpolation between minimum thicknesses and those required from maximum design values is based on the original connector research (117).

C13.2.3 Penetration Depth Factor, C_d

Adjustments for reduced lag screw penetration depths are permitted to be interpolated between the values for $C_d = 1.0$ and $C_d = 0.75$ using the corresponding penetrations, respectively, for each species group.

C13.2.4 Metal Side Plate Factor, C_{st}

Increases for metal side plates used with 4 inch shear plate connectors are based on original connector research involving claw plates (117). The increased values for 4 inch shear plates loaded parallel to grain are still limited by footnote 2 of NDS Table 13.2B.

C13.2.5 Load at Angle to Grain

Use of the standard bearing angle to grain equation (NDS Equation 13.2-1 and NDS Appendix J) to determine reference design values for split ring and shear plate connectors located in a shear plane that is loaded at an angle to grain between 0° and 90° are based on claw plate con-

necter research (117). In this same study, tests of split ring connectors showed the relationship between maximum design value and grain angle could be described by a linear relationship without appreciable error. For consistency with the provisions for other fastener types, the standard angle to grain equation is conservatively used in the Specification to adjust both split ring and shear plate connector reference design values for grain angle.

C13.2.6 Split Ring and Shear Plate Connectors in End Grain

Design of connectors in end grain surfaces are frequently encountered in practice, such as those at the peak of A-frames or similar arches. Reference design values for split ring and shear plate connectors in end grain surfaces are keyed to use of a reference design value for connectors in square-cut end surfaces equal to sixty percent of the reference design value for connectors in side grain surfaces loaded perpendicular to grain.

The use of 0.60 Q' as the reference design value for a square-cut end surface was originally based on experience with connector design with glued laminated timber (140). Available data from a comprehensive study of the capacity of shear plates in sloping grain end surfaces in Douglas fir (80) generally confirm the use of the 0.60 ratio. This ratio is slightly more conservative than the 0.67 value assumed for square-cut end surface design values in Canada (32, 75).

For split ring and shear plate connectors used in sloping end grain surfaces, the thickness of the member is taken as the distance between the edge of the connector and the nearest point on the outside edge of the member located on a line parallel to the bolt or lag screw axis. Where the end grain surface is square cut, the thickness of the member may be taken as the length of the dowel in the member.

C13.3 Placement of Split Ring and Shear Plate Connectors

C13.3.1 Terminology

Edge and end distances and spacings for split ring and shear plate connectors are referenced to the center not the edge of the connectors.

C13.3.2 Geometry Factor, C_Δ , for Split Ring and Shear Plate Connectors in Side Grain

The geometry factor adjusts reference design values for split ring and shear plate connectors installed in side grain with end distances, edge distances, and/or spacings less than those required for $C_\Delta = 1.0$. The smallest geometry factor for any split ring or shear plate connector in a joint

is to be applied to all connectors in that joint regardless of their alignment relative to one another.

C13.3.2.1 Values for geometry factors presented in Table 13.3 were determined from original connector research on split rings and claw plates (117). Most values have remained unchanged for decades, but geometry factor values for edge distances were modified in the 2012 edition of the Specification as discussed below.

Geometry Factors for Edge Distance

Prior to the 2012 edition of the Specification, the minimum edge distance was 1.75 inches for 2-1/2" split rings and 2-5/8" shear plates and 2.75 inches for 4" split rings and 4" shear plates. This was based on the assumption that the smaller connectors would be used in lumber with a nominal width of 4 inches (3.5 in.) or larger, and the larger connectors would be used with in lumber with a nominal width of 6 inches (5.5 in.) or larger. However, these connectors are commonly used in structural glued laminated timbers with widths as narrow as 3 inches and 5 inches for the smaller and larger connectors, respectively.

The original research (117) included tests on material with widths as small as the outside diameter of the connector. This data was reviewed to determine appropriate geometry factors for connectors with edge distances as small as 1.5 inches for the smaller connectors and 2.5 inches for the larger connectors. These are considered to be practical minimums for good practice.

Significant differences were observed in the behavior of claw plates and split rings with decreasing loaded edge distances, so separate geometry factor values are tabulated for split rings and shear plates (which are similar to claw plates) for minimum loaded edge distance. Interpolation is permitted for unloaded edge distances between the minimum value and the values corresponding to a geometry factor of 1.0.

Geometry Factors for Spacing

Spacing requirements in NDS Table 13.3 are based on original connector research (17, 142) with the following exceptions:

The factor for the perpendicular loading and spacing case was reduced to 0.50 as part of an effort to simplify adjustment of connector design values for end distances and longitudinal spacing (181).

C13.3.2.2 (Connectors Loaded at an Angle to Grain) NDS 13.3.2.2 was introduced in the 2012 edition of the Specification and clarifies requirements for split ring and shear plate connectors loaded at an angle to grain. Separate geometry factors for end and edge distance are calculated for the parallel and perpendicular components of the resistance.

The original connector research indicated that the load-carrying capacity of a joint made with two or more connectors aligned parallel-to-grain but loaded perpendicular-to-grain, was less than the sum of the maximum design values for the same connectors acting singly (181, 117). Staggering or offsetting connectors along the grain of the transverse loaded member was found to give somewhat higher design values (181). When such offsetting is used, the line connecting the centers of two or more connectors located in the same contact face—the connector axis, ϕ —may not be oriented parallel or perpendicular to the grain of the member or to the direction of load, θ . Prior editions of the NDS referenced methods presented in the TECO design manual (142) and the AITC Timber Construction Manual (140) for determining spacing between connectors with connector axes at angles other than parallel-to-grain or perpendicular-to-grain. NDS Equation 13.3-1 and Table 13.3.2.2 were taken from the AITC Timber Construction Manual (140) and provide identical results to the graphical method presented in the TECO design manual (142). Equation 13.3-1 and Table 13.3.2.2 define elliptical transition functions between parallel-to-grain spacing requirements and perpendicular-to-grain requirements for various angles of load.

C13.3.3 Geometry Factor, C_{Δ} , for Split Ring and Shear Plate Connectors in End Grain

NDS 13.3.3 clarifies that a single geometry factor, C_{Δ} , is determined and applied to both the parallel and perpendicular components of the resistance. Beginning with the 2012 edition of the Specification the same elliptical transition equation for spacing of connectors in side grain is applied to connectors in sloped end grain surfaces. These new provisions remove a large discontinuity from previous requirements and provide a better transition between the parallel-to-grain provisions and the perpendicular-to-grain provisions. Prior to the 2012 edition of the Specification, the perpendicular-to-grain spacing, end distance, and edge distance requirements were applied for all sloped cuts between 45 and 90 degrees and the parallel-to-grain provisions for sloped cuts less than 45 degrees. These provisions resulted in large differences in spacing and end/edge distance requirements between members with sloped cuts of 45 and 46 degrees.

C13.3.3.1 Procedures for establishing geometry factors, C_{Δ} , for connectors in end grain surfaces follow the same logic as that employed to establish reference design values for such configurations in NDS 13.2.6.

