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American Wood Council
Engineered and Traditional Wood Products

ADDENDUM to the

1991 COMMENTARY

on the

NATIONAL DESIGN SPECIFICATION® (NDS®) FOR WOOD CONSTRUCTION

Commentary on Changes in the 1997 Edition of the
National Design Specification for Wood Construction

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FOREWORD

The Addendum provides commentary on changes that have been made in the 1997 Edition of the *National Design Specification for Wood Construction* since the last edition was published in 1991.

In addition to providing background and interpretative information on new or revised provisions, the Addendum includes discussion of other requirements in the Specification that user inquiries have identified as needing further clarification.

The comprehensive 1991 *NDS Commentary* is applicable to the 1997 Edition of the Specification except where superseded by this Addendum.

American Forest & Paper Association

To obtain a copy of the 1991 *NDS Commentary*, call 1-800-890-7732 or visit the American Wood Council website at www.awc.org for details.

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CHAPTER I. GENERAL REQUIREMENTS FOR STRUCTURAL DESIGN

1.1-SCOPE

1.1.1- Practice Defined

The structural design provisions in the Specification are based on working stress or deterministic design principals that have been in general use since 1944. Further discussion of the use of other criteria, such as reliability-based design standards or results of full-scale test results, is given in the 1991 Commentary.

The words "practice" and "method" are used interchangeably in the Specification.

Most of the advisory provisions in the 1991 edition, identified by the phrase "shall be permitted", either have been made mandatory in the 1997 edition or have been deleted. Deleted advisory text is discussed in this Addendum where appropriate to supplement information on the subject in the 1991 Commentary.

1.4-DESIGN LOADS

1.4.2-Governed by Codes

The Specification now requires use of minimum design loads from recognized design load standards, such as ANSI/ASCE Standard 7-95, when there is no building code governing the design of the structure.

1.4.4-Load Combinations

The load duration factors, C_D, in 2.3.2.3 of the Specification are independent of load combination factors and both may be employed in design analyses (see 1991 Commentary 1.4.4).

1.5-SPECIFICATIONS AND PLANS

In the design of wood structures, it is good practice to indicate on applicable plans and specifications the normal load duration design values being used and the moisture conditions to which the design values apply (see 1991 Commentary 1.5).

CHAPTER II. DESIGN VALUES FOR STRUCTURAL MEMBERS

2.1-GENERAL

2.1.2-Responsibility of Designer to Adjust for Conditions of Use

The Specification provides factors to adjust design values for wood members and connections for specific conditions frequently encountered in service. It does not set forth general requirements for adjusting design values for all possible applications and related conditions of use, particularly those involving extreme loading and service exposures (see 1991 Commentary 2.1.2 for example). Such inclusivity would require use of overly conservative and economically prohibitive adjustment factors not required for most applications. It is the designer's responsibility to determine the design value adjustment factors that are appropriate for each application.

2.3-ADJUSTMENT OF DESIGN VALUES

2.3.2-Load Duration Factor, C_D

In the 1997 edition, footnote 2 to Table 2.3.2 has been revised to limit load duration adjustments for members pressure treated with water-borne preservatives or fire retardant chemicals to no more than 1.6. The revision extends the exclusion on use of the 2.0 impact duration of load factor to all members pressure-treated with water-borne preservatives, not just those members treated to the heavy retentions (2 pcf or more) required for marine use. This extension is based on new research which indicates the impact resistance of wood pressure-treated with water-borne preservative to retentions as low as 0.4 pcf and redried is less than that of untreated wood (24).

The revised footnote to Table 2.3.2 not only precludes use of the impact duration of load adjustment with water-borne preservative and fire-retardant treated wood, but also excludes use of any adjustment greater than 1.6 (wind and earthquake) for these materials.

Footnote 2 to Table 2.3.2 continues to note that use of the impact load duration factor is not allowed with connections.

2.3.4-Temperature Factor, C_t

Tabulated design values in the Specification are applicable to members used under ordinary ranges of temperatures and occasionally heated in use to temperatures up to 150°F. Wood increases in strength when cooled below normal temperatures and decreases in strength when heated. Up to 150°F, these changes are immediate and generally reversible when the wood returns to normal temperature levels. Prolonged exposure over 150°F can result in permanent strength loss.

The temperature adjustment factors given in Table 2.3.4 are applicable to those applications where members are exposed to elevated temperatures up to 150°F for extended period of times, such as in industrial applications. Design values for structural members in roof systems meeting building code ventilation requirements are not generally adjusted for temperature as the elevated temperature exposures that can occur in such applications as a result of solar radiation are transient and generally accompanied by offsetting decreases in moisture content. (See 1991 Commentary 2.3.4 for discussion of reversible and permanent temperature effects, and the diurnal temperature fluctuations of members in roof systems.)

2.3.5-Pressure-Preservative Treatment

Duration of load adjustments greater than 1.6 are not permitted for structural members pressure-treated with water-borne preservatives (see 2.3.2 Commentary Addendum). Prior to the 1997 edition, use of the impact duration of load adjustment was not permitted for structural members pressure-treated with water-borne preservatives to the heavy retentions required for “marine” exposure.

2.3.6-Fire Retardant Treatment

The 1997 edition now limits duration of load adjustments for structural members pressure-treated with fire retardant chemicals to a maximum of 1.6, the factor considered applicable to the results of short-term static load tests. Previous editions excluded use of only the tabulated 2.0 impact load duration factor.

2.3.11-Incising Factor, C_i

Adjustment factors to account for the effect of incising on allowable design values for sawn lumber have been introduced in the 1997 edition. Incising involves making shallow, slit-like holes parallel to the grain in the surfaces of material to be preservative treated in order to obtain deeper and more uniform penetration of preservative. It is used to improve the effectiveness of treatment of members having heartwood surfaces and of species which tend to be resistant to side penetration of preservative solution, such as Douglas fir, Engelmann spruce, and hemlock.

The effect of the incising process has been found to be dependent on the depth and length of individual incisions and the number of incisions (density) per square foot of surface area. (17,14,23). The incising adjustment factors for E, F_b, F_t and F_c given in Table 2.3.11 of the Specification are limited to patterns in which the incisions are not deeper than 0.75", not longer than 0.375" and no more than 375 per square foot in number. Where these limits are exceeded, it is the designer's responsibility to determine, from authoritative literature or special tests, the incising adjustment factors that should be used with the structural material being specified.

Adjustments given in Table 2.3.11 are based on reductions observed for incised dimension lumber (e.g., 2 inch and 4 inch nominal thickness). The adjustment factors in Table 2.3.11 are based on the assumption that the incised preservative treated lumber will be used in wet service conditions where tabulated design values have been adjusted by the wet service factors, C_M, given in Tables 4A, 4B, 4C, 4D and 4E of the Specification Supplement. A summary of early testing (17) of timbers and railway ties indicates that a slight decrease in strength properties for timbers may be expected. In some cases, no strength reductions were reported. Consequently, use of reductions in Table 2.3.11 is considered to be conservative for larger members such as solid sawn timbers.

The adjustments given in Table 2.3.11 are less severe than those reported for incision patterns which are denser than defined in the Specification (14). For double density incised 2 inch nominal lumber (density of 1,090 per square foot and incision depths ranging from 0.35 to 0.40 inches) where mean modulus of elasticity was reduced as much as 6 percent, mean and fifth percentile modulus of rupture were reduced as much as 21 percent and 25 percent , respectively, compared to the control group. These reductions were observed for the lumber incised at moisture contents greater than 25%. Mechanical properties of all specimens were evaluated at approximately 12% moisture content.

PART III. DESIGN PROVISIONS AND EQUATIONS

3.2 BENDING MEMBERS - GENERAL

3.2.3-Notches

3.2.3.1, 3.2.3.2, 3.2.3.3, 3.2.3.4 Although these provisions concerning notching on tensions faces, effect of notches on stiffness, limits on notches in sawn lumber beams and effect of notches on shear strength have been editorially revised in the 1997 edition, no substantive changes have been made from the previous edition. In addition to limiting notches in bending members, notching of members subject to tension or combined bending and tension should be avoided whenever possible.

