Only one primary building material comes from a renewable resource; cleans the air and water, providing habitat, scenic beauty and recreation as it grows; utilizes nearly 100% of its resource for products; is the lowest of all in energy requirements for its manufacturing; creates fewer air and water emissions than any of its alternatives; and is totally reusable, recyclable and 100% biodegradable: wood. And it has been increasing in US net reserves since 1952, with growth exceeding harvest in the US by more than 30%.

Wood Works™, the argument is growing every day.

American Wood Council
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 of wood products.

The American Wood Council (AWC) is the wood products division of the American Forest & Paper Association (AF&PA). AF&PA is the national trade association of the forest, paper, and wood products industry, representing member companies engaged in growing, harvesting, and processing wood and wood fiber, manufacturing pulp, paper, and paperboard products from both virgin and recycled fiber, and producing engineered and traditional wood products. AF&PA represents a segment of industry which accounts for over 8% of the total U.S. manufacturing output.
This Commentary and its Addendum were prepared for the American Forest & Paper Association by Edward G. King, Jr., Wood Construction Technologies, Inc., McLean, Virginia. He was Director and Assistant Vice President of Technical Programs for the Association from 1970 to 1987. Technical information for Part V of the Commentary was contributed by Thomas E. Brassell, P.E., technical advisor and consultant to the glued laminated timber industry since 1960. Design examples were prepared by Clifford G. King, Wood Construction Technologies, Inc.
COMMENTARY ON THE
NATIONAL DESIGN SPECIFICATION
FOR Wood Construction

FOREWORD

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 1991 edition is the eleventh edition and sixteenth revision of the publication. The history of the development of this national standard of practice is given in a subsequent section.

For many years, a commentary on the Specification has been requested by architects, engineers, product manufacturers, researchers and other users. As the demand for more efficient and more reliable wood structures has grown, calls for background information and interpretative discussion of the provisions of the Specification have increased commensurately. The Commentary presented herein is intended to respond to these user needs.

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. Interpretative discussion of how a provision should be applied is given where users have suggested the intent of a requirement is ambiguous. One or more examples of the application of a specific provision may be given to illustrate the scope of conditions covered by the requirement. The examples are not meant to be inclusive of all design considerations for a given application, but are intended to illustrate the provisions being discussed in that particular section of the Commentary.

Only those provisions of the Specification whose application by the user would benefit from elaboration of background and interpretation are addressed in the Commentary. Provisions of a self-explanatory nature are omitted.

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

American Forest & Paper Association
FOREWORD to the Addendum

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
# Commentary on the National Design Specification for Wood Construction

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HISTORY OF THE DEVELOPMENT OF A NATIONAL STANDARD OF PRACTICE FOR WOOD DESIGN

Non-Uniform Local Practices

In the early part of the century, structural design with wood was based on general engineering principles using working stresses or design values published in engineering handbooks and in local building codes. These design values were often not in agreement, even for the same species of wood. Further, in most cases, the assigned values were not related to lumber grade or quality level (193).

First Uniform Working Stresses

To meet the growing need for a national standard of practice, the Forest Products Laboratory, an agency of the Forest Service, U.S. Department of Agriculture, in cooperation with the National Lumber Manufacturers Association (now the American Forest & Paper Association), prepared a guide for grading and determining working stresses for structural grades of timber (60). This guide was issued in 1933 and subsequently published as U.S. Department of Agriculture Miscellaneous Publication 185, "Guide to the grading of structural timbers and the determination of working stresses" (207). It provided basic working stresses for clear, straight-grained material of the important commercial species and presented strength ratios for adjusting basic strength values for each species for the effect of any size and location of knots or other natural characteristics permitted in a structural grade. The basic stresses and the strength ratios established in Miscellaneous Publication 185 were based on extensive test data for small, clear specimens and structural timbers obtained from over twenty years of testing and evaluation at the Forest Products Laboratory. The procedures of Miscellaneous Publication 185 were employed by the lumber industry to establish working stresses for the commercially important species and grades of lumber manufactured in accordance with American Lumber Standards for Softwood Lumber, Simplified Practice Recommendation R16-29 and R16-39, promulgated by the U.S. Department of Commerce (60, 188, 193).

Wood Structural Design Data

In 1934, the National Lumber Manufacturers Association (now the American Forest & Paper Association) assembled the information given in Miscellaneous Publication 185 and working stresses derived therefrom together with engineering design equations and other technical information on wood in the publication "Wood Structural Design Data" (WSDD) (127). Included as supplements to the publication was design information for timber fastenings. The information contained in this publication was based largely on information developed over the years at the Forest Products Laboratory and subsequently published under one cover as the first edition of the "Wood Handbook" in 1935 (57). A major component of the WSDD was extensive span and load tables for various sizes of timber beams and columns. A second edition of the WSDD was issued in 1939 with a revised second edition being issued in 1941 (127).
First Edition of National Design Specification

With the initiation of World War II, the need for a comprehensive national design standard for timber structures, including wood connections, became more urgent. The Technical Advisory Committee of the National Lumber Manufacturers Association undertook a more than three year effort to develop the necessary specification in close consultation with the Forest Products Laboratory (128). The first result of this initiative was the issuance of "National Emergency Specification for the Design, Fabrication and Erection of Stress Grade Lumber and Its Fastenings in Buildings", Directive 29, by the Conservation Division, War Production Board in August 1943 (194). The Directive was prescribed for all federal departments and agencies involved in war construction. In 1944, the copyrighted first edition of the "National Design Specification for Stress-Grade Lumber and Its Fastenings" was published by the National Lumber Manufacturers Association (128). The Specification, which had the same content as Directive 29, included allowable unit stresses for stress graded lumber, design formulas, and design loads and provisions for timber connector, bolted, lag screw, nail and wood screw joints. Also included were guidelines for the design of glued laminated structural members.


The scope of the Specification has remained essentially unchanged since the first edition was issued in 1944. Information on machine stress rated lumber and timber piles was introduced into the 1971 edition and expanded in the 1973 edition. The name of the Specification was changed to the "National Design Specification for Wood Construction" in the 1977 edition (123).

Use of ASTM Standards

As indicated by the early history of structural design with wood, wood strength properties or design values are an essential component of a national standard of practice. The material design values incorporated in the National Design Specification are those developed by the rules writing agencies or wood product interest involved. Whereas the authoritative source of the procedures used by these organizations to establish design values prior to World War II was the U.S. Department of Agriculture's Forest Products Laboratory, in the years following the war the standards of the American Society for Testing and Materials (ASTM) began to be utilized to an increasing extent by the manufacturers and consumers of structural lumber to achieve uniformity in and acceptance of the technical aspects of establishing structural grades of lumber and related design values. Procedures and data developed by the Forest Products Laboratory for this purpose were incorporated into ASTM standard methods. In 1960, the Forest Products Laboratory determined that it would no longer continue to recommend and publish basic stresses for wood, but rather would provide test data and related information to ASTM Committee D-7 on Wood to enable that body to make the final determination as to the appropriate stresses to assign to the commercially important species and species groups (63). Since that time, ASTM technical standards have served as the authoritative basis for the establishment of design values for lumber, glued laminated timber and timber piles by the various lumber grading rule agencies and product manufacturers.
Size and Grade Standards

National standardization of the sizes, grades and inspection of lumber began in 1924 with the issuance by the U.S. Department of Commerce of Simplified Practice Recommendation R16 for Softwood Lumber (188). Design values incorporated in 1970 and earlier editions of the National Design Specification were based on the application of Forest Products Laboratory procedures, and subsequently ASTM standards, to the grades defined in Simplified Practice Recommendation R16. R16 was superseded in 1970 by the U.S. Department of Commerce Voluntary Product Standard PS 20-70, American Softwood Lumber Standard. Product Standard PS 20, which was approved by a general concurrence of producers, distributors, users and manufacturing consumers of softwood lumber, provides for the uniform application of ASTM standards in the development of design values published in lumber grading rules (190). The design values for lumber contained in American Lumber Standards Committee approved grading rules are the lumber design values tabulated in the 1971 and subsequent editions of the National Design Specification.

Review and Improvement Process

Since the first edition was published in 1944, the design formulas and other provisions contained in the body of the National Design Specification, including those related to the design of mechanical connections, have been subject to continued review and evaluation by the National Forest Products Association's Technical Advisory Committee. Appropriate revisions have been made in the Specification to reflect new information from laboratory tests, new developments in design, and experience with wood construction in service. Research and engineering data developed over more than eight decades by the Forest Products Laboratory continue to be the principal basis for the provisions in the Specification. However, to an increasing degree in recent years, the results of research conducted at universities and other private and public laboratories, in Canada and other countries as well as the U.S., have been utilized to improve the Specification as a national standard of practice for wood construction.

The 1982, 1986 and 1991 editions of the Specification contain numerous revisions recommended by a Special Advisory Committee composed of consulting design engineers; representatives of user groups, research laboratories and educational institutions; and wood and engineering specialists representing manufacturers of various wood products. The reviews conducted and the changes proposed by this balanced Committee have helped to assure the technical soundness and broad applicability of the Specification.

PART I: GENERAL REQUIREMENTS FOR STRUCTURAL DESIGN

1.1-SCOPE

1.1.1-Practice Defined

1.1.1.1 The Specification defines a national standard of practice for the structural design of lumber, glued laminated timber, piles and wood mechanical connections based on working stress or deterministic design principals. Alternative design approaches based on reliability-based concepts may give different results than those obtained from the Specification. The choice of which design methodology to use should be based on the availability of code approved alternatives, knowledge of the bases of the alternative approaches, and experience with the performance of wood structures designed for different occupancies or other specific purposes.

1.1.1.2 The provisions of the Specification apply to lumber, glued laminated timber and timber piles that are subject to loads transferred from attached panel materials. The structural design of the panel products themselves shall be based on recognized design specifications for the materials involved, and/or in accordance with accepted engineering practice.

Where the structural performance of assemblies of panel products and lumber, glued laminated timber or piles is dependent upon the capacity of the connections between the materials, such as in shear diaphragms, the design provisions for mechanical connections in the Specification may be used for such assemblies when the panels are made of solid wood materials, or when such application is accepted engineering practice, or when experience has demonstrated such application provides for satisfactory performance in service.

1.1.1.4 The data and engineering judgments on which the Specification are founded are those related to working stress design. The general approaches contained in the provisions of the Specification have been successfully employed to design wood structures since 1944.

Components, assemblies or structures may be qualified for use employing criteria other than the working stress design requirements of the Specification, or, if available, the provisions of a reliability-based design standard. Such other criteria may involve use of full-scale test results, use of verified computer design models, application of generally recognized theoretical principles, and broad in-service experience. Examples are testing to establish adequate chord-web connections at the supports of top-hung parallel chord trusses, computer models for determining floor system performance, and computerized design packages for determining the load-carrying capacities of glued laminated timber beams.

An example of the use of a combination of criteria is the design procedure for metal-plate connected roof trusses. This procedure utilizes beam-column equations, sheathing contribution factors and duration of load adjustments from the Specification in combination with theoretical principles, computer modeling and test results to establish appropriate truss spans.

The other criteria for demonstrating satisfactory performance in use may be proprietary to a specific organization, 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 authorities in the jurisdiction in which the product is used or the structure is located.

1.1.2-Competent Supervision

The competent supervision requirement is particularly relevant to joint details and placement of fastenings. Design values for connections are dependent upon use of accurate end, edge, and spacing dimensions. Special attention also should be given to end details of columns and beam-columns to assure that design assumptions related to load eccentricity are meet in construction.

1.2-GENERAL REQUIREMENTS

1.2.2-Framing and Bracing

Adequate bracing and anchorage of trusses and truss members to assure appropriate resistance to lateral loads is particularly important. Good practice recommendations (179) for installation between trusses of vertical sway (cross) bracing, continuous horizontal bottom chord struts and bottom chord cross bracing are given in Appendix A.10. These recommendations have been part of the Specification since 1944.

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.

1.4 DESIGN LOADS

1.4.1 Loading Assumptions

Accidental overload factors are not incorporated in any of the design procedures of the Specification. Should there be the possibility of overload on the structure, this should be directly taken into account in the loading assumptions.

Further, the design provisions in the Specification are not based on any quantified expectation that code specified design loads will generally exceed typical loads likely to be encountered in service. Where such differences in fact do occur, they are part of the successful experience record on which the Specification is based.

1.4.4 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 in ASCE 7-88 (formerly ANSI A58.1) Minimum Design Loads for Buildings and Other Structures (10), in model codes, and in state and local codes. 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 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 load combination factors that apply.

All modifications for load combinations are entirely separate from adjustments for duration of stress or load that are directly applicable to wood design values. Wood strength properties are related to the time period over which the induced stress is sustained: the shorter the duration, the greater the design stress applicable to the member.

Because the duration of stress or load adjustment previously applicable to wood design values for wind or earthquake loads for many years was 1.33, this adjustment was often confused with the $1/3$ modification factor for wind or earthquake loads that is permitted in some codes for all materials. This confusion should be minimized in the future as new research has established that the duration of maximum ANSI wind and earthquake design loads is much shorter than previously assumed, thereby substantiating the establishment of a 1.6 duration of load adjustment of wood product design values for these particular loads (see Commentary for Section 2.3.2). It should be emphasized that reduction of design loads to account for the probability of simultaneous occurrence of load components, and the adjustment of design loads to account for the effect of the duration of the induced stress or applied load are independent of each other and both may be employed in the design calculation.

1.5 SPECIFICATIONS AND PLANS

1.5.1 Design Values

The recommendation for expressing design value requirements on working drawings and plans in terms of normal duration of load is to facilitate design review and implementation. Normal loading is defined as that loading which will fully stress a member, either cumulatively or continuously, for a period of ten years during the life of the structure in which the member is used. Design values for normal load duration are those tabulated for all wood structural materials in design specifications, material references and grading rules. Stating design value requirements in terms of normal loading in plans and drawings provides for ready identification of qualifying species and grades of material.

Identifying in plans and specifications the moisture conditions to which the design values apply also facilitates procurement and use of the proper material for the job.

1.5.2 Sizes

The use of nominal dimensions in the distribution and sale of lumber products has been a source of confusion to some designers, particularly those unfamiliar with wood structural design practices. 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. Alternate bases are the standard nominal or standard net sizes established for each product in national product standards (190); or special sizes applicable to proprietary or made-to-order products.

1.6 NOTATION

The many new symbols added to the Specification in the 1991 edition are a result of the conversion of design requirements to a consistent and comprehensive equation format. All adjustment factors and coefficients that may influence a member or connection
design are identified by symbol to facilitate the designer's review and determination of their applicability.

The system of notation used in the Specification helps to identify the meaning of certain frequently used symbols. Factors, identified by the symbol "C", are those adjustments used to modify tabulated design values for conditions of use, geometry or stability. The subscripts "D", "F", "H", 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, respectively). There is no particular significance to the use of the same letter with different cases for different adjustment factors. Coefficients, identified by the symbol "K", are characteristic values which, in most cases, are used to determine specific values of "C". The symbols "F" and "F'" denote tabulated and allowable strength values, respectively; where the latter values represent tabulated values multiplied by all applicable C factors. The symbol "f" indicates the actual or induced stress caused by the applied loads. The subscripts "b", "t", "c", "v", and "cl" indicate bending, tension parallel to grain, compression parallel to grain, shear and compression perpendicular to grain stress, respectively.
PART II: DESIGN VALUES FOR STRUCTURAL MEMBERS

2.1-GENERAL

2.1.2-Responsibility of Designer to Adjust for Conditions of Use

The Specification identifies design value adjustment requirements for service conditions generally encountered in wood construction. However, this national standard of practice does not provide generic requirements that address all possible design applications or conditions of use. Such inclusive provisions would require the use of excessively conservative and economically prohibitive reduction factors.

Final responsibility for evaluation of the loading and exposure conditions the member or structure will be subjected to and determination of the design values that are appropriate to those conditions rests with the designer. Particular attention is required by the designer 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, that the structural members will be consistently exposed to water at elevated temperatures, and that the structural connections will be subjected to biaxial forces and moments. 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 and design values.

2.2-DESIGN VALUES

Design values tabulated in the Specification and its Supplement are based on normal load duration, dry conditions of service, and other normal environmental and material conditions considered represented by the product and fastener strength tests used to establish the design values.

2.3-ADJUSTMENT OF DESIGN VALUES

2.3.1-Applicability of Adjustment Factors

The Specification requires adjustment of tabulated design values for specific conditions of use, geometry and stability. Such modifications are made through application of adjustment or C factors. For structural sawn lumber and glued laminated timber, adjustment factors defined in the Specification text or footnotes to Table 2.3.1 and applied to one or more of the design values are: load duration, wet service, temperature, beam stability, size, volume, flat use, repetitive member, form, curvature, shear stress, buckling stiffness, column stability and bearing area. The uses of adjustment factors with design values for round timber piles and connections are treated separately in the Specification.

The adjustment or C factors are cumulative except where specifically indicated otherwise. Example C2.3-1 demonstrates this provision.

Example C2.3-1

A 2x6 framing member having a tabulated \( F_b \) of 1200 psi is to be used as a stud spaced 16 in. on-center in a wood foundation. Special site conditions indicate that moisture content in service will exceed 19 percent for an extended period of time. The stud will resist 6 feet of soil load. The allowable \( F_b' \) for the member is:

\[
C_D = 0.90 \text{ (2.3.2 and Appendix B)} \\
C_M = 0.85 \text{ (2.3.3 and Table 4A factor)} \\
C_t = 1.00 \text{ (2.3.4)} \\
C_L = 1.00 \text{ (2.3.7 and 3.3.3.2)} \\
C_F = 1.30 \text{ (4.3.2 and Table 4A factor)} \\
C_{fu} = 1.00 \text{ (4.3.3 and Table 4A factor)} \\
C_r = 1.15 \text{ (4.3.4 and Table 4A factor)} \\
C_f = 1.00 \text{ (2.3.8 rectangular section)} \\
C_V = \text{ not applicable (Table 2.3.1 Footnote 3)} \\
F_b' = 1200 \times (0.90)(0.85)(1.00)(1.00)(1.30)(1.00)(1.15)(1.00) \\
\text{ = } 1372 \text{ psi}
\]

In addition to the adjustment factors given in Table 2.3.1 of the Specification, other adjustments of tabulated design values for special conditions or requirements of use may be required. Such additional adjustments may include modifications for creep effects, variability in modulus of elasticity, and fire retardant treatment. Guidelines for accounting for some of these special conditions are given in the Specification.

2.3.2-Load Duration Factor, \( C_D \)

Historical Development

The characteristic behavior of wood structural members to carry a greater maximum load for short durations than for long load durations was reported as early as 1841 (177). Reductions in strength of 30 and 40 percent after a year or more under load were cited in early investigations. By the beginning of the century
it was generally believed that long-term loads should not exceed about 75 percent of the proportional limit or elastic limit of the member as determined from short term tests (176). However, because proportional limit as well as ultimate strength was found to be affected by speed of testing (175), and because of the subjective element involved in determining proportional limit, ultimate strength came to be considered the most dependable base on which to establish load duration by ultimate strength came to be considered the most element involved in determining proportional limit, it was generally believed that long-term loads should not exceed about 75 percent of the proportional limit or elastic limit of the member as determined from short term tests (176). However, because proportional limit as well as ultimate strength was found to be affected by speed of testing (175), and because of the subjective element involved in determining proportional limit, ultimate strength came to be considered the most dependable base on which to establish load duration adjustments (137).

Permanent Loading Factor. By the 1930's, a factor of 9/16 the short term strength had become established as the safe working stress level for long-term loads (57). Such loads were considered to be those associated with dead and design live loads. However, an increase of 50 percent over this permanent load level was considered appropriate when designing for short load durations, such as wind loads, in conjunction with dead and live loads. These recommendations were based on the results of load duration tests on small, clear specimens in bending and compression parallel to grain, supplemented by field experience with structural members. As the short term tension strength of clear wood is about 50 percent greater than short term bending strength, use of the same working stress for tension as for bending, including the same load duration adjustments, was considered justified.

Wartime Provisions. During World War II, as part of the wartime effort to conserve scarce materials, the War Production Board authorized a twenty percent increase in design values for extreme fiber in bending, tension and horizontal shear and a ten percent increase in compression properties. A twenty percent increase in allowable design values for long columns also was authorized. These increases were based on a reevaluation of available clear wood strength data (193) and load duration considerations. In conjunction with these increases in permanent load design values, increases of 15 percent for snow, 50 percent for wind and earthquake and 100 percent for impact also were authorized provided member sizes were sufficient to carry each combination of longer term loads when the load duration adjustment applicable to that combination was considered (194). For example, wind plus live plus dead loading could be checked against a design value of 1.5 times the permanent design value provided that the live plus dead load did not exceed the permanent design value. The load duration adjustments authorized in 1943 also were applied to fastenings.

Normal Loading Established. The permanent load design values authorized by the War Production Board, including the adjustments for shorter load durations, were published in the first edition of the National Design Specification for Stress-Grade Lumber and Its Fastenings in 1944. Following World War II, one-half (10 percent) of the wartime increase for bending-tension and shear design values was retained as it was a result of reevaluation of available strength data (60,193). The remaining one-half of the war-time increase for these properties and the ten percent increase for compression parallel to grain was removed by reducing the permanent load design values for these properties by 10 percent (National Design Specification 1948). However, a new loading condition, designated "normal loading", was established as the load duration basis for wood design values. This new loading condition, applicable to live loads other than snow, wind, earthquake and impact, and equal to 1/0.9 of permanent load design values, was defined as that in which a member is fully stressed to its maximum allowable design value cumulatively or continuously for a period of ten years or less during the life of the structure in which the member is used (National Design Specification 1951). Additionally, application of 90 percent of the full maximum "normal" design load continuously during the life of the structure is a normal condition. The new normal loading base was applied to all design values for wood members except modulus of elasticity, and to design values for fastenings when the capacity of the connection was limited by the strength of the wood. One-half of the twenty percent war-time increase in allowable design values for long columns also was retained by applying the normal load concept to this design value as well.

Short-term Load Adjustments. In conjunction with the introduction of the ten year or normal load base for tabulated design values in 1948, increases of normal design values of 1.15 for two months duration as for snow, 1.25 for 7 day duration as for construction, 1.33 for one day duration as for wind or earthquake and 2.00 for impact were established. The appropriateness and consistency of the new load duration adjustments for normal and other conditions of loading were substantiated by U.S. Forest Products Laboratory analysis (224). The relationship between strength and load duration established by this analysis is shown in Appendix B. The equation for the curve shown in Figure B1 is

\[ C_D = \frac{1.75192}{(DOL)^{0.04635}} + 0.29575 \]  \hspace{1cm} (C2.3-1)

where:
\[ C_D = \text{load duration factor} \]
\[ DOL = \text{duration of load, seconds} \]
New Design Loads. The adjustments for the effect of load duration on strength established after World War II were used in wood structural design without change for the next 38 years. However, the adoption of new design snow and wind loads and related new analytical procedures by ASCE 7-88 (formerly ANSI A58.1) Minimum Design Loads for Buildings and Other Structures (10), impacted the design of many light frame wood structures which had a long record of satisfactory performance. Analysis of the data bases for the new loads and the probability methods used to establish them indicated the long standing assumptions that all snow loads could be categorized as 2 month loads and that peak wind design loads had a cumulative duration of 1 day were no longer appropriate.

New Provisions for Snow Loads. Snow load records showed that the duration of the ASCE 7-88 snow loads varied significantly from location to location and, in some high snow load areas, was much shorter than the traditional two months assumed. Beginning with the 1986 edition of the Specification, use of a larger load duration adjustment than 1.15 for new snow design loads is allowed when information is available on the actual duration of the load in the specific location being considered. Such adjustments are to be selected from the standard relationship characterizing the effect of load duration on strength in Appendix B (see Commentary Equation C2.3-1).

New Wind Load Adjustment. In 1987, an evaluation of the degree of conservatism in the traditional 1.33 or one day load duration adjustment for wind was initiated in response to proposed new design requirements for structures in high wind coastal areas. The proposed requirements, which called for comprehensive engineering design for ASCE 7-88 wind loads, would have substantially increased the sizes and/or numbers of structural members and connections in low rise wood frame buildings which had a long history of satisfactory performance in coastal areas when constructed in accordance with recognized good building practices. The evaluation took into account that the design wind loads given in ASCE 7-88 are based on a mean recurrence interval of once in 50 years, with the specified wind loads having a duration of from one to ten seconds, rather than the 1 day previously assumed for wood design. The standard relationship between strength and load duration (Appendix B) showed that the load duration factor associated with such short load periods was in excess of 1.85. As a result of this evaluation, a new wind load duration adjustment factor of 1.85 was established in a November 1987 revision of the Specification for engineering design of wood members based on ASCE 7-88 wind loads and related procedures.

Wind and Earthquake Adjustments in the 1991 Edition. Subsequent to the 1987 revision, studies of the duration of code specified maximum earthquake and live floor design loads also have shown that the durations of these loads are much shorter than the 1 day and 10 years periods, respectively, that traditionally have been assumed (39,121). These new findings were taken into account in conjunction with other new information and considerations in the development of load duration factors in the 1991 edition of the Specification. Other influential factors included the availability of new design values for visually graded structural lumber based on major test programs of full size, in-grade lumber in the United States and Canada; the advances made in quantification of the system contributions of framing and sheathing materials in the performance of light frame wood assemblies; and simplicity of load duration adjustments for wood strength properties.

Although new information showing the time floor live loads are at or above the code specified design level is less than 50 days (121), the conservative position of retaining a 10-year normal loading basis for such loads is continued in the 1991 edition. Further, until additional information is developed on the duration of the ASCE 7-88 snow loads, the recommendations for load duration adjustments of snow loads in the 1986 edition also are continued in the current edition.

For purposes of simplification and conservatism, load duration adjustments for ASCE 7-88 earthquake loads (longest on record less than 5 minutes, with most less than 1 minute duration of ground motion) (39) and wind loads are consolidated at 1.6. This adjustment corresponds to the general time of tests to failure used to establish the strength properties of structural wood products and is the factor used to reduce short time test results to a normal loading basis. Use of the 1.6 factor in conjunction with improved system design procedures and analytical techniques for light frame wood assemblies may be used to rationalize the performance of these assemblies under the highest wind loads. Application of the various load duration factors of the current edition are further discussed in Appendix B of the Specification.

Results of In-grade Lumber Tests. Load duration tests conducted on in-grade lumber in bending, tension and compression and reported in 1986 and 1988 indicate the standard load duration curve is a generally adequate representation of the effect of load duration...
on strength. In-grade bending tests on a number of species show most to be slightly less influenced by load duration than indicated by the standard curve while one species was found to be somewhat more affected (74, 75, 96). Similar species effects were observed earlier in load duration bending tests on clear wood specimens (224). Results from load duration tests of in-grade lumber in tension and compression on one species show a greater effect of time under load for these two properties than for in-grade lumber bending strength for the same species (96). However, the load duration effects for the direct stress properties were found to be comparable to that indicated by the standard strength-load duration curve.

The long record of satisfactory performance with wood structures designed using load duration adjustment factors, and the results of load duration tests on full-size members, substantiate the general applicability of the standard strength-load duration relationship (Appendix B). As more information is developed on the frequency of occurrence and the duration of maximum design loads specified in building regulations, use of different adjustment factors than those currently specified may be appropriate for individual load cases. Such changes would be associated with improved definition and recognition of the duration of the loads involved rather than a revision of the underlying relation between strength and time under load.

Damage Accumulation. Adjustments of wood design values for load duration are keyed to the cumulative period of time the design load is expected to be applied to the member over its service life. Such maximum loads are considered to represent the most extreme condition expected over a long period; for example, once in a 50 year interval (10). The peak loading on the member from year to year is much lower than this design load. For this reason, coupled with the exponential relationship that exists between the strength of wood and load duration, the accumulated effects of loadings less than the maximum load the member is designed for are considered negligible and noncritical. The satisfactory performance of all types of wood structures designed on such a basis for more than 40 years attests to the appropriateness of the methodology. Recent studies (52,121) of the damage accumulation in wood structural members using new information on the intensity, frequency and duration of applied loads and one of the new models characterizing the relationship between load level and time to failure (76) further substantiate the procedure. It is reported that "practically all damage occurs when the live load intensity is equal or nearly equal to the" code specified design load (121).

2.3.2.1 Load duration factors ($C_D$) are applicable to all tabulated design values for lumber and glued laminated timber except modulus of elasticity and compression perpendicular to grain. The 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. Increases in deflection or deformation are a separate consideration, independent of ultimate strength. However, prior to 1977, the Specification provided for load duration adjustments to allowable compression parallel to grain design values for all columns, including those with $l/d$ ratios greater than 11 (intermediate and long columns). Because allowable design values for the latter columns were a function of modulus of elasticity, some users began to interpret this application of load duration adjustments as indicating the factors should be applied to modulus of elasticity values. Although the use of load duration adjustments for intermediate and long column allowable design values had provided for satisfactory performance over many years, this provision was removed from the Specification in the 1977 edition to obtain technical consistency and avoid misinterpretations.

Compression perpendicular to grain design values were subject to adjustment for load duration when such values were based on proportional limit test values. When the basis for such design values was changed to a deformation limit in the 1982 edition, the application of the load duration factor to compression perpendicular design values was eliminated.

Table 2.3.2 Frequently Used Load Duration Factors

**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 under national standards of practice.

**Permanent Loads.** In addition to materials of construction dead loads, 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...
Two Month Loads. A 2 month load duration adjustment factor of 1.15 was used for all code specified snow loads prior to 1986. New maximum snow loads published in ASCE 7-88 (10) 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 7-88 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 7 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 1 day or 1.33 factor was used as the load duration adjustment for wind and earthquake loads. In the 1991 Specification, the load duration factor for these loads is based on a 10 minute load duration.

Ten Minutes. The 10-minute or 1.6 load duration factor is to be used with wind and earthquakes in the current Specification. The wind loads and procedures given in ASCE 7-88 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 1 day duration traditionally assigned. The 10-minute load duration factor is a conservative adjustment for 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 (Appendix B).

Pressure treatment of wood with preservative oxides or fire retardant chemicals to retentions of 2.0 pcf or more may reduce energy absorbing capacity as measured by work-to-maximum-load in bending as much as one-third in some cases. Application of the 2.0 load duration adjustment factor for impact was discontinued for oxide treated piles and other members intended for salt water exposure in 1977, and for wood pressure treated with fire retardant chemicals in 1982.

2.3.2.2 Design of structural members in terms of size and resistance is based on the critical combination of loads representing different durations. For fully laterally supported members, this critical combination may be determined by dividing the summation of design loads by the shortest load duration factor applicable to any load included in the combination. The combination with the highest quotient is the critical total load on which the member design is to be based. Example C2.3-2 demonstrates this provision.

Example C2.3-2
Consider a fully laterally supported bending member subject to a dead load of 20 plf, a roof construction live load of 60 plf and a wind load of 40 plf. The normalized load combinations for the member are:

- dead load: \((20)/0.9 = 22\)
- dead plus roof live (7-day): \((20 + 60)/1.25 = 64\)
- dead plus roof live plus wind: \((20 + 60 + 40)/1.6 = 75\)

The critical load combination is the dead plus roof live plus wind case. The actual bending stress \(f_b\) should be calculated with the full 120 plf loading with no load duration adjustment, then checked against an allowable \(F_b\) having a \(C_D\) of 1.60.

Note that load duration adjustments are not applicable to modulus of elasticity (see Commentary 2.3.2.1), hence, a member subject to buckling should be analyzed for the critical load combination after the critical buckling design value has been calculated. This is illustrated in Example C2.3-3.

2.3.2.3 Load combination adjustment factors provided in ASCE 7-88 and in building codes account for the reduced probability that two or more loads, other than dead loads, acting concurrently will each attain its maximum at the same time. Such adjustment factors for load combinations are applicable to all materials. The effect of load duration on the individual ultimate strength properties of wood is unique, relative to other major construction materials, in terms of its magnitude and application to both ultimate strength and proportional limit stresses. Adjustment of wood design values for load duration is an adjustment to the working stress irrespective of whether one or more
Example C2.3-3

Consider a column subject to a 350 lb dead load, a 1300 lb contents live load and a 400 lb roof live load. Assume an 8 ft column height, 2x4 spruce-pine-fir stud grade:

\[
\begin{array}{cccc}
C_D & F^*_c & C_P & F^*_c'A \\
0.90 & 638 & 0.585 & 373 & 1959 > 350 & \text{ok} \\
1.00 & 709 & 0.545 & 386 & 2027 > 1650 & \text{ok} \\
1.25 & 886 & 0.461 & 409 & 2145 > 2050 & \text{ok}
\end{array}
\]

The critical load combination is the dead plus live plus roof live case.

external loads are applied. Reductions allowed in design loads on the basis of load combination probabilities are to be applied concurrently with adjustment of design values for load duration. The former affects the magnitude of the actual load while the latter establishes the maximum resistance available to carry that load. Example C2.3-4 demonstrates this provision.

Example C2.3-4

Consider a column subject to a 400 lb dead load, a 1000 lb contents live load, a 600 lb snow load, and a 400 lb wind (additive) load. Assume an 8 ft column height, 2x4 spruce-pine-fir stud grade:

\[
\begin{array}{cccc}
C_D & F^*_c & C_P & F^*_c'A \\
0.90 & 638 & 0.585 & 373 & 1959 > 400 & \text{ok} \\
1.00 & 709 & 0.545 & 386 & 2027 > 1400 & \text{ok} \\
1.15 & 815 & 0.492 & 401 & 2105 > 2000 & \text{ok} \\
1.60 & 1134 & 0.377 & 427 & 2241 > 1800 & \text{ok}
\end{array}
\]

Note that the ASCE 7-88 load combination factor of 0.75 applies to the dead plus live plus snow plus wind case: \((400 + 1000 + 600 + 400)(0.75) = 1800\). The critical load combination is, therefore, the dead plus live plus snow case.