C13.3.3.2 Shear capacity of members supported by connectors in end grain surfaces should be checked using provisions of NDS 3.4.3.3. Where the slope of the surface cut, α , is other than 90° , the component of the vertical force on the connector shear plane that is normal to the outside or uncut edge of the member should be taken as the shear force, V . The effective depth of the member, d_e , should be taken as the component of the distance from the loaded edge of the member to the unloaded edge of the connector that is normal to the outside or uncut edge of the member.

C13.3.4 Multiple Split Ring or Shear Plate Connectors

C13.3.4.1 The group action factor, C_g , applies only to a row of two or more connectors which are in the same shear plane, are aligned in the direction of load, and are on separate bolts or lag screws (see C11.3.6). The factor need not be applied to connections involving two or more connector units on two or more contact faces concentric to the same bolt axis.

C13.3.4.2 When two sizes of split ring grooves are cut concentrically on the same wood surface and rings are installed in both grooves, the total load on the joint is limited to the reference design value for the larger ring only.

C13.3.4.3 Localized wood stresses should be checked in accordance with NDS 11.1.2.

Table 13.3 Geometry Factors, C_{Δ} , for Split Ring and Shear Plate Connectors

Geometry factors, C_{Δ} , in Table 13.3 were revised in the 2012 edition of the Specification to account for reduced edge distance associated with structural glued laminated timbers with faces as narrow as 3" and 5" rather than 3.5" and 5.5" assumed previously (e.g., edge distances of 1.5" and 2.5" rather than 1.75" and 2.75"). Tabular values for loaded edge distance were removed for the Parallel to Grain Loading case as technically there is no loaded edge in parallel to grain loading. Values had been provided previously in order to accommodate linear interpolation, however new provisions described in NDS 13.3.2.2 eliminate the need for these values.

Based on a re-evaluation of the Scholten data (117) for narrower widths, the minimum unloaded edge distance for 2-1/2" split rings and 2-5/8" shear plates was reduced from 1-3/4" to 1-1/2" with the corresponding geometry factor, C_{Δ} , being reduced from 1.0 to 0.88. Similarly, the minimum unloaded edge distance for 4" split rings and 4" shear plates was reduced from 2-3/4" to 2-1/2", with the corresponding geometry factor, C_{Δ} , being reduced from 1.0 to 0.93.

Also, as a consequence of using narrower widths, Table 13.3 was revised in the 2012 edition of the Specification to tabulate separate geometry factors, C_{Δ} , for split rings and shear plates with regard to loaded edge distance for perpendicular to grain loading.

C14 TIMBER RIVETS

C14.1 General

C14.1.1 Scope

The Specification presently limits use of timber rivets to attachment of steel side plates to structural glued laminated timber.

Timber rivets, also known as glulam rivets, were originally developed in Canada more than 35 years ago to connect pre-drilled steel plates to structural glued laminated timber (41). Typical applications include tension splices, beam hangers, and moment splices. The rivets have flattened-oval shanks with tapered heads that, when driven, wedge tightly into holes in the steel plate (see Appendix M in the Specification). The resulting head fixity adds to the strength and stiffness of the connection. The number of rivet rows in each plate and the number of rivets per row can both range from 2 to 20 (see NDS Figure 14A and NDS Tables 14.2.1A through 14.2.1F and Tables 14.2.2A and 14.2.2B).

C14.1.2 Quality of Rivets and Steel Side Plates

Provisions of the Specification are applicable only to timber rivets that are hot-dipped galvanized. Rivets are made with fixed shank cross-section and head dimensions (NDS Appendix M) and vary only by length.

Steel plates used in timber rivet connections must be a minimum of 1/8" thick and, when used in wet service conditions, must be hot-dipped galvanized. Strength reductions apply for steel plates less than 1/4" thick (see NDS Table 14.2.3). Due to rivet and plate hole dimensions and tolerances, fabrication of joints with plates greater than 1/4" is not practical and is generally avoided. Also, the reduced penetration of the rivet into the wood associated with greater plate thickness can limit connection capacity by reducing the area of wood available to resist the tension and shear loads being applied around the rivet group.

C14.1.3 Fabrication and Assembly

C14.1.3.1 Rivets, whose shank dimensions are nominally 1/4" by 1/8", must be driven with the wider dimension oriented parallel to the grain of the wood member. This orientation provides maximum connection capacity for both parallel and perpendicular to the grain loading and

minimizes any splitting that may occur (41). Further, rivets are not driven flush with the plate but only to the point where the tapered heads wedge tightly into the predrilled holes in the plate. It is assumed that approximately 1/8" of the rivet head will protrude from the face of the plate after driving (see NDS Appendix M).

To minimize splitting in rivet groups involving more than two rows and more than two rivets per row, rivets are driven around the perimeter first and then in successive inner rectangles toward the center.

C14.1.3.2 The limit on maximum penetration of rivets of 70% of wood member thickness is intended to prevent through splitting of the piece.

C14.1.3.3 Connections in which rivets driven through plates on both sides of a member penetrate beyond the midpoint of the member are not generally used. Where such overlap of rivets does occur, the length of overlap is limited to 20% of the member thickness (see NDS 14.1.2.2) and the rivets on both sides are required to be spaced (see NDS 14.3.1) as though they were all driven from one side. The capacity of the connection is then determined as if all rivets were driven from one side and with spacings parallel (s_p) and perpendicular (s_q) to grain (see Figure 14A in the Specification) determined as the distances between adjacent rivets (one from each side but assumed on one side) at their points. Under these provisions, NDS Equations 14.2-1 and 14.2-2 and NDS Tables 14.2.1A through 14.2.1F and 14.2.2A and 14.2.2B are entered with twice the number of rows and twice the number of rivets per row as those actually driven from one of the sides. Also, NDS Tables 14.2.1A through 14.2.1F are entered with the member dimension of a connection with only one plate, which as footnoted in these tables, is twice the thickness of the wood member.

The procedure for determining the capacity of plates on two sides with rivets overlapping is based on the derivation of the design methodology and supporting data for single plate connections.

C14.2 Reference Design Values

C14.2.1 Parallel to Grain Loading

Design equations for timber rivets are based on Canadian research (24, 47, 45, 48, 69). 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 (45). As load is applied to the connection, end rivets carry a larger portion of the load than rivets in the center. As yielding occurs, the load is redistributed to the less-loaded fasteners, until at maximum connection load, all the individual rivets are considered to have reached their ultimate bearing capacity (45). This mode of failure will occur as long as the tension and shear strengths of the wood around the group of rivets is sufficient to resist the total applied load. However, if shear failure of the wood on the side and bottom of the rivet group occurs, followed by tension failure at the interior end of the group perimeter, the block of wood into which the rivets have been driven can be pulled out of the member before the maximum rivet bending load has been reached (45). Thus timber rivet design loads are based on the lower of the maximum rivet bending load and the maximum load based on wood strength.

In the 2012 edition of the Specification, the constant in NDS Equation 14.2-1 for rivet connection design values parallel to grain was changed from 280 to 188. This adjustment results in NDS timber rivet design values that are more consistent with other NDS connection designs for both ASD and LRFD. The constant of 188 in NDS Equation 14.2-1 is based on tests of single rivets in Douglas fir at penetrations of 1, 2, and 3 inches (47). The rivet connection design values (P_r and Q_r) obtained from the equation using a constant of 188 represents the average ultimate test values reduced by a factor of 5.0 to better account for the proportional limit basis of NDS dowel connector values under lateral loads. This adjustment enables consistent application of the load duration factor to rivet capacity which is influenced by both fastener bending and wood bearing stresses. The factor of 5.0 increases the original adjustment of 3.36 (45) on ultimate test values.