3.2.3.5 From 1977 to the 1991, the Specification limited notching of glued laminated timber bending members to good engineering practice. Thus responsibility was placed on the designer to determine specific limits for notching in these members using common practices described in the literature as a guide (see 1991 Commentary 3.2.3.3).

Specific provisions governing notching of glued-laminated timber bending members have been introduced in the 1997 edition. These provisions reflect current standard design practice (1,2) which has a record of satisfactory performance in the field. The limitation for compression side notches in the outer third of the span of 2/5 the depth is based on early experimental research (9). In addition to limiting the depth and location of a notch, notch length should be kept to a minimum. Recent guidelines on notching of glued laminated timber beams (25) suggests that compression side notch length should not exceed three times the depth of the member remaining at the notch or 1/8 of the simple span length. It should be noted that the notch limitation of 1/6 the depth for dimension grade lumber bending members in 3.2.3.3 applies to both compression and tension side notches in the outer third of the span. This lumber limitation reflects reductions taken for lumber edge knots and also is supported by a record of satisfactory field performance (see 1991 Commentary 3.2.3.2).

The occurrence of stress reversals shall be considered when applying notch provisions.

3.3 BENDING MEMBERS - FLEXURE

3.3.3-Beam Stability Factor, C_L

3.3.3.5 A new footnote 2 has been added to Table 3.3.3 that requires effective lengths (ℓ_e) for multiple span applications to be based on the same equations as those given for cantilever and single span beams in the table, including the equations in footnote 1 for unspecified loading conditions.

Based on comparison of moment and deflection diagrams, use of effective length equations for single span members with comparably loaded continuous span members is considered conservative. Because of this assessment, the new footnote provides the alternate of using specific engineering analysis to establish effective lengths for multiple span applications (see Reference 20 for exact equations on the stability of single span beams with different end restraint conditions and general theory in Reference 19).

3.3.3.8 The 1997 edition includes an equation for calculating the Euler buckling constant (K_{bE}) for long bending members based on the coefficient of variation (COV_E) associated with the modulus of elasticity design value (E) of the particular wood product involved. Values of K_{bE} published in the 1991 edition of 0.438 for visually graded and machine evaluated lumber associated with a COV_E of 0.25, and 0.609 for products with a COV_E of 0.11 or less, were based on the equation:

$$[1-1.645(COV_E)][(1.2)(1.03)/(1.66)] \qquad \text{(CA3.3-1)}$$

where:

1.2 is the equivalent of the Euler buckling coefficient of 0.822 for columns,
1.03 adjusts tabulated E values to a pure bending basis,
1.66 represents a factor of safety, and,
1.645 represents the normal deviate associated with a 5 percent lower exclusion value.

(See 1991 Commentary 3.3.3 and 3.7.1.5)

The simplified form of Equation CA3.3-1

$$K_{bE} = 0.745 - 1.225(COV_E) \qquad \text{(CA3.3-2)}$$

was included in the 1997 edition to enable calculation of the buckling constant for long bending members made with different products. However, the use of rounded constants in CA3.3-2 results in slight changes in buckling constants previously given in the Specification. The constant for visually graded lumber, based on a COV_E of 0.25, has been changed from 0.438 to 0.439, and that for COV_E 0.11 products has been changed from 0.609 to 0.610.

Also in the 1997 edition, a separate buckling constant of 0.561 is given for machine evaluated lumber. This value is based on a COV_E of 0.15 which is applicable to the product.

3.4 BENDING MEMBERS - SHEAR

3.4.1-Strength in Shear Parallel to Grain (Horizontal Shear)

3.4.1.1 Although both parallel and perpendicular to grain shear occur simultaneously in wood bending members, parallel to grain shear strength is always the limiting case (see 1991 Commentary 3.4.1.1).

3.4.3-Shear Force

3.4.3.1 (b) The provision governing placement of moving loads has been revised to clarify that the largest single moving load is to be placed at a distance from the support equal to the depth of the member, rather than the wording used previously that required only that a moving load that was "considerably larger than any of the others" be placed in this location. Also, the new edition specifically requires that shear forces be checked at each support to assure that the maximum shear force associated with unequal wheel loads and spacings be considered.

3.4.4-Shear Design for Notched Bending Members

3.4.4.1, 3.4.4.2, 3.4.4.3 The 1997 edition requires the actual shear stress in all bending members notched on the tension face at the end to be determined. In earlier editions, this check was limited to only short, relatively deep members, generally considered to be those with a span to depth ratio of 12 or less based on the depth of the unnotched bending member.

3.4.4.5 An equation for calculating shear stress in rectangular beams notched on the compression face at the end has been added to the 1997 edition. This equation, based on early beam tests (9), accounts for stress concentration effects by reducing the unnotched depth of the beam, d, by the amount $(d-d_n)(e/d_n)$, where d_n is the depth of the member remaining at the notch and e is the distance the notch extends into the beam from the face of the support. This adjustment is less than the reduction accounting for stress concentrations in beams with tension face end-notches. For example (see 1991 Commentary 3.4.4.1), a beam with a compression face end-notch of one-quarter the beam depth extending one-half the beam depth from the support has an effective depth of (5/6)d, 48 percent greater than that of (9/16)d for a beam with a tension face one-quarter end-notch.

For circular cross sections having compression side end-notches, application of a similar adjustment results in the following equation for calculating actual shear stress

$$f_v = 3V/[2(A-(A-A_n)e/dn)] \quad \text{(CA3.4-1)}$$

where:

A = cross-sectional area of circular bending member,
A_n = net area of the circular bending member,
d_n = depth of the member remaining at the notch, and
e = distance the notch extends into the beam from the face of the support.

3.5 BENDING MEMBERS -DEFLECTION

3.5.2-Long Term Loading

An equation for calculating total deflection to account for time-dependent deformation has been added to the 1997 edition. Previous editions provided the same information in text form. The creep factors applied to the initial deflection associated with the long term component of the design load, 1.5 for seasoned and 2.0 for unseasoned material, have not been changed. (See 1991 Commentary 3.5.2 for discussion of loading conditions where creep may be a design consideration.)

3.7 SOLID COLUMNS

3.7.1-Column Stability Factor, C_P

3.7.1.5 An equation for calculating the Euler buckling coefficient (K_{cE}) for long columns based on the coefficient of variation (COV_E) associated with the modulus of elasticity of the material involved has been added to the 1997 edition. Values of K_{cE} published in the 1991 edition of 0.300 for visually graded and machine evaluated lumber associated with a COV_E of 0.25, and 0.418 for products with a COV_E of 0.11 or less, were based on the equation

$$[1-1.645COV_E)][(0.822)(1.03)/(1.66)] \quad \text{(CA3.7-1)}$$

where:

0.822 is the Euler buckling coefficient,

1.03 is the adjustment of tabulated E values to a pure bending basis,

1.66 represents a factor of safety, and,

1.645 represents the normal deviate associated with a 5 percent lower exclusion value.

The simplified form of Equation CA3.7-1

$$K_{cE} = 0.510 - 0.839(COV_E) \quad \text{(CA3.7-2)}$$

was included in the 1997 edition to provide for calculation of the buckling coefficient for any material.

In recognition of the lower COV_E of 0.15 associated with machine evaluated lumber compared to that for visually graded lumber, the 1997 edition provides a separate buckling constant of 0.384 for the former. The buckling constant of 0.418 associated with a COV_E of 0.11 is applicable to glued laminated timber and machine stress rated lumber. A K_{cE} of 0.30 may be applied to round timber piles.

3.8 TENSION MEMBERS

3.8.1 Notching of members subject to tension should be avoided.

3.8.2 Because of the variable effects of checking and splitting that can occur as result of drying in service, sawn lumber tension perpendicular to grain design values are not published in the Specification. However, radial tension perpendicular to grain design values are provided for curved, pitched and other shapes of glued laminated timber bending members in which radial stresses are induced from normal bending loads. Glued laminated timber members are made of dry material whose quality is controlled at the time of manufacture.