2.3.3-Wet Service Factor, \(C_M\)

The different moisture service conditions for which design values or design value adjustments are provided in the Specification are:

Sawn lumber:
- dry, less than 16 percent (Tables 5A - 5C)
- wet, 16 percent or greater (factors, Tables 5A - 5C)

Glued laminated timber:
- dry, less than 16 percent (Tables 5A - 5C)
- wet, 16 percent or greater (factors, Tables 5A - 5C)

Round timber piles:
- dry, 19 percent or less (tabulated by type)
- wet, 30 percent or more (Table 7.3.3)
- partially seasoned, greater than 19 percent and less than 30 percent (Table 7.3.3)

A moisture content of 19 percent has long been recognized as an appropriate upper limit for a dry condition of service for lumber used in wood structures. This maximum level coincides with the moisture content requirement for dry dimension lumber given in the American Softwood Lumber Standard PS20-70.

Uses involving maximum moisture contents of 19 percent are traditionally considered to average 15 percent or less. Those involving maximum moisture contents of 15 percent are considered to average 12 percent or less. Glued laminated timber and wood panel products have moisture content less than 16 percent at manufacture and therefore design values for these products are appropriate for maximum service conditions associated with such a moisture content.

Moisture contents of 19 percent or less are generally obtained in covered structures or in members protected from the weather, including wind blown moisture. Roof, wall and floor framing, and attached sheathing are considered to be such dry applications except where special conditions exist (62,66). These dry conditions of service are generally associated with an average relative humidity of 80 percent or less. Framing and sheathing in properly ventilated roof systems, which are periodically exposed to relative humidities over 80 percent for short periods, meet moisture content criteria for dry conditions of use. Floor framing over properly ventilated crawl space which include a vapor retarder to cover exposed soil meet moisture content criteria for dry conditions of use (238).

The wet service factor, \(C_M\), in the 1991 edition is an adjustment to tabulated design values for wet use conditions. The designer need only determine the expected moisture content of the product in use to establish the applicable value of \(C_M\).
2.3.4-Temperature Factor, $C_t$

Prior to 1977, the effects of temperature on design values for lumber and timber were not addressed in the Specification. Decades of satisfactory experience with wood buildings and bridges have shown that adjustment of design values for ordinary temperature fluctuations encountered by these structures in service is not required. The use of wood construction in non-building industrial applications such as cooling towers where members are subjected to heavy loads and hot water exposures, as well as special cold weather building applications and use of wood for material handling of cryogenic materials, motivated the inclusion of information on temperature effects in the 1977 and subsequent editions of the Specification.

Reversible Effects. 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 (72). The magnitude of the increase or decrease, however, varies with moisture content. The higher the moisture level, the larger the increase with decreasing temperature and the larger the decrease 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 (66). 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 (66). Exposure in air at the same temperature would result in a smaller permanent strength loss.

Temperature Adjustments. Adjustments for reversible temperature effects first given in the 1977 edition of the Specification applied to 0 percent and 12 percent in-service moisture conditions. Separate heating and cooling adjustment coefficients were given for each condition of use for modulus of elasticity and for strength properties. These adjustments, based on summary information published by the Forest Products Laboratory (65), were continued in the 1982 edition.

In the 1986 edition, the temperature adjustments were revised to reflect the results of a comprehensive analysis of available world literature by the Forest Products Laboratory (72), and expanded to include wet conditions of service (24 percent moisture condition).

1991 Factors. In the 1991 Specification, strength loss coefficients for each 1°F increase in temperature above 68°F at 0, 12 and 24 percent moisture content have been replaced with simplified temperature factors, $C_t$, for dry and wet service conditions and two levels of elevated temperature: 101°-125° and 126°-150°. As in previous editions, one set of factors is given for modulus of elasticity and tension parallel to the grain and a separate set for other design values. The use of temperature factors for dry (average 12 percent) and wet (greater than 19 percent) conditions of service follows the same basis used for other design value adjustments related to moisture.

Mandatory Reductions. The temperature adjustments in the 1991 Specification are mandatory 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 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 (66). As discussed below, roof systems and other assemblies subject to diurnal temperature fluctuations from solar radiation are not applications that normally require adjustment of tabulated design values for temperature.

Cold Temperatures. Adjustments for increasing tabulated design values for cooling below normal temperatures, which were included in Appendix C of the Specification in the three previous editions, have been
eliminated in the 1991 edition. The appropriateness of such increases is 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 must be resisted. Increases in design values are not applicable for this common occurring situation. To avoid misapplication of increases for below normal temperatures, and in recognition of the relatively few design situations where such increases are appropriate, temperature factors for cooling have been dropped from the Specification. 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 (66,72).

_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 (89) 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 recedes.

When wood structural members are subjected to temperatures above normal levels, the moisture content of the members decreases. At the highest temperatures reached, moisture contents in properly ventilated and maintained roof systems will approach moisture contents of 6 percent. Associated with such a decrease in moisture content is an increase in strength property which offsets the decrease resulting from temperature. For example, based on available data (72), the bending strength of a member at 140°F and 6 percent moisture is reduced 20 percent relative to strength at 70°F and 6 percent moisture content. However, the increase in strength resulting from a change in an average service moisture content of 12 percent to 6 percent is 30 percent (72), which more than offsets the immediate temperature effect. Further, structural framing members are subjected to elevated temperature exposures in the warm months of the year when uniform roof snow loads are not present.

The foregoing considerations and successful field experience are the basis for the long standing practice of applying the design values tabulated in the Specification without adjustment for temperature to structural wood roof members in systems designed to meet building code ventilation requirements. Tabulated 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.

2.3.5-Pressure-Preservative Treatment

Structural wood members in conditions where the moisture content will exceed 19 percent in service, or members which will be in proximity to damp wood, soil or other sources of moisture, may be susceptible to decay (88). Where such service conditions exist, it is the responsibility of the designer to determine if pressure-preservative treated wood should be used.

Application of tabulated design values for lumber and glued laminated timber pressure treated with standard preservative formulations and processes (25) has been a provision of the Specification since the first edition. Early practice with water-borne preservatives was to treat material that had been air-dried or kiln dried at temperatures of 180°F or lower. Treated material was generally air redried or redried in service. Where kiln redrying was employed, generally temperatures less than 190°F were used.

In the past several decades, kiln redrying of waterborne preservative treated wood has been increasing, particularly with the development and use of wood foundations. During the same period, use of high temperature kilns to initially dry dimension lumber also increased. As a result of studies showing that kiln redry temperatures of 170°F and higher can reduce the strength of small, clear preservative treated wood specimens (210,212,215), a redry temperature limit of 190°F for waterborne preservative treatments was added to AWPA treatment standards for sawn lumber and timber in 1988. Subsequent research showed that the effect of redrying on the strength of treated lumber was related to the grade and quality of the material, with the higher strength grades and higher strength pieces within a grade being more significantly affected than the lower strength grades and lower strength pieces within a grade (211,214).

More recently it has been shown that the combination of preservative treatment of lumber that was initially dried in high-temperature kilns followed by redrying at 190°F can reduce bending and tensile strength properties up to 10 percent for some species and higher grades of

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material (213). As a result of these new findings, the maximum temperature for kiln drying material after treatment in AWPA Standards C2, Lumber, Timbers and Ties - Preservative Treatment by Pressure Processes, and C22, Lumber and Plywood for Permanent Wood Foundations - Preservative Treatment by Pressure Processes, recently was reduced from 190°F to 165°F (26). The provision in the Specification for use of tabulated design values with lumber that has been preservatively treated is applicable to material that has been treated and redried in accordance with AWPA 1991 Standards C2, C22, C28, Structural Glued Laminated Members and Laminations Before Gluing, Pressure Treatment or C31, Lumber Used Out of Contact with the Ground and Continuously Protected from Liquid Water. Tabulated design values for borate treatments in accordance with C31 apply to retention levels of 0.3 lbs/ft³ and less. Borate containing treatments at retention levels of 3.0 lbs/ft³ and higher have shown embrittlement (237).

New preservative treatments for lumber are being developed and should be tested by the manufacturer of the treatment or the company providing the treating and redrying service to evaluate the effects on lumber strength. The designer should consider potential effects on strength of lumber from new preservative treatments and should evaluate the basis and adequacy of the manufacturer’s recommendations for those products not yet having a history of satisfactory performance.

Recent research has indicated that modulus of elasticity and bending design values in incised, pressure-treated lumber may be reduced compared to untreated, unincised specimens (236). 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.

2.3.6-Fire Retardant Treatment

Fire retardant treated wood has been in use since before 1900 for applications ranging from ships, blimp hangers, scaffolding, doors and trim in high rise buildings, interior panelling, interior partitions, roof construction and balconies and stairways (51). Fire retardant treatments are proprietary and chemical formulations vary between manufacturers. All pressure treatments which are accepted by building codes, however, meet minimum flame spread resistance requirements (38, 91, 169). Except for certain special formulations, fire retardant treatments are intended for use in dry conditions of service.

Introduction of 10 Percent Treatment Factor for Lumber. In 1962, following the first building code acceptance of the use of fire retardant treated wood for roof construction in certain non-combustible types of buildings, the Specification recognized the structural use of such material through introduction of a 10 percent reduction in tabulated design values for lumber pressure treated with fire retardant chemicals. Available test data indicated that such a reduction, which was applied to fastener design values as well as strength properties, could be generally used with all treatments (51, 81, 95). The reduced strength of the fire retardant treated wood was associated with the elevated temperatures used in kiln drying after treatment. Strength properties of material air-dried after treatment were reported to be little affected (71). The 10 percent reduction factor applied to lumber design values for fire retardant treatment was continued in the Specification for the next 20 years. The satisfactory performance of components and systems made with fire retardant treated lumber over this period attested to the general adequacy of Specification provisions for the type of treatments that were commercially available.

Glued Laminated Timber. Prior to 1977, the Specification did not contain provisions specifically addressing design values for fire retardant treated glued laminated timber. Because certain species of lumber could be adequately glued after fire retardant treatment, glued laminated timber made with such treated lumber could qualify as a fire retardant treated product. The 10 percent design value adjustment for fire retardant treated lumber was assumed applicable to the properties of the treated laminations.

In 1977, the practice of using the 10 percent reduction factor for fire retardant treated lumber with glued laminated timber was discontinued. This change was based on recognition of the effect of a number of variables: the differences between pressure treatment with fire-retardant chemicals before and after gluing; and differences in the effects of individual treatments on different species and different properties. Particularly noted was information that fire retardant treatment could reduce the bending strength of glued laminated timber more than 10 percent in some cases due to the effect of the treatment on the strength of end joints in the laminations. Beginning in the 1977 edition, the Specification has referred users to the manufacturer of the treatment or to the company providing the treating and redrying service for design value recommendations for glued laminated timber pressure treated with fire retardant chemicals.

Applicable Load Duration Factors. The same load duration adjustments applicable to untreated wood have traditionally been applied to fire retardant treated
wood (129). However, since 1982, the Specification has disallowed the 2.0 increase for impact loading (load duration less than one second) for fire retardant treated lumber. This change was made in recognition of the reduced energy absorbing capacity or toughness of fire retardant treated wood relative to that of untreated material. The property work-to-maximum load obtained from a bending test, which provides a measure of energy absorbing capacity, is more sensitive to treating chemicals and redrying than any of the other properties. Although impact resistance is not generally a design consideration, the reduced energy absorbing capacity of fire retardant treated wood should be recognized where it may be important in material handling, in resisting construction loads, or in resisting special service loads on completed structures. In this regard, cases have been reported of breakage of trusses made with fire retardant treated lumber when being off-loaded in bundles at the construction site and of similar occurrences with other treated products during transportation.

The use of the 1.6 (ten minute) and other load duration factors with fire retardant treated material is supported by the fact that the effects of fire retardant treatments on strength properties are evaluated using standard static tests to failure of treated and untreated wood that involve average times to failure of 5 to 10 minutes.

New Treatment Formulations. In the early 1980's, new fire retardant chemical formulations began to be introduced which had lower affinities for moisture thus reducing the corrosion potential on metal parts of the treated material when exposed to higher relative humidities. These exposure conditions were being encountered in applications considered a dry condition of service but which involved periodic exposure to humidities over 80 percent, such as those occurring in ventilated roof systems in the southeast. The lower hygroscopicity treatments have become the predominate fire retardant treatments available.

In the absence of extensive strength test data for and experience with the new fire retardant treatments, the generic 10 percent reduction in lumber design values was dropped from the Specification by amendment in 1984. Users of fire retardant treated lumber were referred to the company providing the treating and redrying service for appropriate design value adjustments. However, in response to standardization needs expressed by code agencies, users and manufacturers, new provisions for evaluating and qualifying fire retardant treated lumber for specific design value adjustments were introduced in the 1986 edition of the Specification.

Adjustments in the 1986 Edition. The 1986 edition provided that each treatment be qualified for certain tabulated design value adjustments by meeting specific testing, inspection and marking criteria (Appendix Q of the Specification). The prescribed test procedures utilized matched samples of treated and untreated small, clear, straight-grained material to determine the effects of treatment on strength properties. Strength test information from the same type of clear material has long been used in the establishment of design values for visually graded lumber and for development of load duration and other adjustment factors for condition of use.

Strength data for several of the treatment formulations developed in accordance with the specified testing procedures were considered in establishing design value adjustments for fire retardant treated lumber tabulated in the 1986 Specification. The factors were based on tests for the properties of extreme fiber in bending, modulus of elasticity, compression parallel to grain, tension parallel to grain and horizontal shear. Adjustment classes were established in 0.05 increment classes with the highest class being 0.90 to maintain consistency with past practice where possible. Adjustments in the 1986 edition were 0.85 and 0.80 for bending (Fb) and tension (Ft) design values respectively, with all other properties assigned a factor of 0.90. It was recognized that treatments qualifying for these adjustments might have higher treatment ratios than those tabulated for one or more properties. Further, the adjustment factor for compression perpendicular to grain was conservatively assumed equal to that for compression parallel to grain; and, as fastener loads are related to compression properties, and indirectly to shear strength because of end distance considerations, a design load adjustment factor of 0.90 for connectors was used.

Quality Marking Provisions in the 1986 Edition. The effect of fire retardant treatments on wood strength properties is affected by a number of process variables, including amount and penetration of chemicals retained and the time and temperature used to redry the material after treatment. Under AWPA Standard C20 (26), and in accordance with building code requirements, fire retardant treated lumber is required to be dried to a moisture content of 19 percent or less after treatment. The conditions used to achieve this level of drying have a significant influence on strength properties. The 1986 Specification therefore required quality marking by an approved inspection agency to validate that production material had been treated and redried in accordance with the process procedures established by the chemical manufacturer and which were used to qualify the treatment for the design value adjustments.
tabulated in the Specification. This required quality marking was separate from that required by building codes for flamespread classification.

Adjustment Factors Based on Ambient Temperature Conditions. The adjustment factors for fire retardant treated lumber given in the 1986 and earlier editions of the Specification were based on strength tests of material tested at normal room temperature conditions. Although treated lumber and plywood made with the older traditional formulations have given satisfactory performance in roof system applications, significant problems have been reported with plywood roof sheathing made with certain of the new fire retardant treatment formulations, particularly in town house type construction (105). A few cases of problems involving treated lumber roof framing also have been reported. Testing has now shown that fire retardant treated wood generally is more affected by prolonged exposure in service to elevated temperatures of 150°F and higher than untreated wood. Such conditions are encountered in wood roof systems where the effects of solar radiation on the roof are not offset by natural ventilation of ambient air.

The cumulative periods of time fire retardant treated wood is subject to temperatures above ambient in the life of the structure must therefore be taken into account when establishing design value adjustments for treated material used in roof applications. Although roof sheathing is subject to higher thermal loads than supporting framing, the effects of elevated temperature exposure over time must be considered for the latter as well as the former.

The major fire retardant treatment manufacturers have established design value adjustment recommendations for their present commercial formulations for both roof system and ambient temperature applications involving both treated plywood and treated lumber. The testing methodology employed for lumber evaluation is reported to be similar to that given in the 1986 Specification but includes testing after extended exposure at near maximum roof framing temperatures. These recommendations account for the effects of expected cumulative exposure to elevated temperatures over the service life of the structure.

Publication of Design Value Adjustments for Fire Retardant Treated Lumber Discontinued in the Specification. In 1990, the National Forest Products Association made the determination that design value adjustment factors for fire retardant treated lumber should no longer be published in the National Design Specification. This action was reflected in a revision to the 1986 edition, promulgated May 1990, that required the effects of fire retardant chemical treatments on strength to be considered in design, and required design values, including fastener design loads, for fire retardant treated lumber as well as glued laminated timber to be obtained from the company providing the treating and redrying service. This change in the provisions for fire retardant treated lumber was made on the basis that the design value adjustments and test procedures given in the Specification for this material did not address the effects of elevated temperature exposure, that standard methodology necessary to evaluate elevated temperature effects through accelerated testing and to convert results from such tests into long-term durability assessments was not available, and that the development and promulgation of the required methodology was outside the scope of Association functions.

The 1990 revision is carried forward in Section 2.3.6 of the 1991 Specification. It is expected that the lumber design values to be obtained from the treating companies will reflect use of testing methodologies similar to those given in Appendix Q of the 1986 edition, or equivalent; will apply to material treated in accordance with AWPA Standard C20; and will apply to material that bears agency quality monitoring marks similar to those called for in the 1986 Specification. It should be noted that use of individual company design value recommendations for fire retardant treated lumber and glued laminated timber is subject to the approval of the applicable building code jurisdiction.

2.3.7-Beam Stability Factor, $C_L$

The beam stability factor accounts for the tendency of laterally unsupported deep beams to rotate or buckle under bending or combined axial and bending loads. The adjustment of tabulated bending design values for beam stability or slenderness given in 3.3.3 was first incorporated into the Specification in 1968 for glued laminated timber. It was subsequently extended to cover sawn lumber beams in 1977.

Prior to the 1991 edition, the beam stability factor was not applied simultaneously with size adjustment for beams that were larger than 12 inches in depth. In the current edition, the stability factor, $C_T$, and the size factor, $C_F$, are applied simultaneously to all sawn lumber bending design values, including those for beams greater than 12 inches in depth. However, no stability or slenderness adjustment of tabulated design values for sawn lumber beams is necessary if such beams are laterally supported in accordance with the approximate rules given in section 4.4.1. These approximate rules for providing restraint against rotation and
lateral displacement have been in the Specification since the 1944 edition.

The beam stability factor, \( C'_b \), is not applied simultaneously with the volume factor, \( C_v \), for glued laminated timber bending members in the 1991 Specification. This continuation of past accepted practice considers tabulated design values for deep glued laminated timber beams to be controlled by tension zone failures whereas beam buckling characteristics are related to compression zone properties.

2.3.8-Form Factor, \( C_f \)

Adjustment of tabulated bending design values for form are based on early research at the Forest Products Laboratory (136). Results of tests of round, diamond, I and Box beam sections showed that the strengths of these members differed from that expected by their section properties and the strength of matching rectangular beams.

In the case of round beams, it was found that the average modulus of rupture of these beams as calculated by standard beam formulas was 1.15 times the modulus of rupture of matched rectangular beams. However, the section modulus \( (S=I/c) \) of a round beam is about 1/1.18 times smaller than that for a rectangular beam of equivalent area. Thus a beam of circular section has about the same load-carrying capacity as a rectangular beam of equal cross-sectional area; or alternatively, has a \( C_f \) of 1.18.

In the case of square sections loaded with diagonal vertical (diamond section), tests showed that the loads carried by these sections were slightly larger than those for similar sections tested with sides vertical. Although the moment of inertia, \( I \), of a square about its diagonal is the same as that of a square about a neutral axis perpendicular to its sides, the distance from the neutral axis to the extreme fiber is \( \sqrt{2} \) or 1.414 greater. Thus the diamond section, which would be expected to have a 1/1.414 lower strength than the square section by the ordinary beam formula, is assigned a \( C_f \) of 1.414.

In contrast to the circular and diamond sections, tests of I and Box sections showed that these beams had lower strengths than those expected from conventional beam equations. The behavior of the various beam sections was explained in terms of the difference in the compression parallel to grain strength and the bending strength of clear, straight grain wood and the amount of lower stressed fibers available in a beam to support higher stressed fibers near the compression face. Equations were developed at the Forest Products Laboratory to adjust tabulated bending design values, based on tests of rectangular sections, for use with I and Box beams (57,136). These equations were used in the design of wing struts and similar non-rectangular members used in wood aircraft structures (61). Because the effect of beam depth on bending strength also was considered a function of the relative support available from lower stressed fibers nearer the neutral axis, both size effect and adjustment for form were accounted for in form factor equations (136).

Form Factor Provisions in the Specification.

The equivalence of circular and rectangular beams having the same cross-sectional area has been recognized in the Specification since 1944. The 1.18 form factor for circular sections, the 1.414 factor for diamond sections and a form factor equation for I and Box sections were added to the Specification in 1957. The form factor equation, which included a size effect for deep beams, was discontinued in the 1977 edition. This was a result of the introduction of a new size effect equation for sawn lumber beams in the 1973 edition and changes in commercial practice.

Application of the form factor equation for I and Box beams with built-up structural members used in buildings gradually decreased in the years following World War II as commercial lumber grades rather than aircraft grades of lumber were used as flange material. Rather than employ form factors in conjunction with bending design values to establish allowable strength values for these beams, compression parallel to grain design values were assumed to be the limiting criterion; or, more recently, design values were based on tests of full-size beams made with particular grades of flange material.

Form factors for circular and diamond sections have continued unchanged in the specification to the present edition. Because these factors are based on the strength of equivalent rectangular sections which vary with size, they are applied cumulatively with size factor, \( C_f \).

Application of Form Factors. In view of the basis of the values of \( C_f \) for circular and diamond sections, use of these factors should be limited to naturally round material, such as piles, or to grades of material in which compression parallel to grain strength values are lower than bending (modulus of rupture) strength values.

Circular wood structural members generally will be tapered. The variable section properties of such members must be taken into account. Deflection and moment equations are available for tapered circular
beams under quarter-point loading (196). A procedure for relating the deflection of round tapered beams (log beams) under uniform load to that of equivalent area (mid-span) square beams also has been developed (147).

For guidance in design of I and Box sections using special high grades of lumber, the original research publication on the subject should be consulted (136). For I and Box sections in which the top and bottom edges of the beam are not perpendicular to the vertical axis of the beam, test results from this research show the strength of such beams is equivalent to that of an I or Box section whose height equals the mean height of the original section and whose width and flange areas equal those of the original section.

2.3.9 Column Stability Factor, $C_p$

The column stability factor is the $l/d$ or $l/r$ based adjustment of tabulated compression parallel to grain design values to account for buckling. This factor is applied simultaneously with the size factor, $C_F$.

2.3.10 Bearing Area Factor, $C_b$

Provisions for increasing tabulated compression perpendicular design values for length of bearing have been included in the Specification since the 1944 edition. Tabulated values are based on loading a two-inch wide plate bearing on a two-inch wide by two-inch deep by six-inch long specimen. Early research at the 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 (183,217). Early research conducted in Australia and Czechoslovakia confirmed the nature and magnitude of the bearing length effect (217).

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 (112,217). Because of the localized nature of the edge effect, the contribution provided decreases as the length of the area under compressive load increases. Alternatively, when the bearing plate covers the entire surface of the supporting specimen (full bearing), values lower than those obtained in the standard two-inch plate test are obtained.

Formal bearing length adjustments, indexed to tabulated compression perpendicular to grain design values at a bearing length of six or more inches, were first recommended in 1935 (57). The adjustments published in the 1944 edition of the Specification and continued to the present edition are slightly revised from those first proposed in 1935, being about 10 percent lower for bearing lengths less than four inches. No adjustments are allowed for bearings less than three inches from the end of a member in recognition of the fact the adjustments assume support from two edges and the standard bearing test has a two inch unloaded length on each side of the loaded plate.

Application of Bearing Area Factors. 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. In the latter case of perpendicular grain bearing on perpendicular grain, see Commentary for 4.2.6 for discussion of deformation occurring in this support condition relative to metal or end grain bearing on side or face grain.

For the case of complete surface or full bearing (bearing length equals supporting member length), such as may occur in a pressing operation, 75 percent of the tabulated compression perpendicular design value may be used.
PART III: DESIGN PROVISIONS AND EQUATIONS

3.1-GENERAL

3.1.2-Net Section Area

3.1.2.1 Specific provisions pertaining to notches in bending members are given in section 3.2.3. Provisions for calculation of actual shear stresses parallel to grain in notched bending members are given in section 3.4.4. Section 3.6.3 concerning compression parallel to grain 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.

The effect of eccentric loading of compression members can be evaluated using the equations given in Section 15.4 of the Specification.

3.1.2.2 A specific criterion for determining when staggered fasteners in adjacent rows should be considered in the same critical section was first introduced in the 1960 edition. The criterion was expressed in terms of a minimum spacing of fasteners parallel to grain in the same row, or less than 8 fastener diameters. This provision, which was continued to the 1986 edition, assumed symmetrical placement of fasteners in the joint. However, the requirement provides no minimum parallel to grain distance between fasteners in adjacent rows if a non-symmetrical staggered fastener pattern is used. To avoid possible misapplication when non-uniform patterns are used, the provision has been changed in the 1991 edition to require 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.

3.1.2.3 The limiting parallel to grain distance of one or less connector diameters between staggered split ring or shear plate connectors in adjacent rows that is used to determine net area of critical section has been in the Specification since 1960. The limit should be applied to the parallel to grain offset or stagger of rings or plates in adjacent rows.

The alternate procedure for checking the adequacy of net section at split ring and shear plate connector joints in compression members and sawn lumber tension members given in Appendix A.12 assumes that the net section area has been reduced for the cross-sectional area of any knots occurring within 1/2 a connector diameter of the critical section (62,65,178). When this criterion is met, actual tension stress parallel to grain ($f_t$) and actual compression stress parallel to grain ($f_c'$) values are checked against bearing design values parallel to grain ($F_b'$) (see Table 2A). The use of clear wood compression parallel to grain design values for checking net section of sawn lumber tension members joined with connectors is based on test results that show the concentration of stresses in the net section caused failure in tension at levels approximately equal to the maximum compressive strength of the material parallel to grain (62,159,178,183).

The alternate procedure for checking the net section at split ring and shear plate connections was first introduced for tension members in the 1944 edition of the Specification and subsequently extended to compression members in the 1951 edition. Prior to 1982, the clear wood allowable bearing design values parallel to grain used to check the net section were obtained through use of connector net section factors tabulated for connector species groups (178). Use of the special connector species group values was discontinued in the 1982 edition in favor of the species specific bearing design values parallel to grain (see Table 2A).

3.1.3-Connections

Inadequate design of the connections between members is a frequent cause of unsatisfactory performance in wood structures. Important connection designs include those between beams and columns, between members of built-up trusses, between roof structural members and vertical supporting walls, and between load-bearing walls and foundations.

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 Commentary for Section 7.1.1.1).

Requirements for (i) design and fabrication of connections so as to insure all members at the location of the joint carry their proportional load and (ii) use of symmetrical members and fasteners at connections unless induced moments resulting from unsymmetrical design are taken into account have been included in various forms in the Specification since the first edition. Section 3.1.3 in the 1991 edition consolidates two separate provisions relating to this subject in the 1986 and earlier editions. Also, the reference to stresses induced by unsymmetrical arrangement of members...
and/or fasteners as "secondary stresses" has been discontinued to avoid confusion with the common usage of this term in engineering practice. Lapped joints have been referenced as examples of unsymmetrical connection design to add emphasis to the need to consider the bending moments induced in such configurations (see Commentary for 7.1.2). The effects of eccentricity of loads at critical net section also are referenced in paragraph 3.1.2.1 of the Specification.

Joists attached to one side of a built-up girder made of two or more pieces of dimension lumber of the appropriate width is a common design configuration where the connection between the members of the girder and between the girder and the joist are important to insure the entire carries its proportional share of the load. If joists are attached to or bear on ledger strips attached to the outside member of the girder only, members of the girder should be bolted, clinched nailed or otherwise connected to each other such that all will deflect the same amount under the distributed load from the intersecting joists.

3.1.4-Time Dependent Deformations

Consideration of time dependent deformations in built-up members of structural frames was first introduced in the 1960 edition of the Specification in the context that the arrangement of connections between the leaves or sections of a member shall provide for equal inelastic deformation of the components. The provision was generalized and made advisory in the 1977 edition.

One type of application being addressed by 3.1.4 is the use of a leaf or flange to strengthen or stiffen a single main member in a truss without increasing the size of other members in the same plane. Mechanically fastening additional layers of material to a compression web to form an I or T section, or to otherwise increase the least dimension of the web, is such a practice (179). Because the components of such built-up members do not provide full composite action, judgment must be used to establish the level of contribution of the layers of material attached to the main member. Creep effects should be considered in making this assessment.

3.1.5-Composite Construction

Structural composites of lumber and other materials utilize the superior 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 (83,179). Detailed design and fabrication information for plywood-lumber structural components are available from the American Plywood Association (11). The American Institute of Timber Construction provides design information for composites involving glued laminated timber (4).

3.2-BENDING MEMBERS-GENERAL

3.2.1-Span of Bending Members

Use of the clear span plus one-half the required bearing length at each reaction to establish the design span for simple bending members has been a provision of the Specification since 1944. This definition provides a reasonable design standard for applying the assumption of knife edge supports used in the derivation of general equations for moment and reactions.

Prior to 1986, the span of continuous bending members was taken as the distance between the centers of supports over which the beam was continuous. To avoid unrealistic moment determinations where wider supports than those required for bearing were used, and to provide consistent treatment of all bending members, the definition of span length for continuous bending members was first made the same as simple members in the 1986 edition. For completeness and clarity, cantilevered members were included under the provision at the same time.

When determining shear forces on bending members resisting uniform loads, the provisions of 3.4.3 and 4.4.2 with regard to loads at or near the supports apply.

3.2.2-Lateral Distribution of Concentrated Load

Generally all designs involving multiple parallel bending members which 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 for dimension lumber in paragraph 4.3.4 indirectly and partially accounts for such load redistribution. Comprehensive computer design methodology is available to establish the extent of load redistribution or sharing that is obtained in light frame wood floor systems (125). Such methodology is based on the stiffness, size, spacing and spans of the joist and panel members in the system and the rigidity of the connections between them.

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 section 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 (2). 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 (53).

3.2.3-Notches

3.2.3.1 Avoidance of notches in bending members has been a recommendation of the Specification since the earliest editions. 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. The reduction in these stress concentrations that can result by using gradually tapered rather than square corner notches (62) was formally noted in the Specification in the 1977 edition.

The assumption that 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 has little practical effect on bending member stiffness (57,62) has been a provision of the Specification since the 1944 edition. For example, the reduction in stiffness of a 2x4 on 6 ft. span, a 2x8 on 12 ft. span and a 2x12 on a 16 ft. span of such a notch at midspan, when calculated by integration of the elastic curve for the variable cross-section bending member under a uniform load, is 1.9, 2.0 and 2.3 percent respectively. Research has shown that the effect of notches on bending member stiffness is somewhat greater than that indicated by the notch dimensions, and can be approximated by using a notch length equal to the actual length of the notch plus twice the depth of the notch (65,66,120). When this approximation is applied to the 1/6 depth by 1/3 length notch used in the example bending members above, the reduction in stiffness of the 2x4, 2x8 and 2x12 bending members is 3.8, 3.9 and 4.6 percent respectively, or about twice the effect obtained using the actual notch dimensions.

3.2.3.2 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 (57,62). It was recognized even at that time that stress concentrations at the corners of the notch caused lowered 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 (57,62).

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 (119,120,172). 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 (66).

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 the span is consistent with good practice recommendations for light frame construction (91,126). 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 tabulated 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 that only end notches are permitted 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.
Tension perpendicular to grain stresses also occur with shear stresses at end notches to make a bending member more susceptible to splitting at the corner of such notches. The design provisions for shear in end notched bending members given in 3.4.4 include a magnification factor to account for this condition. 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. Restricting the length of rectangular end notches, measured from the end of the bending member, to the depth of the member is advisable.

3.2.3.3 Prior to 1977, notching provisions for glued laminated timber and sawn lumber were the same. As a result of field experience with notched large glued laminated bending members and the difficulty of accurately quantifying stress concentration effects, application of sawn lumber notch provisions to glued laminated timber bending members was discontinued in the 1977 and subsequent editions of the Specification. The designer has the responsibility of determining if 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 notching at the supports is necessary. This end notching is generally limited to the smaller bending members and to notch depths not exceeding 1/10 of the bending member depth (4). The methods of 3.4.4 are used to calculate actual shear stresses parallel to grain ($f_s$) of tension side end notches in glued laminated timber members (4).

3.3-BENDING MEMBERS-FLEXURE

3.3.3-Beam Stability Factor, $C_L$

**Background**

Equations, verified experimentally, for calculating critical lateral buckling loads of any length and thickness of wood bending members under various loading and end fixity conditions were available as early as 1931 (184). However, in reflection of designer preference, the early editions of the Specification addressed the subject of lateral buckling or twisting of bending members by referencing approximate bending member depth to thickness rules (Section 4.4.1) for determining when and what type of lateral support should be provided (62). In 1968, general provisions for determining the adequacy of lateral support of bending members to prevent lateral buckling or displacement of the compression face were incorporated in the Specification for glued laminated timber bending members. The provisions were extended to sawn lumber bending members in the 1977 edition.