NDS Equation 14.2-1 also includes an additional adjustment of 0.88 to account for specifying use of rivets of lower hardness and associated lower ultimate tensile and yield strength than rivets used in the original research (48, 41). The change in rivet specification was made to avoid the possibility of hydrogen embrittlement occurring in

service conditions involving high temperature and high humidity (48).

Because of the complexity of the equations used to check wood capacity in timber rivet connections loaded parallel to grain, only tabular values for a range of rivet penetrations, spacings and rivet group sizes are given in the Specification (NDS Tables 14.2.1A through 14.2.1F). In the 2018 edition of the Specification, the number of rivet rows per side in Tables 14.2.1A through to 14.2.1F was revised to reflect the 12" limit on maximum distance perpendicular to grain between outermost rows of rivets per section 14.3.1. The loads in these tables are the lesser of the reference wood tension capacity or the reference wood shear capacity as determined from the equations developed in the original research and verified by tests of full-size connections representing a range of rivet group sizes and spacings in Douglas fir structural glued laminated timbers (45).

The maximum normal (tension) stress is checked assuming an area equal to the rivet penetration times the width of the rivet group. The induced stress on this area is calculated as a function of coefficients which are derived from equations involving the variables of rivets per row, number of rows, spacing between rivets, spacing between rows, and the ratio of member thickness to rivet penetration (45). The lower the ratio, the larger the load component resisted by the normal stress and the lower the load component resisted by shear stress. It is this effect that is being accounted for by entering NDS Tables 14.2.1A through 14.2.1F with a wood member dimension for a single plate connection which is twice the member thickness of a connection with plates on both sides.

In the original research involving evaluation of rivet connections made with Douglas fir members, an average ultimate tension stress parallel to grain of 5600 psi was found in connections whose ultimate load was either a result of rivet bending or wood shear failure (45). For determination of reference connection capacity limited by normal stress, this tension ultimate was reduced to 1600 psi to account for variability (1.6) and load duration and factor of safety (approximately 2.1).

The maximum shear stress in the rivet connection is checked assuming an area equal to twice the rivet penetration times the length of the rivet group. The load on this area is calculated as a function of coefficients which are based on different equations but involving the same variables as those used to determine normal stress plus end distance. These equations account for shear resistance

on the bottom of the rivet group acting on the plane at the rivet tips as well as the lateral shear on the sides by proportioning the total shear loads carried by the bottom and side surfaces (45).

Rather than use shear stress values based on the ASTM D 143 block shear specimen, the reference shear stress used in the shear checking equation for rivet connections was developed using a Weibull weakest link model in which strength is inversely related to volume. Based on experimental data, it was determined that the shear strength of a unit volume of Douglas fir under uniform shear at 0.5 survival probability was 2526 psi (45). Employing this value in the equation developed in the original research for maximum lateral shear stress and reducing the equation constants by a factor of 3.36 (1.6 variability and 2.1 load duration and factor of safety) gives a reference shear stress for evaluating shear loads in rivet connections of 745 psi. As verification of the shear checking equation, a mean ratio of estimated to observed ultimate loads of 1.03 was obtained for eight rivet connection configurations in Douglas fir that exhibited wood shear failure. Test connections involved configurations containing 25, 50, 100, and 150 rivets and rivet spacings of 0.5", 1", and 1.5" (45).

It is to be noted that calculated P_r values and P_w values tabulated in NDS Tables 14.2.1A through 14.2.1F apply to connections made with 1/4" side plates and to one plate with associated rivets. For connections with thinner side plates, the adjustments in NDS Table 14.2.3 apply. Where connections involve plates on two sides of the wood member, the limiting P_r or applicable tabular P_w value is doubled to determine the reference capacity of the connection.

Because of the species test results and property values used to develop the rivet bending and wood capacity equations, use of reference design values based on the provisions of NDS 14.2.2 should be limited to Douglas Fir-Larch and Southern Pine structural glued laminated timber.

C14.2.2 Perpendicular to Grain Loading

As with parallel to grain loading, design loads for timber rivet connections in which the loads act perpendicular to the grain of the wood member are based on the lower of the maximum rivet bending load and the maximum load based on wood strength. However, strength in tension perpendicular to grain is the controlling wood property rather than tension parallel and shear strength properties. The mode of wood failure in the perpendicular load case is a separation along the grain just above the first line of rivets nearest the unloaded edge, as contrasted to the pull out of the block of wood containing the rivet group that occurs in the parallel load case (45).

NDS Equation 14.2-2 is the same as that for the parallel to grain loading case (NDS Equation 14.2-1) except for the value of the constant, 108 compared to 188. The ratio of the two values (0.57) represents the ratio of the average ultimate lateral load-carrying capacities of single rivet joints in Douglas fir structural glued laminated test specimens loaded perpendicular to grain and parallel to grain (47, 69). In the 2012 edition of the Specification, the constant in NDS Equation 14.2-2 for rivet connection design values perpendicular to grain was changed to 108 (see C14.2.1).

The wood capacity of rivet connections loaded perpendicular to the grain is a function of penetration, number and configuration of rivets, rivet spacings, and unloaded edge distance (45). Checking equations assume the connection load acts on an area equal to the width of the rivet group times the rivet penetration. However, the distribution of stress is not uniform over this area, but is a maximum at the surface of the member and decreases sharply along the penetration depth and on either side of the center of the rivet group (24). This non-uniform distribution is accounted for in the basic design equations.

Based on tests that showed tension perpendicular to grain strength decreases with increase in cross-section area and/or length, a Weibull brittle fracture model was used to establish a reference wood stress for checking wood capacity in rivet connections loaded perpendicular to grain. Using results from tests of blocks cut from Douglas fir structural glued laminated timber beams and ranging from 16 to 3600 in.³ in volume, a tension perpendicular to grain strength for unit volume under uniform stress at a 95% survival probability of 267 psi was established (24). Reducing this value by a factor of 2.1 for load duration and factor of safety gives a reference tension perpendicular to grain stress of 127 psi. This unit value is adjusted in the checking equations for volume through introduction of a variable based on the distance between the unloaded edge of the member and the first line of rivets in the connection.

In lieu of presenting the complex equations required to determine wood capacity for perpendicular to grain loading, a simplified equation (NDS Equation 14.2-3) is given in the Specification enabling such capacity to be calculated for any rivet penetration and plate thickness using loads and factors from NDS Tables 14.2.2A and 14.2.2B that account for the effects of a range of rivet configurations, spacings, and unloaded edge distances. The unit load values given in NDS Table 14.2.2A include an adjustment factor to account for stress distribution effects in connections with two side plates; thus the load values in this table are conservative for a single plate application. In the 2018 edition of the Specification, the number of rivets per row in Table 14.2.2A was revised to

reflect the 12" limit on maximum distance perpendicular to grain between outermost rows of rivets per section 14.3.1. It is to be noted that Equations 14.2-2 and 14.2-3 in the Specification provide reference design values for connections with one side plate. Reference design values obtained from either equation are doubled for connections having two side plates.