Wood is relatively weaker in tension perpendicular to grain than in other properties. Designs involving applied loads that induce this stress in both sawn lumber and glued laminated timber beams are to be avoided. Examples of designs that induce tension stress perpendicular to grain include the hanging of loads below the neutral axis of a beam and the use of wood members to resist loads which induce cross grain bending. Mechanical reinforcement shall be considered for all designs where induced tension perpendicular to grain stresses cannot be avoided (see discussion of reinforcement in 1991 Commentary 3.8.2).

3.9 COMBINED BENDING AND AXIAL LOADING

3.9.1-Bending and Axial Tension
3.9.2-Bending and Axial Compression

Although 3.9.1 and 3.9.2 in the 1997 edition no longer specifically address the use of the load duration factor, C_D, associated with the shortest duration load in a combination of loads when calculating F_b and F_t, or F_b and F_c values, such use is provided for under 2.3.2.2 of the Specification.

All combinations of design load components, from the shortest to the longest duration, shall be considered when determining values of F_b, F_t and F_c used in combined load equations (3.9-1), (3.9-2) and (3.9-3). See 1991 Commentary 3.9.1 for discussion of the use of either (i) the shortest load duration factor for both axial and bending stresses even though the load of shortest duration is associated with only one of these stresses, or for use of (ii) different factors for bending and axial stresses depending on the loads that are associated with each stress.

3.10 DESIGN FOR BEARING

3.10.1-Bearing Parallel to Grain

3.10.1.2 The provision for use of an insert when the actual bearing stress, f_g, is 0.75 or more of the allowable bearing stress, F_g', has been revised in the 1997 edition to clarify that it is the stiffness of the insert material that is the critical property to assure uniform distribution of load between end-to-end bearing members. Twenty-gage steel plate is generally considered to have adequate stiffness as well as strength for this purpose.

CHAPTER IV. SAWN LUMBER

4.1. GENERAL

4.1.2-Identification of Lumber

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

4.1.3-Definitions

4.1.3.4 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.

4.1.4-Moisture Service Condition of Lumber

Design values tabulated for Southern Pine timbers and Mixed Southern Pine timbers in 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.

4.1.6-End-Jointed or Edge-Glued Lumber

End- 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 "VERT 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 "VERT USE ONLY".

4.2.6-Compression Perpendicular to Grain, F_{c2}

In the 1997 edition, the equation for calculating an allowable F_{c2} associated with a deformation level of 0.02" has been simplified from

$$F_{c20.02} = 5.6 + 0.73F_{c2} \qquad \text{(CA4.2-1)}$$

to

$$F_{c20.02} = 0.73F_{c2} \qquad \text{(CA4.2-2)}$$

4.3 ADJUSTMENT OF DESIGN VALUES

4.3.3-Flat Use Factor, C_{fu}

The flat use factor, C_{fu}, is to be used cumulatively with the size factor, C_F.

4.4 SPECIAL DESIGN CONSIDERATIONS

4.4.1-Stability of Bending Members

4.4.1.2 The alternate lateral support requirements have been revised in the 1997 edition to be more consistent with the original form of the approximate rules (9) and the applications of these rules in the early editions of the Specifications (see background and discussion in 1991 Commentary 4.4.1).

The main changes made in the approximate rules in the new edition are the specific requirement for rotation and lateral displacement restraint at points of bearing for depth to width ratios greater than 4 and the requirement that the compression edge of members having a depth to breadth ratio greater than 4 and equal to or less than 5 be held in line for their entire length. In earlier editions, either the tension or the compression edge of members with a depth to breath ratio of 5 could be held in line. The approximate rules place no limit on member length. Also, information from early research (20) indicates that wrinkling of one edge of a compression member can occur when the other edge is restrained (T section) along its length.

4.4.3-Wood Trusses

4.4.3.1 The buckling stiffness factor, C_T, applicable to 2x4 or smaller compression chords is inversely related to the tabulated design modulus of elasticity, E, adjusted to a nominal 5th percent exclusion value, or

$$E_{0.05} = K_T\, E_{table} \qquad \text{(CA4.4-1)}$$

where:

$$K_T = [1 - 1.645(COV_E)] \qquad \text{(CA4.4-2)}$$

COV_E = coefficient of variation, percent

The 1997 edition introduces equation CA4.4-2 for K_T to enable calculation of a C_T factor for any lumber COV_E. Also added in the new edition is the K_T value 0.75 for machine evaluated lumber based on a COV_E of 0.15. Previously listed values were 0.59 for visually graded lumber (COV_E of 0.25) and 0.82 for products with a COV_E of 0.11 or less.

CHAPTER V. STRUCTURAL GLUED LAMINATED TIMBER

5.2 ADJUSTMENT OF DESIGN VALUES

5.3.2-Volume Factor, C_V

The volume factor, C_V, applies when glued laminated timber bending members are loaded perpendicular to the wide face of the laminations. The factor is applied only to the tabulated F_{bxx} design values in Tables 5A and 5B. The C_V factor and the flat use factor, C_{fu}, (5.3.3) are not applied cumulatively.

5.3.3-Flat Use Factor, C_{fu}

The flat use factor, C_{fu}, applies when glued laminating timber bending members are loaded parallel to the wide face of the laminations. The C_{fu} factors given in Tables 5A and 5B 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 lamination. For bending members loaded parallel to the wide face of the laminations and the dimension of the member in this direction is greater than 12", a flat use factor based on Equation 4.3-1 of the Specification should be used.

5.4.2-Lateral Stability for Glued Laminated Timber

5.4.2.1 The modulus of elasticity for beams loaded parallel to the wide face of the laminations, E_{yy}, is less than that for beams loaded perpendicular to the wide face of the laminations, E_{xx}. For this reason, all glued laminated beam stability calculations are made using values of modulus of elasticity for bending about the y-y axis, E_{yy}, modified by applicable adjustment factors.

CHAPTER VII. MECHANICAL CONNECTIONS

7.1 GENERAL

7.1.1-Scope

7.1.1.4 Design values for connections loaded in single and double shear tabulated in Chapters 8, 9, 11 and 12 are based on the fastener bending yield strengths, F_{yb}, given in the footnotes of the respective tables. Other fastener bending yield strengths may be used with the yield mode equations in these Chapters to calculate design values for the connections involved. However, bolts, lag screws and wood screws must conform to the applicable ANSI/ASME Standard referenced for these fasteners in 8.1.1, 9.1.1 and 11.1.1; and nails and spikes must meet the requirements specified in 12.1.2. Bending yield strength of nails and spikes may be determined in accordance with ASTM F1575-95 (see Appendix I of the Specification).

7.1.1.5 This new provision in the 1997 edition clarifies that tabulated lateral load design values for all fastener types (i) apply to connections in which the members are brought into contact at the time of fabrication and (ii) allow for dimensional changes of members associated with seasonal variations in moisture content. The effects of relatively large changes in moisture content, such as occur when connections are fabricated with wood members at moisture contents greater than 19 percent and/or will be exposed to conditions which will cause wood members to exceed 19 percent moisture content at any time in service are accounted for by the adjustment factors, C_M, given in Table 7.3.3. Other than these adjustments, no further modification of design values are needed for member dimensional changes associated with seasonal variations in moisture content that occur within the dry service condition of use class (19 percent or less), or within the partially seasoned or wet condition of use class (greater than 19 percent).

7.2.4-Design of Concrete or Masonry Parts

Directions for designing concrete or masonry parts involved in wood connections have been added to the 1997 edition. They parallel those given for the design of metal parts in 7.2.3.

As with metal parts, connection adjustment factors shown in Table 7.3.1 are not to be applied when the capacity of the connection is controlled by the concrete or masonry part. Also, strength of concrete or masonry parts in a connection are not to be increased 1/3 for wind or earthquake loadings if the design load on the connection has been reduced for load combinations as provided for in the applicable building code or national standard.