The procedures used to adjust bending design values for slenderness factor that were introduced in the Specification in 1968, and continued through the 1986 edition, were based on Canadian research involving verification of the applicability of lateral buckling theory to wood bending members and the development of simplified adjustment procedures for slenderness effects (86). The verification tests involved 3/4 inch wide simply supported and cantilever bending members up to 8 inches in depth and from 24 to 204 inches in length. The tests confirmed that the following simplified general equations based on a modulus of elasticity to modulus of rigidity ratio ($E/G$) of 16 gave reasonably good estimates of critical stress:

$$f_b = \frac{1.20E}{S^2}$$  \hspace{1cm} (C3.3-1)

where:

- $f_b =$ critical bending stress
- $E =$ modulus of elasticity

and

$$S^2 = \frac{T\ell_u d}{b^2} \left[1 - C\left(\frac{d}{\ell_u}\right)\left(1 + \frac{2b}{3d}\right)\right]$$  \hspace{1cm} (C3.3-2)

with

- $T, C =$ constants depending on type of load and support condition
- $\ell_u =$ unsupported length
- $d, b =$ depth and breadth of bending member

The critical bending stress equation is similar to that for long columns except with the Euler constant of 0.822 replaced by 1.20 and $\ell/d$ replaced by $\ell d/b^2$, the latter ratio being similar to the comparable ratio for steel bending members of $\ell d/b t$. Where $t$ is defined as the flange or web thickness of the steel bending member.

Based on test results and steel design practice in England, the general equation was applied to critical stresses above the proportional limit by assuming the transition stress-strain curve for bending above the proportional limit was the same shape as that for compression. Thus, applying the established intermediate column formula defining this region for compression loading, the level at which inelastic beam buckling
began was established as 2/3 of the tabulated bending design value $F_b$. Also analogous to column design provisions in the 1977 to 1986 editions of the Specification, an $S$ value of 10 was selected as the level below which the full allowable bending design value could be used. Thus long, intermediate and short beam buckling criteria were established comparable to long, intermediate and short columns.

The equation for $S$ was further simplified for design use by defining

$$S^2 = \frac{\ell_e d}{b^2}$$  \hspace{1cm} (C3.3-3)

and an effective length

$$\ell_e = 1.15 \ell_u + 3d$$  \hspace{1cm} (C3.3-4)

for loads applied to the top of the bending member. For loads applied at the center of gravity of the bending member, the $3d$ term in the $\ell_e$ equation is deleted; and for loads applied on the bottom of the bending member the $3d$ term is replaced with $-d$. The 1.15 factor in the equation for $\ell_e$ represents a 15 percent increase in the actual $f_u$ to account for less than full torsional restraint at lateral support points (86).

Based on the foregoing analyses and simplifications, bending design values adjusted for slenderness factor ($F'_b$) incorporated in the 1968 and 1977 editions of the Specification were:

Slenderness factor

$$C_s = \sqrt[\frac{1}{3}] {\frac{\ell_e d}{b^2}}$$  \hspace{1cm} (C3.3-5)

For $C_s \leq 10$

$$F'_b = F_b$$  \hspace{1cm} (C3.3-6)

For $11 < C_s \leq C_k$

$$F'_b = F_b \left[ 1 - \frac{1}{3} \left( \frac{C_s}{C_k} \right)^4 \right]$$  \hspace{1cm} (C3.3-7)

in which

$$C_k = \sqrt[\frac{1}{3}] {3E/5F_b}$$  \hspace{1cm} (C3.3-8)

For $C_s > C_k$

$$F'_b = \frac{0.40 E}{(C_s)^2}$$  \hspace{1cm} (C3.3-9)

The Euler buckling constant for long bending members of 0.40 used in the 1968 and 1977 editions of the Specification, corresponded to a factor of safety of 3.0 on $E$. This constant was changed in the 1982 edition to 0.438 to correspond to the factor of safety of 2.74 associated with the Euler buckling constant of 0.30 for long columns. (Note: The beam constant of 0.438 = the column constant of 0.30 times the ratio 1.20/0.822). The constant in the formula for $C_k$ defining the slenderness factor above which inelastic stresses begin to occur also was changed at the same time from $\sqrt[\frac{1}{3}] {75}$, or 0.775, to 0.811 to reflect the same change in factor of safety basis. In addition, the 1982 and 1986 editions incorporated changes in the specific equations for $\ell_e$, as noted in the specific commentary given under 3.3.3.5.

In the 1991 edition, the short, intermediate and long beam equations for adjustment of bending design values for slenderness factor were replaced with a single continuous beam equation comparable to that introduced for columns in section 3.7.1. This change and further modifications and additions to the $\ell_e$ equations made in the 1991 edition are addressed in the Commentary for 3.3.3.5 and 3.3.3.8.

3.3.3.1 When the breadth of the bending member is equal to or greater than the depth, lateral buckling of the bending member is not a factor and bending moment capacity is limited by the applicable bending design value adjusted for factors other than slenderness (57,86).

3.3.3.2 The approximate depth to breadth rules for determining lateral support requirements for sawn lumber bending members given in 4.4.1 are alternate provisions to those of 3.3.3. The lateral support guidelines represented by the approximate rules have proven satisfactory in service for more than 40 years. The provisions of 3.3.3.2 and 4.4.1 may not give equivalent combinations of lateral support and allowable bending design values. Specific span and loading conditions should be checked to compare the relative restrictiveness of each.

3.3.3.3 When the compression edge of a bending member is continuously supported along its length and end bearing points are also restrained from rotation or displacement, lateral buckling under loads inducing compressive stresses in the supported edge is not a concern. However, the possibility of stress reversal, such as that associated with wind loadings, should be
fully considered to assure that what is the tension side of the bending member under the predominant loading case also is adequately supported to carry any expected compressive forces. Bending members with very large depth to breath ratios, such that perpendicular to grain buckling of the member along its depth (web buckling) may be a factor, should be avoided.

3.3.3.4 The unsupported length \( l_u \) of a bending member laterally supported at points of end bearing and loaded through uniformly spaced purlins or similar framing members that are appropriately connected to the top face of the bending member, or the side of the bending member by hangers or ledgers, to prevent lateral translation is the distance between such purlins or framing members (85).

3.3.3.5 Formulas for determining the effective span length, \( l_e \), from the unsupported length, \( l_u \), in the 1968 and 1977 editions of the Specification were for an \( l_u/d \) ratio of 17. Formulas for other \( l_u/d \) ratios were added in 1982. To correct a misinterpretation of a recommendation of the original researchers that resulted in unnecessarily conservative \( l_e \) values, the formulas for determining \( l_e \) were revised and generalized to apply to any \( l_u/d \) ratio in the 1986 edition. Also included in this edition were new formulas for single span and cantilever beams with any load condition.

The 1991 edition continues the \( l_e \) formulas from the 1986 edition but limits \( l_e \) values for relatively short, deep bending members to those for bending members having an \( l_u/d \) of 7. This limit was added because use of the general equations resulted in unrealistically large \( l_e \) values for certain short, deep bending members and the fact that this beam length to depth ratio was the lower limit of the experimental data used to verify the methodology (86). In addition, the 1991 edition provides \( l_e \) formulas for single span beams with various numbers of equally spaced and equal concentrated loads where lateral support occurs at each load point. These formulas, based on analysis of the effect of equally spaced purlins on beam buckling (85), show the increased capacity that is obtained by having lateral support at the point of application of a concentrated load.

The constants in the formulas for effective length in Table 3.3.3 include the 15 percent increase in \( l_u \) for incomplete torsional restraint at lateral support points. The formulas given in the table apply to the condition of loads applied to the top 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.

3.3.3.6 The beam slenderness ratio, \( R_B \), calculated as:

\[
R_B = \sqrt{\frac{l_e d}{b^2}}
\]

is comparable to the slenderness ratio for solid columns, \( l_e/d \), in terms of its effect on allowable design strength.

3.3.3.7 Limiting the beam slenderness ratio of bending members, \( R_B \), to a maximum value of 50 is a good practice recommendation that has been a requirement in the beam stability provisions of the Specification since 1968 to preclude the design of bending members with high buckling potential. For example, a 2x16 bending member having an unsupported length \( l_u \) of 16 feet has a slenderness ratio of approximately 50. The limit was originally recommended to parallel the limit on the \( l_e/d \) slenderness ratio for columns of 50 (86).

3.3.3.8 The single beam stability factor equation is applicable to all bending member slenderness ratio values \( R_B \) and replaces the short, intermediate and long beam equations given in previous editions for determining the effects of slenderness on allowable bending design values. The equation is of the same form as the continuous equation for calculating the column stability factor in 3.7.1.5. This column equation was first proposed in alternate form as a new method of determining the buckling stress of columns of any material in the elastic and inelastic range by a Finnish researcher in 1956 (226). The column equation has as its upper limit the tabulated compression design value parallel to grain associated with a crushing mode of failure at small \( l_e/d \) ratios and the critical buckling design value (Euler column buckling stress) at large \( l_e/d \) ratios. Continuity between these extremes of slenderness is obtained by assuming a curvilinear stress-strain relationship where the degree of nonlinearity caused by inelasticity, nonuniform material structure and initial eccentricity can be modeled through a single parameter "\( e \)". This continuous column equation form has been applied to bending members through substituting tabulated bending design values for tabulated compression parallel to grain (crushing) design values, critical (Euler) beam buckling design values for critical (Euler) column buckling design values, and selection of a value of "\( e \)" considered representative of bending member behavior. The general beam stability factor equation thus becomes
\[ C_L = \frac{1 + \left( F_{be}/F_b^* \right)}{2c} \]

(C3.3-11)

where:

- \( F_b^* \) = tabulated bending design value multiplied by all applicable adjustment factors except \( C_f, C_V, C_L \), psi
- \( F_{be} \) = critical buckling design value for bending members, psi

\[ F_{be} = \frac{K_{be} E}{R_B^2} \]

- \( K_{be} \) = Euler buckling coefficient for beams
- \( c \) = nonlinear parameter for beams
  - \( c = 0.95 \)

The 0.95 value for the parameter "c" is based on the generally satisfactory experience with bending members designed using the slenderness factor adjustment provisions included in previous editions of the Specification. A value of 0.95 for the parameter gives \( C_L \) values reasonably similar to equivalent values obtained from the previous specified short, intermediate and long beam equations.

For visually graded lumber, the Euler buckling coefficient for beams, \( K_{be} \), is 0.438. As shown by the above equation, the critical (Euler) beam buckling design value, \( F_{be} \), and the adjusted bending design value used in the beam stability factor equation are the same as the comparable values in the slenderness factor equations in the 1986 and earlier editions (see Equations C3.3-5 - C3.3-9). As shown in Figure C3.3-1, the continuous beam stability factor equation (Equation C3.3-11) gives lower values of \( C_L \) in the intermediate slenderness ratio ranges than comparable values obtained from earlier editions of the Specification.

Beam stability factors obtained from the equation in the 1991 edition are in general agreement with comparable values developed using the beam buckling equations developed in 1931 (57,184) when the latter are entered with a 15 percent increase of \( l_u \), an \( E/G \) ratio of 16 is assumed, and a 2.74 reduction factor on \( E \) is used (see Commentary under 3.3.3 for discussion of these assignments). For example, a 2x14 bending member with \( F_b \) of 1200 psi, \( E \) of 1,800,000 psi with an unsupported length of 168 inches and carrying a uniformly distributed load has a \( C_L \) of 0.347 based on 3.3.3.8 of the 1991 Specification and an equivalent \( C_L \) of 0.357 based on the 1931 methodology. For the same bending member and span carrying a concentrated load at the center, the \( C_L \) from 3.3.3.8 is 0.400 compared to a value of 0.426 based on the 1931 methodology. The differences between the two methodologies is attributed to the conservative simplifications made in the formulas for \( l_u \) (86) and the use of the more conservative continuous beam equation for the long beam examples rather than direct application of the Euler beam buckling equation.

For sawn lumber bending members, the \( C_L \) equation of 3.3.3.8 is entered with an \( F_{be}^* \) adjusted for all \( C \) factors except \( C_f \) and \( C_{fl} \). The latter is the flatwise use adjustment factor, which is applied independently to the bending design value when lumber is used in the flatwise orientation. A bending member is not subject to lateral buckling in this orientation since the breadth of the member is greater than its depth, therefore, \( C_L = 1.0 \) (see Commentary section 3.3.3.1). Thus, size adjustment factors for sawn lumber, \( C_F \), are to be applied simultaneously with the beam stability factor, \( C_L \). This is a more conservative practice than that used in earlier editions of the Specification, wherein size adjustments for sawn lumber bending members over 12 inches in depth were not applied simultaneously with slenderness factor adjustment. The previous approach was based on transfer of the original application of the slenderness factor methodology for glued laminated timber in the 1968 edition to large sawn lumber bending members. In the 1991 edition, the practice of not applying volume (size) adjustments,
The 0.438 value for the Euler buckling coefficient for beams is applicable to visually graded lumber and machine evaluated lumber (MEL). It includes a reduction of 2.74 on tabulated modulus of elasticity values and is related to the equivalent Euler buckling coefficient for columns of 0.30 by the ratio 1.2/0.822 (see background discussion in Commentary for 3.3.3). The modulus of elasticity of visually graded sawn lumber and MEL is considered to have a coefficient of variation of 0.11 or less, the 2.74 factor thus represents an approximate 5 percent lower exclusion value on pure bending modulus of elasticity (1.03 times tabulated \( E \)) and a 1.66 factor of safety.

For glued laminated timber, machine stress rated lumber (MSR) or other products having a coefficient of variation in modulus of elasticity of 0.11 or less, the Euler buckling coefficient for beams is 0.609. This also represents an approximate 5 percent lower exclusion value on pure bending modulus of elasticity and a factor of safety of 1.66. The 0.609 coefficient is related to the 0.438 coefficient for visually graded lumber by the ratio

\[
\frac{[1 - 0.11(1.645)]}{[1 - 0.25(1.645)]} = 1.39
\]

Examples C3.3-1 and C3.3-2 illustrate the use of beam stability provisions for bending members. Other design considerations for bending members (i.e., shear, compression perpendicular to grain, etc.) are not covered in these examples, but are addressed in other sections of the Commentary.

3.4-BENDING MEMBERS - SHEAR

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

3.4.1.1 Shear strength perpendicular to the grain, alternatively termed 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 in structural wood bending members, parallel to grain shear strength is always the limiting case. Tabulated shear design values, \( F_{y} \), 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. Such 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.

3.4.1.2 The limitation on application of shear design provisions to solid flexural members such as sawn lumber, glued laminated timber and mechanically laminated timber bending members was added to the Specification in the 1986 edition. Built-up components, such as trusses, were 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. Because of the difficulty of making a general determination for all truss designs of the effects of stress concentrations and embedded metal connectors at such locations, and of the applicability of the general practice of ignoring loads close to supports, shear design at supports for proprietary built-up components is required to be based on testing, theoretical analysis and/or documented experience related to specific design configurations.

3.4.2-Shear Design Equations

Actual shear stress parallel to grain, \( f \), in a circular bending member may be determined as:

\[
f_{v} = \frac{4V}{3A}
\]

where:

\( V = \) shear force

\( A = \) cross-sectional area of circular member

3.4.3-Shear Force

3.4.3.1 (a) Ignoring loads within a bending member depth of the support face for purposes of calculating shear force has been a provision of the Specification since the 1944 edition. The allowance assumes such loads are carried directly to the support by diagonal compression through the depth of the
Example C3.3-1
A Select Structural Southern Pine 4×16 beam on a 20 ft span supports a hoist located at the center of the span. Determine the maximum allowable load on the hoist (including its weight) based on bending. Assume normal load duration. Lateral support is provided at the ends only.

\[ F_b = 1900 \text{ psi} \quad E = 1,800,000 \text{ psi} \quad (\text{Table 4B}) \]

\[ C_r = (0.9)(1.1) \quad C_o = 1.0 \quad A = 53.38 \text{ in}^2 \quad S = 135.7 \text{ in}^3 \]

**Beam Stability Factor** \( C_L \) (3.3.3)

\[ F_b^* = F_b C_o C_r = (1900)(1.0)(0.9)(1.1) = 1880 \text{ psi} \]

\[ \frac{\ell_c}{d} = \frac{(20)(12)}{(15.25)} = 15.7 > 7 \]

\[ K_{Kf} = 0.438 \]

\[ F_{Kf} = \frac{K_{Kf} E'}{R_b^2} = \frac{(0.438)(1,800,000)}{(21.6)^2} = 1691 \text{ psi} \]

\[ C_L = 1 + \frac{(F_{Kf}/F_b^*)}{1.9} - \frac{1 + (F_{Kf}/F_b^*)^2}{1.9} - \frac{F_{Kf}/F_b^*}{0.95} \]

**Allowable Bending Design Value,** \( F_b' \) (Table 2.3.1)

\[ F_b' = F_b C_o C_r C_f = (1900)(1.0)(0.770)(0.9)(1.1) = 1448 \text{ psi} \]

**Maximum Moment**

Assume density of beam = 37.5 lb/ft<sup>3</sup>

weight of beam = (37.5)(53.38/144) = 13.9 lb/ft

hoist plus payload = \( P \)

\[ M_{tur} = P /4 + w \ell^2 /8 \]

= ((1448)(13.57) - 8340) /60

= 3136 lb - 3100 lb

**Maximum Allowable Load**

\[ M_{allow} = F_b' S \]

Substituting,

\[ P = (F_b' S - 8340)/60 \]

= ((1448)(13.57) - 8340) /60

= 3136 lb - 3100 lb

Total allowable concentrated load = 3100 lb (hoist plus payload)

---

Example C3.3-2
A Select Structural Southern Pine 2×14 beam on a 16 ft span carries five 500 lb (DL+SL) concentrated loads from purlins spaced at 32 in. on center (1/6 points). Determine if the member is adequate for bending. Lateral support is provided at the purlins and the supports.

\[ F_b = 1900 \text{ psi} \quad E = 1,800,000 \text{ psi} \quad (\text{Table 4B}) \]

\[ C_r = 0.9 \quad C_o = 1.15 \quad A = 19.88 \text{ in}^2 \quad S = 43.89 \text{ in}^3 \]

**Beam Stability Factor** \( C_L \) (3.3.3)

\[ F_b^* = F_b C_o C_r = (1900)(1.15)(0.9) = 1967 \text{ psi} \]

\[ \ell_c = 32 \text{ in.} \]

\[ \ell_c = 1.73 \ell_c = 1.73(32) = 55.4 \text{ in.} \quad (\text{Table 3.3.3}) \]

\[ R_b = \frac{\ell_c d}{b^2} = \frac{(55.4)(13.25)}{(15.25)^2} = 18.1 \]

\[ K_{Kf} = 0.438 \]

\[ F_{Kf} = \frac{K_{Kf} E'}{R_b^2} = \frac{(0.438)(1,800,000)}{(18.1)^2} = 2418 \text{ psi} \]

\[ C_L = 1 + \frac{F_{Kf}/F_b^*}{1.9} - \frac{1 + (F_{Kf}/F_b^*)^2}{1.9} - \frac{F_{Kf}/F_b^*}{0.95} \]

\[ = 1 + 2418/1967 - \frac{1 + (2418/1967)^2}{1.9} - \frac{2418/1967}{0.95} \]

= 0.886

**Allowable Bending Design Value,** \( F_b' \) (Table 2.3.1)

\[ F_b' = F_b C_o C_r C_f = (1900)(1.15)(0.886)(0.9) = 1742 \text{ psi} \]
Example C3.3-2 (cont.)

Maximum Moment

Assume density of beam = 37.5 lb/ft³
weight of beam = (37.5)(19.88/144) = 5.2 lb/ft
Purlin loads = P (five at 1/6 points) = 500 lb

\[ M_{\text{max}} = P(0.75P) + \frac{wL^2}{8} \]
\[ = (500)(0.75)(16)(12) + (5.2)(16)^2(12)/8 \]
\[ = 72,000 + 1,988 = 73,988 \text{ in-lb} \]

Actual Bending Stress, \( f_b \)

\[ f_b = \frac{M_{\text{max}}}{S} \]
\[ = \frac{73,988}{43.89} \]
\[ = 1686 \text{ psi} < F_p = 1742 \text{ psi} \text{ ok} \]

2x14 member satisfies NDS bending criteria

bending member. The clarification that such practice is permitted only for bending members fully supported on one surface and loads applied to the opposite surface, the condition obtained with solid single structural members, was added in the 1982 edition.

3.4.3.1 (b) Earliest editions of the Specification placed the critical moving load at three beam depths from the support. This procedure was based on shear analysis of checked or end-split sawn lumber bending members that shows part of the shear force near the support is carried to the support by the upper and lower portions of the bending member independently of the portion of the bending member at the neutral axis (see Appendix E of the Specification). The general shear stress equation does not account for such a redistribution of stress. Beginning with the 1960 edition, placement of the critical moving load was changed to one beam depth from the support to cover the general case of shear force for any material. The special case of critical moving loads in checked or end-split lumber bending members is treated separately in the Specification (see Section 4.4.2).

3.4.3.2 This provision of the Specification establishes a limiting condition for use of the two-beam shear force equations for sawn lumber given in 4.4.2. The criterion requires the actual shear stress parallel to grain, \( f_v \), calculated in accordance with 3.4.2 and 3.4.3.1 using conventional shear force values to be equal to or lower than the maximum allowable shear design value for an unsplit or unchecked member of the species and grade of lumber being considered. Such stresses are equivalent to twice the shear design values parallel to grain tabulated in the Supplement to the Specification. This same limitation was included in the 1986 edition.

Interpretation of how the two-beam provisions of the Specification are applied differed in the 1982 and earlier editions from that in the 1986 and 1991 editions. However, these early provisions also contained limits on the maximum shear design value parallel to grain that could be used. From 1944 through the 1973 editions, the shear design value parallel to grain for an equivalent unsplit member also was the upper limit. In the 1977 and 1982 editions a limit equivalent to 75 percent of the tabulated shear design value parallel to grain was employed. The two-beam provisions and associated limitations in the current Specification were first published in the 1986 edition after reevaluation of the original two-beam research and review of early interpretative analyses (see Commentary for 4.4.2).

3.4.4 Shear Design for Notched Bending Members

3.4.4.1 The equation for calculating actual shear stress parallel to grain in rectangular bending members containing end notches on the tension face has been a provision of the Specification since the 1944 edition. In this calculation, the actual shear stress determined by entering the normal shear equation with the depth of the bending member above the notch is increased by the ratio of original (depth away from the notch) to notched depth. Thus for a given shear force and depth of notched member, shear capacity is reduced as the amount of material below the notch is increased. This behavior, verified by tests of bending members of clear wood of two species notched to various depths (158), is related to the concentration of tension and shear stresses occurring at the reentrant corner of the notch.

The equation is applicable to any rectangular bending member notched on the tension face. Because of the reduction in shear strength associated with the stress concentrations occurring at the notch, shear capacity is more likely to govern overall bending member capacity of notched beams with span to depth ratio of 12 or less than equivalent unnotched bending members. End notches in sawn lumber bending members are limited to 1/4 the beam depth (see 3.2.3.2). Limiting the length of rectangular end notches
(measured from the end of the bending member) to the original depth of the member is advisable.

Recent research on tension-side end notched beams has examined the combined effects of loading type, end support, and beam and notch variables such as beam height, fractional notch depth, notch radius, and notch location (235). Results indicate that the effects of these factors can be represented by a material parameter established from destructive tests of notched beams. The strength equation developed from this research provides design criteria for beams notched on the tension face that explicitly consider notch geometries and the relative proportion of moment and shear at the notch sections. Current provisions of the Specification do not provide for this type of analysis. Designers who use the alternative new methodology have the responsibility for establishing its appropriateness and adequacy for particular design cases.

Shear strength of bending members is less affected by rectangular end notches on the compression face than on the tension face. Tests have shown that the actual shear stress parallel to grain in end-notched beams on the compression face may be determined from the following equation (62):

\[ f_v = \frac{3V}{2bg} \]  

(C3.4-2)

where:

- \( V \) = shear force
- \( b \) = width of bending member
- \( g = [d - (d - d_n)(e/d_n)] \)

with

- \( d \) = depth of bending member
- \( d_n \) = depth of bending member below the notch
- \( e \) = distance between end of notch and face of support

For a bending member with a compression face end-notch of one-quarter the beam depth extending one-half the depth of the bending member from the support, the value of the effective depth \( g \) in the foregoing equation is \((15/18)d\). For a tension face end-notch of one-quarter the beam depth, the effective beam depth from Equation 3.4-3 is \((9/16)d\).

In all cases, the shear force, \( V \), used in the calculation of actual shear stress parallel to grain in end-notched bending members shall not be based on the alternate two-beam shear provisions of 4.4.2 for sawn lumber and shall include all loads located within a beam depth from the face of the support. These limitations were first introduced in the 1977 and 1982 editions, respectively.

3.4.4.2 The equation for calculating actual shear stress parallel to grain in members of circular cross section end-notched on the tension face is a new provision of the Specification. This equation parallels that for end-notched rectangular bending members with the area of the circular member \( A_n \) at the notch replacing the width \( b \) and depth at the notch \( d_n \) in the equation for the rectangular beam equation as shown below.

rectangular cross section:

\[ f_v = \frac{3V}{2bd_n} \left( \frac{d}{d_n} \right) \]  

(C3.4-3)

circular cross section:

\[ f_v = \frac{3V}{2A_n} \left( \frac{d}{d_n} \right) \]  

(C3.4-4)

where:

- \( d_n \) = depth of bending member above the notch
- \( A_n \) = cross-sectional area of circular bending member at notch

and

other symbols as defined under Equation C3.4-2

Although it is known that the boundary conditions assumed in the derivation of the standard \( VQ/Ib \) equation are not met by unnotched circular cross sections, it has been shown that shear stresses parallel to grain near the neutral axis of an unnotched circular member that are calculated using this standard theory, or \( 4V/3A \), are within 5 percent of actual stresses (149).

The equation for end-notched members of circular cross section (Equation C3.4-4) is an approximate expression that is considered to provide for reasonable allowable shear loads. The cross-sectional area of a circular bending member above a notch, \( A_n \), may be calculated from the formula:

\[ A_n = \frac{d^2}{4} (\pi - \alpha_j + \cos \alpha_j \sin \alpha_j) \]  

(C3.4-5)

where:

- \( d \) = diameter of unnotched circular bending member
\[ \alpha_i = \text{one-half the angle subtending the chord located at } d/2 \text{ minus the notch depth } (c) \text{ below the unnotched center of the circular bending member} \]
\[ = \cos^{-1}(1 - 2c/d) \]

For a circular member containing a 1/4 depth (1/2 radius) end notch, the approximate equation, C3.4.4, without the application of the magnification factor of \( d/d_n \), results in actual shear stresses parallel to grain that are 8 percent greater than those determined through application of the conventional \( VQ/Ib \) theory to truncated circular sections. Also, because of the curvature of the member in the perpendicular plane, the stress concentrations occurring at end notches in members of circular cross section are viewed as somewhat less severe than those in end-notched rectangular sections. Thus the provisions of 3.4.4.2, which utilize the same magnification factor as that used for notches in rectangular members, are judged a conservative basis for shear design of end-notched circular bending members.

3.4.4.3 Procedures used to calculate actual shear stresses parallel to grain 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.

3.4.4.4 Use of tapered rather than squared end notches have been shown by test to greatly reduce the magnification factor for stress concentrations represented by the \( d/d_n \) factor in the shear stress equation for end notched members in 3.4.4.1. Rounding of the cut to the center of the bending member effectively eliminates the effect of stress concentrations and the actual shear stress parallel to grain is reduced to that which occurs with a bending member equivalent to the depth \( (d_n) \) of the notched bending member (158).

3.4.5 Shear Design for Bending Members at Connections

3.4.5.1 Provisions for calculating actual shear stress parallel to grain in bending members at connections were made progressively more conservative between the 1944 and 1977 editions of the Specification but have remained essentially unchanged since that time. In the 1944 edition, a 50 percent increase in tabulated shear design values parallel to grain, \( F_v \), was allowed for shear design associated with connections and use of two-beam shear procedures (see 4.4.2) was permitted.

In 1947, actual shear stress parallel to grain at all connections was required to be calculated as

\[ f_v = \frac{V}{bd_e} \]

where \( d_e \) was the depth of the member less the distance from the unloaded edge to the edge or center of the nearest fastening, the same effective distance as defined in the present edition. In the 1957 edition, the equation was changed to

\[ f_v = \frac{3V}{2bd_e} \]

In 1973, the 50 percent increase in tabulated shear design values parallel to grain was limited to only those connections which were at least five times the depth of the member from its end. Additionally, the actual shear stress parallel to grain determined on the basis of the full cross section at such joints was required to be equal to or less than the tabulated shear design value parallel to grain without the 50 percent increase.

In 1977, a modified equation for calculating actual shear stress parallel to grain at connections located less than five times the depth of the member from its end was introduced. This equation:

\[ f_v = \frac{3V}{2bd_e} \left( \frac{d}{d_e} \right) \]

is similar to that for end-notched rectangular bending members where the ratio \( d/d_e \) is comparable to the magnification factor \( d/d_n \). The disallowance of the 50 percent increase in tabulated shear design values parallel to grain when checking actual shear at connections less than five times the depth of the member from its end was not changed with introduction of the new equation.

Also in 1977, use of the alternate two-beam shear provisions for determining \( V \) in sawn lumber at connections at any location was specifically excluded. Use of the general provision allowing loads within a beam depth from the support to be neglected in shear at connections was specifically disallowed in 1982.

Provisions for shear design at connections, as revised through the 1982 edition, have been carried forward to the 1991 edition.

The 50 percent increase in tabulated shear design values parallel to grain allowed in some joint details is considered to be based on the judgment that the stress concentrations present at such joints are of lower
magnitude than those assumed (4/9 reduction on block shear specimen values) in the establishment of tabulated shear design values parallel to grain. The more restrictive provisions introduced for shear design at connections since 1944, including the limitation on application of the increase to tabulated shear design values parallel to grain and the addition of the $d/d_e$ stress magnification factor, reflect conservative assessments of (i) field experience with large bending members and (ii) the effects of shrinkage or potential splitting and excessive tension perpendicular to grain stresses at connections, particularly those near the ends of the members.

Although neither the provisions of 3.4.3.1 allowing loads within a beam depth from the support to be ignored nor the two-beam provisions of 4.4.2 are applicable to shear design at connections, an allowable shear design value parallel to grain, $F'_{v}$, for members with limited splits, shakes or checks ($C_H$ of 2.0) may be used for connections in sawn lumber bending members. This adjusted allowable shear design value ($F_{v} \times C_H$) is cumulative with the 50 percent increase in tabulated shear design values parallel to grain provided in 3.4.5.1 (b) when checking the actual shear stress from Equation 3.4-6 (Commentary Equation C3.4-8); as well as when checking the actual shear stress based on the gross section area of the member as required by 3.4.5.1 (b).

When the ratio $d/d_e$ exceeds 0.67 for a connection located five or more times the depth of any member from its end, the actual shear stress parallel to grain, $f_{v}$, based on gross section will limit the design rather than the actual stress based on Equation 3.4-6 and the related 50 percent increase in the allowable shear design value parallel to grain. However, in most loading situations, the gross section actual shear stress parallel to grain at locations five or more times the depth of the bending member from its end will not be more limiting than the maximum actual shear stress occurring at supports.

Examples C3.4-1 and C3.4-2 illustrate the use of shear design provisions for bending members at connections.

3.4.5.2 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 3.4.4. This requirement was introduced in the Specification in the 1973 edition.

3.5-BENDING MEMBERS - DEFLECTION

3.5.1-Deflection Calculations

Allowable Modulus of Elasticity. Tabulated modulus of elasticity design values, $E$, in the Specification for sawn lumber, glued laminated timber and round timber are average values for the species and grade combinations designated. Individual pieces within each such combination will have modulus of elasticity values both higher and lower than the tabulated mean value.

Tabulated 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 21 under uniformly distributed load. Assuming a modulus of elasticity to modulus of rigidity ratio $(E/G)$ of 16, pure bending modulus of elasticity may be taken as 1.03 times the tabulated value. Standard methods for adjusting modulus of elasticity to other load and span-depth conditions are available (21).

Example C3.4-1

A No. 1 Douglas Fir-Larch 2×10 beam, supported by two 1/2-in. bolts in a clip angle connection attached to a girder, has an end reaction of 650 lb (DL+LL). Assume a normal load duration. Check shear in the member at the connection. The connection meets NDS criteria for bolt load, spacing and end and edge distance. The bolt on the unloaded edge is 2" from the edge of the member.

$$F_v = 95 \text{ psi} \quad \text{(Table 4A)}$$

$$C_D = 1.0$$

$$b = 1.5 \text{ in.}$$

$$d = 9.25 \text{ in.}$$

Allowable Shear Design Value Parallel to Grain, $F'_{v}$

$$F'_{v} = F_v C_D = (95)(1.0) = 95 \text{ psi} \quad \text{(Table 2.3.1)}$$

Actual Shear Stress Parallel to Grain at Connection, $f_{v}$

$$\text{end distance to bolt } = 2.5 \text{ in. } < 5d = 5(9.25) = 46.25 \text{ in. therefore, Equation 3.4-5 controls}$$

$$\text{edge distance to bolt } = 2.0 \text{ in. } > 1.5d = 1.5(0.5) = 0.75 \text{ in. therefore, ok} \quad \text{(8.5.3)}$$

$$d_e = d - \text{distance from unloaded edge to nearest bolt}$$

$$= 9.25 - 2.0 = 7.25 \text{ in.}$$

$$V = \text{reaction } = 650 \text{ lbs} \quad \text{(cont.)}$$
Example C3.4-1 (cont.)

Recompute \( f_v \)

\[
d_s = d - \text{distance from unloaded edge to nearest bolt} = 9.25 - 1.25 = 8.0 \text{ in.}
\]

\[
f_v = \frac{3V}{2bd_s} \left( \frac{d}{d_s} \right) = \frac{(3)(650)}{2(1.5)(8.0)} \left( \frac{9.25}{8.0} \right) = 94 \text{ psi} < F_v' = 95 \text{ psi} \quad \text{ok}
\]

Member satisfies NDS shear criteria at the connection if the bottom bolt is located 1.25 in. from the unloaded edge.