Because of the species test results and property values used to develop the rivet bending and wood capacity equations, use of reference design values based on the provisions of NDS 14.2.2 should be limited to Douglas Fir-Larch and Southern pine structural glued laminated timber.

C14.2.3 Metal Side Plate Factor, C_{st}

Supporting experimental data for timber rivet design equations involved tests of connections made with 1/4" thick steel side plates (45, 69). Use of thinner plates reduces the amount of fixity of the rivet head which in turn reduces rivet bending capacity.

Reference design values determined in accordance with NDS 14.2.1 and 14.2.2 assume 1/4" side plates are used. For connections made with 3/16" and 1/8" plates,

reference design values based on rivet capacity (P_r and Q_r) are adjusted by the side plate factors of 0.90 and 0.80 given in NDS Table 14.2.3.

C14.2.4 Load at Angle to Grain

The equation for calculating reference design values for timber rivet connections loaded at angles to grain other than 0° and 90° is the same form as the bearing angle to grain equation (see NDS Appendix J).

C14.2.5 Timber Rivets in End Grain

The 50 percent reduction for timber rivets used in end grain is based on Canadian design practice (41). It can be compared with the end grain adjustment factor of 0.67 for nails and spikes (see C12.5.2).

C14.2.6 Design of Metal Parts

Timber rivet connections can carry relatively high loads. It is the responsibility of the designer to assure the metal side plates on such connections are of adequate strength to carry the total load being transferred.

C14.3 Placement of Timber Rivets

C14.3.1 Spacing Between Rivets

In the 2012 edition of the Specification, a limit on the maximum distance perpendicular to grain between outermost rows of rivets of 12 inches was added to parallel requirements in the new Table 12.5.1F which specifies the maximum spacing between outer rows of dowel-type fasteners in structural glued laminated timber connections.

C14.3.2 End and Edge Distance

Effects of rivet spacing and edge and end distances have been evaluated using the basic rivet design equations

(45). For parallel to grain loading and with other variables constant, wider rivet spacings are associated with the rivet bending failure mode while closer spacings induce wood shear failures. Similarly, with other factors constant, longer end distances allow rivet bending to control while shorter end distances cause wood shear capacity to control.

Minimum spacings and minimum end and edge distance requirements given in NDS 14.3 and NDS Table 14.3.2 minimize the occurrence of premature wood failure in favor of more ductile rivet yielding based on Canadian design standards (41).

C15 SPECIAL LOADING CONDITIONS

C15.1 Lateral Distribution of a Concentrated Load

C15.1.1 Lateral Distribution of a Concentrated Load for Moment

The lateral distribution of concentrated loads is particularly important to obtain efficient design of bending members in structures such as bridges and warehouse or industrial buildings where heavy wheel loads are involved. Easily applied methods for determining the maximum moment and maximum shear in bending members subject to concentrated wheel loads are given in NDS 15.1 of the Specification. These methods, which are based on the thickness of the flooring or decking involved (two to six inches thick) and the spacing of the beams or stringers, have long been used in timber bridge design (129). The procedures have been verified through test and shown to be generally conservative, particularly when the portion of the load distributed to adjacent members is 40 percent or less (42).

The lateral distribution factors for moment in NDS Table 15.1.1 are keyed to the stiffness of the flooring or decking through use of nominal thickness and spacing of beams. These factors are based on recommendations of the American Association of State Highway and Transportation Officials (129). For cases where the lateral load distribution factor exceeds 1.0 (i.e., $S/\text{denominator} > 1.0$) the load is assumed to be fully on the beam. Where the concentrated load is applied to the deck between the beams, the load is distributed to the adjacent beams assuming the deck acts as a simply-supported beam. For cases where the factor is less than or equal to 1.0 and the concentrated load is applied to the deck between the beams, provisions of NDS 15.1.1 can be conservatively used or a more rigorous method of analysis should be considered.

The two-inch plank floor refers to one made of pieces of lumber laid edge-to-edge with the wide faces bearing on the supporting beams or stringers. The four-inch and six-inch laminated floors refer to those made of pieces of lumber laid face-to-face with the narrow edges bearing on the supporting beams or stringers, with each piece being nailed to the preceding piece (129). Nails typically penetrate into two adjacent pieces, are staggered and are

alternated on the top and bottom edges (42). Flooring is typically attached to stringers by toe-nailed connections.

The lateral distribution factors apply to bridges designed for one traffic lane and to interior beams and stringers only. The computed factor gives the fraction of the wheel load (both front and rear of tractor or trailer axles on one side) positioned to give maximum bending moment at mid-span of the beam or stringer closest to the wheel load (129, 42).

The live load bending moment for outside beams or stringers is calculated using a load equal to the reaction of the wheel load assuming the flooring or decking between the outside and adjacent stringer is acting as a simply-supported beam (129).

Lateral distribution factors determined in accordance with NDS Table 15.1.1 can be used for any type of fixed or moving concentrated load. The lateral distribution factors determined from the table have been verified by field tests on five timber bridges ranging from 15 to 46 feet in span and by laboratory tests on three full-size bridge deck and stringer assemblies 16 to 28 feet in span (42). These tests indicate the factors are somewhat conservative, particularly at ratios greater than 0.60.

For bridges of two or more traffic lanes, the American Association of State Highway and Transportation Officials (129) provides other lateral distribution factors.

Generally all designs involving multiple parallel bending members that are loaded through transverse elements such as flooring, decking, or sheathing are capable of some lateral distribution of a concentrated load on one member to adjacent members on either side. The repetitive member factor (see NDS 4.3.9, 7.3.6, and 8.3.7) partially accounts for such load redistribution.

C15.1.2 Lateral Distribution of a Concentrated Load for Shear

The lateral distribution factors for shear 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.

The tabulated values of the percentage of a concentrated load on the center beam at the quarter point of the span and the percentage of the same load on the center beam at mid-span is closely described by the following relation:

$$P_{1/4} = -1.807 + 1.405 \log(P_m) \quad (\text{C15.1-1})$$

where:

$P_{1/4}$ = percentage of load at 1/4 point of center beam

P_m = percentage of load at mid-span of center beam

= S/denominator from NDS Table 15.1.1 or other basis

Values of $P_{1/4}$ from NDS Table 15.1.2 are used to determine the actual shear stress from the wheel or other concentrated load being considered. Field and laboratory tests of full-size timber bridges verify the appropriateness of the NDS Table 15.1.2 values and indicate they are conservative at S/denominator ratios above 0.50 (42).

C15.2 Spaced Columns

C15.2.1 General

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 (181). The end fixity developed by the connectors and end blocks increases the buckling resistance of the individual members in the direction perpendicular to the wide faces when loaded in compression parallel to grain (parallel to the d_1 dimension in Figure 15A of the Specification).

C15.2.1.1 In the design of spaced columns, the adjusted compression stress for an individual member is determined in accordance with the provisions of NDS 15.2 and other applicable provisions of the Specification. The actual compression stress parallel to grain, f_c , on the members of the spaced column is not to exceed the adjusted compression design value parallel to grain, F_c' , for these members based on all provisions of NDS 3.6 and 3.7 except as modified or extended by the provisions of NDS 15.2. The net section requirements of NDS 3.6.3 are to be applied to the members of spaced columns.