7.3 ADJUSTMENT OF DESIGN VALUES

7.3.1-Applicability of Adjustment Factors

Timber rivets have been added to Table 7.3.1, Applicability of Adjustment Factors for Connections, in the 1997 edition. Lateral load design values for this type of connection used with glued laminated timber are given in Part XIII of the Specification. In rivet connections, the adjustments for load duration and geometry factor apply only when capacity is controlled by the wood members. The metal side plate factor applies only when rivet strength is limiting.

7.3.3-Wet Service Factor, C_M

Table 7.3.3 giving wet service factors for different fastener types has been simplified in the 1997 edition.

Previously, three moisture conditions at time of fabrication were recognized. These were designated as follows:

Dry - wood moisture content is equal to or less than 19%
Wet- wood moisture content is equal to or greater than 30%
Partially seasoned (PS) - wood moisture content is greater than 19% and less than 30%.

In the current edition, the wet and partially seasoned conditions have been combined and the moisture conditions redesignated as "≤19%" and ">19%".

The five in-service moisture conditions considered in earlier editions are as follows:

Dry - wood moisture content is equal to or less than 19%
Wet- wood moisture content is equal to or greater than 30%
Partially seasoned (PS) - wood moisture content is greater than 19% and less than 30%
Exposed to the weather (Exposed) - wood moisture content will vary from greater than 19% to less than 30%
Subject to wetting and drying (W&D) - wood moisture content will vary from greater than 19% to less than 30% or >19% to over 30%, and the reverse

These have been reduced to two conditions of ≤19% and >19%. The latter condition includes both continuous or occasional exposure at moisture levels greater than 19%.

These consolidations eliminated the PS-Dry (fabrication to in-service combination) for shear plates, bolts and lag screws, which previously were assigned a C_M proportionate between that for the Dry-Dry combination and that for the Wet-Dry combination.

As part of the simplification, the C_M factor previously assigned to the Dry or Wet fabrication condition to the PS or Wet in-service condition for bolts, drift pins, drift bolts and lag screws of 0.67 was assigned a value of 0.7 for use with the new in-service condition of >19%. Also, bolts, lag screws, and wood screws in the fabricated Dry or Wet service condition and used in an Exposed service condition and previously assigned a C_M of 0.75 now fall in the >19% use class assigned the C_M of 0.7. Similarly, nails and spikes previously assigned a C_M of 0.75 for fabricated PS or Wet and used Dry or Wet, or fabricated Dry and used PS or Wet now are assigned a value of 0.7 for any combination involving a >19% condition either at fabrication or in use.

For comparable fabrication and in-service moisture content conditions, C_M values in the 1997 edition for metal connector plates, drift pins, drift bolts and threaded and hardened nails are unchanged from those previously assigned. Also, for comparable fabrication and in-service moisture conditions, C_M values for nail and spike withdrawal loads remain unchanged from previous editions. However, it should be noted that a new reduction has been introduced in the 1997 edition for the case of withdrawal loads on lag and wood screw connections having in-service moisture contents greater than 19%. This C_M of 0.7 is a conservative adjustment to account for the increase in the screw hole that occurs as a result of dimensional changes associated with increases in moisture content from dry to wet (19%) conditions.

The C_M factors for bolts and lag screws of 1.0 for connections used in dry (≤19%) service conditions apply to the following arrangements: single fastener, two or more fasteners placed in a single row parallel to grain, and fasteners placed in two or more rows parallel to grain with separate metal side plates for each row. The eligibility of these arrangements for the 1.0 C_M factor is the same as that recognized in previous editions.

The 0.4 service factor applies to all connections made in unseasoned wood and used in dry service conditions where the pattern of bolts, drift pins, drift bolts, or lag screws used may cause splitting of the main or side member due to restraint of shrinkage across the grain.

Wet service factors for timber rivets have been added to Table 7.3.3. Only the adjustments for connections fabricated in dry material and used in dry service conditions, C_M = 1.0, or in wet service conditions, C_M = 0.8, are applicable to rivet design values for glued laminated timber given in Part XIII of the Specification. Such members are fabricated of dry material and generally are not exposed to wet conditions before connections are installed. The 0.8 wet service adjustment accounts for the reduction in wood stiffness where rivet bending limits allowable loads or for the reduction in wood strength (shear and tension) where these properties are limiting.

CHAPTER VIII. BOLTS

8.1 GENERAL

8.1.2-Fabrication and Assembly

8.1.2.1 Forcible driving of bolts because of undersized holes, misalignment of members or other installation factors is specifically prohibited.

8.1.2.2 Bolt design values assume that holes in main members and side plates of all materials, not only steel, are aligned.

8.2 DESIGN VALUES FOR SINGLE SHEAR CONNECTIONS

8.2.1-Wood-to-Wood Connections

The equations used to established dowel bearing strength values, $F_{e//}$ and $F_{e\perp}$, tabulated in Table 8A have been added to the legend for the yield mode equations and as a new footnote 2 to Table 8A (see 1991 Commentary 8.2.1 for the basis of these equations).

Also in the 1997 edition, two new species combinations have been added to Table 8A: Engelmann Spruce-Lodgepole Pine overall species group and Spruce-Pine-Fir (E of 2,000,000 psi and higher grades of MSR and MEL). With regard to the latter case, some MSR and MEL grades in certain species combinations are assigned different specific gravity values than those for the overall species where specific gravity has been determined on a mill-specific basis or where such values have been found to be consistently higher for the grade(s) at all producing mills.

Table 8.2A in the 1997 edition includes a fourth column of design values, $Z_\perp$, for wood-to-wood single shear bolted connections in which both members are loaded perpendicular to grain. Examples where such tabulated values apply to connections between ledgers and band joists, between girders and ledger strips and similar applications.

8.2.3-Wood-to-Concrete Connections

The procedure for establishing design values for wood-to-concrete connections has been changed in the 1997 edition. In the 1991 edition, yield mode equations in 8.2.1 were entered with the concrete assumed to be twice the thickness of the wood member and to have a dowel bearing strength equal to that of the wood member (see 1991 Commentary 8.2.3). However, the yield mode equations show that this approach, which assigns concrete a different dowel bearing strength depending upon the specific gravity and dowel bearing strength of the wood member, inappropriately penalizes species that have lower values for these properties.

The 1997 edition requires that the dowel bearing strength of concrete be used as the main member dowel bearing strength, F_{em}, in the yield equations. Theoretical and experimental studies show that the ultimate dowel bearing strength of concrete can be related to its compression strength (21,4). These studies and comparison of steel to concrete connection tests (22) generally show that the ultimate dowel bearing strength of concrete can be considered to be 5 or more times its compressive strength. Lower ratios can be obtained depending on test conditions, particularly when premature splitting of the concrete prevents full bearing strength from being developed. A study of bolted wood-to-concrete connections (18) suggests assuming a ultimate dowel bearing strength of concrete equal to 5 times the compressive strength improves calculated estimates of connection strength.

Based on the available research and the satisfactory field experience of concrete to southern pine lumber connections designed using the yield equations and the dowel bearing strength for that species of 6150 psi for concrete, a dowel bearing strength of 6000 psi for concrete was used to establish the single shear concrete-to-wood bolt design values given in Table 8.2E. This dowel bearing value is assumed to be applicable to concrete with compressive strengths of 2000 psi and greater. Values in Table 8.2E are all based on a bolt embedment depth of 6", a minimum depth specified in some building codes for anchor bolts connecting wood wall plates to concrete foundations or piers.

Values for wood-to-masonry (assumed to be concrete masonry) connections are not specifically tabulated in the 1997 NDS. Lacking specific research on the dowel bearing strength of masonry, an assumed dowel bearing strength for masonry is not provided. Satisfactory field experience with connections of wood to masonry and a comparison of the capacities of equivalent fasteners in concrete and masonry indicates a dowel bearing strength similar to that assumed for concrete may be appropriate for some applications. In all cases, the concrete and masonry, including connections in concrete and masonry, are to be designed in accordance with accepted practices to support the applied loads (See 7.2.4 and 8.2.3.2 of the Specification).