Example C3.4-2

A Select Structural Southern Pine 4x10 beam, on a span of 141 in., supports a uniform load of 80 lb/ft (DL+LL) plus two equal concentrated loads of 1600 lb (LL) at the one-third points from tension stirrups supporting a catwalk. The tension stirrups are connected to the member with a steel U-plate with two 1/2 in. bolts at each connection as shown. Check shear in the member at one of the connections. Assume 3 in. supports and normal load duration.

Allowable Shear Design Value Parallel to Grain, \( F_v' \)

\[
F_v' = F_i C_D C_H = (90)(1.0) = 90 \text{ psi}
\]

Actual Shear Stress at Connection, \( f_v \)

\[
d_s = d - \text{distance from unloaded edge to nearest bolt} = 9.25 - 1.25 = 8.0 \text{ in.}
\]

\[
f_v = \frac{3V}{2bd_s} \left( \frac{d}{d_s} \right) = \frac{(3)(1757)}{2(3.5)(6.625)} = 113.7 \text{ psi} < 1.5 F_v' = 1.5(90) = 135 \text{ psi} \quad \text{ok}
\]

Member satisfies NDS shear criteria at the connection if the bottom bolt is located 1.25 in. from the unloaded edge.
The use of average modulus of elasticity values for the deflection design of wood bending members has been general practice for more than 50 years. Experience has shown that such values provide an adequate measure of the immediate deflection of bending members used in normal wood structural applications. It should be noted that a reduced modulus of elasticity value is used in beam stability analyses. The reduction is incorporated in the Euler buckling coefficient for beams, \( K_{BE} \), and is equivalent to that incorporated in the buckling coefficient for columns (see Commentary for 3.3.3).

**Floor Joist or Beam Deflection.** In the particular case of floor joists or beams, maximum allowable spans are commonly associated with a deflection limitation of \( l/360 \) under uniform live load, or

\[
\frac{l}{360} = \frac{5wL^4}{384EI} \tag{C3.5-1}
\]

For constant \( w \) and \( E \), this standard criterion requires that \( I \) change in proportion to \( L^3 \) as the span increases. This means that the deflection under a constant concentrated load \( P \) applied at the center of the span of a joist or beam having the same \( E \), or

\[
\Delta_c = \frac{PL^3}{48EI} \tag{C3.5-2}
\]

will remain constant as span increases. Thus, for a given species and grade combination and constant uniform load, there is no change in the absolute deflection associated with a given concentrated load as span is increased if the \( l/360 \) criterion is met.

Notwithstanding the concentrated load deflection limit imposed by the \( l/360 \) requirement, the deflection performance of floors subjected to walking traffic loads and designed to meet the \( l/360 \) criterion for code specified uniform load may not be acceptable to some owners or occupants, particularly where beam spans exceed 14 feet. A comprehensive study of home owner response to levels of floor performance conducted in Canada found that deflection under a concentrated load is the best available measure of a floor’s acceptability (142). The study indicated that an increasingly lower absolute deflection under a constant concentrated load was required as spans increase above 10 feet if the range of preferences of most home owners was to be covered. The use by many builders in the United States of joists a size deeper than those required by the code for a given span also is indicative of such performance preferences.

A comprehensive computer design methodology for light frame wood floor systems, which accounts for the composite action of sheathing and framing members joined with varying connector stiffness, is available that provides optional spans that are shorter than those permitted by the \( l/360 \) criterion in order to meet the stiffness performance desired by many users (125). Differences between the concentrated load deflection permitted by the optional criterion and the \( l/360 \) criterion increase as the span increases. For example, the \( l/360 \) criterion imposes the same absolute deflection limit for a fixed concentrated load applied to joists used on spans of 10, 12 and 20 feet (see Commentary related to Equation C3.5-2). The optional methodology limits such concentrated load deflections at spans of 12 feet and 20 feet to 72 percent and 38 percent, respectively, of that for a 10 foot span.

In view of the foregoing, it should be understood that the use of the \( l/360 \) deflection criterion for uniform load and the modulus of elasticity values tabulated in the Specification do not necessarily provide dynamic floor performance that will be found acceptable to all users. Such performance must be evaluated under other criteria.

**Critical Applications.** In certain applications, such as in uses of closely engineered structural components on long spans, deflection may be a critical design consideration. If a maximum immediate deflection under load must be assured, use of a reduced modulus of elasticity value may be appropriate. Such values may be developed using the following coefficients of variation:

<table>
<thead>
<tr>
<th>Material Type</th>
<th>COV_E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visually graded sawn lumber and machine evaluated lumber (MEL)</td>
<td>0.25</td>
</tr>
<tr>
<td>Machine stress-rated sawn lumber (MSR)</td>
<td>0.11</td>
</tr>
<tr>
<td>Glued laminated timber (six or more laminations)</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Machine evaluated lumber (MEL) is assumed to have the foregoing coefficient of variation (\( COV_E \)) unless a different coefficient is specifically included in the grade mark.

Reducing average modulus of elasticity design values tabulated in the Specification for a species-grade combination by the product of the average value and 1.0 or 1.65 times the applicable coefficient of variation provides an estimate of the modulus of elasticity that is nominally exceeded by 84 percent or 95 percent, respectively, of the individual pieces in the combination.
3.5.2-Long Term Loading

The modulus of elasticity values tabulated in the specification 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 (creep). At moderate to low levels of sustained stress and under stable environmental conditions, the rate of creep will decrease over time. Such creep can be described (73,87) by a power function of the form

\[ \Delta_{cr} = At^B \]  

where:

- \( \Delta_{cr} \) = relative creep
- \( t \) = time
- \( A, B \) = constants

Where creep is decreasing over time, total creep occurring in a specific period of time is approximately proportional to stress level (168,221). Total bending creep increases with increase in moisture content (44,176) and temperature (154); and is greater under variable compared to constant relative humidity conditions (154). 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 (13,18,176,221).

Such estimated reductions are given by the following formulas:

\[ E_{0.16} = E - E(1.0)(COV_E) \]  
\[ E_{0.05} = E - E(1.65)(COV_E) \]  

(3.5-3)  
(3.5-4)

3.6-COMPRESSION MEMBERS - GENERAL

3.6.2-Column Classifications

3.6.2.1 Simple solid columns are defined as single piece members or those made of pieces properly 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.

3.6.2.2 Design provisions for spaced columns have been a part of the Specification since 1944.

3.6.2.3 Built-up mechanically laminated columns are not designed as solid columns. The strength of such columns is the sum of the strengths of the individual laminations except where specific additional procedures apply. The provisions of 15.3 for the design of bolted or nailed built-up mechanically laminated columns are new to the Specification. Previous editions referenced other methods (62).

3.6.3-Strength in Compression Parallel to Grain

Calculated actual compression stress parallel to grain, \( f_c \), based on net section area is required not to exceed the tabulated compression design value parallel to grain, \( F_c \), multiplied by all applicable adjustment factors except the column stability factor, \( C_p \). This provision is included to cover the case where the net or reduced section does not occur at the column length location most subject to potential buckling; and actual compression stress, \( f_c \), at such points are calculated on the basis of gross section area. Whether \( f_c \) at the critical buckling location is based on net or gross section area, it is to be understood that this stress shall not exceed the tabulated compression parallel to grain design value multiplied by all applicable adjustment factors, including the column stability factor, \( C_p \).

Prior to 1977, the Specification provided that calculated allowable compression design values parallel to grain which included the effect of slenderness ratio, \( \ell_c/d \), could be adjusted by the load duration factor. This early practice was based on the assessment that the reduction factor incorporated in the Euler buckling
coefficient for columns, $K_{ce}$, accounted for any long-time loading effects and that increases could be made for short-time loads without encroaching on the factor of safety (62). The practice began to be interpreted by some users to mean that modulus of elasticity design values were subject to the same load duration adjustments as strength design values. To avoid such misinterpretations and to clearly separate load duration factors for strength from creep effects or deflection under long term loading, the 1977 and subsequent editions of the Specification have provided for application of load duration adjustments only to tabulated compression design values parallel to grain. Thus no adjustments for load duration are embedded in column stresses limited by stiffness or buckling potential. This is evidenced by the column stability factor, $C_p$, computation procedure of 3.7.1.5 and the separation of this factor from the load duration factor, $C_d$.

**3.6.4-Compression Members Bearing End to End**

Bearing stresses occurring at the ends of compression members are limited by bearing design values parallel to grain ($F_{ce}$) and differ from compression design values parallel to grain ($F_{ge}$) which are applicable to columns. The latter include the effects of knots and other permitted grade characteristics while the former are based on clear wood properties. As provided in 3.10.1, use of bearing design values parallel to grain is predicated on the presence of adequate lateral support to resist moments at the joint.

**3.6.6-Column Bracing**

Alternative methods of bracing columns supporting trusses or beams are described in Appendix A.9 of the Specification. Unless fixity at the column base or diaphragm action of walls and connecting roof sheathing is provided, columns resisting lateral loads on the side walls should be braced in the direction of the trusses by knee braces (179) or by extending the column to the top chord of a sufficiently deep parallel chord truss. If the sheathing material does not provide adequate resistance to end-wall lateral loads, cross or knee bracing between column members should be provided. Related information on bracing of trusses is given in Appendix A.10.

Designing vertical cross bracing and horizontal struts between trusses to withstand a horizontal compression load equivalent to 10 percent of the stress (load) on the bottom chord of the truss divided by the product of the number of lines (along the span of the truss) of cross bracing and the number of cross braces in each line has been recommended (179).

---

**3.6.7-Lateral Support of Arches, Studs and Compression Chords of Trusses**

Provisions for determining slenderness ratio, $\ell_c/d_c$, of laterally supported arches and compression chords have been part of the Specification since the first edition. These provisions were removed from the body of the Specification and placed in the Appendix in the 1982 edition. At this time the guidelines were revised to recognize lateral support provided by purlins (members spaced more than 24 inches apart) as well as roof joists and expanded to include studs used in light frame construction.

The provisions of Appendix A.11 provide that where roof joists or purlins are used between arches or compression chords, the larger of (i) the slenderness ratio based on the depth of the arch or chord and its length or (ii) the ratio based on the breadth of the beam and the distance between purlins or joists is used in the calculation of column stability factor, $C_p$. Where the depth of the arch or chord is used in the slenderness ratio calculation, the effective length, $\ell_c$, is the length between points of lateral support and/or points of contraflexure on the deflection curve in the plane of the depth dimension. Thus the slenderness ratio based on depth is that represented by the ratio $\ell_c/d_c$ in Figure 3F of the Specification. The ratio based on the breadth of the arch or chord and the distance between purlins or joists is that represented by the ratio $\ell_c/d_c$ in Figure 3F of the Specification.

Use of the depth of the arch or compression chord in the determination of $\ell_c/d_c$ rather than the breadth or thickness of the member was limited to roof joists prior to the 1982 edition. In these earlier editions, purlins (members spaced more than 24 inches apart) were specifically excluded from this provision. The spacing of roof joists (24 inches or less) was considered such that when these members were used, the $\ell_c/d_c$ ratio rather than the $\ell_c/d_c$ ratio was the limiting case. In the 1982 and subsequent editions, buckling about both the strong and weak axis of the arch or chord is specifically examined and therefore the lateral support provided by purlins as well as roof joists is taken into account.

When continuous decking or sheathing is installed on the top of the arch or compression chord, it is long standing practice to use the ratio of the depth of the arch or chord and the length between points of lateral support and/or points of contraflexure in the plane of the depth dimension, or $\ell_c/d_c$, as the slenderness ratio.
Use of the depth of the stud as the least dimension in calculating the slenderness ratio in determining the axial load-carrying capacity of normally sheathed or clad light frame wall systems also is long standing practice. Experience has shown that any code allowed thickness of gypsum board, hardwood plywood or other interior finish adequately fastened directly to studs will provide adequate lateral support of the stud across its thickness irrespective of the type or thickness of exterior sheathing and/or finish used.

3.7-SOLID COLUMNS

3.7.1-Column Stability Factor, $C_p$

3.7.1.2 Some standard practices for determining effective column length are given in Appendix A.11 of the Specification. 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, and/or between points of contraflexure on the deflection curve. 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 than translation fixed and rotation free, adjustment of actual column lengths by the recommended coefficients, $K$, in Appendix G of the Specification are permitted.

As shown in Table G-1 of 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 $K$ are the same as those used in steel design (3) 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. The level of conservatism used in the sixth case is equivalent to that used for the third case (rotation and translation fixed both ends). 

3.7.1.4 The limitation on the slenderness ratio of solid columns in permanent structures to 50, which has been a provision of the Specification since 1944, is a good practice that precludes the use of column designs susceptible to potential buckling from slight eccentricities in loading or nonuniform cross sectional properties. The $l_e/d$ limit of 50 is comparable to the $l/r$ limit of 200 ($l/d$ of 87) used in steel design (3).

Allowing a temporary $l_e/d$ ratio of 75 during construction is a new provision in the 1991 edition. This allowance is based on 15 years of satisfactory experience with temporary bracing of trusses installed in accordance with truss industry standards (185); 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, an $l/r$ ratio of 300 ($l/d$ of 87), or 50 percent higher than the permanent design maximum of 200 (3) is permitted during construction with cold-formed steel structural members (8). The allowable load on a column with an $l_e/d$ ratio of 75 is approximately 45 percent that of an equivalent column with an $l_e/d$ ratio of 50.

3.7.1.5 Background. Design provisions in the Specification for wood columns prior to 1991 generally were based on formulas for three classes of wood columns, defined in terms of slenderness ratio, $l_e/d$, as follows:

Short columns: $l_e/d \leq 11$

$$F_{c'} = F_c$$ (C3.7-1)

Intermediate columns: $11 < l_e/d \leq K$

$$F_{c'} = F_c \left[ 1 - \frac{1}{3} \left( \frac{l_e/d}{K} \right)^4 \right]$$ (C3.7-2)

in which

$$K = K_{ef} \sqrt{E/F_c}$$ (C3.7-3)

Long columns: $l_e/d > K$

$$F_{c'} = \frac{K_{ce}E}{(l_e/d)^2}$$ (C3.7-4)

where:

$F_c$ = tabulated compression parallel to grain design values adjusted for condition of use

$E$ = tabulated modulus of elasticity design value adjusted for condition of use

$K_{ce}$ = Euler buckling coefficient for columns

$K_{ef}$ = intermediate column coefficient

$$K_{ef} = \left[ \frac{3}{2} K_{ce} \right]$$
In the above formulas, $K$ is the minimum $\ell_c/d$ ratio at which the column will behave as an Euler column. This is defined as occurring at a stress of $2F_c/3$, the assumed elastic limit. At $K$, the 4th power intermediate column equation and the Euler formula are tangent.

The foregoing column formulas, established in 1930, were based on tests of one hundred and sixty 12 inch by 12 inch by 24 foot Douglas fir and southern pine timbers representing a full range of density and grade; and were supported by the findings of earlier column research (133). The results of the large column tests showed that limiting Euler column behavior to stresses below $2F_c/3$ was a conservative assumption for all but green or unseasoned material. Dry or seasoned columns showed Euler behavior extending to stresses over $4F_c/5$, which was consistent with the elastic limit for that material. Although assuming an elastic limit of 80 percent of ultimate would have resulted in use of an 8th power parabolic equation for intermediate columns, the 4th power equation was used in order to cover all seasoning conditions (133). The large column tests also showed that $\ell_c/d$ ratios of 11 or below had a negligible effect on column capacity.

Provisions in Earlier Editions. Column design provisions in the 1944 edition of the Specification were based on the short, intermediate and long column equations using constants of

- $K_{cE}$ (Euler buckling coefficient for columns) = 0.329
- $K_{dc}$ (intermediate column coefficient) = 0.702

These coefficients incorporate the wartime increase (20 percent) in permanent load design values authorized by the War Production Board (see Commentary for 2.3.2). The coefficient of 0.329 included a 2.5 reduction or safety factor on the Euler constant for rectangular sections of 0.822.

In 1953, the buckling coefficient in the long column equation, $K_{cE}$, was reduced to 0.300 to remove the war-time increase and to reflect the application of the normal loading concept (10 percent increase over permanent load values) to long column stresses (see Commentary for 3.6.3). Simplified column design procedures also were introduced in the 1953 edition. These new methods provided for use of the lower of the Euler buckling stress or the short column stress. Although the simplified procedure resulted in stresses up to 15 percent higher than those calculated by the intermediate column equation for intermediate slenderness ratios (62), the simplification was considered reasonable based on (i) the assessment that the property values used in the short and intermediate column equations had factors of safety of approximately four compared to a factor of safety of three for those used in the long column equation (57); (ii) that columns used in dry conditions of service had a higher elastic limit than that assumed by the intermediate column equation (133); and (iii) the low probability of having limiting strength reducing grade characteristics present at the location most subject to potential buckling. The increases in 1991 tabulated compression parallel to grain design values, $F_c$, for dimension lumber based on the results of extensive testing of in-grade full size pieces is supportive of these earlier assessments.

The column design provisions of the 1953 edition were carried forward essentially unchanged until 1977 when the intermediate column formula was reinstated. Structural columns in buildings designed using the simplified provisions had performed satisfactorily for more than 20 years. However, a change to a more conservative procedure was considered appropriate in view of the use being made of the Specification in certain industrial design applications. Particular attention was given to use of the provisions of the Specification in the design of wood cooling towers which involved subjecting spliced columns continuously to full design loads in a hot water environment. Although the column design and related procedures of the Specification were not tied to field problems encountered with one tower design, the use of the Specification for such a specialized and severe application indicated the more conservative design methodology should be reinstated.

The intermediate column equation was reintroduced in the 1977 edition with a $K_{dc}$ coefficient of 0.671 to correspond to the Euler buckling coefficient, $K_{cE}$, of 0.300 used in the long column equation. In addition, load duration factors for columns were limited only to values of $F_c$, thus disallowing any adjustment to column stresses controlled by buckling (see Commentary for 3.6.3). The 1977 provisions were continued in the 1982 and 1986 editions.

New Continuous Column Formula. The single column stability factor equation in 3.7.1.5 of the 1991 edition of the Specification is applicable to all slenderness ratios, $\ell_c/d$, and replaces the short, intermediate and long column equations given in earlier editions. In addition to facilitating column and beam-column design, the new continuous column equation takes into account research that shows the 4th power intermediate column formula, which was based on tests of 12 by 12 inch columns, overestimates the strength of columns made with lumber of two inch nominal thickness in the
The new column equation is

\[ C_p = \frac{1 + (F_{ce}/F^*_{c})}{2c} \]

(C3.7-5)

where:

- \( F^*_c \): tabulated compression design value multiplied by all applicable adjustment factors except \( C_p \)
- \( F_{ce} \): critical buckling design value for compression members
  \[ = \frac{K_{ce} E'}{(\theta_e/d)^2} \]
- \( K_{ce} \): Euler buckling coefficient for columns
- \( c \): nonlinear parameter for columns

The foregoing equation was first proposed in alternate form as a new method of determining the buckling stress of columns of any material in the elastic and inelastic range by a Finnish researcher in 1956 (226). The equation has as its upper limit the allowable compression parallel to grain design value associated with a crushing mode of failure \( F_c \) at very small \( \ell_e/d \) ratios and the critical (Euler) buckling design value \( F_{ce} \) at large \( \ell_e/d \) ratios. Continuity between these extremes of slenderness, where crushing and buckling interact, is obtained by assuming a curvilinear stress-strain relationship where the degree of nonlinearity caused by inelasticity, nonuniform material structure and initial eccentricity can be modeled through a single parameter "\( c \)". In this modeling, the slope of the stress-strain relationship is considered proportional to level of stress (226), with the rate of change of slope being constant.

Evaluation of the applicability of the proposed new procedure to short and intermediate in-grade wood columns in 1964 showed that when the parameter "\( c \)" was empirically established from the stress-strain relationship of very short columns (\( \ell_e/d \) of 2.5), the equation could provide a good approximation of column strength if the short prism tests adequately characterized the properties and nonuniformities of the longer columns (143). It subsequently was found that establishment of the parameter "\( c \)" by empirical fitting of the equation to column strength data resulted in a predictive equation that closely followed test results at all \( \ell/d \) ratios (229,232,233). A significant advantage of the methodology is that by selecting column test material representative of the nonuniform 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 "\( c \)" (233).

The critical buckling design value used in the continuous column equation is based on the same Euler buckling coefficients, \( K_{ce} \), as those used in the 1986 and previous editions: 0.300 for visually graded lumber and 0.418 for products having a coefficient of variation in modulus of elasticity of 0.11 or less. The 0.300 factor includes the Euler constant for rectangular sections (0.822) and a reduction of 2.74 on tabulated modulus of elasticity values. Assuming a coefficient of variation of 0.25 in the modulus of elasticity for visually graded sawn lumber or machine evaluated lumber (MEL), the 2.74 factor represents an approximate 5 percent lower exclusion value on pure bending modulus of elasticity (1.03 times tabulated \( E \) values) and a 1.66 factor of safety.

For glued laminated timber and machine stress rated lumber (MSR) or other products having a coefficient of variation in modulus of elasticity of 0.11 or less, the Euler buckling coefficient of 0.418 also represents an approximate 5 percent lower exclusion value on pure bending modulus of elasticity with a factor of safety of 1.66. The 0.418 coefficient is related to the 0.300 coefficient for visually graded lumber by the ratio

\[ \frac{1 - 0.11(1.645)}{1 - 0.25(1.645)} = 1.39 \]

A "\( c \)" value of 0.95 in the continuous column equation gives column stresses close to those obtained with the short, intermediate and long column equations used in previous editions (see Figure C3.7-1). Comparing the values of "\( c \)" specified for use in the new equation of 0.80 for sawn lumber, 0.85 for round timber piles and 0.90 for glued laminated timber with this baseline value indicates the relative reduction in column values resulting from adoption of the new design equation. The 0.80 factor for lumber has been established from column tests of dimension lumber (223,233) and, based on early large timber tests (133), is considered to be a conservative value for posts and timbers and beams and stringers. The values of "\( c \)" for piles and glued laminated timber are based on assessment of the sizes of these products relative to dimension lumber and the relative degree of warp and

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nonuniformity in density and grain slope that can occur as a result of permitted grade characteristics.

Example C3.7-1 illustrates the use of the column stability factor in calculating the allowable compression design value parallel to grain.

**Comparison with other Column Equations.** The column stability factor equation of 3.7.1.5 is somewhat similar to the column equation used in the British Standard Code of Practice CP112 and included in the 1988 draft standard of Eurocode No. 5 (45140). As given in the British Code, this equation has the form:

\[
F_c' = \frac{F_c + [1 + n (\ell_e/d)] F_{cE}}{2} \tag{C3.7-6}
\]

where:

- \(F_c\) = tabulated compression parallel to grain design value
- \(F_{cE}\) = critical buckling design value for compression members
- \(K_{cE}E'\) = \(\frac{n (\ell_e/d)^2}{4 - F_c F_{cE}}\)
- \(n\) = eccentricity factor
- 0.006928

Whereas Equation of 3.7.5 is based on the Engesser tangent modulus theory and assumes a curvilinear stress-strain curve, the European equation (C3.7-6) assumes linear elastic behavior to ultimate load, linear interaction between axial load and bending failure and an initial eccentricity of axial load (232). The eccentricity factor is assumed to represent nonuniformity and initial lack of straightness of the member due to grade characteristics (140). The value of 0.006928 for the factor is an average based on column tests of various grades. At the level of design stresses, the European (British Code of Practice) column equation gives results approximately the same (generally within \(+/-5\) percent) as those obtained from 3.7.1.5 with \(c\) equal to 0.80 (see Figure C3.7-1). As previ-

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**Figure C3.7-1 Column Formula Comparisons**

- 1977-1986 NDS
- 1991 NDS (\(c = 0.95\))
- 1991 NDS (\(c = 0.8\))
- British Code (\(n = 0.006928\))
- Parabolic, 2nd power

\[E = 1,600,000 \text{ psi} \]
\[F_{cE} = 1200 \text{ psi} \]
Adjustment factors assumed = 1.0
ously noted, when the value \(c\) is based on column tests of material representative of commercial grades, the effects of small eccentricities associated with crook or other warp as well as with nonuniform placement of knots and variable slopes of grain are embedded in the \(c\) value.

The design equation of 3.7.1.5 also is compared in Figure C3.7-1 with the results obtained with the short, intermediate and long column equations given in the 1977-1986 editions, and with the 2nd power parabolic equation for inelastic buckling proposed in the 1920's for wood (216) and presently used in steel design (3, 153). The 2nd power parabolic is tangent with the Euler curve at \(F_c/2\). It can be seen from Figure C3.7-1 that the new continuous equation with \(c\) of 0.80 gives significantly lower stresses than those obtained from the 1977-1986 equations at slenderness ratios less than about 35. For various \(E/F_c\) ratios at the design level, the maximum difference between the two criterion is about 21 percent. The new equation also gives lower values than the 2nd power parabolic equation in the intermediate and low Euler slenderness ratio range.

For visually graded dimension lumber in a number of the more important species groups, the reduction in column stresses associated with the new column equation will be partially offset by new higher compression parallel to grain design values, \(F_c\), based on in-grade test results. This is illustrated in Figure C3.7-2 where column design values from 3.7.1.2 based on 1.40\(F_c\) and 1.60\(F_c\) (representative of the increases reflected in the new design values) are compared with those from 1982 and 1986 column equations based on 1.00\(F_c\), assuming no change in \(E\).

3.7.1.6 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

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**Example C3.7-1**

A No. 2 Spruce-Pine-Fir 2x6 interior bearing wall stud, 91.5 in. long, sheathed on both sides with gypsum board, carries dead load + snow load from the roof. Determine \(C_p\) and the allowable compressive stress, \(F_c'\) for the stud. Assume 16" on center spacing.

\[
F_c = 1100 \text{ psi} \quad E = 1,400,000 \text{ psi} \quad (\text{Table 4A})
\]

\[
C_p = 1.1 \quad C_p = 1.15
\]

**Column Stability Factor \(C_p\)** (3.7.1)

\[
F_c' = F_c C_p C_p = (1100)(1.15)(1.1) = 1392 \text{ psi}
\]

\[
\ell_e/d_e = (91.5)/(5.5) = 16.64 \quad (\text{controls})
\]

\[
\ell_e/d = 16.65 < 50 \quad \text{ok}
\]

\[
K_{ce} = 0.3 \quad c = 0.8
\]

\[
\begin{align*}
F_{ce} &= \frac{K_{ce}E'}{(\ell_e/d)^2} = \frac{(0.3)(1,400,000)}{(16.64)^2} = 1517 \text{ psi} \\
C_p &= \frac{1 + (F_{ce}/F_c^*)}{2c} - \sqrt{\frac{1 + (F_{ce}/F_c^*)^2}{2c} - \frac{F_{ce}/F_c^*}{c}} \\
&= \frac{1 + 1517/1392}{2(0.8)} - \sqrt{\frac{1 + 1517/1392}{2(0.8)} - \frac{1517/1392}{0.8}} \\
&= 0.72
\end{align*}
\]

**Allowable Compression Design Value, \(F_c'\)** (2.3.1)

\[
F_c' = F_c C_p C_p = (1100)(1.15)(0.72)(1.1) = 1002 \text{ psi}
\]
the need to design the member as a beam-column (see section 15.4 of the Specification).

### 3.7.2-Tapered Columns

Use of a minimum diameter (dimension) plus one-third the difference between the minimum and maximum diameters (dimensions) as the representative dimension of a tapered column for determining effect of slenderness ratio was a general provision of the Specification from the 1944 through the 1986 editions. The rule was considered to give the diameter of a round column that could be used in the long column (Euler) equation to prevent buckling (57). This representative dimension was limited to no more than 1.5 times the minimum diameter or dimension until the 1960 edition when this upper limit was dropped.

Equation 3.7-2 in Section 3.7.2 for determining the representative dimension of tapered columns for specific support conditions is new in the 1991 edition. It was added following new analyses (47) that showed the general one-third rule was overly conservative for some end support conditions but unconservative for another. The use of a dimension 1/3 the length from the smaller end was estimated to give a buckling load that was 35 percent too low for a tapered column fixed at the large and unsupported at the small end, and a load that was 16 percent too low for a tapered column simply supported (translation fixed) at both ends. Alternatively, the 1/3 rule was shown to give buckling loads that were 13 percent too high for a tapered column fixed on the small end and unsupported on the large end. These estimates were for a minimum to maximum diameter (dimension) ratio of 0.70. The new equation provides more realistic estimates of column performance for these specific support conditions while the general 1/3 equation is retained for other support arrangements.

The support conditions referenced in 3.7.2 are related to the end condition modes given in Appendix G of the Specification as follows:

- **Fixed**: rotation fixed - translation fixed
- **Unsupported**: rotation free - translation free
- **Simply supported**: rotation free - translation fixed

The one end fixed - one end free or simply supported conditions referenced in 3.7.2 correspond to the fifth and sixth buckling mode cases in Appendix G. The condition of both ends simply supported corresponds to the fourth case. Values for the constant "a", given under "Support Conditions" in 3.7.2, are considered applicable when the ratio of minimum to maximum diameter equals or exceeds 1/3 (47).

The effective length factor, $K_e$, from 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 $F_c$ adjusted for all applicable adjustment factors except $C_p$.

### 3.7.3-Round Columns

The required size of a round column may be determined by first designing a square column having the same taper and then using a round member diameter that will give the same cross-sectional area as the square. This procedure is based on the equivalence of the bending and compression load-carrying capacities of round and square wood members having the same cross-section area (see Commentary for 2.3.8).

### 3.8-TENSION MEMBERS

#### 3.8.2 Average strength values for tension perpendicular to grain that are available in reference documents (62,66) 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 (20). However, because of undetectable ring shake and 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. In the 1982 edition, cautionary provisions about the use of design configurations which induce such stresses were introduced. Avoidance of these design configurations wherever possible is now required. Particular attention should be given in the design of bending member connections to have incoming vertical loads applied above the neutral axis or distributed across the entire cross-section through hangers or other means.

If perpendicular to grain tension stresses are not avoidable, use of stitch bolts or other mechanical reinforcement to take such loads is to be considered. When such a solution is used, care should be taken to insure the reinforcement itself does not cause splitting of the member as a result of drying in service (4). In any case, it is to be understood that the designer is responsible for avoiding the introduction of tension perpendicular to grain stresses or to assuring that the
methods and practices used to account for such stresses are adequate.

Radial stresses are induced in curved, pitch tapered and certain other shapes of glued laminated timber beams. Such beams are made of dry material which is controlled for quality at the time of manufacture. The Specification has provided allowable radial tension design values perpendicular to grain for curved glued laminated timber bending members since 1944. The present radial tension design values perpendicular to grain given in 5.2.2 were established in 1968 and have been shown to be adequate by both test (30,155) and experience.

3.9-COMBINED BENDING AND AXIAL LOADING

3.9.1-Bending and Axial Tension

The linear interaction equation for combined bending and tension stresses

\[
\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1
\]  

(C3.9-1)

where:

\[F_{b1}^*, F_{b2}^*\] = the tabulated bending design values for the two directions multiplied by all applicable adjustment factors except CL

Tabulated bending design values, \(F_b\), are not adjusted for slenderness, \(C_L\), in the bending-tension interaction equation because the tension load acts to reduce buckling propensity and the combined stress is not the critical buckling condition. Buckling is checked separately through the second equation given in 3.9.1 or

\[
\frac{f_b - f_t}{F_{b1}^{**}} \leq 1
\]  

(C3.9-3)

where:

\[F_{b1}^{**}\] = is the tabulated bending design value multiplied by all applicable adjustment factors including \(C_L\) (excluding \(C_T\))

Although inferred in earlier editions, the specific requirement that actual net bending stress (stress on the compression face of the member) not exceed the allowable bending design value adjusted for slenderness was added to the Specification in the 1982 edition. The moment effect of any eccentric axial load is to be included in \(f_b\) when the buckling check is made. Where biaxial bending is involved, actual net bending stress in each direction, \((f_{b1} - f_t')\) and \((f_{b2} - f_t')\), is checked against the slenderness adjusted design value, \(F_{b1}^{**}\) and \(F_{b2}^{**}\), for that direction.

Load Duration Adjustments. The allowable design values, \(F_t', F_b^*\) and \(F_b^{**}\) used in the bending-tension interaction equations are to be adjusted for load duration. The Specification specifically permits the use of the same \(C_D\) factor for both properties based on the shortest load duration in the combination of loads, even though the load of shortest duration may be associated with only one of the actual stresses. For example, if the actual tension stress parallel to grain is a result of roof snow load and dead load and the actual bending stress is associated with floor live load and dead load, a \(C_D\) of 1.15 (assumed applicable snow load duration) may be used to obtain both \(F_t'\) and \(F_{b1}^*\) or \(F_{b2}^*\) for use in the interaction equations. However, the design must also be evaluated without the snow load using a \(C_D\) of 1.0 for both properties, and without both snow and live loads using a \(C_D\) of 0.9 for both properties, to assure these conditions also meet the acceptance criteria of the interaction equations.
The provision for use of a load duration factor, \( C_D \), associated with the shortest duration of load in a combination of loads is based on field performance and long-standing practice in the design of trusses and glued laminated timber arches (4). Example C3.9-1 illustrates the use of these load duration provisions in a combination of loads.