C15.2.1.2 The advantage of a spaced column is the increase in the critical buckling design value for compression members obtained by the partial end fixity of the individual members. This increase in capacity, 2-1/2 or 3 times the value for a solid column with the same slenderness ratio, applies only to buckling in the direction perpendicular to the wide face of the members (buckling limited by the ℓ_1/d_1 ratio). If there was no slip in the end connections and full fixity of the ends were provided by the end block fastenings, the buckling stress would be 4 times that of a solid column because of the 50 percent reduction in effective column length (141).

The increase in the critical buckling stress associated with the ℓ_1/d_1 slenderness ratio obtained through the use of spaced column design may make capacity in the direction parallel to the wide face of the members (buckling associated with the ℓ_2/d_2 ratio) the limiting case. The adjusted compression design value parallel to grain in this direction is not affected by spacing the individual members and, therefore, must be checked in accordance with NDS 3.7.

C15.2.2 Spacer and End Block Provisions

C15.2.2.1 Where more than one spacer block is used, the distance ℓ_3 (see NDS Figure 15A) is the distance from the center of one spacer block to the centroid of the connectors in the nearest end block.

C15.2.2.2 Spacer blocks located within the middle one-tenth of the column length are not required to be joined to the compression members by split ring or shear plate connectors. Such blocks should be fastened to assure the compression members maintain their spacing under load (181). A web member joined by connectors to two truss chords making up a spaced truss chord (spaced column) may be considered a spacer block.

Where it is not feasible to use a single middle spacer block, two or more spacer blocks joined to compression members by split ring or shear plate connectors may be required to meet the ℓ_3/d_1 ratio limit of 40 (see NDS 15.2.3.2). Connectors used in such spacer blocks must meet the same requirements as those applicable to end blocks and the distance between two adjacent spacer blocks is not to exceed one-half the distance between the centroids of connectors in the end blocks. Connectors are required for spacer blocks not located in the middle of the column length to provide the shear resistance necessary to assure the two members act as a unit under load.

C15.2.2.3 Spaced columns are used as compression chords in bowstring and other large span trusses (141). In this case, the web members of the truss serve as the end blocks. The distance between panel points, which are laterally supported, is taken as the length of such columns. Spaced-column web members may be designed using the procedures of NDS 15.2 if the joints at both ends of the web member are laterally supported.

C15.2.2.4 The thickness of end and spacer blocks is required to be equal to or larger than the thickness of the compression members and meet the minimum requirements for split ring or shear plate connections in NDS Chapter 13 (181). The length of end blocks and spacer blocks located at other than mid-length of the column should be sufficient to meet the end distance requirements for split ring or shear plate connectors given in NDS Chapter 13. In this regard, the load on the connectors in the end blocks shall be considered applied in either direction parallel to the longitudinal axes of the compression members.

C15.2.2.5 Connectors used in spaced columns are designed to restrain differential displacement between the individual compression members. Since the forces causing differential movement decrease as the ℓ/d of the individual members decrease, connector design value requirements vary with slenderness ratio (181).

The equations for end spacer block constants in NDS 15.2.2.5 are based on K_S of zero when $\ell_1/d_1 \leq 11$ and a K_S equal to one-fourth of the clear wood compression design value parallel to grain for the species group when $\ell_1/d_1 \geq 60$ (181). The equations give K_S values for intermediate slenderness ratios based on linear interpolation between these limits.

The limiting K_S values of 468, 399, 330, and 261 for species groups A, B, C, and D (defined in Table 13A of the Specification), respectively, represent one-fourth the normal load, unseasoned clear wood compression design value parallel to grain applicable to representative species in each group in 1955 (181). The representative species were dense Douglas fir and dense southern pine for Group A, Douglas fir and southern pine for Group B, western hemlock for Group C, and white fir-balsam fir for Group D.

The connector or connectors on each face of each end spacer block should be able to carry a load equal to the cross-sectional area of one of the individual compression members (without reduction for cuts made to receive connectors) times the end spacer block constant, K_S .

C15.2.3 Column Stability Factor, C_P

C15.2.3.1 Effective column length for spaced columns is determined in accordance with NDS Figure 15A and adjusted by any applicable buckling length coefficient, K_e ,

greater than one as specified in NDS Appendix G. It is to be noted that ℓ_1 is the distance between points of lateral support restraining movement perpendicular to the wide faces of the individual members, and ℓ_2 is the distance between points of lateral support restraining movement parallel to the wide faces of the individual members. ℓ_1 and ℓ_2 are not necessarily equal.

C15.2.3.2 The slenderness ratio (ℓ_1/d_1) limit of 80 for the individual members is a conservative good practice recommendation recognizing that the individual members are continuous at the bracing locations. The limit of 50 on the slenderness ratio ℓ_2/d_2 is the limit applied to solid columns (see NDS 3.7.1.4). The limit of 40 on the ℓ_3/d_1 ratio also is a conservative good practice recommendation to assure the length between end and spacer blocks in a spaced column is not a controlling factor in the column design.

C15.2.3.3 The column stability factor for an individual member in a spaced column is calculated using the slenderness ratio ℓ_1/d_1 and the same equation as that applicable to solid columns (see NDS 3.7.1.5) except that the critical buckling design value for compression, F_{cE} , is modified by the spaced column fixity coefficient, K_x .

The actual compression stress parallel to grain, f_c , calculated by dividing the total load on the spaced column by the sum of the cross-sectional areas of the individual members, is checked against the product (F_c') of the column stability factor (C_P), the reference compression design value parallel to grain (F_c), and all other applicable adjustment factors (see NDS 2.3). If connectors are required to join spacer (interior) blocks to individual members, and such blocks are in a part of the column that is most subject to potential buckling, f_c is to be calculated using the reduced or net section area remaining at the connector location (see NDS 3.1.2) when comparing with the C_P adjusted compression design value parallel to grain, F_c' .

In spaced-column designs, the actual compression stress parallel to grain, f_c , based on the net section area of the individual members at the end blocks is checked against the product of the reference compression design value parallel to grain and all applicable adjustment factors except the column stability factor (see NDS 3.6.3).

C15.2.3.4 Use of the lesser adjusted compression design value parallel to grain, F_c' , for a spaced column having members of different species or grades for all members is conservative. Where the design involves the use of compression members of different thicknesses, the F_c' value for the thinnest member is to be applied to all other members.

C15.2.3.5 The actual compression stress parallel to grain, f_c , in spaced columns also is to be checked in all cases against the adjusted compression design value parallel to grain, F_c' , based on the slenderness ratio ℓ_2/d_2 and a

C_p factor calculated in accordance with the provisions of NDS 3.7 without use of the spaced column fixity coefficient, K_x . Use of connectors to join individual compression members through end blocks is assumed to only increase the load-carrying capacity of spaced columns in a direction perpendicular to the wide face of the members. When the ratio of the width to thickness of the individual compression members is less than the square root of the spaced column fixity coefficient, K_x , the adjusted compression

stress parallel to grain, F_c' , based on the slenderness ratio ℓ_2/d_2 may control.

C15.2.3.6 See C3.7.1.6.