New design values for concrete-to-wood connections made with Southern Pine and Spruce-Pine Fir and ½", 3/4" and 1" bolts are compared with those for equivalent connections based on 1991 provisions (main member thickness of 3" and a side member thickness of 1-1/2") in Table CA8.2-3 below.

As with wood-to-metal connections, bolt design values for concrete-to-wood connections are to be adjusted by the applicable factors in Table 7.3.1 and values for one species are applicable to other species having the same or higher dowel bearing strength.

8.2.3.2 It is the responsibility of the designer to confirm that the concrete or masonry involved in the connection has sufficient strength to support the applied loads. This includes providing appropriate edge and end distances for the bolt diameters being used.

8.3 DESIGN VALUES FOR DOUBLE SHEAR CONNECTIONS

8.3.1-Wood-to-Wood Connections

As with single shear connections, the general equations for establishing species dowel bearing strengths based on specific gravity have been added to the legend for the yield mode equations for the convenience of the user. Also added for clarification to this section in the 1997 edition is Equation 8.3-5 for determining the dowel bearing strength of a member included in the connection which is loaded at an angle to grain. This is the same equation as Equation 8.2-7 for single shear connections. In the 1991 edition, the latter equation was considered to apply to members in double shear connections as well but this intent was not specifically stated.

8.5 PLACEMENT OF BOLTS

8.5.1-Terminology

8.5.1.5 This new section in the 1997 edition has been added to further clarify that, while end distance, edge distance and spacing requirements in 8.5 are applicable to the wood members in metal-to-wood and concrete-to-wood connections, the strength properties of metal and concrete parts also must be checked to assure that end and edge distances and spacings in these materials are adequate to carry the applied load.

8.5.3-Edge Distances

8.5.3.3 The provision in the 1991 edition on avoiding suspension of heavy or medium concentrated loads below the neutral axis of beams has been clarified in the new edition for single sawn lumber or glued laminated timber beams. Designs with such loads only are permitted where stitch bolts or other mechanical or equivalent reinforcement is used to fully resist tension perpendicular to grain stresses. (See 1991 Commentary 3.8.2 and 8.5.3.3 for discussion of reinforcement considerations and light loading conditions.) Built-up girders made of multiple, similar size, parallel members in contact designed to carry loads from joists supported on ledger strips are not included under the requirements of this section because of their long record of satisfactory performance.

Table CA8.2-3 - Comparison of 1991 and 1997 NDS Wood-to-Concrete Single Shear Bolt Design Values

	Bolt	Bolt Design Value[1], lbs					
	Diam.	$Z_{//}$			Z_2		
Species	in.	1991	1997	Ratio	1991	1997	Ratio
Southern Pine	½	660	660	1.00	400	400	1.00
	3/4	1270	1270	1.00	660	660	1.00
	1	1740	2140	1.23	770	760	0.99
Spruce-Pine-Fir	½	540	570	1.06	320	330	1.03
	3/4	1000	1140	1.14	450	450	1.00
	1	1330	1760	1.32	530	520	0.98

1. Side member thickness, t_s=1.5 in.; main member thickness, t_m=6" and 3" for 1997 and 1991 NDS, respectively.

CHAPTER IX. LAG SCREWS

9.2 WITHDRAWAL DESIGN VALUES

9.2.1-Withdrawal from Side Grain

Equation 9.2-1 for calculating lag screw withdrawal design values in pounds per inch of thread penetration into the main member on the basis of tabulated specific gravity (oven-dry weight and volume) and lag screw unthreaded shank diameter, has been added in the 1997 edition as a convenience to users. This equation is the same as that used to establish the design values in Table 9.2A and in similar tables in previous editions (see 1991 commentary 9.2.1). It is to be noted that the equation is not to be used for main member specific gravities outside the range of 0.31 to 0.73, nor to lag screw diameters outside the range of 1/4" to 1-1/4"in the 1997 edition.

9.3 LATERAL DESIGN VALUES

9.3.1-Wood-to-Wood Connections

The species combinations Engelmann Spruce-Lodgepole Pine and Spruce-Pine-Fir (E of 2,000,000 psi and higher grades of MSR and MEL) have been added to the listing of dowel bearing strengths in Table 9A (see Addendum Commentary 8.2.1).

Equations for calculating dowel bearing strength are now included in the legend for the yield mode equations (see Addendum Commentary 8.2.1). Also, it is now required that the equations be entered with D (diameter) equal to the root diameter of the threaded portion of the lag screw when the threaded length extends into the shear plane of the connection. Previously, the wording of the provision indicated that root diameter be used only when the threaded length of the screw was greater than that specified in Appendix L. (See 1991 Commentary 9.3.1 for discussion of how the different yield moments of threaded and unthreaded shanks are accounted for in the Mode III and IV equations.)

Lateral load values given in Table 9.3A assume that the unthreaded shank diameter extends beyond the shear plane. For situations where this does not occur, design loads should be calculated using the yield mode equations directly with D equal to the root diameter of the threaded portion of the shank.

9.3.2-Wood-to-Metal Connections

9.3.2.1 Design values in Table 9.3B apply only to connections where the unthreaded shank diameter extends beyond the shear plane. If this condition does not occur, the yield mode equations should be used to establish design loads using the root diameter as D.

9.3.3-Penetration Depth Factor, C_d

The provisions for adjustment for length of lag screw in the main member, which are unchanged from the 1991 edition, are based on earlier research which showed that penetration depth was related to the ultimate load carried by the connection. In the 1986 and earlier editions, this research was reflected in the penetration requirements established to develop full design load, which increased from 7D to 11D as specific gravity decreased.

The introduction of the yield mode equations in 1991 required adjustment of equation values based on 5 percent dowel bearing offset values to the design load levels assigned lag screw connections in previous editions of the Specification. This was done by assuming that the full design load is developed when the penetration into the main member is 8D or more regardless of species specific gravity. The assumption was considered reasonable on the basis that lag screw design loads in earlier editions represented average proportional limit test values divided by 1.8 and that proportional limit test values were less affected by penetration depth than were ultimate loads (see 1991 Commentary 9.3.1). Thus use of the lag screw penetration depth requirements of 9.3.3 is keyed to the methodology used to establish present lag screw design values. This methodology does not require checking yield modes that involve main member penetration.

9.4 PLACEMENT OF LAG SCREWS

9.4.1-Geometry Factor, Edge Distance, End Distance, and Spacing for Lag Screws Loaded Laterally

Placement requirements for laterally loaded lag screws and lag screws under combined lateral and withdrawal loads are identical to those for bolts with the same unthreaded shank diameter.

In the 1997 edition the provision that placement requirements of laterally loaded lag screws meet those for bolts with the same diameter as the unthreaded shank diameter has been extended to specifically include lag screws under combined lateral and withdrawal loads.

9.4.2-Edge Distance, End Distance, and Spacing for Lag Screws Loaded in Withdrawal and Not Loaded Laterally

This new section has been added in the 1997 edition to assure that all lag screw connections are designed with adequate edge and end distances and spacing to avoid splitting of wood members. The specific placement requirements in Table 9.4.2 follow those for bolts loaded laterally parallel to grain and carrying full design load.

CHAPTER X. SPLIT RING AND SHEAR PLATE CONNECTORS

10.1 GENERAL

10.1.2-Quality of Split Ring and Shear Plate Connectors

10.1.2.3 A provision has been added in the 1997 edition requiring bolts used in split ring and shear plate connectors to have an unreduced nominal or shank diameter in accordance with ANSI/ASME Standard B18.2.1. This new requirement was introduced to prevent the use of undersize fasteners that did not provide full bearing with the connectors.

10.1.2.4 The same provision for use of unreduced nominal or shank diameter for bolts also has been applied to lag screws in the new edition. In this case, lag screws are required to have both unreduced shank diameter and threads in accordance with the ANSI/ASME Standard B18.2.1. As this Standard only recognizes cut thread lag screws (outside diameter of thread same as shank diameter), specific wording prohibiting the use of lag screws with rolled threads (root diameter equal to shank diameter) that was included in previous editions of the Specification has been deleted (see 1991 Commentary 10.1.2.4).