For other applications, a different load duration adjustment may be used with each property, \( F'_1, F'_b \) and \( F''_b \), in the equation depending on the nature of the load or loads that are responsible for the actual stress associated with that property. For example, if the maximum actual tension stress on a member is a result of a roof snow load and the actual bending stress is a result of a permanent load, a \( C_D \) of 1.15 (assumed applicable snow load duration) is used to obtain \( F'_1 \) and a \( C_D \) of 0.9 to obtain \( F'_b \) or \( F''_b \). The factor associated with the longest total load duration is not applied to both properties when evaluating the maximum actual stresses.

3.9.2-Bending and Axial Compression

Background

Design provisions for members subject to bending and axial compression were essentially the same in the 1986 and earlier editions of the Specification except for location in the body of the standard or the appendix and in specific application requirements. These provisions provided two general interaction equations, one for short and one for long columns, as shown below

For \( \ell_e/d \leq 11 \):

\[
\frac{f_c}{F'_e} + \frac{f_b}{F'_b} \leq 1 \tag{C3.9-4}
\]

For \( \ell_e/d \geq K \):

\[
\frac{f_c}{F'_e} + \frac{f_b}{F'_b - f_c} \leq 1 \tag{C3.9-5}
\]

in which:

\[
K = 0.671 \sqrt{E/F_e} \tag{C3.9-6}
\]

For intermediate columns, \( 11 < \ell_e/d < K \), (see Commentary for 3.7.1.5), allowable loads were determined through linear interpolation between those established for \( \ell_e/d \) of 11 and those for \( \ell_e/d \) of \( K \). The foregoing two linear interaction equations were developed from analysis of test results of clear Sitka spruce columns subject to combined bending and concentric and eccentric axial compression loads (135). These tests indicated (223) that, when the additional moments resulting from member deflection where taken into account, the strength of members subject to combined stress could be closely estimated by the relationship

\[
\frac{P/A}{C} + \left( \frac{M/S}{F} \right)^2 = 1 \tag{C3.9-7}
\]

where:

\[
P/A = \text{applied axial stress}
\]

\[
M/S = \text{applied flexural stress, including additional moments associated with deflection of the member}
\]

\[
C, F = \text{ultimate strengths in flexure and compression}
\]

For short columns, where lateral deflections are small, it was concluded that the equation could be conservatively applied at the design stress level by dropping the exponent on the flexure ratio term (132,223). The resulting linear equation, the one used in the 1986 and earlier editions of the Specification, is comparable to that used with other structural materials (161,223).

In the case of long columns, however, neither the original nor the simplified form of the equation describing test results were applicable to design conditions where the additional moments associated with deflections due to eccentric axial and bending loads are not readily calculated. To obtain a useable equation that would account for these additional moments, the assumption was made that the ultimate strength of long columns under combined compression and bending loads was equal to the bending strength of the members (223). Under this assumption, which was shown to be reasonable by Sitka spruce column test data (135,223), the minimum bending strength available under combined loading to resist actual bending stress is the bending strength of the material less the long-column (Euler) buckling stress (223). For the general case where actual compression stress parallel to grain, \( f_c \), is less than the Euler buckling stress, the available bending strength becomes the allowable bending design value less the actual compressive stress parallel to grain, thus giving rise to the general equation

\[
\frac{f_c}{F'_e} + \frac{f_b}{F'_b - f_c} \leq 1 \tag{C3.9-8}
\]

Formulas for eccentric axial loads in combination with bending and concentric axial loads were developed following the same general assumption, and further
Example C3.9-1

A No. 2 Southern Pine 2x8, used as the bottom chord of a 28-ft roof truss (14 ft between panel points), is subject to a uniform dead load of 10 psf (4 ft truss spacing), as well as tension forces (assuming pinned connections) of 1680 lb from roof wind loads, 1680 lb from roof live loads and 1560 lb from dead loads. Check the adequacy of the member between panel points for bending and tension for all load combinations.

\[ F_b^* = 1200 \text{ psi} \quad E = 1,600,000 \text{ psi} \quad (\text{Table 4B}) \]
\[ F_t^* = 650 \text{ psi} \quad C_F = 1.0 \quad A = 10.88 \text{ in}^2 \quad S = 13.14 \text{ in}^3 \]

**Load Case 1: DL+RLL+WL, } C_D = 1.6**

**Tension**

\[ F_t' = F_t C_D C_F = (650)(1.6)(1.0) = 1040 \text{ psi} \]

Tension force in chord, \( T = 1560+1680+1680 = 4920 \text{ lb} \)

\[ f_t = T/A_g = 4920/10.88 = 452 \text{ psi} < F_t^* = 1040 \text{ psi} \quad \text{ok} \]

**Bending**

\[ F_b^* = F_b C_D C_F = (1200)(1.6)(1.0) = 1920 \text{ psi} \]

\[ \ell_e = 1.63 A_g + 3d = 1.63(14)(12) + 3(7.25) = 295.6 \text{ in.} \]

\[ R_B = \frac{\ell_e d}{b^2} = \frac{295.6(7.25)}{(1.5)^2} = 30.9 \]

\[ K_{bf} = 0.438 \]

\[ F_{bf} = \frac{K_{bf} F_b^*}{R_b^2} = \frac{(0.438)(1,600,000)}{(30.9)^2} = 736 \text{ psi} \]

\[ C_L = \frac{1 + (F_{bf}/F_b^*)}{1.9} - \frac{1 + (F_{bf}/F_b^*)^2}{1.9} = \frac{F_{bf}/F_b^*}{0.95} \]

\[ = 1 + \frac{1736/1920}{1.9} - \frac{1 + (1736/1920)^2}{1.9} = 0.372 \]

\[ F_b^{**} = F_b C_D C_L C_F = (1200)(1.6)(0.372)(1.0) = 715 \text{ psi} \]

\[ M_{max} = wd^2/8 = (10)(4)(14)(12)/8 = 11,760 \text{ in-lb} \]

\[ f_b = M_{max}/S = 11,760/13.14 = 895 \text{ psi} < F_b^* = 1920 \text{ psi} \quad \text{ok} \]

**Combined Bending and Axial Tension**

\[ \frac{f_t + f_b}{F_t^*} = \frac{452 + 895}{1040} = 0.90 < 1.0 \quad \text{ok} \]

\[ \frac{f_b - f_t}{F_b^{**}} = \frac{895 - 452}{715} = 0.62 < 1.0 \quad \text{ok} \]

**Load Case 2: DL+RLL, } C_D = 1.25**

**Tension**

\[ F_t' = F_t C_D C_F = (650)(1.25)(1.0) = 812.5 \text{ psi} \]

Tension force in chord, \( T = 1560+1680 = 3240 \text{ lb} \)

\[ f_t = 3240/10.88 = 298 \text{ psi} < F_t' = 812.5 \text{ psi} \quad \text{ok} \]

**Bending**

\[ F_b^{**} = F_b C_D C_F = (1200)(1.25)(1.0) = 1500 \text{ psi} \]

\[ F_{bf} = 736 \text{ psi} \]

\[ C_L = \frac{1 + \frac{1736/1500}{1.9}}{1.9} - \frac{1 + \frac{1736/1500}{1.9}^2}{1.9} = 0.470 \]

\[ F_b^{**} = F_b C_D C_L C_F = (1200)(1.25)(0.470)(1.0) = 705 \text{ psi} \]

\[ f_b = 895 \text{ psi} < F_b^{**} = 1500 \text{ psi} \quad \text{ok} \]

**Combined Bending and Axial Tension**

\[ \frac{f_t + f_b}{F_t^*} = \frac{298 + 895}{812.5} = 0.97 < 1.0 \quad \text{ok} \]

\[ \frac{f_b - f_t}{F_b^{**}} = \frac{895 - 298}{705} = 0.85 < 1.0 \quad \text{ok} \]

**Load Case 3: DL only, } C_D = 0.9**

**Tension**

\[ F_t' = F_t C_D C_F = (650)(0.9)(1.0) = 585 \text{ psi} \]

Tension force in chord, \( T = 1560 \text{ lb} \)

\[ f_t = 1560/10.88 = 143 \text{ psi} < F_t' = 585 \text{ psi} \quad \text{ok} \]

**Bending**

\[ F_b^* = F_b C_D C_F = (1200)(0.9)(1.0) = 1080 \text{ psi} \]

\[ F_{bf} = 736 \text{ psi} \]

\[ C_L = \frac{1 + \frac{1736/1080}{1.9}}{1.9} - \frac{1 + \frac{1736/1080}{1.9}^2}{1.9} = 0.628 \]

\[ F_b^{**} = F_b C_D C_L C_F = (1200)(0.9)(0.628)(1.0) = 678 \text{ psi} \]

\[ f_b = 895 \text{ psi} < F_b^{**} = 1080 \text{ psi} \quad \text{ok} \]

**Combined Bending and Axial Tension**

\[ \frac{f_t + f_b}{F_t^*} = \frac{143 + 895}{585} = 1.07 > 1.0 \quad \text{ng} \]

\[ \frac{f_b - f_t}{F_b^{**}} = \frac{895 - 143}{678} = 1.11 > 1.0 \quad \text{ng} \]

(Cont.)
Example C3.9-1 (cont.)

At this point, the web configuration might be changed, member size could be increased or a higher grade selected. Try No. 1 Dense Southern Pine. Recheck the controlling case of dead load only.

\[ F_b = 1650 \text{ psi} \quad F_c = 875 \text{ psi} \quad E = 1,800,000 \text{ psi} \]

Recheck Load Case 3: DL only, \( C_D = 0.9 \)

**Tension**

\[ F'_{t} = F_c C_D C_P = (875)(0.9)(1.0) = 787.5 \text{ psi} \]  

Tension force in chord, \( T = 1560 \text{ lb} \)

\[ f'_t = 1560 / 10.88 = 143 \text{ psi} < F'_{t} = 787.5 \text{ psi} \quad \text{ok} \]

**Bending**

\[ F^* = F_b C_D C_P = (1650)(0.9)(1.0) = 1485 \text{ psi} \]

\[ F_{bf} = \frac{K_{b} E}{R_b^2} = \frac{(0.438)(1,800,000)}{30.9^2} = 826 \text{ psi} \]

assuming a sinusoidal deflection curve in which deflection is proportional to stress (132,223). Such interaction equations were included in the Appendix of the Specification in the 1986 and all earlier editions.

Prior to the 1971 edition, only the short column interaction curve was included in the body of the Specification; with reference being made to the more exact equations for intermediate and long columns given in the Appendix. In the 1971 edition, application of the short column equation in the body of the Specification was limited to columns with \( \ell / d < K \).

In the 1977 edition, the long column equation was brought into the body of the Specification and the factor \( J \) was introduced to facilitate interpolation of allowable loads for intermediate columns, as shown below

\[ \frac{f_c}{F'_{c}} + \frac{f_b}{F'_{b} - Jf_c} \leq 1 \quad \text{(C3.9-9)} \]

where:

\[ J = \frac{\ell_c / d - 11}{K - 11} \quad \text{(C3.9-10)} \]

and

\[ 0 \leq J \leq 1 \]

In 1982, specific wording was added to the Specification to require checking the effect of stress interaction about both principal axes. In 1986, this provision was extended to define what slenderness values should be used in the calculation of \( F'_{c}, F'_{b}, \) and \( J \). The extension required the coupling of beam buckling propensity due to loads acting on one face of the member with column buckling propensity perpendicular to the plane of bending.

**New Bending and Axial Compression Interaction Equation.** Although the beam-column interaction criteria in the 1986 and earlier editions have provided for satisfactory performance, it was recognized that (i) the linear equations were developed without direct consideration of beam buckling, (ii) the adjustment of moment to account for the interaction of axial load and deflection is not consistent with more recent theoretical analyses, and (iii) the equations do not address bending about both principal axes. The new interaction equation given in 3.9.2 of the Specification corrects these limitations and closely describes recent beam-column test data for in-grade lumber as well as similar earlier data for clear wood material (229,230).

For the case of bending load applied to the narrow face of the member and concentric axial compression load, the new interaction equation reduces to

\[ \left( \frac{f_c}{F'_{c}} \right)^2 + \frac{f_{bl}}{F_{bl'} (1 - f_c/F_{edl})} \leq 1 \quad \text{(C3.9-11)} \]

where:

\[ f_c = \text{actual compression stress parallel to grain} \]

\[ f_{bl} = \text{actual edgewise bending stress} \]
$F_c' = \text{allowable compression design value parallel to grain including adjustment for largest slenderness ratio}$

$F_{bl}' = \text{allowable bending design value including adjustment for slenderness ratio}$

$F_{cEI} = \text{critical buckling design value in the plane of bending}$

$$= \frac{K_{cE} E'}{\left( \frac{\ell_{el}}{d_l} \right)^2}$$

in which:

$d_l = \text{dimension of wide face}$

$\ell_{el} = \text{distance between points of support restraining buckling in the plane of bending}$

In the new equation the ratio of actual to allowable compression stress is squared where previously the exponent on this term was one. This change is based on tests of southern pine, spruce-pine-fir and western hemlock 2x4 and 2x6 southern pine short beam-columns (94,228,229). The moment magnification factor in the denominator of the bending ratio term of the new equation is $(1 - F_c/F_{cEI})$. This factor in the 1986 and earlier editions was $(1 - F_c/F_{cEI}')$ for the case of long columns ($J = 1$). The new magnification term, which is consistent with that used for other structural materials, is based on theoretical analysis and confirmed by test results for intermediate and long beam-columns made with the same species and sizes of lumber used to establish the first ratio term in the equation (229,230). The magnification factor in the new equation is larger than the equivalent adjustment used in earlier editions when (i) the higher bending strength grades are involved, (ii) the slenderness ratio for bending is small and (iii) the column slenderness ratio in the plane of bending is large.

A comparison of the new interaction equation with that used previously for the case of a bending load applied on the narrow face and a concentric axial compression load is shown in Figure C3.9-1 for different beam and column slenderness ratios and different bending and column strength combinations. The ordinate $(f_c/F_c)$ and abscissa $(f_b/F_b)$ ratios in this figure are based on tabulated compression and bending design values, $F_c$ and $F_b$, both unadjusted for slenderness ratio. Thus the values of the ratios for pure compression are less than 1.00 and are limited by the slenderness ratio for the member being used in the example. Similarly, the values of the ratios for pure bending are less than 1.00 when the 1986 slenderness factor, $C_s$, for the member used in the example exceeds
zero. The maximum differences shown in the comparisons are the maximum difference between the 1991 and 1986 \(f_c/F_c\) ratio for any value of the \(f_p/F_p\) ratio; and the maximum difference between the 1991 and 1996 \(f_c/F_c\) ratios for any value of the \(f_p/F_p\) ratio. It can be seen from Figure C3.9-1 that for the loading case being used (bending load on narrow face and concentric axial compression load), and at the level of design stresses, generally larger allowable loads are obtained with the new interaction equation than those obtained from the equation given in earlier editions. In these comparisons, the same values of \(F'_c\) and \(F'_b\) are used in both the 1991 and 1986 interaction equations in order to illustrate the different allowable design values obtained from each, independent of the changes made in column and beam slenderness provisions in the 1991 edition.

The complete interaction equation in 3.9.2 provides for any combination of axial concentric compression load applied either on the narrow or wide face. The equation facilitates design of members subjected to these load combinations and accounts for the amplification, which was not previously considered, of the bending moment associated with load on the wide face due to bending load on the narrow face. The general interaction equation is shown below with the moment magnification factors identified by the symbol \(C_m\).

\[
\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{bl}}{C_{m1} F_{bl}'} + \frac{f_{b2}}{C_{m2} F_{b2}'} \leq 1 \quad (C3.9-12)
\]

where:

- \(f_c\) = actual compression stress parallel to grain
- \(f_{bl}\) = actual edgewise bending stress
- \(f_{b2}\) = actual wideface bending stress
- \(F_c'\) = allowable compression design value including adjustment for largest slenderness ratio
- \(F_{bl}'\) = allowable bending design value, including adjustment for slenderness ratio, for load applied to the narrow face
- \(F_{b2}'\) = allowable bending design value, including adjustment for slenderness ratio, for load applied to the wide face
- \(C_{m1}\) = moment magnification factor = \(1 - f_c/F_{ce1}\)
- \(C_{m2}\) = moment magnification factor = \(1 - f_c/F_{ce2} - (f_{bl}/F_{bl})^2 - (f_{b2}/F_{b2})^2\)

in which:

- \(F_{ce1}\) = critical (Euler) buckling design value for compression member in plane of bending from edgewise load
- \(F_{ce2}\) = critical (Euler) buckling design value for compression member in plane of bending from wideface load

\[
\frac{K_{ce}E'}{(l_{bl}/d_1)^2} = F_{ce2}
\]

\[
\frac{K_{ce}E'}{(l_{b2}/d_2)^2} = F_{ce2}
\]

and

\[
d_1 = \text{dimension of wide face}
\]

\[
d_2 = \text{dimension of narrow face}
\]

\[
\ell_{el} = \text{distance between points of support restraining buckling in plane of bending from edgewise load}
\]

\[
\ell_{e2} = \text{distance between points of support restraining buckling in plane of bending from wideface load}
\]

\[
F_{bl} = \text{critical buckling design value for bending member}
\]

The third term, \((f_{bl}/F_{bl})^2\), of \(C_{m2}\) represents the amplification of \(f_{b2}\) from \(f_{bl}'\). This term is based on theoretical analysis and certain simplifying assumptions (230). The appropriateness of the term is closely verified by the early beam-column tests made on clear Sitka spruce (135,230). The \(C_{m2}\) equation conservatively models cantilever and multispans beam-columns subject to biaxial loads (231).

For bending loads on the narrow and wide faces only, the equation reduces to

\[
\frac{f_{bl}}{F_{bl}'} + \frac{f_{b2}}{C_{m2} F_{b2}'} \leq 1 \quad (C3.9-13)
\]

where:

- \(C_{m2}\) = moment magnification factor
  
  \[= 1 - (f_{bl}/F_{bl})^2\]

For a concentric axial load and bending load on the wide face, the equation reduces to

\[
\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b2}}{C_{m2} F_{b2}'} \leq 1 \quad (C3.9-14)
\]

where:

- \(C_{m2}\) = moment magnification factor
  
  \[= 1 - f_c/F_{ce2}\]
In all cases, \( F' \) is to be determined in accordance with the provisions of 3.6.3 and 3.7.1; and \( F'_{bl} \) and \( F'_{b2} \) are to be determined in accordance with 3.3.1 and 3.3.3. The load duration factor, \( C_D \), used in the determination of these allowable values may be that associated with the shortest load duration in any combination of loads. Alternatively, the value of \( C_D \) associated with each load (axial compression, narrow face bending, wide face bending) may be used for the allowable design value associated with that load. The Commentary for 3.9.1 on load duration factor adjustments for the case of bending and axial tension also applies to bending and axial compression.

Examples C3.9-2 and C3.9-3 illustrate the use of the bending and axial compression interaction equation

---

**Example C3.9-2**

A No. 1 Southern Pine 2x6 beam-column carries axial compression loads of 840 lb (snow) and 560 lb (dead) plus a 25 psf wind load on the narrow face. Column spacing is 4 ft. and column length is 10 ft. Lateral support is provided at the ends and on the narrow face throughout the length of the member. Check the adequacy of the member for bending and compression for all load combinations.

- \( F_c = 1650 \text{ psi} \)  
- \( E = 1,700,000 \text{ psi} \) (Table 4B)
- \( F_c = 1750 \text{ psi} \)  
- \( C_p = 1.0 \)  
- \( A = 8.25 \text{ in}^2 \)  
- \( S = 7.563 \text{ in}^3 \)

**Load Case 1: DL+SL+WL, \( C_D = 1.6 \)**

**Compression** (3.6, 3.7)

\[
F_c^* = F_c C_D C_p = (1750)(1.6)(1.0) = 2800 \text{ psi} \quad (3.7.1.5)
\]

\[
\epsilon/d = 0 \quad \text{(fully supported)}
\]

\[
K_e = 1.0 \quad \text{(assume pin-pia end conditions)} \quad \text{(App. G)}
\]

\[
F_{ef} = \frac{K_{ef} E'}{(\epsilon/d)^2} = \frac{(0.3)(1,700,000)}{(21.8)^2} = 1073 \text{ psi}
\]

\[
C_p = \frac{1 + (F_{ef}/F_c^*)}{2 c} = \frac{1 + (1073/2800)}{2 \cdot 0.8} = 0.346
\]

\[
F_c' = F_c C_D C_p = (1750)(1.6)(1.0)(0.346) = 969 \text{ psi}
\]

Axial Force, \( P = 560 + 840 = 1400 \text{ lb} \)

\[
f_c = P/A_g = 1400/8.25 = 170 \text{ psi} < F_c' = 969 \text{ psi}, \; F_{ef} \; \text{ok}
\]

**Bending** (3.3)

\[
F_{b1}' = F_b C_D C_p = (1650)(1.6)(1.0) = 2640 \text{ psi}
\]

\[
M_{max} = wL^2/8 = (25)(4)(10)^2/8 = 15,000 \text{ in-lb}
\]

\[
f_b = M_{max}/S = 15,000/7.563 = 1983 \text{ psi} < F_{b1}' = 2640 \text{ psi} \; \text{ok}
\]

---

**Combined Bending and Axial Compression** (3.9.2)

\[
\left( \frac{F_c}{F_c'} \right)^2 + \frac{f_{bl}}{F_{b1}'(1 - f_c/F_{ef})} \leq 1.0
\]

\[
(170/969)^2 + \frac{1983}{2640(1 - 170/1073)} = 0.92 < 1.0 \; \text{ok}
\]

**Load Case 2: DL+SL, \( C_D = 1.15 \)**

**Compression** (3.6, 3.7)

\[
F_c^* = F_c C_D C_p = (1750)(1.15)(1.0) = 2013 \text{ psi} \quad (3.7.1.5)
\]

\[
C_p = 1 + \frac{1073/2013}{2(0.8)} = \frac{1 + 1073/2013}{2(0.8)} = 0.456
\]

\[
F_c' = F_c C_D C_p = (1750)(1.15)(0.456) = 918 \text{ psi}
\]

Axial Force, \( P = 560 + 840 = 1400 \text{ lb} \)

\[
f_c = P/A_g = 1400/8.25 = 170 \text{ psi} < F_c' = 918 \text{ psi}, \; F_{ef} \; \text{ok}
\]

**Load Case 3: DL only, \( C_D = 0.9 \)**

**Compression** (3.6, 3.7)

\[
F_c^* = F_c C_D C_p = (1750)(0.9)(1.0) = 1575 \text{ psi} \quad (3.7.1.5)
\]

\[
C_p = 1 + \frac{1073/1575}{2(0.8)} = \frac{1 + 1073/1575}{2(0.8)} = 0.548
\]

\[
F_c' = F_c C_D C_p = (1750)(0.9)(0.548) = 863 \text{ psi}
\]

Axial Force, \( P = 560 \text{ lb} \)

\[
f_c = P/A_g = 560/8.25 = 68 \text{ psi} < F_c' = 863 \text{ psi}, \; F_{ef} \; \text{ok}
\]

Load case 1 (DL+SL+WL) controls

No. 1 Southern Pine 2x6 satisfies NDS criteria for combined bending and axial compression.
Example C3.9-3

Assuming pinned connections for purposes of illustration, a 3 ft section (between panel points) of a No. 2 Southern Pine 4x2, top chord of a gable end, parallel chord truss is subjected to axial compression forces of 300 lb (DL) and 600 lb (SL), plus concentrated loads of 60 lb (DL) and 120 lb (SL) applied at the center of the 3 ft span on the wide face, and a concentrated load of 120 lb (WL) at the center of the span on the narrow face. Lateral support is provided at the ends only. Check the adequacy of the member for bending and compression for all load combinations.

\[ F_b = 1500 \text{ psi} \quad F_c = 1650 \text{ psi} \quad (\text{Table 4B}) \]

\[ E = 1,600,000 \text{ psi} \quad C_p = 1.0 \quad C_f = 1.1 \]

\[ A = 5.25 \text{ in}^2 \quad S_x = 3.063 \text{ in}^3 \quad S_y = 1.313 \text{ in}^3 \]

**Load Case 1: DL+SL+WL, \( C_D = 1.6 \)**

**Compresson**

(3.6, 3.7, 3.9.2)

\[ F_e^* = F_cc_s = (1650)(1.6)(1.0) = 2640 \text{ psi} \]

\[ \ell_x/d_x = (3)(12)/(3.5) = 10.3 > 50 \text{ Controls} \]

\[ K_{ef} = 0.3 \]

\[ F_{ef1} = K_{ef}E' \left( \frac{\ell_x}{d_x} \right)^2 = \frac{(0.3)(1,600,000)}{(24)^2} = 4537 \text{ psi} \]

\[ F_{ef2} = \frac{K_{ef}E'}{\ell_x/d_x} = \frac{(0.3)(1,600,000)}{24^2} = 833 \text{ psi} \]

\[ F_{ef2} \text{ controls for } C_p \]

\[ C_p = \frac{1 + (F_{ef}/F_c^*)}{2} - \sqrt{\left[ \frac{1 + (F_{ef}/F_c^*)}{2} \right]^2 - \frac{F_{ef}/F_c^*}{c}} \]

\[ = \frac{1 + 4537/2640}{2} - \frac{4537/2640}{0.8} = 0.292 \]

\[ F_c^* = F_cC_pC_f = (1650)(1.6)(1.0)(0.292) = 770 \text{ psi} \]

Axial Force, \( P = 300 + 600 = 900 \text{ lb (DL+SL)} \)

\[ f_c = P/A = 900/5.25 = 171 \text{ psi} < F_c^* = 770 \text{ psi}, \quad F_{ef1}, \quad F_{ef2} \quad \text{ok} \]

**Narrow Face Bending** (load parallel to wide face) (3.3)

\[ F_e^* = F_eC_pC_f = (1500)(1.6)(1.0) = 2400 \text{ psi} \]

\[ \ell_x = 3 \text{ ft} \quad d_x = 3.5 \text{ in.} \]

\[ \ell_x/d_x = (3)(12)/(3.5) = 10.3 > 7 \]

\[ \ell_x = 1.37(\ell_x + 3d) \quad (3.3.3.3) \]

\[ R_p = \frac{\ell_x/d_x}{\sqrt{b^2}} = \sqrt{\frac{(59.8)(3.5)}{(1.5)^2}} = 9.65 \]

\[ K_{ef} = 0.438 \]

\[ F_{ef} = \frac{K_{ef}E'}{R_p^2} = \frac{(0.438)(1,600,000)}{(9.65)^2} = 7526 \text{ psi} \]

\[ F_{bl} = \frac{1 + (F_{ef}/F_c^*)}{1.9} - \left[ \frac{1 + (F_{ef}/F_c^*)}{1.9} \right]^2 - \frac{F_{ef}/F_c^*}{0.95} \]

\[ = \frac{1 + 7526/2400}{1.9} - \frac{7526/2400}{0.95} = 0.978 \]

Alternatively, using the approximate rules of 4.4.1.2, with \( d/b = 2, \quad C_l = 1.0 \) according to 3.3.3.2; however the calculated \( C_l \) of 0.978 will be used here (see 4.4.1.1).

\[ F_{bl} = F_bC_lC_pC_f = (1500)(1.6)(0.978)(1.0) = 2347 \text{ psi} \]

Bending Load, \( P = 120 \text{ lb (WL)} \)

\[ M_{max} = Pl/4 = (120)(3)(12)/4 = 1080 \text{ in-lb} \]

\[ f_{bl} = M_{max}/S_y = 1080/3.063 = 353 \text{ psi} < F_{bl} = 2347 \text{ psi}, \quad F_{bl} \text{ ok} \]

**Wide Face Bending** (load parallel to narrow face) (3.3)

Since \( d < b \) (1.5 < 3.5 in.), \( C_l = 1.0 \)

\[ F_{bl} = F_bC_lC_pC_f = (1500)(1.6)(1.0)(1.0) = 2640 \text{ psi} \]

Bending Load, \( P = 60 + 120 = 180 \text{ lb (DL+SL)} \)

\[ M_{max} = Pl/4 = (180)(3)(12)/4 = 1620 \text{ in-lb} \]

\[ f_{bl} = M_{max}/S_y = 1620/1.313 = 1234 \text{ psi} < F_{bl} = 2640 \text{ psi ok} \]

**Combined Bending and Axial Compression** (3.9.2)

\[ \left( \frac{f_c}{F_c^*} \right)^2 + \frac{f_{bl}}{F_{bl}'} + \frac{f_{b2}}{F_{b2}'} = \frac{171}{770} + \frac{353}{2347(1-171/4537)} + \frac{1234}{2640(1-171/833-353/7526)^2} = 0.796 < 1.0 \quad \text{ok} \]

**Load Case 2: DL+SL, \( C_D = 1.15 \)**

**Compression** (3.6, 3.9.2)

\[ F_e^* = F_eC_D = (1500)(1.15)(1.0) = 1898 \text{ psi} \]

\[ F_{ef2} = 833 \text{ psi since } \ell_x/d_x \text{ controls} \]

(cont.)
Example C3.9-3 (cont.)

\[
C_p = \frac{1+833/1898}{2(0.8)} - \sqrt{\frac{1+833/1898}{2(0.8)}} - \frac{833/1898}{0.8} = 0.389
\]

\[
F_e' = F_cC_DC_F = (1650)(1.15)(1.0)(0.389) = 738 \text{ psi}
\]

Axial Force, \( P = 300+600 = 900 \text{ lb (DL+SL)} \)

\[
f_c = P/A = 900/5.25 = 171 \text{ psi} < F_e' = 738 \text{ psi}, F_{cd} \text{ ok}
\]

Wide Face Bending (no narrow face bending) (3.3)

Since \( d < b \) (1.5 < 3.5 in.), \( C_L = 1.0 \) (3.3.3.1)

\[
F_{bd} = F_cC_DC_LC_F = (1500)(1.15)(1.0)(1.0)(1.1) = 1898 \text{ psi}
\]

Bending Load, \( P = 60+120 = 180 \text{ lb (DL+SL)} \)

\[
M_{\text{max}} = Pd/4 = (180)(3)(12)/4 = 1620 \text{ in-lb}
\]

\[
f_{bd} = M_{\text{max}}/S_y = 1620/1.313 = 1234 \text{ psi} < F_{bd} = 1898 \text{ psi ok}
\]

Combined Bending and Axial Compression (3.9.2)

\[
\left( \frac{171}{738} \right)^2 + 0 + \frac{1234}{1898(1 - 171/833 - 0)} = 0.872 < 1.0 \text{ ok}
\]

Load Case 3: DL only, \( C_D = 0.9 \)

Compression (3.6, 3.7, 3.9.2)

\[
F_e^* = F_cC_DC_F = (1650)(0.9)(1.0) = 1485 \text{ psi (3.7.1.5)}
\]

\[
F_{cd2} = 833 \text{ psi controls for } C_p
\]

\[
C_p = \frac{1+833/1485}{2(0.8)} - \sqrt{\frac{1+833/1485}{2(0.8)}} - \frac{833/1485}{0.8} = 0.475
\]

\[
F_e' = F_cC_DC_F = (1650)(0.9)(1.0)(0.475) = 705 \text{ psi}
\]

Axial Force, \( P = 300 \text{ lb (DL)} \)

\[
f_c = P/A = 300/5.25 = 57 \text{ psi} < F_e' = 705 \text{ psi}, F_{cd2} \text{ ok}
\]

Wide Face Bending (no narrow face bending) (3.3)

Since \( d < b \) (1.5 < 3.5 in.), \( C_L = 1.0 \) (3.3.3.1)

\[
F_{bd} = F_cC_DC_LC_F = (1500)(0.9)(1.0)(1.0)(1.1) = 1485 \text{ psi}
\]

Bending Load, \( P = 60 \text{ lb (DL)} \)

\[
M_{\text{max}} = Pd/4 = (60)(3)(12)/4 = 540 \text{ in-lb}
\]

\[
f_{bd} = M_{\text{max}}/S_y = 540/1.313 = 411 \text{ psi} < F_{bd} = 1485 \text{ psi ok}
\]

Combined Bending and Axial Compression (3.9.2)

\[
\left( \frac{171}{705} \right)^2 + 0 + \frac{411}{1485(1 - 57/833 - 0)} = 0.304 < 1.0 \text{ ok}
\]

Load case 2 (DL+SL) controls

No. 2 Southern Pine 2" x 4" satisfies NDS criteria for combined bending and axial compression.

including use of load duration provisions for the shortest duration of load in a combination of loads.

3.10-DESIGN FOR BEARING

3.10.1-Bearing Parallel to Grain

Tabulated bearing design values parallel to grain, \( F_e' \), are based on clear wood properties and represent typical quality material present across various product grades. For certain of the higher grades of sawn lumber 4 inches or less in thickness, tabulated compression design values parallel to grain, \( F_g \), may exceed \( F_e' \) values for the species. Such \( F_e' \) values are based on in-grade lumber tests and reflect the presence of a higher quality of clear wood material in such grades than that characteristic of the species as a whole. Where end grain bearing is a design consideration, actual bearing stress parallel to grain, \( f_g \), shall not exceed the appropriate allowable bearing design value parallel to grain, \( F_e^* \), regardless of the value of \( F_e' \) for the grade. This criterion represents a continuation of design practice that has been part of the Specification since 1944.

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. Prior to 1953, all end bearings were required to be on metal plates or straps or on metal inserts. This requirement was modified in the 1953 and subsequent editions to permit direct end-to-end bearing of wood surfaces when the actual bearing stress parallel to grain is less than or equal to 75 percent of the allowable bearing design value parallel to grain (\( F_e' = 3/4 F_g \)), abutting end surfaces are parallel and appropriate lateral support is provided. The required use of a metal plate or equivalent strength material as an insert in higher
loaded end-to-end bearing joints is to assure a uniform distribution of load from one member to another.

3.10.2-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. This practice was first addressed in the Specification in the 1977 edition.