C15.2.3.7 Design provisions for spaced beams joined by end blocks and connectors are not included in the Specification. The beam-column equations of NDS 3.9, therefore, apply only to those spaced columns that are subject to loads on the narrow edges of the members that cause bending in a plane parallel to their wide face.

C15.3 Built-Up Columns

As with spaced columns, built-up columns obtain their efficiency by increasing the buckling resistance of the individual laminations. The smaller the amount of slip occurring between laminations under compressive load, the greater the relative capacity of that column compared to a solid column of the same slenderness ratio made with the same quality of material. Based on tests of columns of various lengths (114, 116), the capacity of two equivalent column types can be expressed as a percentage of the strength of a solid column made with material of the same grade and species. For mechanically-connected built-up columns, efficiencies ranged from a value of 82 percent at an ℓ/d ratio of 6, decreasing to a low of 65 percent at an ℓ/d of 18, and then increasing to 82 percent at an ℓ/d of 26.

The NDS design provisions for built-up columns made with various types of mechanical fasteners are based on more recent modeling and testing (82, 83). This model can be used to determine the strength of any built-up column on the basis of the slip between members of the column in both the elastic and inelastic ranges. The theoretical formulas were verified through extensive testing including 400 column tests and evaluation of the load-slip properties of 250 different types of connections. The formulas are entered with fastener load-slip values based on beam-on-elastic-foundation principles (71).

C15.3.1 General

The provisions of NDS 15.3 apply only to multi-ply columns in which the laminations are of the same width and are continuous along the length. The limitations on number of laminations are based on the range of columns that were tested (83) that met the connection requirements of NDS 15.3.3 and 15.3.4. The minimum lamination thickness requirement assures use of lumber for which reference design values are available in the Specification.

C15.3.2 Column Stability Factor, C_p

Provisions in NDS 15.3.2 are the same as those applicable to solid columns in NDS 3.7.1 except for the addition of the column stability coefficients, K_f , in NDS Equation 15.3-1.

When nailed in accordance with the provisions of NDS 15.3.3, the capacity of built-up columns has been shown to be more than 60 percent of that of an equivalent solid column at all ℓ/d ratios (82). Efficiencies are higher for columns in the shorter ($\ell/d < 15$) and longer ($\ell/d > 30$) slenderness ratio ranges than those for columns in the intermediate range.

The efficiency of bolted built-up columns conforming to the connection requirements of NDS 15.3.4 is more than 75 percent for all ℓ/d ratios (82). As with nailed columns, efficiencies of short and long bolted built-up columns are higher than those for intermediate ones. The greater efficiency of bolted compared to nailed columns is reflective of the higher load-slip moduli obtainable with bolted connections.

In accordance with NDS 3.7.1.3, NDS Equation 15.3-1 is entered with a value of F_{ce} based on the larger of ℓ_{e1}/d_1 or ℓ_{e2}/d_2 , where d_2 is the dimension of the built-up member across the weak axis of the individual laminations (sum of the thicknesses of individual laminations). Research (82) has shown that buckling about the weak axis of the individual laminations is a function of the amount of slip and load transfer that occurs at fasteners between laminations. When the controlling slenderness ratio is the strong axis of the individual laminations, ℓ_{e1}/d_1 , then $K_f = 1.0$. It is also necessary to compare C_p based on ℓ_{e1}/d_1 and $K_f = 1.0$ with C_p based on ℓ_{e2}/d_2 and $K_f = 0.6$ or 0.75 to determine the adjusted compression design value parallel to grain, F_c' .

Due to the conservatism of using a single factor for all ℓ_{e2}/d_2 ratios, F_c' values for individual laminations designed as solid columns can be greater than F_c' values for built-up columns for relatively small ℓ_{e2}/d_2 ratios. In these cases,

the column capacity should not be limited to the built-up column capacities.

C15.3.3 Nailed Built-up Columns

C15.3.3.1 Nailing requirements (a), (b), and (g) and the maximum spacing requirements of (d) and (e) are based on the conditions for which the column stability coefficient, K_{c} , of 60 percent was established (82). The maximum spacing between nails in a row of 6 times the thickness of the thinnest lamination minimizes the potential for buckling of the individual laminations between connection points. End, edge, and minimum spacing requirements are good practice recommendations for preventing splitting of members (32) and for assuring fasteners are well distributed across and along the face of the laminations.

The requirement for adjacent nails to be driven from opposite sides of the column applies to adjacent nails aligned both along the grain of the laminations and across their width.

The requirement that all nails penetrate at least $\frac{3}{4}$ of the thickness of the last lamination was further clarified in the 2012 edition of the Specification by making clear that all nails need to penetrate all laminations including the last outermost lamination.

In the nailing requirements of NDS 15.3.3.1, a nail row refers to those nails aligned parallel to the grain of the laminations and in the direction of the column length. Where only one longitudinal row of nails is required, such

nails are required to be staggered along either side of the center line of the row. Adjacent offset nails in such a configuration should be driven from opposite faces.

Where three rows of nails are required by spacing and edge distance requirements, nails in adjacent rows are to be staggered and adjacent nails beginning with the first in each row driven from opposite sides as if nails were aligned across the face of the laminations.

C15.3.4 Bolted Built-up Columns

C15.3.4.1 Maximum spacing limits for bolts and rows, and number of row requirements in (d), (e), and (g), respectively, are based on conditions for which the bolted built-up column efficiency factor, K_{c} , was established (82). Maximum end distance limits in (c) are good practice recommendations (32) to assure end bolts are placed close to the ends of the column where interlaminar shear forces are largest. Minimum end distance, spacing between adjacent bolts in a row, spacing between rows, and edge distance in (c), (d), (e), and (f) correspond to provisions governing bolted joints in NDS 12.5.

As with nailed columns, a bolt row refers to those bolts aligned parallel to the grain of the laminations and in the direction of the column length. The maximum spacing of bolts in a row of six times the lamination thickness minimizes the potential for buckling of individual laminations between connection points.

C15.4 Wood Columns with Side Loads and Eccentricity

C15.4.1 General Equations

Equations for wood columns are based on theoretical analyses (186). The equation in NDS 15.4.1 for combined bending and eccentric axial compression loads is an expansion of the interaction equation given in NDS 3.9.2 to the general case of any combination of side loads, end loads, and eccentric end loads (189).

For the case of a bending load on the narrow face and an eccentric axial load producing a moment in the same direction as the bending load, the general interaction equation in NDS 15.4-1 reduces to:

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1} + f_c(6e_1/d_1)[1 + 0.234(f_c/F_{cE1})]}{F_{b1}[1 - (f_c/F_{cE1})]} \leq 1.0 \quad (\text{C15.4-1})$$

or

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1} + f_c(6e_1/d_1)[1.234 - 0.234C_{m1}]}{C_{m1}F_{b1}'} \leq 1.0 \quad (\text{C15.4-2})$$

where:

$$C_{m1} = \text{moment magnification factor} = 1 - f_c / F_{cE1}$$

A limit on $(f_c/F_{cE2}) + ([f_{b1} + 6f_c e_1/d_1]/F_{bE})^2$ is given in Equation 15.4-2 to clarify a limitation on flatwise bending in the stress interaction equation (see C3.9.2).