10.2 DESIGN VALUES

10.2.1-Tabulated Nominal Design Values

Additional species combinations have been added to the connector groups in Table 10A in the 1997 edition. Species are classified into the four load groups in this table on the basis of specific gravity (see 1991 Commentary 10.2.1 for specific gravity ranges of species in each connector group). The four species combinations added to Table 10A are:

B. Spruce-Pine-Fir (E of 2,000,000 psi and higher grades of MSR and MEL)
C. Engelmann Spruce-Lodgepole Pine (MSR 1650f and higher grades)
D. Engelmann Spruce-Lodgepole Pine (MSR 1500f and lower grades)
D. Engelmann Spruce-Lodgepole Pine

The first two species combinations represent MSR or MEL grades which have been assigned specific gravity values different from those for the overall species combination on the basis of specific gravity evaluation of the identified grades at an individual mill or where specific gravity values have been found to be consistently higher for the grades at all producing mills.

10.3 PLACEMENT OF SPLIT RING AND SHEAR PLATE CONNECTORS

10.3.7-Multiple Split Ring and Shear Plate Connectors

10.3.7.1 This section has been revised in the 1997 edition to correct language inadvertently introduced when the group action factor (C_g) was first made a part of the Specification in 1977. The revised wording makes clear that connector loads are subject to the group action factor only when two or more connectors are aligned in the direction of load on the same shear plane. The factor is not applicable to two or more connectors on two or more contact faces concentric to the same bolt axis (see 1991 Commentary 10.3.7.1).

CHAPTER XI. WOOD SCREWS

11.2 WITHDRAWAL DESIGN VALUES

11.2.1-Withdrawal from Side Grain

The equation used to calculate withdrawal design loads tabulated in the Specification is now given in the text as a convenience to users (see 1991 Commentary 11.2.1 for basis of this equation). It is entered with the tabulated specific gravity (oven-dry weight and volume) of the wood member and unthreaded shank diameter of the screw being used. The equation is not to be used for main member specific gravities outside the range of 0.31 to 0.73, nor to screw diameters outside the range of 0.138 inch (6g) to 0.372 inch (24g).

11.3 LATERAL DESIGN VALUES

11.3.1-Wood-to-Wood Connections

Species combinations for the Engelmann Spruce-Lodgepole Pine group and for Spruce-Pine-Fir (E of 2,000,000 psi and higher grades of MSR and MEL) have been added to the dowel bearing strength table (see Addendum Commentary 9.3.1). Also, the equation used to calculate dowel bearing strength is now given in the legend for the yield mode equations (see 1991 Commentary 11.3.1 for the basis of the bearing strength equation).

As with lag screws, the yield mode equations for wood screws in the 1997 edition require the use of D equal to the root diameter of the threaded portion of the screw when the threads extend into the shear plane of the connection. It is to be noted that the lateral design values for wood screws in Table 11.3A apply only to the case where the unthreaded shank of the screw extends beyond the shear plane.

Lateral design values for wood screws tabulated in Chapter 11 apply to cut thread wood screws which have a shank diameter equal to the outside diameter of the thread. The design load for connections made with rolled thread screws, which have a shank diameter equal to the root diameter, may be conservatively established using the 11.3.1 yield mode equations with D equal to the shank-root diameter. This application is appropriate because the Mode III_s equation assumes only yielding of the threaded portion of the screw in the main member and the Mode IV equation assumes only yielding of the shank portion in the side member and the threaded portion in the main member. These simplifications, which reduced the three possible yield mode conditions in each of Modes III and IV to one in each, were accomplished by assuming a constant ratio of yield moment of the threaded portion to yield moment of the shank portion of 0.75. This ratio is embedded in the Modes III_s and IV equations (see 1991 Commentary 9.3.1); and, therefore, when these equations are applied to rolled thread screws the resulting loads will be conservative.

11.3.2-Wood-to-Metal Connections

Design values in Table 11.3B apply only to connections where the unthreaded portion of the screw shank extends beyond the shear plane. Where this condition is not meet, design values must be based on fastener diameter, D, equal to the threaded portion of the screw.

11.3.3-Penetration Depth Factor, C_d

The 1997 edition clarifies that wood screw penetration length for lateral design values includes both the threaded and unthreaded length of the screw in the member. Prior to the introduction of the yield mode equations in the 1991 edition, just the threaded portion of the screw was considered in determining conformance to penetration requirements.

CHAPTER XII. NAILS AND SPIKES

12.2 WITHDRAWAL DESIGN VALUES

12.2.1-Withdrawal from Side Grain

The equation used to calculate nail and spike withdrawal design values tabulated in the Specification since the 1944 edition is now given in the text for the convenience of users (see 1991 Commentary 12.2.1). It is entered with the tabulated specific gravity (oven-dry weight and volume) of the wood member and diameter of the nail being used. Where the equation is used in lieu of Table 12.2A, it is not to be used to establish design values for member specific gravity and nail diameters outside the ranges provided in Table 12.2A.

12.3 LATERAL DESIGN VALUES

12.3.1-Wood-to-Wood Connections

The fastener bending yield strengths, F_{yb}, used to establish the nail and spike design values given in Tables 12.3A-H of the Specification are specified in the footnotes of each table. Other fastener bending yield strengths may be used in the yield mode equations to determine design loads for nail and spike connections. Bending yield strength of nails and spikes may be determined in accordance with ASTM F1575-95 (see Appendix I of the Specification).

The application of the yield mode equations to toe-nailed connections is clarified in the 1997 edition by indicating the use of the vertically projected length of the fastener in the side member which is equal to 1/3 the fastener length in place of the thickness of the side member in the Mode I_s and Mode III_s equations. This equivalent side-member thickness is based on toe-nail connection requirements (Figure 12A in the Specification). Similarly, the length of penetration in the main member equals the vertically projected length of the fastener in the main member (See Equation C12.3-3 and Figure C12.3-1) .

As with the other connector types, two additional species combinations have been added to the dowel bearing strength table for nails and spikes (see Addendum commentary 9.3.1) and the dowel bearing strength equation has been added to the legend for the nail and spike yield mode equations (see 1991 Commentary 12.3.1 for basis).

12.3.3-Double Shear Wood-to-Wood Connections

The 1991 edition (revised 1992) of the Specification provided double shear wood-to-wood connections made with main member thickness greater than 6D, and with 12D or smaller nails extending at least three diameters beyond side members 3/8" or thicker and which are clinched, to be assigned a design value 100 percent larger than the applicable single shear design value (see 1991 Commentary 12.3.3). The basis for this provision was early research involving single and double shear connections made with 3/8" and ½" plywood and 8D, 10D and 12D common nails with and without clinching (15).

In the 1997 edition, the increase in applicable single shear design values for double shear connections made with 12d or smaller nails extending 3D beyond 3/8" or thicker slide plates and clinched has been reduced to 75 percent. This more conservative assignment was considered appropriate because a few test combinations involving 3/8" plywood side members with nails clinched perpendicular to the applied load had increases over matching single shear connections of less than 100 percent; and because the provision is applied to all sheathing types and to box as well as common nails.

12.3.7-Toe-Nail Factor, C_{tn}

Clarification of penetration lengths used for establishing lateral connection values is provided in 12.3.1 of this Addendum.

12.3.8-Combined Lateral and Withdrawal Loads

In the 1986 and earlier editions, lag screws, wood screws and nails or spikes subject to combined lateral and withdrawal loads were analyzed separately for the resistance to each load. In the 1991 edition, an interaction equation for lag and wood screws subject to combined loading was introduced. This equation, similar to the form of the bearing angle to grain equation in 3.10.3, was based on lag screw tests which showed there was an interaction of the withdrawal and lateral load components for certain joint configurations at the design load level (see 1991 Commentary 9.3.5).