3.10.3-Bearing at an Angle to Grain

The equation for calculating the allowable compressive stress on an inclined surface, \( F_{g}' \), has been a provision of the Specification since 1944. Developed from tests on Sitka spruce in 1921, the general applicability of the equation has been confirmed by more recent tests on other species (57,77,84).

The equation

\[
F_{g}' = \frac{F_{g}' F_{c_{l}}'}{F_{g}' \sin^2 \theta + F_{c_{l}}' \cos^2 \theta}
\]  

(C3.10-1)

applies when the inclined or loaded surface is at right angles to the direction of load. The equation has limits of \( F_{g}' \) when the angle between direction of grain and direction of load, \( \theta \), is 0° and \( F_{c_{l}}' \) when this angle is 90°. When the force is at an angle other than 90° to the inclined surface, the equation is entered with \( \theta \) equal to the angle between the direction of grain and the direction of the load component that is normal to the surface. In this case the allowable stress obtained from the equation, \( F_{g}' \), is that component of the total force that is perpendicular to the surface. Example C3.10-1 illustrates the use of these provisions. Stresses on both inclined surfaces in a notched member should be checked if the limiting case is not apparent.

Example C3.10-1

A 2x10 member is loaded in bearing at an angle to grain from a 2x4, such that the angle between the perpendicular (to 2x10 bearing surface) component of the resultant force and the direction of grain (2x10) is 38°, while the angle between the total load direction and grain direction is 46°. For \( F_g' = 1670 \) psi and \( F_{c_{l}}' = 410 \) psi, determine \( F_g' \) and the total allowable load based on bearing in the 2x10.

![Diagram showing angle calculations](image)

### Allowable Bearing Design Value

**Parallel to Grain, \( F_{g}' \)**

For Equation 3.10-1, \( \theta = 38° \)

\[
F_{g}' = \frac{F_{g}' F_{c_{l}}'}{F_{g}' \sin^2 \theta + F_{c_{l}}' \cos^2 \theta}
\]

(Eq. 3.10-1)

\[
= \frac{(1670)(410)}{(1670) \sin^2(38) + (410) \cos^2(38)}
\]

= 771 psi

**Total Allowable Load, \( P_{total} \)**

\( \alpha \) = angle between total force and component perpendicular to 2x10 bearing surface

\( = 46° - 38° = 8° \)

\( A_{bearing} = bd/(1/\cos \alpha) \)

\( = (1.5)(3.5)/(1/\cos(8°)) \)

\( = 5.30 \text{ in}^2 \)

\( P_{\text{to } 2x10 \text{ b.s.}} = F_{g}' A_{bearing} \)

\( = (771)(5.30) \)

\( = 4088 \text{ lb} \)

\( P_{total} = (P_{\text{to } 2x10 \text{ b.s.}})(1/\cos \alpha) \)

\( = (4088)(1/\cos(8°)) \)

\( = 4128 \text{ lb} \)
PART IV: SAWN LUMBER

4.1-GENERAL

4.1.2-Identification of Lumber

4.1.2.1 The local building code body having jurisdiction over the structural design is the final authority as to the competency of the lumber grading or inspection bureau or agency, and to the acceptability of its grademark and certificate of inspection. Most code jurisdictions accept the grademarks of those agencies that have been certified by the Board of Review of the American Lumber Standards Committee, established under the U.S. Department of Commerce's Voluntary Product Standard PS20-70 (190).

The design provisions of the Specification applicable to sawn lumber are based on (i) use of material identified by an agency that has been certified by the Board of Review and (ii) use of the design values tabulated in the Specification which have been taken from grading rules approved by that same Board (190). Those agencies publishing approved grading rules are given in the Design Value Supplement to the Specification under "List of Sawn Lumber Grading Agencies". Where other agencies and their grademarks are accepted, it is the responsibility of the designer to assure that the design values given in the Specification are applicable to the material so identified. If design values other than those tabulated in the Specification 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.

4.1.3-Definitions

4.1.3.2 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 PS20-70 (190). The rule provides standard use categories, grade names, and grade descriptions. The latter 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 (18).

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

Structural Joists & Planks 2"-4" thick, 5" and wider
Select Structural
No. 1
No. 2
No. 3

Appearance Framing 2"-4" thick, 2" and wider
Appearance (A)

Concurrent with the introduction in the 1991 edition of new design values for dimension lumber based on in-grade tests of full-size pieces, design values for Structural Light Framing and Structural Joists and Planks are no longer separately tabulated in the Specification. In the 1991 edition, values for these classifications 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 Tables 4A and 4B). Also in the 1991 edition, listing of design values for the Appearance grade, equivalent to the No. 1 grade in terms of strength ratio, has been discontinued. 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.

4.1.3.3 "Beams and Stringers" are uniformly defined in certified grading rules as lumber that is 5" (nominal) or more thick with width more than 2" greater than thickness. Such members, for example 6x10, 6x12, 8x12, 8x16 and 10x14, are designed for use on edge as bending members. Grades for which design values are given in this Specification (Table 4D) are:

Select Structural
No. 1
No. 2
4.1.3.4 "Posts and Timbers" are defined as lumber that is 5" (nominal) or more in thickness but with width not more than 2" greater than thickness. These members, such as 6x6, 6x8, 8x10, and 12x12, are designed to support axial column loads. Grades of lumber in this classification are the same as those for "Beams and Stringers".

4.1.4-Moisture Service Condition of Lumber

Design values tabulated in the Specification (Table 4) 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 the 1991 edition for uses where this limit will be exceeded for a sustained period of time or for repeated periods.

Prior to 1971, the Specification defined moisture conditions of service as either (i) continuously dry or (ii) at or above the fiber saturation point (approximately 30 percent moisture content) or submerged. These criteria were intended to be applied to the condition of the full cross-section of the member. Beginning in the 1971 edition, the dry service condition was set at the maximum 19 percent moisture content level to correspond to provisions in the then new American Softwood Lumber Standard PS20-70 (190). In the same edition, a second dry service condition of 15 percent maximum moisture content for uses involving dimension lumber surfaced at a maximum moisture content of 15 percent was introduced. Separate design value adjustment factors were given where such material was used in service conditions involving higher moisture levels. Provision for the 15 percent moisture content use condition was continued in the Specification until the current edition. With the introduction of new design values based on in-grade tests of full-size pieces, this provision has been discontinued.

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 base for the design. Additional information on moisture conditions in use is given under the Commentary for 2.3.3.

4.1.5-Lumber Sizes

4.1.5.1 The minimum lumber sizes given in Table 1A of the Specification are minimum-dressed sizes established in the American softwood Lumber Standard, PS20-70 (190).

4.1.5.2 Dry net sizes are used in engineering computations for dimension lumber surfaced in the Green condition. This material is manufactured oversize to allow for shrinkage to the maximum 19 percent moisture content condition (190).

4.1.5.3 Beams and Stringers and Posts and Timbers are manufactured in the Green condition to standard Green dimensions (190). The tabulated 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.

4.1.6-End-Jointed or Edge-Glued Lumber

Design values tabulated in the Specification 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 4.1.2.1). This identification indicates the glued product is subject to ongoing quality monitoring, including joint strength evaluation, by the agency.

4.1.7-Resawn or Remanufactured Lumber

Material that has been regraded after resawing qualifies for design values tabulated in the Specification only when identified by the grademark or inspection certificate of an approved agency (see 4.1.2.1).

4.2-DESIGN VALUES

4.2.1-Tabulated Values

Design values tabulated in Tables 4A-4E of the Supplement to the Specification have been taken from grading rules that have been certified by the Board of Review of the American Lumber Standards Committee as conforming to the provisions of the American Softwood Lumber Standard, PS20-70 (190). Such grading rules may be obtained from the rules writing agencies listed in the Supplement.

Tabulated bearing design values parallel to grain, $F_{gb}$, given in Table 2A are based on the provisions of ASTM Standard D245 (18). Values are applicable to clear, straight grain wood (see Commentary for 3.10.1).
Stress-rated boards of nominal 1", 1¼" and 1½" thickness, 2" and wider, of most species, are permitted the design values shown for Select Structural, No.1, No.2, No.3, Stud, Construction, Standard, Utility, Clear Heart Structural and Clear Structural grades as shown in the 2" to 4" thick categories of the Specification, when graded in accordance with the stress-rated board provisions in the applicable grading rules. Table 4A, footnote 2 outlines these provisions in the current edition. Southern Pine stress-rated boards are an exception to this rule as noted in Table 4B, footnote 2 of the Specification. Information on stress-rated board grades applicable to the various species is available from the respective grading rules agencies.

4.2.2-Other Species and Grades

Where design values other than those tabulated in the Specification are to be used, it 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 Commentary for 4.1.2.1).

4.2.3-Basis for Design Values

4.2.3.2-Visually Graded Lumber

Background. General information on the early history of the development of standardized design values for structural lumber is given in the History section at the beginning of the Commentary. From the first edition of the Specification in 1944 through the 1986 edition, design values for visually graded lumber have been based on the provisions of ASTM D245 (13,18). Under this standard, design values are established by reducing the strength properties of clear, straight-grained wood for variability, load duration, and a factor of safety; and then further reducing the resultant values, or basic stresses, to account for the effects of size and permitted grade characteristics such as knots, slope of grain and splits and checks. The reductions for allowable grade characteristics are made through the application of strength ratios which relate the ratio of the strength of a piece containing a permitted characteristic with that of a comparable piece without that characteristic. Such ratios, which are tabulated in the standard, are established by relating the size and location of a permitted characteristic to the width or thickness of the piece in which it occurs.

The methodology of ASTM D245 was originally developed by the U.S. Department of Agriculture's Forest Products Laboratory (60,207). Results of early tests of full-size structural members (42,43,133,222) validated the appropriateness of the approach. Because of the extensive clear wood property data already available for individual species (112), the introduction of the standard procedures enabled the establishment of structural grades and related design stresses for structural lumber without the necessity of testing full-size pieces of each species, size and grade.

Prior to 1966, basic stresses (equivalent to design values for clear, straight-grained material) recommended by the Forest Products Laboratory for individual species and species groups or combinations (57,62) were tabulated in ASTM D245. This practice was discontinued when a new standard concerned with the establishment of clear wood strength values, ASTM D2555, was published in 1966 (20). This standard presents average clear wood strength values and related measures of variability for individual species grown in the U.S. and Canada. Values are based on tests conducted in accordance with the test methods given in ASTM D143 (15) and, when wood density survey are available, additional procedures contained in D2555. Also given in D2555 are standard criteria for establishing the strength value assignments for any combination of species grouped together for marketing purposes.

Excluding changes in individual species values in D2555 as a result of new or additional clear wood property information, and changes in group assignments as a result of application of the D2555 grouping criteria; design values based on D245 methods remained relatively stable from 1944 to 1986 with certain noteworthy exceptions.

One of the aforementioned exceptions has been the basis for tabulated tension design values parallel to grain, $F_t$. In the 1962 and earlier editions of the Specification, tabulated bending design values, $F_b$, were also used for tension design values. This practice, provided for by ASTM D245, was based on the fact that clear wood tension strength parallel to grain is greater than clear wood bending strength, or modulus of rupture (66). This difference was considered to be on the general order of 50 percent. The appropriateness of continuing to assign $F_t$ values equal to $F_b$ values was reevaluated in the 1960's as a result of field experience with connections in tension chords of large bowstring trusses. Subsequent tension tests on full-size pieces of dimension lumber showed that knots and associated distorted grain occurring in commercial grades had a more significant effect on strength in tension than on bending strength (49). As a result of these findings, ASTM D245 was revised to establish strength ratios for tension parallel to grain as 55 percent of the corresponding bending strength ratio for all visually graded lumber. New $F_t$ values reflecting...
this change were published in the 1968 edition of the Specification.

Subsequent to the 1968 edition, additional tension test data for commercial sizes of dimension lumber became available from a number of different sources. Analysis of this new information indicated use of tension strength ratios equal to 55 percent of corresponding bending strength ratios might not reflect the full effect of knots and other permitted characteristics in the wider widths and lower grades of dimension lumber (122). Pending the development of more comprehensive information on the bending and tension strength of full-size dimension lumber, tabulated tension design values parallel to grain in published grading rules were reduced to conservatively reflect available data. The changes involved using a tension strength ratio for Select Structural grades of 8 inch and 10 inches and wider dimension equal to 50 percent and 44 percent, respectively, of the corresponding bending strength ratio; and a tension strength ratio for all other grades of 8 inch and 10 inch and wider dimension equal to 44 percent and 33 percent, respectively, of the corresponding bending strength ratio. These special adjustments in \( F_t \) values were reflected in the 1977 edition of the Specification and continued through the 1986 edition.

Another change of note in design values for visually graded lumber developed under ASTM D245 provisions was the introduction of a grade effect adjustment on modulus of elasticity, \( E \). Based on stiffness tests of three species, three grades and two sizes of joists (225), reductions of 10 percent and 20 percent in average modulus of elasticity were assigned to No. 2 (45 to 54 percent bending strength ratio) and to all lower grades (bending strength ratios less than 45 percent) respectively. Design \( E \) values reflecting these adjustments were first published in the 1968 edition of the Specification. Prior to this time, \( E \) values were considered unaffected by grade characteristics.

A third significant change in design values based on ASTM D245 provisions occurred in 1970 with the adoption of the National Grading Rule for Softwood Dimension Lumber which was authorized by the American Softwood Lumber Standard PS20-70 (190). Prior to this time, grade descriptions for various species of lumber were established primarily to provide material in particular bending strength classes, such as 1200f, 1500f, 1700f. Because the basic strength properties of each species are different, this resulted in different allowable knot sizes (variable visual quality levels) being used with different species to achieve the same general strength ratings. With the introduction of the National Grading Rule in 1970, all grade names, grade descriptions and grade strength ratios became standardized for dimension lumber of all species. Thus, under ASTM D245 procedures, material of a given grade had the same general appearance in terms of visual quality but different design values depending upon the clear wood properties of the species or species group being used. Design values based on full implementation of the National Grading Rule were first published in the 1971 edition of the Specification.

Two additional changes of note took place in tabulated design values between 1944 and 1986. One of these was the use of a maximum reduction for splits and checks (50 percent strength ratio) in the development of tabulated shear design values parallel to grain (horizontal shear), \( F_v \), for most sawn lumber grades in 1968. Previously, tabulated shear design values parallel to grain were keyed to the maximum split or check allowed in the grade at the time of manufacture. Because this procedure did not provide for the increase in split or check length that could occur as a result of drying in service, the conservative position was adopted of assuming a split or check of maximum strength reducing length was present in each piece (see additional discussion in the Commentary for 4.2.3.2 - Shear Stress Factor). Also of note was the change in tabulated compression design values perpendicular to grain, \( F_c \), to a deformation limit basis in 1982 (see Commentary under 4.2.4). Prior to this time, \( F_c \) design values were based on proportional limit stresses.

1991 Design Values. 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 dimension lumber made from most commercially important species in North America (90). The testing program conducted over an eight year period, involved the destructive testing of 70,000 pieces of lumber from 33 species or species groups. A new test method standard, ASTM D4761, was developed to cover the mechanical test methods used in the program (19). A new standard practice, ASTM D1990, was developed to codify procedures for establishing design values for visually graded dimension lumber from test results obtained from in-grade test programs (23).

Design values for bending, \( F_b \), tension parallel to grain, \( F_t \), compression parallel to grain, \( F_c \), and modulus of elasticity, \( E \), for dimension lumber of 14 species or species combinations listed in Tables 4A and 4B of the Supplement to the 1991 edition are based on in-grade test results. These new values replace those previously developed for the four properties using
ASTM D245 procedures. 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 still being established by D245 methods. Further information on the species evaluated in the in-grade program and the general application of D1990 provisions to all dimension grades and species is given in subsequent commentary under this section.

Strength design values based on in-grade test results are generally higher than previous assignments except for \( F_b \) values for the lower grades and larger widths. A significantly greater effect of size (width) on bending design values is reflected in the new values relative to past assignments. However, the same size relationship is applicable to tension design values parallel to grain, resulting in a relative increase in tension design values parallel to grain for the lower grades and wider widths compared to previous assignments. Values of \( F_b \) and \( F_t \) for the smaller widths of Select Structural are in most cases substantially higher than previous values. Although a moderate size effect is recognized for \( F_c \), new design values for this property are generally much higher than previous values for all grades and sizes. Differences between new \( F_b \), \( F_t \) and \( F_c \) values for the No. 1 and No. 2 grades are smaller than previous assignments and, for some species combinations, the same values are used for both grades (23).

Design values for Beams & Stringers, Posts & Timbers and Decking in the 1991 edition continue to be based on ASTM D245 provisions.

**Dimension**

Design values for visually graded dimension lumber grades are given in Tables 4A and 4B of the Specification. All design values for shear parallel to grain, \( F_{\gamma} \), and compression perpendicular to the grain, \( F_{\epsilon_{cl}} \), in these tables are based on ASTM D245 provisions (18). Design values for bending, \( F_b \), tension parallel to grain, \( F_t \), compression parallel to grain, \( F_c \), and modulus of elasticity, \( E \), for the following species or species combinations are based on in-grade tests of full-size pieces using the provisions of ASTM D1990 (23):

**Table 4A:**
- Douglas Fir-Larch
- Douglas Fir-Larch (North)
- Douglas Fir-South
- Eastern Hemlock-Tamarack
- Eastern Softwoods
- Eastern White Pine
- Hem-Fir
- Hem-Fir (North)
- Northern Species

**Spruce-Pine-Fir**  
**Spruce-Pine-Fir (South)**  
**Western Woods**

**Table 4B:**
- Southern Pine
- Mixed Southern Pine

Values of \( F_b \), \( F_t \), \( F_c \), and \( E \) for other species and species combinations (Redwood, cedars and hardwoods) listed in Table 4A are based on ASTM D245 (18) for the grades of Select Structural and No. 2, with values for other grades derived from these index values using the grade models set forth in ASTM D1990.

**Size Adjustments.** Values of \( F_b \), \( F_t \) and \( F_c \) values in Table 4A for all species and species combinations are adjusted for size using the size factors, \( C_F \), tabulated in the front of the table. These factors and those used to develop the size specific values given in Table 4B for certain species combinations are based on the adjustment equation for geometry given in ASTM D1990 (23). 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). Tabulated values in Tables 4A and 4B for \( F_b \) and \( F_t \) are based on the following standardized lengths:

<table>
<thead>
<tr>
<th>Width, in.</th>
<th>Length, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 - 6</td>
<td>12</td>
</tr>
<tr>
<td>8 - 10</td>
<td>16</td>
</tr>
<tr>
<td>12 &amp; wider</td>
<td>20</td>
</tr>
</tbody>
</table>

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 new. Additionally, the modification of \( F_b \) and \( F_t \) for length in the D1990 equation also represents a new adjustment. Based on the total conservatism of these combined adjustments relative to past practice, use of the design values in Tables 4A and 4B for any member span length is considered appropriate.

**Flat Use Factors.** Values of \( C_{fu} \) given in the adjustment factor tables of Table 4A and 4B are based on the size adjustment equation given in ASTM D245. Use of this equation, which also was the basis for flatwise use adjustments in previous editions of the Specification, has been continued because of the limited availability of in-grade data for flatwise bending.
Relative to the test results that are available, the D245 equation gives conservative $C_{fu}$ values.

Wet Service Factors. Values of $C_M$ for $F_b$, $F_t$, $F_c$ and $E$ in Tables 4A and 4B are based on provisions of ASTM D1990 (23). The new wet service reduction for $E$ is somewhat greater and the new wet service reductions for $F_t$ and $F_c$ are somewhat smaller than those given in previous editions of the Specification. Earlier adjustments for these properties as well as those for tabulated shear design values parallel to grain, $F_v$, and for compression design values perpendicular to grain, $F_{cv}$, which remain unchanged, are based on ASTM D245 provisions. The wet service factors for $F_b$, $F_t$, $F_c$ and $E$ account for the increase in cross-section dimensions associated with this exposure.

Shear Stress Adjustment Factor. Adjustment factors, $C_H$, for length of split or shake given in Tables 4A and 4B are based on ASTM D245 provisions (18) and are unchanged from previous editions. These factors provide increases in tabulated shear design values parallel to grain, $F_v$, and for compression design values perpendicular to grain, $F_{cv}$, which remain unchanged, are based on ASTM D245 provisions. The wet service factors for $F_b$, $F_t$, $F_c$ and $E$ account for the increase in cross-section dimensions associated with this exposure.

Application of the shear stress adjustment factors, $C_H$, for Beams and Stringers generally will be limited to the evaluation of the strength of members in service. These grades together with Posts and Timbers are manufactured in the Green condition and reach equilibrium moisture content after seasoning in service. The length of split or shake expected to occur can not be established at the time of design.

Decking

Design values for Decking in 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. Tabulated design values are the same as those published in the 1986 edition except for the wet service and shear stress adjustment factors given in Table 4D for Beams and Stringers and Posts and Timbers are based on the provisions of ASTM D245 (18). Values are unchanged from those published in the 1986 edition except that southern pine and mixed southern pine timber design values were revised subsequent to the publication of the 1991 Specification. These new design values are published in a March, 1992 Errata/Addendum.

Application of the shear stress adjustment factors, $C_H$, for Beams and Stringers generally will be limited to the evaluation of the strength of members in service. These grades together with Posts and Timbers are manufactured in the Green condition and reach equilibrium moisture content after seasoning in service. The length of split or shake expected to occur can not be established at the time of design.

4.2.3.3-Machine Stress Rated and Machine Evaluated Lumber

Design values for $F_b$, $F_t$, $F_c$ and $E$ given in 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. Stress rating machines are set so that pieces passing through the machine will have the average $E$ desired. Values of $F_b$ are based on correlations established between minimum bending strength for lumber loaded on edge and $E$. $F_t$ and $F_c$ values are based on test results for lumber in each $F_b$ grade. Machine settings are monitored and routinely verified through periodic
stiffness and strength testing. Mechanically rated lumber is also 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 \((F_b, F_t, \text{ and } F_c)\) 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 grade 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 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 \(F_b\).

The category of Machine Evaluated Lumber (MEL) lumber is material that is categorized in classes of regularly increasing strength \((F_b, F_t, \text{ and } F_c)\) 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 grade 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 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 \(F_b\).

The category of Machine Evaluated Lumber (MEL) is new to the Specification with the 1991 edition. Design values for this material are characterized by several different levels of \(E\), \(F_t\), and/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.

Tabulated shear design values parallel to grain, \(F_{\gamma}\), and compression design values perpendicular to grain, \(F_{cl}\), for mechanically graded lumber are the same as those tabulated for No. 2 visually graded lumber of the same species in Tables 4A and 4B. The same adjustment factors applicable to visually graded lumber in Tables 4A and 4B are applicable to MSR and MEL lumber except the size factor, \(C_P\), modification.

4.2.4-Modulus of Elasticity, \(E\)

For discussions of basis of shear deflection component in tabulated \(E\) values, floor beam deflection as measured by average \(E\) values, dynamic floor performance, coefficients of variation for modulus of elasticity, and effects of creep and long term loading, see Commentary for 3.5.

4.2.5-Bending, \(F_b\)

4.2.5.1 When tabulated \(F_b\) 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.

4.2.5.4 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, tabulated bending design values, \(F_{bd}\), for Beams and Stringers in Table 4D can not be applied to check loads applied on the wide face and lumber of this designation is not to be used where biaxial bending occurs. Posts and Timbers are graded for bending in both directions and can be used in biaxial bending design situations.

4.2.6-Compression Perpendicular to Grain, \(F_{cl}\)

Tabulated 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 (18). Since 1982, lumber \(F_{cl}\) values tabulated 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 Background commentary under 4.2.3.2). Implementation of this information and the grouping criteria through ASTM D245 in 1971 resulted in a significant reduction in the \(F_{cl}\) 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 (124). This test consists of loading a two-inch wide steel plate bearing on the middle of a 2 by 2 inch by 6 inch long
wood specimen (15). 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 (31,32). This methodology was coupled with field experience to establish a deformation limit of 0.04 inches in the standard 2 inch ASTM D143 test as an appropriate design stress base for applied loads of any duration (124). Stresses at 0.04 inches of deformation for individual species were subsequently 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 (124). 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 (124). The $F_{cL}$ 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 4.2.6 for adjusting tabulated $F_{cL}$ 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 (32). 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.

**4.3-ADJUSTMENT OF DESIGN VALUES**

**4.3.2-Size Factor, $C_F$**

4.3.2.1 Values of $C_F$ for $F_b$, $F_c$, and $F_t$ design values given in Table 4A and used to develop the size specific values for these properties in Table 4B are new adjustments based on the results of in-grade test of dimension lumber (see History and Size adjustments commentary under 4.2.3.2). Prior to the 1991 edition, an adjustment for width was incorporated in the tabulated bending design values for dimension lumber. This adjustment was based on the general equation

$$\left(\frac{d_2}{d_1}\right)^{1/9}$$

from ASTM D245. The only other size adjustment for dimension lumber design values prior to 1991 was a special adjustment factor for $F_t$ values for 8 inch and 10 inch and wider members in the 1977 edition (see Background commentary under 4.2.3.2).

The size adjustments referenced in Table 4B reflect the fact that the tabulated values apply only to 2 and 3 inch thick material and to material up to 12 inches wide. The Dense Structural grades listed in Table 4B are industrial grades governed by special product rules. Design values for these grades are based on ASTM D245 provisions and are unchanged from previous editions.

4.3.2.2 Bending design values for Beams & Stringers and Posts & timbers in Table 4D apply to a 12 inch depth. The size factor equation for adjusting these values to deeper members is based on the formula given in ASTM D245 that is expressed in terms of a 2 inch deep member, or

$$C_F = \left(\frac{2}{d}\right)^{1/9}$$

The equation, developed in 1966 from tests on bending members one inch to 32 inches deep (33), was introduced in the Specification for glued laminated timber beams over 12 inches in depth in the 1971 edition and then to sawn lumber of the same sizes in the 1973 edition. In the 1957 to 1968 editions, an adjustment factor for deep bending members based on the following formula was given in the Specification:

$$C_F = 0.625 \left(\frac{h^2 + 143}{h^2 + 88}\right)$$

This equation, also referenced to a 2 inch bending member depth, was based on tests of bending members up to 16 inches deep (68). Prior to 1957, the Specification contained no size adjustment equations for deep bending members but referenced other publications giving the following size adjustment equation based on tests of bending members up to 12 inches deep (136).
For bending members between 12 and 40 inches deep, the \((d_2 / d_1)^{1/9}\) equation gives values of \(C_{F}\) intermediate to those obtained from the two earlier formulas.

4.3.2.3 (See Commentary for 2.3.8.)

4.3.2.4 Values of \(F_b\) tabulated for decking in 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 size equation

\[
C_F = 1.07 - 0.07 \sqrt[1/9]{\frac{d}{2}} \quad \text{(C4.3-4)}
\]

Adjustment factors for flat use of bending members are based on the \(1/9\) power size equation shown under the commentary for 4.3.2.4. (Also see Commentary for 4.2.3 - Flat Use Factors.)

4.3.3-Flat Use Factor, \(C_{fu}\)

4.3.4-Repetitive Member Factor, \(C_r\)

The 15 percent repetitive member increase in tabulated bending design values, \(F_b\), for lumber 2 to 4 inches thick has been a provision of the Specification since the 1968 edition. The adjustment, recommended in ASTM D245 (18), 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 (34,148,192,227). It reflects two interactions: load-sharing or redistribution of load between framing members and partial composite T or I beam action of the framing member and the covering materials (192). 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 conservative (152,219,220).

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 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 (34). In this case the transverse elements may be mechanical fasteners such as through nailing, 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. Examples of application of the repetitive member factor, \(C_r\), to built-up framing sections are shown in Figure C4.3-1.
member in such an assembly which is adjacent to an opening that is wider than 24 inches also qualifies.

A stud or studs adjacent to openings, including jack or jamb studs (studs continuous from floor to header) are eligible for the repetitive member increase without regard to the number or location of intersecting windows or door headers, provided such studs are part of an assembly of three or more essentially parallel members of equal size spaced not more than 24 inches apart.

Individual members in a qualifying assembly made of different grades and/or species are each eligible for the repetitive member increase if all spacing, number, size, orientation and distributing element requirements set forth in the foregoing paragraphs are met. It is to be noted that the $C_r$ factor only applies to tabulated bending design values, $F_b$.

Examples of the application of the repetitive member increase to wall sections are illustrated in Figures C4.3-2 - C4.3-4.

The applications of the provisions of section 4.3.4 of the Specification that are discussed in this section of the Commentary have been policies of the American Forest & Paper Association since 1974.

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**Figure C4.3-2 Application of NDS Section 4.3.4 to Wall Framing**

![Diagram showing wall sections qualifying as repetitive member assemblies.]

**Figure C4.3-3 Application of NDS Section 4.3.4 to Wall Framing**

![Diagram showing wall sections qualifying as repetitive member assemblies.]

**Figure C4.3-4 Application of NDS Section 4.3.4 to Wall Framing**

![Diagram showing wall sections qualifying as repetitive member assemblies.]

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**Table C4.3.4.3 Wall Elements Comprising (each consisting of one or more studs) Used for Studs in Element**

<table>
<thead>
<tr>
<th>Wall Element (each consisting of one or more studs)</th>
<th>Allowable Bending Design Value</th>
<th>3-Member Assembly Minimum Number of Studding Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>2 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>3 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>4 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>5 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>6 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>7 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>8 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>9 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>10 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>11 - Repetitive if total number of studs in elements 1 and 2 is three or more</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
<tr>
<td>12 - Single if elements 1 and 2 have only one stud each</td>
<td>1.2</td>
<td>1, 2</td>
</tr>
</tbody>
</table>

---

**Formula**

\[ S = \text{Spacing} \]

- Spacing 24 inches or less
- Spacing over 24 inches, as for openings.
- All studs of the same size.

---

*Figure C4.3-2 Application of ADS Section 4.3.4 to Wall Framing*

*Figure C4.3-3 Application of ADS Section 4.3.4 to Wall Framing*

*Figure C4.3-4 Application of ADS Section 4.3.4 to Wall Framing*

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**Sawn Lumber**

62
4.4-SPECIAL DESIGN CONSIDERATIONS

4.4.1-Stability of Bending Members

4.4.1.1 Approximate rules for providing restraint to prevent lateral displacement or rotation of lumber bending members (62) have been included in the Specification since the 1944 edition. Although equations for calculating critical lateral buckling loads for any size and length of bending member were available (184), the early editions utilized the approximate rules in order to simplify design for general applications while referencing the complex design equations for use in special situations. The rules became an alternate method of dealing with the stability of sawn lumber bending members when procedures introduced in 1968 for adjusting design values for glued laminated timber bending members for slenderness were extended to sawn lumber members in the 1977 edition.

4.4.1.2 Although the approximate rules for providing lateral restraint of bending members given in various editions of the Specification are generally similar, certain changes in the rules have occurred as a result of changes in construction practices and changes in interpretation.

**Background** Until the 1973 edition, separate rules for lateral deflection of floor joists and lateral deflection of beams and roof joists were provided in the Specification. In the case of the former, it was assumed that the compressive edge of the joist was held in line by sheathing or subflooring. In addition to this restraint, bridging at 8 feet or less intervals along the length of the joist, or if the depth to breath ratio was greater than 6, at spacing not more than 6 times the depth, was required. By 1962, the floor joist bridging requirement had been modified to require use of such bracing at 8 foot intervals only when the depth to thickness ratio of the joist was 6 or larger. In 1973, the separate treatment of floor joists was discontinued with all bending members consolidated under the same rules. These 1973 edition rules were for solid sawn rectangular beams and roof joists:

**Depth to breadth ratio** (nominal dimensions)

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 to 1</td>
<td>no lateral support shall be required</td>
</tr>
<tr>
<td>3 to 1</td>
<td>ends shall be held in position</td>
</tr>
<tr>
<td>4 to 1</td>
<td>the piece shall be held in line as in a well-bolted truss chord member</td>
</tr>
<tr>
<td>5 to 1</td>
<td>one edge shall be held in line</td>
</tr>
</tbody>
</table>

6 to 1 bridging shall be installed at intervals not exceeding 8 feet

7 to 1 both edges shall be held in line

The original published source of the rules (62) indicates that the 4 to 1 case was addressing a bolted or spiked vertically laminated or built-up chord member. Also this source used flooring nailed to the edge of a joist as an example of the 5 to 1 rule for a joist, and the combination of rafters (or roof joists) and diagonal sheathing as an example of the 5 to 1 rule for a beam. In the latter case, the beam is the member being checked for stability under loads transmitted through the rafters.

In the 1977 edition, the 4 to 1 case was consolidated with the 3 to 1 case to refer to restraint at the ends of the piece. Also in this edition, the rules for 5 to 1 and 7 to 1 were revised to require one or both edges to be held in line for their entire length; and the 6 to 1 rule was modified to accept either continuous compression edge support and support at bearing points or two edge continuous support as alternates to bridging. The rules in the 1977 edition have been carried forward unchanged to the 1991 edition.

**Application of the Approximate Rules.** The original form of the rules and the history of interpretations indicate the rules should generally be applied cumulatively. This means the requirement for end support for the 3 to 1 and 4 to 1 cases should also be applied to the 5 to 1, 6 to 1 and 7 to 1 cases; and the requirement for one edge held in line for the 5 to 1 case should also be applied to the 6 to 1 case. Further, the requirement for one edge to be held in line should be taken to mean the compression edge. Sheathing, subflooring or decking attached with two or more fasteners per piece is acceptable edge restraint for a joist, rafter or beam loaded through such sheathing, subflooring or decking. Rafters or joists attached to the side of a beam and stabilized through the attachment of sheathing, subflooring or decking are acceptable edge restraint for a beam that is loaded through such rafters or joists.