For the case of a bending load on the wide face and an eccentric axial load producing a moment in the same direction as the bending load, the general interaction equation in NDS 15.4-1 reduces to:

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b2} + f_c(6e_2/d_2)[1 + 0.234(f_c/F_{cE2})]}{F_{b2}[1 - (f_c/F_{cE2})]} \leq \quad (\text{C15.4-3})$$

or,

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b2} + f_c(6e_2/d_2)[1 + 0.234C_{m2}]}{C_{m2}F'_{b2}} \leq 1.0 \quad (\text{C15.4-4})$$

where:

$$C_{m2} = \text{moment magnification factor} = 1 - f_c / F_{cE2}$$

An alternative formulation of Equation C15.4-4 in terms of induced axial load and induced bending moment follows:

$$\left(\frac{P}{P'}\right)^2 + \frac{M_2 + P\Delta_2[1 + 0.234\frac{P}{P_{cE2}}]}{M_2'(1 - \frac{P}{P_{cE2}})} \leq 1.0 \quad (\text{C15.4-5})$$

where:

P = induced axial compression load, lbs

P' = adjusted axial compression resistance (i.e. $F'_c A$), lbs

M_2 = induced bending moment in the weak axis, in.-lbs

M_2' = adjusted weak axis moment resistance (i.e. $F_{b2}' S_2$), in.-lbs

Δ_2 = eccentricity of axial load, measured perpendicular to the plane of the weak axis, in.

P_{cE2} = critical column buckling resistance in the weak axis, lbs (see C3.7.1.5)

For cases where loading is concentric (i.e. $\Delta_2 = 0$ in.), equation C15.4-5 reduces to the following:

$$\left(\frac{P}{P'}\right)^2 + \frac{M_2}{M_2'(1 - \frac{P}{P_{cE2}})} \leq 1.0 \quad (\text{C15.4-6})$$

where

$$P < P_{cE2} = \frac{\pi^2 E_{\min}' I_2}{\ell_{e2}^2} \quad (\text{C15.4-7})$$

C15.4.2 Columns with Side Brackets

The procedure for calculating the portion of an axial load applied through a bracket that is assumed to act as a side load at mid-height of the column is based on early recommendations (184). The application of a side load, P_s , acting at mid-span of a simply-supported beam is assumed to produce a maximum moment ($P_s \ell / 4$) equal to three-fourths of the moment produced by the eccentric load on the bracket, $P(a)$, times the ratio of the bracket height (ℓ_p) to the column length (ℓ).

When the bracket is at the top of the column, results obtained by entering NDS Equation 15.4-1 (or NDS Equation 3.9-3) with a concentric axial load and the calculated side load, P_s , will give a 25 percent lower combined stress index than that obtained from the eccentric axial end load formula, NDS Equation 15.4-2. This difference is a result of the latter being based on the assumption of eccentric loads on both ends of the column (constant moment along the length of the column) whereas the procedure in NDS 15.4.2 assumes the moment due to the bracket load decreases linearly from the point of application to zero at the column base.

C

COMMENTARY: SPECIAL LOADING CONDITIONS

C16 FIRE DESIGN OF WOOD MEMBERS

C16.1 General

The design provisions in the Specification are intended for use in allowable stress design (ASD) of structural wood members exposed to fire. These provisions do not address procedures for evaluating members for continued service following fire damage.

The topic of fire safety is broad and is addressed extensively in building codes. Other important topics such as flame spread, fire prevention, control of fire growth, fire suppression, fire extinguishment, and compartmentalization of fire are not addressed in the NDS. Other publications of interest from AWC include:

- *DCA No. 1-Flame Spread Performance of Wood Products*
- *DCA No. 2-Design of Fire-Resistive Exposed Wood Members*
- *DCA No. 3-Fire Rated Wood Floor and Wall Assemblies*
- *DCA No. 4-Component Additive Method (CAM) for Calculating and Demonstrating Assembly Fire Resistance*

Fire-Resistance-Rated Assemblies

Some buildings and occupancies require the use of fire-resistance-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 support design loads during fire exposure or to prevent the spread of fire. 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 using light-frame wood members can be accomplished through the use of fire-resistive membranes such as gypsum wallboard. However, material requirements and construction details are critical for assembly performance and are carefully specified. For some wood assemblies, 2-hour ratings have been achieved.

Fire-resistance ratings are available in building codes and reference documents for several generic assemblies. Fire-resistance ratings for assemblies using proprietary materials are typically supplied by either wood products manufacturers or gypsum manufacturers. Selected fire-re-

sistance-rated assemblies for walls and floors are provided in AWC's *DCA No. 3-Fire Rated Wood Floor and Wall Assemblies*, available at www.awc.org. In addition, DCA 3 includes details for the intersection of fire-resistance-rated walls and floors.

Analytically Rated

As an alternative to determining the fire-resistance-rating of members and assemblies by testing for every situation, building code provisions provide for acceptance of fire-resistance-ratings based upon engineering principles and material properties developed from standardized testing. 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 determining a fire-resistance-rating for 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*, and *Technical Report 10: Calculating the Fire Resistance of Exposed and Protected Wood Members* (136), available at www.awc.org.

Exposed Timber Members

Using calculations, exposed timber members of sawn lumber, glued-laminated softwood timber, laminated veneer lumber, parallel strand lumber, laminated strand lumber, and cross-laminated timber can be designed for desired fire-resistance ratings. Additional details regarding the analytical approach are provided in NDS Chapter 16 and AWC's *Technical Report 10* available at www.awc.org.

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 and Protected Wood Members* (136). The design procedure calculates the capacity of exposed wood members using basic wood engineering mechanics. Section properties are computed assuming an effective char depth, a_{eff} , at a given time, t . Reductions of strength and stiffness of wood in the heated zone adjacent to the char layer are accounted for by assuming the effective char depth, a_{eff} , is equal to 1.2 times the char depth, a_{char} (See C16.2.1.4). Average member strength properties are approximated from 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 evaluate wood members for continued use after a fire event.

C16.2.1 Char Rate

C16.2.1.1 Extensive one-dimensional char rate data is available for wood slabs. Two-dimensional char data is also available for 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 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 charring tests of wood slabs 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, a_{char} , to adjust for char rate nonlinearity:

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

where:

t = exposure time (hr.)

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

a_{char} = char depth (in.)

However, application of this model is limited since the char slope (hr./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, the one-dimensional non-linear char rate model was modified to enable values for the slope factor, m , to be estimated using nominal char rate constants (in./hr.), β_n . The nominal char rate constant, β_n , is estimated using measured char depth at approximately one hour. The nonlinear char rate constant, β_t , is estimated from the nominal char rate constant, β_n . The relationship between β_t and β_n is as follows:

$$\beta_t = \beta_n (1 \text{ hr.}) / (1 \text{ hr.})^{0.813} \text{ in./hr.}^{0.813} \quad (\text{C16.2-1})$$

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

$$a_{\text{char}} = \beta_t t^{0.813} \quad (\text{C16.2-2})$$

C16.2.1.2 For sawn lumber, structural glued-laminated softwood timber, laminated veneer lumber, parallel strand lumber, and laminated strand lumber, the char depth can be directly estimated using NDS Equations 16.2-1 and 16.2-2 assuming a nominal char rate constant, β_n , of 1.5 inches/hr. The char depth, a_{char} , for each exposed surface can be calculated as:

$$a_{\text{char}} = 1.5 t^{0.813} \quad (\text{C16.2-3})$$

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

$$t_{\text{gi},i} = \left(\frac{h_{\text{lam},i}}{\beta_t} \right)^{1.23} \quad (\text{C16.2-4})$$

where:

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

h_{lam} = lamination thickness (in.)