Although design loads for nailed connections are substantially lower than those for lag screws, a design equation (Equation 12.3-6) for nails and spikes subject to combined lateral and withdrawal loads has been introduced in the 1997 edition to put the design of such connections on a comparable analytical level with that of lag and wood screws. 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.

Early research provides some information on the effect of combined lateral and withdrawal loading on nailed connections (6). This research 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°, 90° and six intermediate directions were investigated. Two tests were conducted at each load angle. The interaction equation found to best describe ultimate load results for each species and penetration depth was of the form

$$Z'_\alpha = \frac{[(1 + K\sin 2\alpha)(W'pZ')]}{[(W'p)\cos\alpha + (Z')\sin\alpha]} \qquad \text{(CA12.3-1)}$$

where:

- Z'_α = maximum load
- $W'p$ = maximum load at 90° (withdrawal load perpendicular to grain per inch of penetration in the main member times the penetration depth)
- Z' = maximum load at 0° (lateral load)
- α = angle between wood surface and direction of applied load, and
- K = correction factor based on least squares analysis of test data for each species-penetration group

When K=0, Equation CA12.3-1 reduces to Equation 12.3-6 or CA12.3-2 as follows:

$$W/(W'p) + Z/Z' \leq 1 \qquad \text{(CA12.3-2)}$$

where W is the connection withdrawal force and Z is the connection lateral force. In equation 12.3-6 and CA12.3-2, Z'_α, $W'p$ and Z' are associated with allowable values.

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.

A comparison of Equation 12.3-6 with the combined loading equation CA12.3-3 used with lag screws and wood screws

$$Z'_\alpha = (W'p)Z'/[(W'p)\cos^2\alpha + (Z')\sin^2\alpha] \qquad \text{(CA12.3-3)}$$

is shown in Figure C12.3.8 along with Equation CA12.3-1 using an average K of 0.535 and average Z' and $W'p$ values of 231 lbs. and 118 lbs., respectively. Figure C12.3.8 shows that Equation 12.3-6 is a conservative characterization of the average test data for 8d nails with 6d-14d penetration and, also, relative to the screw interaction Equation CA12.3-3.

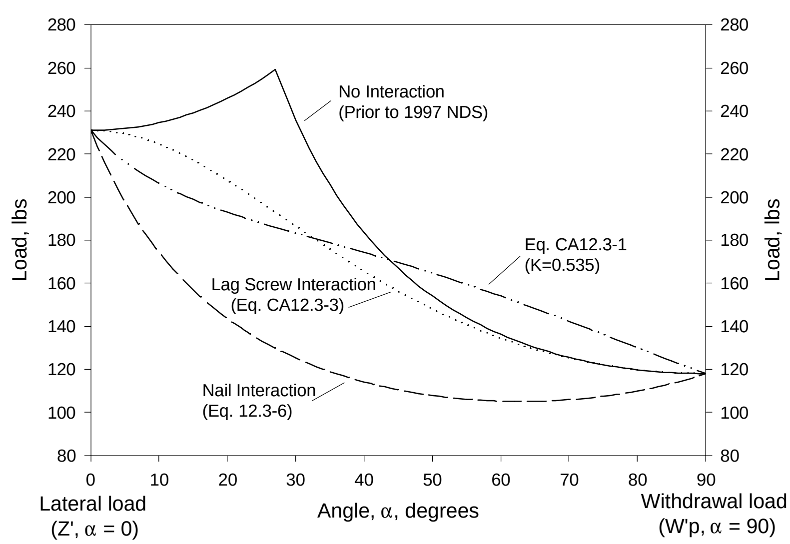

Figure C12.3.8 Combined Lateral and Withdrawal, W'p/Z'=0.51

CHAPTER XIII. TIMBER RIVETS

13.1 GENERAL

Provisions for designing connections made with timber rivets are new to the 1997 edition. Part XIII in previous editions dealt with metal connector plates, which are now covered under Miscellaneous Fasteners in Part XIV.

Timber rivets, also known as Glulam rivets, were originally developed in Canada more than 35 years ago to connect pre-drilled steel plates to glued-laminated timber (5). 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 Figure 13A and Tables 13.2.1 and 13.2.2).

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

13.1.1-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 (Appendix M) and vary only as to 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 Table 13.2.3). 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.

13.1.2-Fabrication and Assembly

13.1.2.1 Fabrication requirements for timber rivets are to be especially noted. 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 (5). 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 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.

13.1.2.2 The limit on maximum penetration of rivets of 70% of wood member thickness is considered a good practice recommendation to prevent through splitting of the piece.

13.1.2.3 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 13.1.2.2) and the rivets on both sides are required to be spaced (see 13.3.1) 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 13A) determined as the distances between adjacent rivets (one from each side but assumed on one side) at their points. Under these provisions, Equations 13.2-1 and 13.2-2 and all Tables 13.2.1 and 13.2.2 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, Tables 13.2.1 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.

Although not based on specific research, this procedure for determining the capacity of plates on two sides with rivets overlapping is logically inferred from the derivation of the design methodology and supporting data for single plate connections.

13.2 DESIGN VALUES

13.2.1-Parallel to Grain Loading

Design equations for timber rivets are based on Canadian research (11,10,12,3,13,) 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 (11). As load is applied to the connection, end rivets carry a larger portion of the load than rivets in the center but, 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 (11). 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 (11). Thus timber rivet design loads are based on the lower of the maximum rivet bending load and the maximum load based on wood strength.

The design rivet capacity for one plate and associated rivets where the load acts perpendicular to the axis of the rivets (lateral loading) is:

$$P_r = 280p^{0.32}n_R n_C \qquad \text{(CA13.2-1)}$$

where p is the actual penetration of the rivet in the wood member and n_R and n_C are the number of rows of rivets parallel to the direction of load and the number rivets per row. Penetration, p, is equal to the actual rivet length minus the thickness of the plate being used minus the protruding portion of the rivet head, assumed to be 1/8". The constant and exponent in Equation CA13.2-1 are based on tests of single rivets in Douglas-fir at penetrations of 1, 2 and 3 inches (10). The rivet capacity obtained from the equation represents average ultimate test values reduced by a factor of 3.36, the same factor used for test values limited by wood capacity and represents a 1.6 reduction for variability and 2.1 factor for duration of loading and factor of safety (11). Equation CA13.2-1 also includes an additional adjustment of 0.88 to account for specifying use of rivet of lower hardness and associated lower ultimate tensile and yield strength than the rivet used in the original research (12,5). The change in rivet specification was made to avoid the possibility of hydrogen embrittlement occurring in service conditions involving high temperatures and high humidities (12).

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 (Tables 13.2.1A-F). The loads in these tables are the lesser of the allowable wood tension loads or the allowable wood shear loads 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 glued laminated members (11).

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 (11). 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 Tables 13.2.1A-F 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 to give connections whose ultimate load was either a result of rivet bending or wood shear failure (11). For determination of allowable connection load limited by normal stress, this tension ultimate was reduced to 1600 psi to account for variability (1.6) and duration of load 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 (11).

Rather than use shear stress values based on the ASTM D143 block shear specimen, the allowable 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 (11). 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 duration of load and factor of safety) gives a reference unit volume allowable shear strength 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" (11).

It is to be noted that calculated P_r values and P_w values tabulated in Tables 13.2.1A-F 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 Table 13.2.3 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 total allowable load on the connection.

Because of the species test results and property values used to develop the rivet bending and wood capacity equations, use of design values based on the provisions of 13.2.2 should be limited to Douglas fir-Larch and southern pine glued laminated timber.

13.2.2-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 (see Commentary Addendum 13.2.1). However, in the perpendicular case, 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 (11).

The design rivet capacity for a connection with one plate and associated rivets when the load acts perpendicular to the axis of the rivets and perpendicular to the grain of the wood member is

$$Q_r = 160p^{0.32}n_R n_C \qquad \text{(CA13.2-2)}$$

where p, n_R and n_C are as defined in Equation CA13.2-1.

This equation is the same as that for the parallel to grain loading case (CA13.2-1) except for the value of the constants, 160 compared to 280. 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 glued laminated test specimens loaded perpendicular to grain and parallel to grain (10,13).