The requirement for bridging in the form of diagonal cross bracing or solid blocking in the 6 to 1 case and the requirement for both edges to be supported in the 7 to 1 case are practices concerned with (i) partial redistribution of traffic or other concentrated loads on beams of long span to adjacent members and (ii) with assuring that deep beams do not cup or twist significantly as a result of drying in service. Therefore the bridging requirement in the 6 to 1 case is not applicable to the 7 to 1 case, being replaced by the...
requirement in the later for both edges to be held in line.

The approximate rules of 4.4.1.2 are equivalent to the beam stability provisions of 3.3.3.3 at the 5 to 1 depth to breadth ratio where edge support along one edge and lateral restraint at bearing points are required. For larger depth to breadth ratios the approximate rules require more restrictive measures than those of 3.3.3.3. For smaller ratios the approximate rules will be less restrictive, with the difference between effective bending stress based on the two methods increasing as $F_b$ increases and $E$ decreases.

4.4.1.3 The rule permitting up to a 5 to 1 depth to breadth ratio for beam-columns having one edge held in line has been a provision of the Specification since the 1944 edition. The provision allowing this breadth to depth ratio to increase to 6 to 1 when the unbraced edge of the member being checked was in tension under all load combinations was introduced in the 1977 edition. In earlier editions, the 6 to 1 ratio for beam-columns was permitted when the dead load on the rafters providing lateral support was sufficient to induce tension on their underside. This latter requirement was concerned with assuring that the rafters, carrying uniform loads transferred from the sheathing, had sufficient reserve capacity to resist any extra load that is induced in them by the tendency of the beam to buckle.

Early tests of heavily stressed biaxial beam-columns requiring lateral support showed that the members providing such support could themselves buckle under the combination of the load directly on them and that induced by the tendency of the beam to buckle (184). Where stabilized rafters are providing lateral support to the compression edge of beam-columns, the rafters, which typically will have one edge unbraced, should have sufficient capacity to carry the compression load produced by the tendency of the beam to buckle.

4.4.2 Shear Parallel to Grain (Horizontal Shear) in Bending Members

4.4.2.1 Cross sections in continuous and cantilever sawn lumber bending members 2 to 4 inches in thickness which are five times or more the depth of the member from the end of the member are assigned allowable shear design values parallel to grain that are twice the applicable values given in Tables 4A and 4C, or twice 90 psi for Mixed Southern Pine and Southern Pine, and twice 80 psi for Redwood. The tabulated or equivalent values assume a maximum 50 percent strength ratio end split, check or shake is present in each piece (see discussion under Shear Stress Adjust-

4.4.2.2 The alternate procedures for calculating the shear force, $V$, in single span, unnotched sawn lumber bending member may be employed only when the shear force calculated in accordance with 3.4.3.1 does not result in an actual shear stress, $f_v$, that exceeds the maximum allowable shear design value parallel to grain for an unsplit or unchecked member as given in 3.4.3.2. This value, before application of other adjustment factors, is twice the tabulated value given in Tables 4A, 4C and 4D, except twice 90 psi for Mixed Southern Pine and Southern Pine and twice 80 psi for Redwood.

For qualifying bending members, the alternate procedures provide for lower values of shear force by accounting for the redistribution of shear stresses that is considered to occur at the ends of members containing splits, checks and shakes. The procedures are referred to as "two-beam shear" design provisions. It is to be noted that the actual shear stress, $f_v$, associated with the reduced shear force, $V$, calculated in accordance with the two-beam shear procedures of 4.4.2.2 is checked against an allowable shear design value parallel to grain that contains the maximum 50 percent reduction for end splits, checks and shakes. Such a design value is the tabulated shear design value parallel to grain, $F_v$, given in Tables 4A, 4C and 4D, except 90 psi for Mixed Southern Pine and Southern Pine and 80 psi for Redwood, multiplied by any applicable adjustment factors.

Background

The two-beam shear design provisions were developed in 1934 to account for the field performance of large beams used in highway and railroad bridges (138). Severely checked beams were observed to carry significantly higher loads than those predicted using traditional shear design methods and the reduced shear areas associated with the checks (131,138). From the results of tests of 200 built-up artificially checked beams ranging from 3/4 by 1-1/2 inches to 8 by 16 inches in cross-section and approximate mathematical analyses, it was determined that the upper and lower halves of a checked beam carry a portion of the shear independently of that at the neutral axis (139). This proportion of the shear load carried by the two halves was found to vary inversely with the square of the distance of the load from the support, or
\[ V = B + \frac{A}{a^2} \]  
\[ V = (2/3) Jbd + \frac{(1/6) Eu bd^3}{a^2} \]

where:

- \( J \) = mean shear stress at the neutral axis
- \( E \) = modulus of elasticity
- \( d \) = depth
- \( b \) = breadth (no reduction for checks)
- \( a \) = distance of load from support
- \( u \) = longitudinal shear displacement

The first component, \( B \), of the shear force or reaction is referred to as the "single-beam" portion and the second component, \( A/a^2 \), is referred to as the "two-beam" portion. The two-beam portion of the reaction increases rapidly as the point of application of the load approaches the support so that at loads close to the support almost all of the reaction is carried by the upper and lower halves of the beam. This two-beam action also accounts for the fact observed in all the tests that the point of application of the minimum shear failure load was three or more times the depth of the beam from the support (139). These relationships are illustrated in Figure C4.4-1.

Test results showed that the values of \( J \) and \( A \) in the foregoing equation were independent of the position of the load; that, for a beam of a given size and placement of load, the ratio of two-beam to total reaction at failure was approximately constant with varying depth of checks; and that with beams of different dimensions the ratio of two-beam to total reaction was approximately constant when the load was placed at a constant \( a/d \) ratio (ratio of distance of load from support to the depth of the beam) (139). Assuming the numerical value of two-beam to total reaction is 2/11 when the load is at a distance from the support of three times the depth of the beam, the following equation for the reaction associated with a concentrated load was developed (139):

\[ V = \frac{P(L-a)(a/d)^2}{L[2+(a/d)^2]} \]

where:

- \( V \) = shear force or reaction
- \( P \) = concentrated load
- \( L \) = span length
- \( d \) = beam depth

For a uniform load over the entire span, the area under the straight line represents the total reaction; the shaded area, the single-beam reaction; and the unshaded area, that portion of the reaction which is carried to the support by two-beam action, and is not associated with shear in the neutral plane.

The following equation for the reaction associated with a uniformly loaded simple beam also was derived:

\[ V = \frac{wL}{2} - \frac{wd}{2.3 \log \left( \frac{L^2+2d^2}{2d^2} \right) \tan(L/2)} \]

where:

- \( w \) = uniform load per unit length of span

An approximation of this uniform load equation was proposed by the developers as

\[ V = \frac{wL}{2} - \frac{wd}{2.3 \log \left( \frac{L^2+2d^2}{2d^2} \right) \tan(L/2)} \]
This simplification indicates that two-beam shear action in checked beams reduces the shear force calculated by neglecting the uniform load within a distance $d$ of the supports by 10 percent.

The foregoing shear force equations were intended for use with the ordinary shear stress equations with the resulting actual shear stress to be compared with allowable shear design values parallel to grain appropriately reduced for splits, checks and shakes. Because the shear design values parallel to grain recommended in 1934 were based on beam tests considered to contain a 10 percent two-beam shear reaction, the developers of the two-beam shear force equations proposed that a 10/9 factor be applied to both formulas. The final two-beam shear equations proposed for design use in 1934 were

$$V = \frac{10P(L-a)}{9L[2+(a/d)^2]}$$  \hspace{1cm} (C4.4-6)

$$V = \frac{wL}{2}\left(1 - \frac{2d}{L}\right)$$  \hspace{1cm} (C4.4-7)

The recommendations (139) for application of the two-beam shear provisions at the time were

(a) Neglect all loads within the height of the beam from each support

(b) Place any heavy concentrated moving load at three times the height of the beam from the support

(c) Treat all other loads in the usual manner

(d) If the beam does not qualify under (a), (b) and (c), calculate the shear load for concentrated loads from the equation (C4.4-6) for this load case

**Application of Two-Beam Shear Provisions in the Specification.** The foregoing recommendations for application of the two-beam shear procedures were introduced in the 1944 edition of the Specification as proposed except that the equation for the uniform load case (C4.4-7) was included under part (d). Actual shear stresses, $f_s$, based on the reduced shear force values were required to be checked against tabulated shear design values parallel to grain which contained adjustments for splits, checks and shakes permitted in the member at the time of manufacture.

The two-beam shear provisions in the 1944 edition were continued unchanged in the Specification until the 1962 edition. In this edition, the procedures were revised to permit actual shear stress values involving two-beam shear methodology to be checked against the shear design value parallel to grain for a member containing no splits, checks or shakes rather than the tabulated shear design values. Shear design values parallel to grain for members without checks or splits were up to 50 percent higher than tabulated shear design values for some species and grades.

The change in the 1962 edition followed a reevaluation of the application of the two-beam shear provisions to sawn lumber and glued laminated timber (141). This reevaluation confirmed that the methodology should not be applied to glued laminated timber or to sawn lumber known to be free of splits and checks. Further, it was noted that in addition to checking the actual stresses associated with two-beam shear forces against tabulated shear design values parallel to grain, the resultant loads in any case should not produce stresses when calculated in the conventional manner that exceeded equivalent values for split and check free material. When this new limitation was introduced into the Specification, it was considered a replacement of the other checking condition which was dropped.

The two-beam shear provisions in the 1962 edition and the checking provisions introduced in that edition were carried forward up to the 1986 edition. In 1971, tabulated shear design values parallel to grain for sawn lumber were revised to reflect implementation of new clearwood strength values and the grouping criteria in ASTM D2555, new drying adjustments for shear in ASTM D245, and the use of a maximum reduction for splits and checks (50 percent strength ratio) for most sawn lumber grades (see Commentary for 4.2.3.2 - Background and Shear Stress Adjustment Factor). As a result of these changes, split and check free shear design values parallel to grain generally increased and became a uniform 2.0 times the comparable tabulated shear design values parallel to grain.

Although the two-beam shear provisions of the Specification continued to give designs having satisfactory performance in all applications where they were known to be employed, such as bridge timbers, headers and concrete formwork, the stresses used to check two-beam shear forces were revised in the 1977 edition to 1.5 times rather than twice tabulated design values. This change, continued in the 1982 edition, was made to bring the checking stresses back to the levels that existed prior to 1971.
In 1986, the two-beam shear provisions of the Specification were revised to correct a number of inconsistencies that were identified as a result of close review of the early literature concerned with the development and application of the two-beam shear methodology. One of the changes reinstated the requirement that actual stress based on the reduced end reaction obtained from the two-beam shear equations be checked against the tabulated shear design value parallel to grain for split or checked members. Concurrent with this change, a second requirement was added that the actual stress based on the conventional formulas, including neglecting loads within the height of the beam from the support, not exceed the shear design value parallel to grain for unsplit and unchecked beams (twice the tabulated). These revisions provide for application of the two-beam methodology as originally intended (141).

A second change made in the 1986 edition was the elimination of the 10/9 factor in the two-beam shear equation for concentrated load (Equation C4.4-6). As previously discussed, this factor was added to both the concentrated load and the approximate uniform load equations by the developers to account for the estimated ten percent of two-beam reaction that was considered present in the large beam tests from which shear design values parallel to grain were derived in the 1934 era (139).

Also in the 1986 edition, the beam length term to be used in the two-beam shear equations was specifically defined as the clear span, \( l_c \), or the distance between the faces of the supports. A similar clarification was made to the load location term, \( x \) (equivalent to \( a \) in Commentary Equations C4.4-3 and C4.4-6), which was defined as the distance between the face of the support and a concentrated load. Under these definitions, the total uniform load, \( W \), used in Equation 4.4-1 of the Specification, became equal to the unit uniform load times the clear span or \( w \ell_c \) (see Commentary Equation C4.4-8).

Since 1944, shear design values parallel to grain have been developed from small, clear block shear specimens (15) which contain no seasoning defects. Further, the standard 4.1 factor for shear in ASTM D245 that is used to establish design values for split and check free sawn lumber includes a reduction factor of 4/9 (44 percent) to account for stress concentration effects associated with checks and shakes (54). In addition to reducing the area of the member resisting shear forces, checks and shakes are considered to have stress concentrations at their edges that significantly reduce the average shear strength of the remaining area below that of unchecked wood as determined from the standard block shear test (112). The 4/9 factor is intended to account for this behavior which is independent of check or shake size. Reductions for specific sizes of checks, shakes and splits are provided separately in ASTM D245.

In view of the methods used to establish present shear design values parallel to grain, the 10/9 calibration factor originally included in the two-beam shear equations was no longer considered applicable and was therefore dropped in the 1986 edition. Because of the confusion caused by the approximate equation for the uniform load case (Equations C4.4-5 and C4.4-7), being only a formula for neglecting loads within a beam depth from the support, and the difference in results obtained when using this approximation instead of the exact expression (Equation C4.4-4) at various span/depth ratios, a partially simplified form of the exact two-beam shear equation for the uniform load case was introduced in the 1986 edition. This was done by applying an adjustment factor based on span/depth ratio to the basic shear force equation neglecting loads within a beam depth from the support, or

\[
V = \frac{W}{2} \left(1 - \frac{2d}{\ell_c}\right) K_v \quad (C4.4-8)
\]

where:

\[
W = \text{total uniform load}
\]

\[
= w \ell_c
\]

with

\[
w = \text{uniform load per unit length of span}
\]

\[
\ell_c = \text{clear span measured from face to face of the supports}
\]

\[
K_v = 0.95 + \frac{\sqrt{\ell_c/d}}{250} - 1.32 \left(\frac{d}{\ell_c}\right) + 11.5 \left(\frac{d}{\ell_c}\right)^3 \leq 1.0 \quad (C4.4-9)
\]

Equations C4.4-8 and C4.4-9 give values of \( V \) within 0.6 percent or less of those obtained from the exact equation (C4.4-4). As can be seen from Figure C4.4-2, values of \( K_v \) range from a low of about 0.79 at an \( \ell_c/d \) of 5 to a high of 0.95 at an \( \ell_c/d \) of 48. Thus the reductions in shear force obtainable with the revised two-beam shear equation for uniform loads relative to the conventional methodology neglecting loads close to the support range from 5 to 21 percent. The two-beam shear equation for concentrated loads, the case for which the methodology was primarily developed, can result in greater reductions.
General Applicability. The general applicability of the two-beam shear provisions as a strength model has been questioned because the procedures are independent of check or crack depth (164). In this regard, it is to be noted that the provisions, based on the results of checked beams, were developed for the purpose of rationalizing the field performance of deeply checked bridge timbers in which the depth and length of checks are unknown and variable. The methodology has been applied to beams containing end splits which also are of unknown and variable length. Because checks and splits can develop or increase in depth and length as a result of drying in service, more rigorous theoretical methods which require knowledge of the split length or check depth and cumulative length are not directly applicable to the practical design situation (see Commentary for 4.2.3.2 - Shear Stress Adjustment Factors). However, one new methodology using linear elastic fracture mechanics shows that the end split adjustment factors given in ASTM D245 for dimension lumber are too conservative (29). It is to be noted that use of two-beam shear provisions with lumber assigned a shear design value parallel to grain that has not been reduced for checks or splits is not permitted.

In evaluation of shear design procedures for lumber, it should be kept in mind that tabulated shear design values parallel to grain assume a split, check or shake equivalent to a 50 percent strength ratio is present in every piece. A greater reduction than 50 percent is not taken because a beam split lengthwise at the neutral axis will still carry as two half-depth beams one-half the bending moment of a comparable unsplit beam, neglecting partially offsetting size effect adjustments and assuming no change in grade effects. The total shear load capacity of the two half beams is the same as that for the original beam (54). In terms of maximum load carrying capacity, bending is the critical property. Reports of lumber beams split full length that have been unable to carry design loads are not generally available. Those beams most likely to check and end split are those having straight grain and a higher proportion of clear wood so that the bending strength of any two resulting beams is likely to be sufficient to carry the design loads (131).

The foregoing is supportive of the field performance record: namely, that heavily checked bridge timbers can carry higher loads than conventional shear design methods would indicate; and that sawn lumber beams designed under two-beam shear provisions, which include a 4/9 stress concentration reduction in the checking stresses, have provided satisfactory service for over four decades.

1991 Provisions. The 1986 two-beam shear provisions have been carried forward unchanged to the 1991 edition. The present provisions are considered to have reduced the relative benefits obtainable from the two-beam methodology from 25 to 40 percent below those obtainable from the procedures in the 1977 and 1982 editions, and by a greater amount compared to those obtainable from the procedures in earlier editions.

It should be noted that the value of \( x \) to be used in the equation for concentrated loads (Equation 4.4-2 of the Specification) is the distance between the face of the support and the load, rather than the distance between the reaction and the load. This is consistent with the use of the face of the support as the reference for the clear span, \( l_c \) (see 1.6 in the Specification). Also to be noted, the value of \( W \) to be used in the equation for uniform loads (Equation 4.4-1 of the Specification) is the unit uniform load times the clear span or \( W l_c \).

In applying the provisions to loading cases involving a combination of concentrated loads, the sum of the reduced reactions from each load, when the combination of loads is placed to give the largest total, is used to determine the actual stress. This methodology is illustrated in Example C4.4-3. Examples C4.4-1 and C4.4-2 are intended to illustrate application of shear parallel to grain design provisions only. Other design provisions such as bending, compression perpendicular to grain and deflection are not considered here but are covered elsewhere in the Specification.
Example C4.4-1

No. 2 Douglas Fir-Larch 2x4 joists, used in concrete formwork, are supported over a single span on 2x6 stringers spaced 27 in. on center. The joists are spaced at 24-in. with a uniform load on each joist of 600 lb/ft. Assume 7-day duration of load and wet service conditions apply. Check to see if joists satisfy NDS shear provisions.

\[ F_v = 95 \text{ psi} \quad C_M = 0.97 \quad C_D = 1.25 \quad (\text{Table 4A}) \]

\[ b = 1.5 \text{ in.} \quad d' = 3.5 \text{ in.} \]

Allowable Shear Design Value

Parallel to Grain, \( F_v' \) (Table 2.3.1)

\[ F_v' = F_v C_D C_M = (95)(1.25)(0.97) = 115 \text{ psi} \]

Shear Force, \( V \) (3.4.3.1)

Neglecting loads within a distance \( d \) of the face of the support:

\[ V_{max} = w(\ell/2 - (d + 1/2 \text{ support})) \]
\[ = (600)(1/12)(27/2 - (3.5 + 1.5/2)) \]
\[ = 463 \text{ lb} \]

Actual Shear Stress, \( f_v \) (3.4.2)

\[ f_v = \frac{3V}{2bd} = \frac{(3)(463)}{(2)(1.5)(3.5)} \]
\[ = 132 \text{ psi} > F_v' = 115 \text{ psi} \quad \text{ng} \]

Member does not satisfy shear provisions based on 3.4.3.1; try alternate two-beam shear provisions.

Two-Beam Shear (4.4.2.2)

two-beam shear qualification check: (3.4.3.2)

\[ f_v < (2.0)F_v C_D C_M C_i \]

\[ f_v = 132 \text{ psi} < (2.0)(95)(1.25)(0.97)(1.0) = 230 \text{ psi} \quad \text{ok} \]

\[ \ell_c = 27.0 - 1.5 = 25.5 \text{ in.} \quad \text{(clear span)} \]

\[ W = w\ell_c = (600)(25.5/12) = 1275 \text{ lb} \]

\[ K_v = 0.95 + \frac{\sqrt{\ell_c/d}}{250} - 1.32 \left( \frac{d}{\ell_c} \right) + 11.5 \left( \frac{d}{\ell_c} \right)^3 \leq 1.0 \]

\[ = 0.95 + \frac{\sqrt{25.5/3.5}}{250} - 1.32 \left( \frac{3.5}{25.5} \right) + 11.5 \left( \frac{3.5}{25.5} \right)^3 \]

\[ = 0.809 \]

\[ V = \frac{W}{2} \left( 1 - \frac{2d}{\ell_c} \right) K_v \quad (\text{Eq. 4.4-1}) \]

\[ = \left( \frac{1275}{2} \right) \left( 1 - \frac{2(3.5)}{25.5} \right)(0.809) = 374 \text{ lb} \]

Actual Two-Beam Shear Stress, \( f_v \) (3.4.2, 4.4.2.2)

\[ f_v = \frac{(3)(374)}{(2)(1.5)(3.5)} < F_v C_D C_M C_i \]

\[ = 107 \text{ psi} < (95)(1.25)(0.97)(1.0) = 115 \text{ psi} \quad \text{ok} \]

2x4 joists satisfy NDS shear criteria
Example C4.4-2

A built-up girder of three select structural Southern Pine 2x10 members, supports a portion of a second story floor of a two-story, 32 ft wide house, plus a load bearing wall along its length. The girder spans 10 ft and carries a uniform load of 865 lb/ft @L+LL. Support lengths are 4.5 in. at each end. Check girder for NDS shear criteria and modify design if necessary.

\[ F_v = 90 \text{ psi} \quad C_M = 1.0 \quad C_D = 1.0 \quad (\text{Table 4B}) \]
\[ b = 1.5 \text{ in.} \quad d = 9.25 \text{ in.} \]

**Allowable Shear Design Value**

**Parallel to Grain, \( F_v' \)**

\[ F_v' = F_v C_D C_M = (90)(1.0)(1.0) = 90 \text{ psi} \]

**Shear Force, \( V \)**

(3.4.3.1)

Considering each 2x10 separately and neglecting loads within a distance \( d \) of the face of the support:

\[ V_{max} = w(L/2 - (d + 1/2 \text{ support})) \]

\[ = (865/3)(1/12)(10)(12)/2 - (9.25 + 4.5/2) \]

\[ = 1165 \text{ lb} \]

**Actual Shear Stress, \( f_v \)**

(3.4.2)

\[ f_v = \frac{3V}{2bd} = \frac{(3)(1165)}{(2)(1.5)(9.25)} \]

\[ = 126 \text{ psi} > F_v' = 90 \text{ psi} \quad \text{ng} \]

Members do not satisfy shear provisions based on 3.4.3.1; try alternate two-beam shear provisions.

**Two-Beam Shear**

(4.4.2.2)

two-beam shear qualification check:

\[ f_v < (2.0)F_v C_D C_M C_t \]

\[ f_v = 126 \text{ psi} < (2.0)(90)(1.0)(1.0) = 180 \text{ psi} \quad \text{ok} \]

\[ \ell_c = (10)(12) - 4.5 = 115.5 \text{ in.} \quad (\text{girder span}) \]
\[ W = w\ell_c = (865/3)(115.5/12) = 2775 \text{ lb} \]
\[ K_v = 0.95 + \frac{\ell_c}{250} \left[ -1.32 \left( \frac{d}{\ell_c} \right)^3 + 11.5 \left( \frac{d}{\ell_c} \right) \right] \leq 1.0 \]
\[ = 0.95 + \frac{115.5/9.25}{250} \left[ -1.32 \left( 9.25 \right)^3 + 11.5 \left( 9.25 \right) \right] \]
\[ = 0.95 + 0.864 \]
\[ = 1.814 \]
\[ V = \frac{W}{2} \left( 1 - \frac{2d}{\ell_c} \right) K_v \quad (\text{Eq. 4.4-1}) \]
\[ = \frac{2775}{2} \left( 1 - \frac{2d}{115.5} \right) (0.864) = 1007 \text{ lb} \]

**Actual Two-Beam Shear Stress, \( f_v \)**

(3.4.2, 4.4.2.2)

\[ f_v = \frac{(3)(944)}{(2)(1.5)(11.25)} < F_v C_D C_M C_t \]
\[ = 84 \text{ psi} < (90)(1.0)(1.0)(1.0) = 90 \text{ psi} \quad \text{ng} \]

Members do not satisfy alternate two-beam shear provisions of 4.4.2.2. Also, since the occurrence and length of splits, etc. near the ends of the members are unknown, an increase in the allowable shear design value parallel to grain by the shear stress adjustment factor, \( C_H' \), is unjustifiable; so members (and girder) fail all NDS shear criteria and design must be modified. Try three No. 1 Southern Pine 2x12 members. No. 1 grade is satisfactory for bending, bearing and deflection criteria; check new design for shear.

\[ F_v = 90 \text{ psi} \quad C_M = 1.0 \quad C_D = 1.0 \quad (\text{Table 4B}) \]
\[ b = 1.5 \text{ in.} \quad d = 11.25 \text{ in.} \]

**Allowable Shear Design Value**

**Parallel to Grain, \( F_v' \)**

(2.3.1)

\[ F_v' = F_v C_D C_M = (90)(1.0)(1.0) = 90 \text{ psi} \]

**Shear Force, \( V \)**

(3.4.3.1)

Considering each 2x12 separately and neglecting loads within a distance \( d \) of the face of the support:

\[ V_{max} = (865/3)(1/12)(10)(12)/2 - (11.25 + 4.5/2) \]
\[ = 1117 \text{ lb} \]

**Actual Shear Stress, \( f_v \)**

(3.4.2)

\[ f_v = \frac{3V}{2bd} = \frac{(3)(1117)}{(2)(1.5)(11.25)} \]
\[ = 99 \text{ psi} > F_v' = 90 \text{ psi} \quad \text{ng} \]

Members do not satisfy shear provisions based on 3.4.3.1; try alternate two-beam shear provisions.

**Two-Beam Shear**

(4.4.2.2)

two-beam shear qualification check:

\[ f_v = 99 \text{ psi} < (2.0)(90)(1.0)(1.0) = 180 \text{ psi} \quad \text{ok} \]

\[ \ell_c = (10)(12) - 4.5 = 115.5 \text{ in.} \quad (\text{girder span}) \]
\[ W = w\ell_c = (865/3)(115.5/12) = 2775 \text{ lb} \]
\[ K_v = 0.95 + \frac{\ell_c}{250} \left[ -1.32 \left( \frac{d}{\ell_c} \right)^3 + 11.5 \left( \frac{d}{\ell_c} \right) \right] \leq 1.0 \]
\[ = 0.95 + \frac{115.5/9.25}{250} \left[ -1.32 \left( 9.25 \right)^3 + 11.5 \left( 9.25 \right) \right] + 11.5 \left( \frac{9.25}{115.5} \right)^3 \]
\[ = 0.95 + 0.845 \]
\[ = 1.795 \]
\[ V = \frac{2775}{2} \left( 1 - \frac{2d}{\ell_c} \right) K_v \quad (\text{Eq. 4.4-1}) \]
\[ = \frac{2775}{2} \left( 1 - \frac{2d}{115.5} \right) (0.845) = 944 \text{ lb} \]

**Actual Two-Beam Shear Stress, \( f_v \)**

(3.4.2, 4.4.2.2)

\[ f_v = \frac{(3)(944)}{(2)(1.5)(11.25)} < F_v C_D C_M C_t \]
\[ = 84 \text{ psi} < (90)(1.0)(1.0)(1.0) = 90 \text{ psi} \quad \text{ok} \]

2x12 members (and girder) satisfy NDS shear criteria.
Example C4.4-3

A No. 1 Douglas Fir-Larch 6x24 stringer from a single lane bridge spanning 20 ft, is subject to a uniform dead load of 208 lb/ft and moving live (wheel) loads of 9360 lb and 2340 lb spaced 10 ft apart. Required bearing length is 6 in. at each end. Assume wet service conditions apply. Check member for NDS shear criteria.

Place the largest moving load at the closest to the support of 3d or the quarter points (4.4.2.2 (a)); neglect loads within a distance d from face of support for other loads.

Distances from center of support:

3d + 1/2 support = ((3)(23.5) + 6/2)/12 = 6.125 ft

quarter points = 20/4 = 5 ft controls

V_DL = 1621 lb

V_LL = (1/2)([9360(20 - 5 ft) + 2340(20 - 10 ft + 5 ft)])
= (1/20)(9360(20 - 5) + 2340(20 - 10 + 5))
= 7605 lb

V_TL = V_DL + V_LL = 1621 + 7605 = 9226 lb

Actual Two-Beam Shear Stress, f_v (3.4.2, 4.4.2.2 (a))

f_v = \frac{(3)(9226)}{(2)(5.5)(23.5)} < F_v C_D C_M C_i
= 107 psi > (85)(1.0)(1.0)(1.0) = 85 psi ng

Member does not satisfy two-beam shear provisions of 4.4.2.2 (a). Try 4.4.2.2 (b) provisions.

Two-Beam Shear (4.4.2.2 (b))

 required bearing length at each end
20 - 2(6/2)/12 = 19.5 ft = 234 in. (clear span)

Dead load:

W = w \ell = (208)(19.5) = 4056 lb

K_v = 0.95 + \sqrt{\frac{\ell_c d}{250}} - 1.32 \left( \frac{d}{\ell_c} \right)^3 + 11.5 \left( \frac{d}{\ell_c} \right) \leq 1.0

= 0.95 + \sqrt{\frac{234}{234}} - 1.32 \left( \frac{23.5}{234} \right) + 11.5 \left( \frac{23.5}{234} \right)^3
= 0.842

V_DL = \frac{W}{2} \left( 1 - \frac{2d}{\ell_c} \right) K_v
(Eq. 4.4-1)

= \left( \frac{4056}{2} \right) \left( 1 - \frac{2(23.5)}{234} \right)(0.842) = 1365 lb

Live load - maximum shear is obtained when the heaviest load is placed at 2.6d from the face of the support (close to quarter point):

V_LL = \frac{(P)(\ell_c - x)(x/d_c)^2}{(\ell_c)(2 + (x/d_c)^2)}

9360 lb at x = (2.6)(23.5)/12 = 5.092 ft from support

(cont.)
Example C4.4-3 (cont.)

\[
V_{360} = \frac{(9360)(19.5 - 5.092)(5.092/(23.5/12))^2}{(19.5)[2 + (5.092/(23.5/12))^2]}
\]
\[= 5337 \text{ lb}\]
\[V_{2340} = \frac{(2340)(19.5 - 15.092)(15.092/(23.5/12))^2}{(19.5)[2 + (15.092/(23.5/12))^2]}
\]
\[= 512 \text{ lb}\]

\[V_{TL} = V_{DL} + V_{9360} + V_{2340} = 1365 + 5337 + 512\]
\[= 7214 \text{ lbs}\]

Actual Two-Beam Shear Stress, \(f'_v\) (3.4.2, 4.4.2.2 (b))

\[f'_v = \frac{(2)(7214)}{(2)(5.5)(23.5)} < F_{CD}C_M C_t\]
\[= 84 \text{ psi} < (85)(1.0)(1.0)(1.0) = 85 \text{ psi ok}\]

6x24 member satisfies NDS shear criteria

4.4.3-Wood Trusses

4.4.3.1 Provisions to recognize the contribution of plywood sheathing to the buckling resistance of compression chords (in the plane of the chord depth) were introduced in the 1977 edition of the Specification. This addition was made to partially offset the reduction of allowable spans of residential roof trusses, which had a long history of satisfactory performance, associated with changes in column design formulas and related duration of load adjustments in the 1977 edition.

Quantification of the increase in chord buckling resistance from plywood sheathing was based on new research involving stiffness tests of sheathing and 2x4 T-beams, nail slip tests, use of existing methodology for estimating the nail slip modulus of combinations of materials and application of a new finite element analysis program for layered wood systems. It was found that sheathing contribution increases with decrease in modulus of elasticity of the chord, with increase in span and with increase in slip modulus. Effects of plywood thickness and chord specific gravity were found to be of lesser significance.

The buckling stiffness factor equations introduced in the 1977 edition were of the form

For lumber at 19 percent or less moisture content at time plywood attached:

\[C_T = 1 + 0.002 \ell_e\] (C4.4-10)

For lumber unseasoned or partially seasoned at time plywood attached:

\[C_T = 1 + 0.001 \ell_e\] (C4.4-11)

where:

\(\ell_e\) = the effective length used in the design of the compression chord

The difference between the two equations reflects the effect on nail slip modulus of the separation or gap that develops between the sheathing and the chord member as the latter dries. The equations apply to chord lengths up to 96 inches, 2x4 inch or smaller chords in trusses spaced 24 inches or less, and 3/8 inch or thicker plywood nailed to the narrow face of the chord using recommended schedules.

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.

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 tabulated design value. The 5th 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.

The 5 percent exclusion value of \(E\) used in the equations was derived assuming a coefficient of variation of 25 percent for tabulated \(E\) values for visually graded lumber (see Commentary for 3.7.1.5). Because
the highest $E$ value grade and species of visually graded lumber was used, the 1977 $C_T$ equations were applicable to all grades and species of this material. Machine stress rated lumber has higher $E$ grades than the highest such grade for visually graded material and the coefficient of variation associated with machine stress rated $E$ values is only 11 percent. Therefore many of the stiffer grades of machine stress rated lumber had a higher 5 percent exclusion value of $E$ than that used in the derivation of the 1977 equations. As a result, the equations were applicable only to those machine stress rated grades having a design $E$ of 1,400,000 psi or less.

Based on additional analyses of the effects of various levels of chord $E$ on the stiffness contribution of sheathing to chord buckling (79), the $C_T$ equations were revised in the 1982 edition to apply to all grades of machine stress rated lumber and to any grade or species of visually graded lumber. As shown below, the equations are entered with a nominal 5 percent exclusion value, $E_{0.05}$, which is computed from tabulated $E$ values (Tables 4A, 4B and 4C) as

$$E_{0.05\text{vis}} = [1 - 1.645(0.25)]E_{\text{table}} = 0.59\, E_{\text{table}}$$

and

$$E_{0.05\text{mas}} = [1 - 1.645(0.11)]E_{\text{table}} = 0.82\, E_{\text{table}}$$

for visually graded and machine stress rated material, respectively.