The number of laminations that could potentially fall off 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:

$$t - \sum_{i=1}^{n_{lam}} t_{gi,i} \geq 0 \quad (C16.2-5)$$

where:

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

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

$$a_{char} = \sum_{i=1}^{n_{lam}} h_{lam,i} + \beta_t \left(t - \sum_{i=1}^{n_{lam}} t_{gi,i} \right)^{0.813} \quad (C16.2-6)$$

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

$$a_{char} = (n_{lam})(h_{lam}) + \beta_t \left(t - (n_{lam})(t_{gi}) \right)^{0.813} \quad (C16.2-7)$$

where:

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

and

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

C16.2.1.4 For sawn lumber, structural glued-laminated softwood timber, 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 the effective char depth, a_{eff} , calculated per NDS Equation 16.2-4 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.

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

C16.2.1.5 For cross-laminated timbers, reduced cross-section dimensions are calculated using the effective char depth, a_{eff} , calculated per 16.2.1.3 and 16.2.1.4; however, due to the cross laminations and proprietary nature of cross-laminated timber layups, the effect of charring on capacity should be checked with the manufacturer. Alternatively, the capacity of the charred cross-laminated timber can be conservatively estimated by limiting the post-fire capacity to that of a panel with the same number of full-depth strong axis laminations remaining after a given fire resistance time.

C16.2.2 Member Strength

To approximate an average member strength using a reference design value, the reference design value is multiplied by an adjustment factor, K , to adjust from a 5% exclusion value allowable design value to an average ultimate value. F_b^* is multiplied by the adjustment factor, K , prior to calculation of the beam stability factor, C_L (Equation 3.3-6). Similarly, 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 sawn lumber, structural glued laminated softwood timber, laminated veneer lumber, parallel strand lumber, laminated strand lumber, and cross-laminated timber, 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 load cannot exceed the average member capacity of a wood member exposed to fire for a given time, t . The average member capacity is 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 Softwood 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 must 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 C16.2.4A(b) and C16.2.4A(c), respectively. For balanced beams, the core laminations adjacent to the tension laminations on both sides need to be replaced as shown in Figure C16.2.4B(b) and C16.2.4B(c), respectively.

Figure C16.2.4A Typical Unbalanced Beam Layout

Unrated	1-hr Fire Resistance Rating	1-1/2 and 2-hr Fire Resistance Rating
Outer Compression	Outer Compression	Outer Compression
Inner Compression	Inner Compression	Inner Compression
Core	Core	Core
Core	Core	Core
Core	Core	Core
Core	Core	Core
Core	Core	Core
Core	Core	Core
Core	Inner Tension	Inner Tension
Inner Tension	Inner Tension	Inner Tension
Inner Tension	Outer Tension	Outer Tension
Outer Tension	Outer Tension	Outer Tension
(a)	(b)	(c)

Figure C16.2.4B Typical Balanced Beam Layout

Unrated	1-hr Fire Resistance Rating	1-1/2 and 2-hr Fire Resistance Rating
Outer Tension	Outer Tension	Outer Tension
Inner Tension	Outer Tension	Outer Tension
Inner Tension	Inner Tension	Outer Tension
Core	Inner Tension	Inner Tension
Core	Core	Inner Tension
Core	Core	Core
Core	Core	Inner Tension
Core	Inner Tension	Inner Tension
Inner Tension	Inner Tension	Outer Tension
Inner Tension	Outer Tension	Outer Tension
Outer Tension	Outer Tension	Outer Tension
(a)	(b)	(c)

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 occurs as members shrink and joints between members open. The degree of exposure is a function of the view angle of the

radiant flame and the ability of hot volatile gases to pass through the joints. When the joint is completely open, such as can occur with butt-jointed timber decking, hot gases will carry into the joint and the sides of the decking members will char. This charring can be conservatively approximated assuming the sides of a member along the joint char to a depth of a_{eff} . 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, such that it chars to a depth of $a_{eff}/3$. 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.

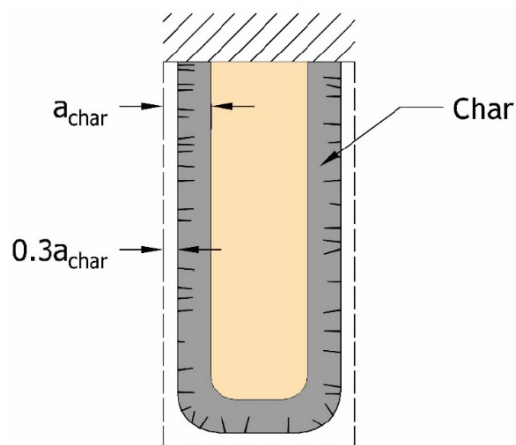
C16.3 Wood Connections

In the 2018 edition of the Specification, this section was revised to clarify that all components of the connection, including wood members, connectors, and fasteners, must be protected from fire exposure. Portions of the wood members included in the connection design must be fully protected against charring during the required fire resistance period to ensure that the design strength of the connection is maintained. Protection must be provided by wood, fire-rated gypsum board, other approved materials, or a combination thereof. Where wood is used as a protective cover, use of a_{char} will ensure that the connection elements are shielded from radiative heating and the temperature behind the char front will be less than 550 degrees F (300 degrees C). If the strength of the wood member in the connection is being evaluated (NDS 16.2.2), then use of a_{eff} will ensure no strength loss in the elevated temperature zone behind the char front. Connections located at ends of members must be detailed to ensure that char contraction is addressed to prevent premature loss of protection.

The loss of thickness at any location within the char layer can be estimated by multiplying the char depth, a_{char} , at that location by a *Char Contraction Factor*, C_{CF} (See Ref. 136), equal to 0.3 as shown in Figure C16.3. For a wood member with a char depth of approximately 1 inch in the face of the member, the loss of char thickness would be approximately 0.3 inches.

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

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

Figure C16.3 Char contraction**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	$1 - 1.645 \text{ COV}_b$	0.16 ²	2.85
Tensile Strength	F_t	2.1 ¹	$1 - 1.645 \text{ COV}_t$	0.16 ²	2.85
Compression Strength	F_c	1.9 ¹	$1 - 1.645 \text{ COV}_c$	0.16 ²	2.58
Buckling Strength	E_{05}	1.66 ³	$1 - 1.645 \text{ COV}_E$	0.11 ⁴	2.03

1. Taken from Table 10 of ASTM D245 *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber*, Table 1 of ASTM D3737 *Standard Practice for Establishing Allowable Properties for Structural Glued Laminated Timber (Glulam)*, and Table 1 of ASTM D5456 *Standard Specification for Evaluation of Structural Composite Lumber Products*.

2. Taken from Table 5-6 of 2010 *Wood Handbook* for bending clear wood values.

3. Taken from Appendices D and H of 2018 *National Design Specification for Wood Construction*.

4. Taken from Appendix F of 2018 *National Design Specification for Wood Construction*.

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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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