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 (11). 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 (3). This nonuniform distribution is accounted for in the basic design equations.

Based on tests that showed tension perpendicular to grain strength decreases with increase in cross-sectional area and/or length, a Weibull brittle fracture model was used to establish an allowable wood stress for checking wood capacity in rivet connections loaded perpendicular to grain. Using results from tests of blocks cut from Douglas fir glued laminated beams and ranging from 16 to 3600 in.^{3} in volume, a tensile perpendicular to grain strength for unit volume under uniform stress at a 95% survival probability of 267 psi was established (3). Reducing this value by a factor of 2.1 for duration of load and factor of safety gives an allowable basic unit volume strength 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 (13.2-3) is given in the Specification enabling such capacity to be calculated for any rivet penetration and plate thickness using loads and factors from

Tables 13.2.2A-B that account for the effects of a range of rivet configurations, spacings and unloaded edge distances. The unit load values given in Table 13.2.2A 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. It is to be noted that Equations (13.2-2) and (13.2-3) in the Specification provide design loads for connections with one side plate. Load 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 design values based on the provisions of 13.2.2 should be limited to Douglas Fir-Larch and southern pine glued laminated timber.

13.2.3-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 (11,13). Use of thinner plates reduces the amount of fixity of the rivet head which in turn reduces rivet bending capacity.

Design loads determined in accordance with sections 13.2.1 and 13.2.2 assume 1/4" side plates are used. For connections made with 3/16" and 1/8" plates, calculated design loads based on rivet capacity (P_r and Q_r) are adjusted by the side plate factors of 0.90 and 0.80 given in Table 13.2.3. These factors have been verified by unpublished Canadian research.

13.2.4-Load at Angle to Grain

The equation for calculating allowable design values for timber rivet connections loaded at angles other than 0° and 90° to the grain is the same form as the bearing angle to grain equation (see Appendix J) and that used for dowel bearing strength (Equation 8.2-7) and for split ring and shear plate connectors (Equation 10.2-1).

13.2.5-Timber Rivets in End Grain

The 50 percent reduction for timber rivets used in end grain is based on Canadian design practice (5). It can be compared with the end grain adjustment factor of 0.67 for nails and spikes (see 12.3.5).

13.2.6-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.

13.3 PLACEMENT OF RIVETS

13.3.1-Spacing Between Rivets
13.3.2-End and Edge Distance

Effects of rivet spacing and edge and end distances have been evaluated using the basic rivet design equations (11). 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 limit allowable load.

Minimum spacings and minimum end and edge distance requirements given in 13.3 and Table 13.3.2 minimize the occurrence of early wood failure in favor of more ductile rivet yielding. They are good practice recommendations based on Canadian design standards (5).

CHAPTER IV. MISCELLANEOUS FASTENERS

14.3 METAL CONNECTOR PLATES

Provisions relating to the design of metal connector plates were previously included in the Specification as a separate Part XIII. In the 1997 edition, this connector type is now included under Miscellaneous Fasteners and specific design provisions have been eliminated. Wood connections involving this type of fastener are to be designed in accordance with ANSI/TPI 1-1995.

CHAPTER XV. SPECIAL LOADING CONDITIONS

15.2 SPACED COLUMNS

15.2.2-Spacer and End Block Provisions

15.2.2.4 With regard to the requirement that the thickness of spacer and end blocks not be less than that of individual members of the spaced column, it should be noted that blocks thicker than a side member do not appreciably increase load capacity.

15.2.3-Column Stability Factor, C_p

15.2.3.1 The effective length factors given in Appendix G are now identified as one method for establishing effective column length when end-fixity conditions are known. This is to clarify that other methods that can be supported by engineering mechanics principles may be used.

15.2.3.3 For user convenience, the 1997 edition includes the equation for calculating the Euler buckling coefficient for columns, K_{cE}, based on the coefficient of variation in E, in the legend for calculating the column stability factor (see Addendum Commentary 3.7.1.5). Also a separate buckling coefficient for machine evaluated lumber (MEL) has been added.

15.3 BUILT-UP COLUMNS

15.3.1-General

The provisions of 15.3 apply only to built-up columns with 2 to 5 laminations which meet the limitations (a) through (e) given in 15.3.1. The 1997 edition clarifies that built-up columns that do not meet these criteria are to have individual laminations designed in accordance with the requirements for solid columns in 3.6.3 and 3.7.

15.3.2-Column Stability Factor, C_p

15.3.2.1 (See Addendum commentary 15.2.3.1)

15.3.2.2 This section has been revised in the 1997 edition to clarify the procedure that is to be used to determine the column stability factor (C_p) for calculating allowable compression load and to be consistent with new definitions of the column stability coefficient for built-up columns, K_f. Previously, the provision called for use of the larger of the fixity adjusted ℓ_{e1}/d_1 or ℓ_{e2}/d_2 ratios in the equation for calculating C_p (Equation 15.3-1). The provision now requires a separate calculation of C_p with each fixity adjusted ℓ_e/d ratio and then use of the smaller factor to determine the allowable compression design value for the column. Although the provision indicates that effective column length is to be determined using the buckling length coefficients (K_e) from Appendix G, it is to be understood from 15.3.2.1 that other methods consistent with the principles of engineering mechanics may be used to establish such lengths.

15.3.2.4 The general equation for calculating the Euler buckling coefficient (K_{cE}) based on the coefficient of variation in E, and a separate coefficient for machine evaluated lumber (MEL) have been added to the legend for the C_p equation (see Addendum Commentary 3.7.1.5).

The column stability coefficients, K_f, defined in the legend for the column stability factor (C_p) equation for built-up columns (Equation 15.3-1) have been redefined in the 1997 edition to make them consistent with design provisions for solid columns and with the supporting research on which built-up column design procedures are based. As discussed in 1991 Commentary 15.3.2, when the controlling slenderness ratio in accordance with 3.7.1.3 is the strong axis of the individual laminations (ℓ_{e1}/d_1), then K_f is equal to 1.0. The K_f factors of 0.65 and 0.75 for nailed and bolted built-up columns, respectively, apply to slenderness ratios based on the weak axis of the individual laminations (ℓ_{e2}/d_2 where d_2 is the sum of the thicknesses of the individual laminations). Buckling about the weak axis of the laminations is related to the amount of slip and load transfer that occur at the fasteners between the laminations. The K_f coefficients of 0.65 and 0.75 account for this interlayer slip between laminations.

The foregoing considerations are addressed in the new edition by limiting the application of the K_f coefficients of 0.65 and 0.70 for nailed and bolted built-up columns only to the calculation of C_p where F_{cE} is based on the weak axis slenderness ratio (ℓ_{e2}/d_2). A new K_f of 1.0 for both nailed and bolted columns is established for calculating C_p values when F_{cE} is based on the strong axis slenderness ratio (ℓ_{e1}/d_1). As provided in 15.3.2.2, the allowable compression design value for the column is based on the smaller of the two C_p values.

15.4 WOOD COLUMNS WITH SIDE LOADS AND ECCENTRICITY

15.4.1-General Equations

As a convenience to users, equations for calculating Euler buckling coefficients for columns (K_{cE}) and for beams (K_{bE}) for any material based on coefficient of variation in E (COV_E) have been added to the legend for the general equations (see Addendum Commentary 3.7.1.5 and 3.3.3.8). Also in the new edition, separate buckling coefficients have been added for machine evaluated lumber (MEL) based on a COV_E of 0.15.

The eccentric load design provisions of 15.4.1 are not generally applied to columns supporting beam loads where the end of the beam bears on the entire cross section of the column. It is standard practice to consider such loads to be concentrically applied to the supporting column. This practice reflects the fact that the end fixity provided by the end of the column (16) is ignored when the usual pinned end condition is assumed in column design. In applications where the end of the beam does not bear on the full cross section of the supporting column, or in special critical loading cases, use of the eccentric column loading provisions of 15.4.1 may be considered appropriate by the designer.

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