For lumber dry at time of plywood attachment:

$$C_T = 1 + \frac{2300 \, f_e}{E_{0.05}} \quad \text{(C4.4-12)}$$

For lumber unseasoned or partially seasoned at time of plywood attachment:

$$C_T = 1 + \frac{1200 \, f_e}{E_{0.05}} \quad \text{(C4.4-13)}$$

The 1982 $C_T$ equations have been carried forward unchanged to the 1991 edition. Application is limited to 2 inch thick lumber 4 inches or less in depth. The equations have not been verified beyond chord lengths of 96 inches and therefore, $C_T$ factors for longer chord lengths should be based on a length of 96 inches.

The $C_T$ adjustment for $E$ is intended for use in checking loads on the chords of trusses subject to combined bending and compression. The interaction equation of 3.9.2 is entered with an $F_{c'}$ allowable stress and Euler buckling stress in the primary $(d_i)$ plane of bending computed with $E^\prime$ equal to $C_T E$.

The equations of 4.4.3 apply only to plywood sheathing panels. Buckling stiffness factors for other types of structural panels, which have different nail load-slip relationships than plywood, have not been established.

4.4.3.2 The design of triangular and parallel chord wood trusses made with metal connector plates are governed by specific national standards of practice for these components (186,187). Such practices employ provisions of this Specification that relate to the performance characteristics of individual pieces of lumber used as chord or web members.
PART V: STRUCTURAL GLUED LAMINATED TIMBER

5.1 GENERAL

Background

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 (62). 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 use of glued laminated timber to bridges, marine construction and other applications involving direct exposure to the weather.

A significant advantage of glued laminated members is the fact they can be made of dry lumber laminations in which the location and frequency of knots and 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 glued laminated timber paralleled that for visually graded lumber (see History at the beginning of the Commentary). The methods published in 1934 in U. S. Department of Agriculture's Miscellaneous Publication 185 for the grading and determination of working stresses for structural timbers (207) were also applied to 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 glued laminated timber being produced.

The earliest comprehensive procedures for establishing design values that were specifically developed for glued laminated timber were published in 1939 in U. S. Department of Agriculture Technical Bulletin 691 (208). 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 glued laminated timber were codified as national standards of practice in 1943 as part of the War Production Board's Directive No. 29 (194) and then in 1944 as part of the first edition of the National Design Specification (128). 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.

As a result of research at the Forest Products Laboratory following World War II, new procedures for establishing grades and design values for glued laminated timber were developed (68,209). By closer control and placement of different grades of laminations, the new procedures provided for higher design values than those previously used. Basic design values for both dry and wet service conditions were recommended. Most significantly, a new strength ratio concept of grading laminated members was introduced which has continued in use to the present time. In the case of tabulated bending design values, the relative strength of the member was defined in terms of a near-maximum (99.5 percent exclusion level) $I_K / I_G$ ratio, where $I_K$ is the sum of the moments of inertia about the neutral axis of the full cross section of all knots within a one-foot length of the critical section and $I_G$ is the moment of inertia of the full cross section. In the case of tabulated compression design values parallel to grain and tension design values parallel to grain, relative strength was defined in terms of a near-maximum $K / b$ ratio, where $K$ is the sum of knots in a three-foot length and $b$ is the laminate width. Under
this methodology, the larger the $I_K/I_G$ or $K/b$ ratio, the lower the strength ratio for the combination (62).

The regional lumber rules writing agencies used the new Forest Products Laboratory procedures (68) to establish specifications for the design and fabrication of structural glued laminated timber 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.

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

Changes in design values for glued laminated timber over the years largely reflect changes in grades and grade combinations being manufactured. However, certain changes in clear wood property assignments and in lumber design values also were paralleled by changes in related values for glued laminated timber (see Commentary for 4.2.3.2 - Background). One set of changes of note was an approximate 20 percent reduction in tension design values parallel to grain in 1968 and 1971. This adjustment paralleled the establishment of tension strength ratios for visually graded lumber as 55 percent of the corresponding bending strength ratio in 1968. Tension design values parallel to grain for glued laminated timber were further reduced in the 1977 edition on the basis of full-size tension tests of laminated members (36). The additional reduction, up to 37 percent for some species and combinations based on 1971 values, was made at the same time that a further reduction was made in tension design values parallel to grain for the wider widths and lower grades of visually graded lumber (see 4.2.3.2 Commentary).

A second change of note in glued laminated timber design values was the introduction in the 1986 edition of new compression design values perpendicular to grain based on a deformation limit. Previous values for this property were based on proportional limit stresses. A similar change was made in lumber design values in 1982 (see Commentary 4.2.6).

Glued Laminated Timber Product Standard. 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 (189). The standard was revised and repromulgated by the Department as Voluntary Product Standard PS56-73 in 1973 (191). 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 (9). 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 glued laminated timber are required to be in conformance to the current editions of two American Institute of Timber Construction specifications: AITC 117 for softwood species (6) and AITC 119 for hardwood species (5).

The provisions of AITC 117 in turn represent implementation of procedures given in ASTM D3737, Standard Test Method for Establishing Stresses for Structural Glued Laminated Timber (24). Procedures embodied in this ASTM standard, first published in 1978, reflect the previously used methodology (68) as modified by data from a succession of more recent full-scale test programs (9). The provisions of AITC 119 for hardwoods are based exclusively on earlier methodology (68).

National Design Specification provisions for glued laminated timber are limited to those products identified as being in conformance with ANSI/AITC A190.1. This consensus standard couples the AITC design and manufacture specifications with the inspection and certification requirements that are necessary to obtain uniform and assured product quality.

5.1.1-Application

5.1.1.1 The design and use of glued laminated timber is similar to that of sawn lumber products. Therefore, the general requirements given in Parts I, II and III of the Specification are applicable to glued laminated timber except where indicated otherwise. Part V of the Specification contains provisions which are particular to glued laminated timber because of the sizes, shapes, moisture content and combinations of grades in a single member employed in the product's manufacture.

The provisions of Part V contain only the basic requirements applicable to engineering design of 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 (4).

5.1.1.2 Where design values others than those given in Tables 5A, 5B and 5C 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 AITC 117 (6) and AITC 119 (5).

The design provisions in the Specification for 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.

5.1.2-Definition

The laminations of glued laminated timber usually are 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, are occasionally used for tension laminations where high tensile strength is required (9).

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

5.1.3-Standard Sizes

5.1.3.1 Standard finished widths of glued laminated members have been given in the Specification since the 1951 edition. Widths were reduced 1/8 inch for nominal widths of 4 to 6 inches and 1/4 inch for larger widths in the 1971 edition to reflect new dry lumber sizes established in the American Softwood Lumber Standard, PS 20-70 (190). The sizes in the 1971 edition have been continued for western species in the 1991 edition of the Specification except the finished width for nominal 3 inch members which has been increased from 2-1/4 inch to 2-1/2 inch. This change reflects current manufacturing practice wherein laminations of this width are now made by splitting nominal 2 by 6 lumber in half. Also in the 1991 edition, separate smaller finished widths for 4, 6, 10 and 12 inch nominal widths for southern pine members are introduced.

The finished widths of glued laminated timber members are 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 1/2 inch of the width from the original lumber width by planing or sanding.

Where necessary, other than standard widths can be specified. However, such special widths (for example, 7 inches) will require use of the nominal lumber width (10 inches) associated with the closest larger standard finished size (8-3/4 inches), which results in significant waste. Where appearance is not a factor, larger than standard widths for the smaller sizes can be provided from some manufacturers by use of laminations that have been split or sawn from larger lumber without further edging.

5.1.3.2 The sizes of 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 (4).

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.

Glued laminated timbers are usually custom manufactured to the specifications of the user. However, an increasing volume of material is being manufactured as non-custom or stock beams. These members, generally of the smaller widths and depths, are shipped to wholesalers who in turn sell them to builders and other users. The non-custom beams are often shipped in long billets and then cut to the length specified by the end user.

5.1.4-Specification

5.1.4.1 It is the responsibility of the designer to specify the moisture content condition to which the members will be exposed during service (see Commentary for 5.1.5). Although glued laminated timbers are made with dry lumber, the manufacturer needs to know the condition of use in order to select an appropriate adhesive.

Grades of glued laminated timber are specified in terms of laminating combination or the design values required. For members to be loaded primarily in bending about the x-x axis (load applied perpendicular to the wide face of the laminations), combinations given in Table 5A of the Specification should be designated; or the design values associated with those combinations should be cited. For members subject
primarily to axial loads (tension or compression), or to bending loads about the y-y axis (loads applied parallel to the wide face of the laminations), combinations given in Table 5B, or associated design values, should be designated.

5.1.5.2 Because laminating grades of hardwood lumber are not generally available, design values for glued laminated members made with hardwood species are established in a different manner than that used to establish values for members made with softwood species. Strength property stress modules for hardwood laminating grades are given in Part B of Table 5C in terms of the ratio of the size of the maximum permitted knot to the finished width of the lamination. These stress modules are multiplied by the species adjustment factors given in Part A of Table 5C to obtain design values for the particular hardwood laminating grade and species being specified. Example C5.1-1 illustrates the application of these adjustment factors.

Example C5.1-1
Consider a hardwood glued laminated member made with 10 laminations of combination B commercial red oak. Design values for the member based on Table 5C are:

Bending: \(770 \times 2.80 = 2156\) or \(2160\) psi
Tension parallel to grain: \(500 \times 2.80 = 1400\) psi
Compression parallel to grain: \(920 \times 2.05 = 1886\) or \(1890\) psi
Modulus of elasticity: \(1,000,000 \times 1.6 = 1,600,000\) psi
Horizontal shear: 230 psi
Compression perpendicular to grain: 800 psi

5.1.5-Service Conditions

5.1.5.1 When the equilibrium moisture content of members in service is less than 16 percent, the dry service design values tabulated in Tables 5A, 5B and 5C apply. The dry service condition for glued laminated timber was first defined in the 1951 edition as moisture contents of less than 15 percent. This was changed to less than 16 percent in the 1962 edition and has remained unchanged since that time.

A dry service condition for 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 tabulated design values.

5.1.5.2 Glued laminated members used in exterior exposures that are not protected from the weather by a roof, overhang or cave are generally considered wet conditions of use. 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.

Design values for wet service conditions are obtained by adjusting tabulated design values in Tables 5A, 5B and 5C by the wet service factors, \(C_M\), given in these tables. Where wet service conditions apply, the susceptibility of the member to decay and the need for preservative treatment (see Commentary for 2.3.5) should be considered.

5.2-DESIGN VALUES

5.2.1-Tabulated Values

The history of glued laminated timber design values published in the Specification is given in the Commentary for 5.1.

Softwood Species. Design values in Tables 5A and 5B are for members made with softwood species. Values in Table 5A are for 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). Values in Table 5B are for 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). Because of the mixing of grades to provide maximum efficiency and the necessity of having special tension laminations, values for a given property may vary with orientation of the loads on the member.

Table 5A. Values in this table apply to members having 4 or more laminations and are divided into western species/visually graded, western species/E-rated, southern pine/visually graded and southern pine/E-rated. The combination symbol in the first column designates a specific combination and lay-up of grades of lumber. For example, 16F-V1 under western species indicates a combination with a bending design value, \(F_b\), of 1600 psi (column 3 - tension zone stressed in tension) made with visually graded lumber (V) and is
the first such combination listed for western species. In the same format, 24F-E3 under southern pine indicates an \( F_b \) of 2400 psi (column 3 - tension zone stressed in tension) made with \( E \)-rated lumber and is the third such combination listed for southern pine.

The second column of Table 5A gives a two letter code indicating the species used for the outer laminations and for the core laminations of the member. For example, DF/WW indicates Douglas fir-Larch is used for the outer laminations and any western softwood species or Canadian softwood species is used for the core laminations. The symbol N3 preceding the core lamination species indicates a No. 3 Structural Joist and Plank or Structural Light Framing grade is used.

Design values in columns 3 through 7 of Table 5A apply when the members are loaded perpendicular to the wide face of the laminations. A higher grade of lumber is required in the tension zone of bending members compared to the compression zone, thus a lower grade of lumber is often used in the latter zone of most combinations. Column 3 of Table 5A gives the \( F_b \) value for use when the member is loaded as a simple beam with the tension zone in tension. When the combinations of Table 5A are used as simple beams with short overhangs that create tension on the top side of the beam over the support, \( F_b \) values in column 4 for the case of the compression zone in tension apply. The compression zone is distinguished from the tension zone of the member by the word "TOP" that is marked on all members except curved members where the top of the member is self evident.

When glued laminated members are used as cantilevered or continuous beams, those combinations which have the same \( F_b \) design values in columns 3 and 4 for tension and compression zones stressed in tension generally should be used.

Tabulated compression design values perpendicular to grain, \( F_{cl} \), depend on the density of the lumber used for the outer laminations. Columns 5 and 6 of Table 5A indicate the applicable \( F_{cl} \) value for the tension and compression faces of the member respectively.

Tabulated shear design values parallel to grain, \( F_{v} \), given in column 7 of Table 5A are based on the species of lumber used as the core laminations.

Tabulated modulus of elasticity, \( E \), given in column 8 of Table 5A represent the average value for the combination. These values are considered to have a shear deflection component equivalent to that occurring in a rectangular beam on a span-depth ratio of 21 under uniformly distributed load (see Commentary for 3.5.1). The coefficient of variation of \( E \) for glued laminated timber decreases as the number of lamination increases. The approximate coefficient of variation of \( E \) is 0.10 for members of 6 or more laminations.

Design values in columns 9 through 13 of Table 5A apply when members are loaded in bending parallel to the wide face of the laminations. The \( F_b \) values in column 9 for bending about the y-y axis are lower than those in column 3 for bending about the x-x axis because of the influence of the lower grade core and compression zone laminations and the lower strength species used in the core laminations of some combinations.

Tabulated compression design values perpendicular to grain, \( F_{cl} \), given in column 10 of Table 5A represent the lowest species and grade value applicable to any lamination used in the combination.

Tabulated shear design values parallel to grain, \( F_{v} \), given in column 11 represent the average value of all laminations in the combination reduced by the possibility of an accumulation of seasoning checks that could occur through the wide faces of the laminations.

Tabulated shear design values parallel to grain given in column 12 apply to members manufactured with multiple piece laminations that are not edge glued. For example, a nominal 6-inch wide piece of lumber (5-1/2 net) may be placed beside a nominal 8-inch wide piece (7-1/2 net) to form a lamination 12-3/4-inches wide which eventually is surfaced to a 12-1/4-inch finished width. The nominal 6-inch and 8-inch wide pieces are alternated in the assembly so that the openings between the pieces are not aligned. When the edges between the pieces are not glued, as is the usual practice, only one-half the cross-section is effective in resisting shear parallel to grain. The design values in column 12 reflect this reduced shear area.

Tabulated shear design values parallel to grain, \( F_{v} \), and compression design values parallel to grain, \( F_{c} \), given in columns 14 and 15 of Table 5A are strongly influenced by the lower strength species and grades of lumber used in the core laminations. The combinations given in Table 5B provide more efficient values for these properties.
Although there is a slight difference in the modulus of elasticity in bending about the y-y axis and that in axial loading, axial load $E$ values given in column 16 have been set equal to the y-y bending design values in column 13 for purposes of simplicity.

**Table 5B.** Design values in this table are for combinations intended primarily for resisting axial loads or for bending about the y-y axis. Each combination consists of a single grade of one species of lumber. The grade associated with each numbered combination can be obtained from the referenced AITC specification (6).

Compression design values parallel to grain, bending design values and shear design values parallel to grain vary with the number of laminations. Differences reflect the relative probability of maximum permitted knots occurring in the same critical section of each lamination or the probability of an accumulation through the member of checks occurring across the wide face of the laminations. Shear design values parallel to grain for members with multiple piece laminations reflect the absence of edge gluing between such pieces (see Table 5A discussion in Commentary for 5.2.1).

Tabulated bending design values, $F_b$, in column 15 are for members up to 15 inches deep made of 2 or more laminations. Values are for members without tension grade laminations. Bending design values in column 16 for members with 4 or more laminations apply when tension grade laminations are used. If such laminations are not used, tabulated values should be reduced 25 percent.

**Table 5C.** Hardwood lumber laminating grades are not generally available and therefore design values for combinations of such grades have not been established. Design values for glued laminated members made with hardwood lumber are determined by multiplying stress modules for a specific knot size to laminating width ratio in Part B of Table 5C times the applicable species strength factors given in Part A of the table (see Commentary for 5.1.4.2).

### 5.2.2-Radial Tension, $F_{rt}$

Because of undetectable ring shake and checking and splitting that can occur as result of drying in service, very low tension design values perpendicular to grain can be encountered in commercial grades of lumber. For this reason, design values for this property are not published in the Specification (see Commentary for 3.8.2).

Radial tension stresses, however, are induced in curved and pitched and tapered glued laminated timber members when bending loads tend to flatten out the curve or increase the radius of curvature. These stresses must be accounted for in design.

Glued laminated timber beams are made of dry material which is controlled for quality, including seasoning defects, during manufacture. The Specification has provided limiting stresses for actual radial tension in glued laminated members since the 1944 edition. The earliest editions limited allowable radial tension design values perpendicular to grain to a maximum of 2-1/2 percent of the applicable sawn lumber tabulated bending design value for softwoods and 4 percent of this value for hardwoods. For softwood lumber grades available at the time, this provision resulted in maximum radial tension design values perpendicular to grain of approximately 30 to 60 psi.

In the 1951 edition of the Specification, the limiting radial tension design value perpendicular to grain was 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 defects (20). As a result of field experience, the radial tension design value perpendicular to grain for Douglas fir-Larch under other than wind and earthquake loading was limited to 15 psi in the 1968 edition. For wind and earthquake loads for this species group and all loadings on other species, the limit on allowable radial tension design value perpendicular to grain of 1/3 the shear design value parallel to grain was retained. Also in the 1968 edition, a provision was added that waived all design value limits when mechanical reinforcement was designed to carry all actual radial tension stress.

In 1977, the general waiver for mechanical reinforcement was eliminated. However, when mechanical reinforcement or all vertical grain laminations was used, allowable radial tension design values perpendicular to grain of 1/3 the shear design value parallel to grain was allowed for Douglas fir-Larch under all types of loading. These radial tension design value provisions were carried forward through the 1986 edition.

In the 1991 edition, the vertical grain alternate for Douglas fir-Larch was discontinued because of the reduced availability of vertical grain material. The previous provisions limiting radial tension design values perpendicular to grain for Douglas fir-Larch under loads other than wind and earthquake to 15 psi, or the use of mechanical reinforcement for such loads up to 1/3 the shear design value parallel to grain (see 5.4.1.2
of Specification), have been extended to all western softwood species. For Southern Pine under all types of loading, and for all other softwood species under wind and earthquake loading, radial tension design values perpendicular to grain continue to be limited to 1/3 the shear design value parallel to grain. The allowable radial tension design value provisions in the 1991 edition are supported by both test results (30,155) and experience.

In calculating values of \( F_{w} \), the appropriate tabulated \( F_{w} \) value obtained from Tables 5A and 5B is to be modified by all adjustment factors specified in Table 2.3.1 that are applicable to glued laminated timber.

5.2.3-Other Species and Grades

(See Commentary on designer responsibility under 5.1.1.2)

5.3-ADJUSTMENT OF DESIGN VALUES

5.3.1-General

All adjustment factors specified in Table 2.3.1 of the Specification are applicable to one or more of the design values for glued laminated timber given in Tables 5A, 5B and 5C except size factor, \( C_F \), repetitive member factor, \( C_r \), and buckling stiffness factor, \( C_T \).

5.3.2-Volume Factor, \( C_V \)

Background

Size or depth adjustment of bending design values for glued laminated timber members has been a provision of the Specification since the 1957 edition (see Commentary for 4.3.2.2). From the 1957 through the 1968 editions, this adjustment for beams over 12 inches deep was made by the following relationship:

\[
C_F = 0.81 \left( \frac{d^2 + 143}{d^2 + 88} \right) \quad (C5.3-1)
\]

The size equation was changed in the 1971 edition to

\[
C_F = \left( \frac{12}{d} \right)^{1/9} \quad (C5.3-2)
\]

This revised equation, based on tests of beams one inch to 32 inches deep (33) and introduced into ASTM D245 in 1968 (14), was continued unchanged through the 1986 edition. Adjustments from the equation were applicable to a simply supported beam, uniformly loaded on a span/depth ratio of 21.

In 1973, loading coefficients for modifying the size adjustment of design values for glued laminated beams to concentrated and third point loading conditions were introduced. The coefficients, derived from the bending moment diagrams associated with each condition and the relative span lengths subject to the highest moment levels, were of the order 1.00 uniform, 1.08 concentrated and 0.97 third point. These loading coefficients also were carried forward to the 1986 edition.

1991 Provisions. The volume factor adjustment for glued laminated beams in the 1991 edition includes terms for the effects of width and length as well as depth. The volume factor \( C_V \) equation, based on recent 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 (118), is

\[
C_V = K_L (21/L)^{1/5} (12/d)^{1/5} (5.125/b)^{1/5} \leq 1.0 \quad (C5.3-3)
\]

in which:

\[ K_L = \text{loading condition coefficient} \]
\[ L = \text{length of bending member between points of zero moment, feet} \]
\[ d = \text{depth of bending member, inches} \]
\[ b = \text{width (breadth) of bending member, inches.} \]

For multiple piece width layups, \( b \) = width of widest piece used in the layup. Thus, \( b \leq 10.75". \)

\[ x = 20 \text{ for southern pine} \]
\[ = 10 \text{ for all other species} \]

The foregoing equation is based on the volume effect equation given in ASTM D3737 (24) except that the width effect exponent for other species was changed to 1/10 from 1/9 for simplicity and to reflect the accuracy of the data, and a separate coefficient of 1/20 was established for southern pine based on recent large beam tests of that species (7).

As indicated by the equation, tabulated \( F_b \) values given in Tables 5A, 5B and 5C apply to members that are 5-1/8 inches wide, 12 inches deep and 21 feet long. When any other sizes are used, tabulated values are to be adjusted by multiplying by the volume factor, \( C_V \).

The loading condition coefficients of 1.09 for concentrated load at mid span and 0.96 for two equal loads at the third points given in the 1991 edition are slightly different than the comparable values given in previous editions as a result of a change in the method of calculation of these values (7).
As indicated in footnote 1 to Table 2.3.1, the volume factor, \( C_v \), is not applied simultaneously with the beam stability factor, \( C_b \). The smaller of the two adjustment factors applies. This provision is a continuation of the practice introduced in the 1968 edition of the Specification of considering stability and 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.

5.3.3-Flat Use Factor, \( C_{fu} \)

Adjustment of bending design values for glued laminated beams loaded parallel to the wide face of the laminations when the wide face of the laminations is less than 12 inches was a footnote provision to Table 5B in the 1982 and 1986 editions of the Specification. Now tabulated as flat use factors, \( C_{fu} \), in the front of both Tables 5A and 5B, the adjustments are based on equation C5.3-2, which is the 1/9 power size equation of ASTM D245.

5.3.4-Curvature Factor, \( C_c \)

When the individual laminations of 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 tabulated bending design values, \( F_b \), to account for the effects of these two conditions.

The curvature factor equation given in 5.3.4 is based on early tests (208) and has been a provision of the Specification since the 1944 edition. The limits on the ratio of laminate 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 material, 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 material, 1.5 inches net, a radius of curvature of 27.5 feet commonly is used for all species.

5.4-SPECIAL DESIGN CONSIDERATIONS

5.4.1-Radial Stress

5.4.1.1 The equation for determining actual radial stress in a curved member of constant rectangular cross section, which is based on research published in 1939 (208), has been a provision of the Specification since the 1951 edition. Although limited to members of constant rectangular cross section, subsequent design practice was to employ the same equation for calculating actual stresses in tapered cross sections. However, new research showed that actual radial stresses in curved tapered members had to be determined by different procedures (67). Such new methodology for curved members having variable cross section was introduced in the 1973 edition and continued through the 1986 edition. This methodology consisted of applying a modification factor, derived from the ratio of member depth to radius of curvature and the slope of the upper edge of the member, to the actual stress based on the constant cross section equation.

More recent research has shown that additional modification factors are needed to establish bending design values and deflection as well as radial tension design values perpendicular to grain of pitched and tapered bending members (80). It was concluded that the design procedures required were too complex for inclusion in the Specification. Thus design methodology for radial stresses in curved bending members of varying cross section has been removed from the 1991 edition. Complete design procedures for such members are available from other authoritative sources (4).

5.4.1.2 When the bending moment acts to reduce curvature, the actual radial stress is to be checked against the allowable radial tension design value perpendicular to grain, \( F_d' \), which includes all adjustment factors applicable to tabulated shear design values parallel to grain. (See Commentary for 5.2.2 for background on allowable radial tension design values perpendicular to grain and mechanical reinforcement requirement.)

5.4.1.3 Actual radial stress is checked against the allowable compression design values perpendicular to grain when the bending moment acts to increase beam curvature. As shown by the tabulated design values for this property in Tables 5A and 5B, actual radial compression stress seldom controls design when it occurs.

5.4.2-Lateral Stability for Glued Laminated Timber

5.4.2.1 Bending design values, \( F_b \), given in Tables 5A, 5B and 5C 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 3.3.3. As the tendency of the compression portion of the beam to buckle is a function of beam stiffness about the y-y axis (bending due to loading parallel to the wide face of the laminations), all glued laminated beam stability factor calculations are to be made with values of modulus of elasticity for bending about the y-y axis, $E_{yy}$, modified by all applicable adjustment factors.

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 (205, 206).

5.4.2.2 The depth to breadth limitations for laterally supported arches have been a provision of the Specification since the 1977 edition. These rules are good practice recommendations based on field experience over many years.

5.4.3-Deflection

(See Commentary for 3.5 and 5.2.1 - Table 5A, modulus of elasticity.)

Example C5.4.1 illustrates the use of design procedures outlined in the Specification for design of a glued laminated timber member.

Example C5.4.1

Design a simple beam spanning 32 ft, with 5000 lb loads (1000 lb DL + 4000 lb SL) applied by purlins at 8 ft on center (1/4 points plus ends). Member has lateral support at the ends and on the compression edge by the purlins. Beam supports are 6 in. long. Assume dry service conditions. Temperature is less than 100°F but occasionally may reach 150°F. Use 24F-V1 Southern Pine glued laminated timber.

$C_D = 1.15 \quad C_M = 1.0 \quad C_t = 1.0$ (Table 5A and 2.3.1)

$F_b = 2400$ psi $F'_b = 200$ psi $F_{cl, compression} = 560$ psi

$F_{cl, tension} = 650$ psi $E_{yy} = 1,500,000$ psi

$E_{xx} = 1,700,000$ psi

Bending (3.3.3, 5.3.2)

$F_b^* = F_b C_D C_M C_t = (2400)(1.15)(1.0)(1.0) = 2760$ psi

$\ell_u = 8$ ft

$\ell_e = 1.54 \ell_u = 1.54(8) = 12.32$ ft (Table 3.3.3)

The slenderness factor, $R_B$, must be determined. Since dimensions are unknown a trial design will be estimated and modified as needed. Try a $5 \times 30-1/4$ beam, $S_{xx} = 762.6$ in$^3$

Beam Stability Factor, $C_L$

$R_B = \frac{\ell_u \sqrt{d}}{b^2} = \sqrt{\frac{(12.32)(12)(30.25)}{(5)^2}} = 13.375$

$K_{be} = 0.609$

$F_{be} = \frac{K_{be} E'}{R_b^2} = \frac{(0.609)(1,500,000)}{(13.375)^2} = 5107$ psi

$C_L = \frac{1+(F_{be}/F_b^*)}{1.9} \left[ \frac{1+(F_{be}/F_b^*)}{1.9} \right]^{12} \frac{F_{be}/F_b^*}{0.95}$

$= 1+5107/2760 / 1.9 \left[ 1+5107/2760 / 1.9 \right]^{12} \frac{5107/2760}{0.95}$

$= 0.950$

Volume Factor, $C_V$

$C_V = K_L (21/L)^{1/5} (12/d)^{1/5} (5.125/b)^{1/5} \leq 1.0$

Assume $K_L = 1.0$; load condition approaches uniform load $x = 20$ for Southern Pine

(cont.)
Example C5.4-1 (cont.)

\[ C_V = (1.0)(21/32)^{1/20} (12/30.25)^{1/20} (5.125/5)^{1/20} = 0.936 \]
\[ C_V < C_L \], therefore \( C_V \) applies

Allowable Bending Design Value, \( F_b' \) (Table 2.3.1)

\[ F_b' = F_P C_D C_{M1} C_{V} = (2400)(1.15)(1.0)(0.936) = \text{2583 psi} \]

Determine Section Modulus Required by Bending

Assume weight of glued laminated timber = 40 lb/ft

Purlin loads

\[ M_{\text{est.}} = P (\ell/2) + w \ell^2/8 \]
\[ = (5000)(32/2)(12) + (40)(32)^2(12)/8 = 1,021,440 \text{ in-lb} \]

\[ S_{\text{required}} = M/F_b' = 1,021,440/2583 \]
\[ = 395.45 \text{ in}^3 < 762.6 \text{ in}^3 \]

Try a 5 \( \times \) 22 member, \( S_{\text{xx}} = 403.33 \text{ in}^3, I_{\text{xx}} = 4437 \text{ in}^4 \)

\[ R_B = \sqrt{\frac{\ell d}{b^2}} = \sqrt{\frac{(12.32)(12)(22)}{(5)^2}} = 11.41 \]

\[ F_{b1} = \frac{K_{b1} E'}{R_B^2} = \frac{0.609(1,500,000)}{(11.41)^2} = 7022 \text{ psi} \]

\[ C_L = \frac{1+7022/2760}{1.9} - \frac{[1+7022/2760]^2 - 7022/2760}{0.95} = 0.970 \]

\[ C_V = (1.0)(21/32)^{1/20} (12/30.25)^{1/20} (5.125/5)^{1/20} = 0.951 \]

\[ C_V < C_L \], therefore \( C_V \) controls

Allowable Bending Design Value, \( F_b' \) (Table 2.3.1)

\[ F_b' = F_P C_D C_{M1} C_{V} = (2400)(1.15)(1.0)(0.951) = \text{2625 psi} \]

With a beam weight of 30 lb/ft for a 5\( \times \)22 beam

\[ M = (5000)(32/2)(12) + (30)(32)^2(12)/8 = 1,006,080 \text{ in-lb} \]

\[ S_{\text{required}} = M/F_b' = 1,006,080/2625 \]
\[ = 383.27 \text{ in}^3 < 403.33 \text{ in}^3 \text{ ok} \]

A 5 \( \times \) 20-5/8 member, \( S_{\text{xx}} = 354.5 \text{ in}^3 \), is too small.

Use 5 \( \times \) 22 beam

Shear (3.4)

The loads from the purlins at the supports are within a
distance \( d \) of the face of the supports and can be neglected
for shear (3.4.3.1 (a)).

\[ V_{\text{purlins}} = 3P/2 = (3)(5000)/2 = 7500 \text{ lb} \]

Assuming a beam weight of 30 lb/ft and neglecting loads
within a distance \( d \) of the support

\[ V_{\text{beam}} = w(\ell/2 - (d + 1/2 \text{ support})) = (30)(32/2 - (22 + 6/2))/12 = 418 \text{ lb} \]

\[ V_{\text{total}} = V_{\text{beam}} + V_{\text{purlins}} = 418 + 7500 = 7918 \text{ lb} \]

Allowable Bending Design Value

Parallel to Grain, \( F_{b1}' \) (Table 2.3.1)

\[ F_{b1}' = F_P C_D C_{M1} C_{V} = (200)(1.15)(1.0)(1.0) = 230 \text{ psi} \]

Actual Shear Stress Parallel to Grain, \( f_v \) (3.4.2)

\[ f_v = \frac{3V}{2bd} \]
\[ = 108 \text{ psi} < F_{b1}' = 230 \text{ psi} \text{ ok} \]

Bearing Perpendicular to Grain (3.10.2)

The purlin on the wall side of the beam transmits all of the
load to the end of the beam. The purlin load at the support is included in determining load in bearing perpendicular
in-lb to grain at the support (6 in. supports):

\[ R_{\text{purlins}} = P + 3P/2 = 5000 + (3)(5000)/2 = 12,500 \text{ lb} \]

Assuming the weight of the beam as 30 lb/ft over the full
length of the beam (end-to-end)

\[ R_{\text{beam}} = w(\ell + 2(1/2 \text{ support}))/2 = 30(32+2(6/2))/12/2 = 488 \text{ lb} \]

\[ R_{\text{total}} = R_{\text{purlins}} + R_{\text{beam}} = 12,500 + 488 = 12,988 \text{ lb} \]

\[ F_{c1}' = F_{c1} \text{ tension} C_M C_1 C_b = (650)(1.0)(1.0)(1.0) = 650 \text{ psi} \]

\[ f_{c1} = 12,988/(5)(6) = 433 \text{ psi} < F_{c1}' = 650 \text{ psi} \text{ ok} \]

The purlins are held by hangers that hold 2 purlins, one on
each side of the beam. The area required under the hanger
on top of the beam is

\[ A = P/F_{c1} \text{ compression} = 5000/560 = 8.93 \text{ in}^2 \]

A 3 in. wide hanger is more than adequate. Note: the
design value, \( F_{c1} \), for the compression edge may be
increased by the use of the bearing area factor (\( C_a \)) in 3.2.10
when the length of bearing along the grain is less than 6 in.
and not nearer than 3 in. to the member end. For a 3 in.
wide strap on the compression edge, \( F_{c1}' = (1.13)(560) = 633 \text{ psi} \)

At this stage of the calculations, the span of the beam can
be reviewed. The 32 ft span used in the trial calculations
was based on the distance from center to center of supports
as is customary. The length of the span to use in design is
the distance from face to face of supports plus 1/2 the
required bearing length at the ends (see 3.2.1). In this
example, the distance between the inside faces is 32 ft - 6
in. = 31.5 ft. The required length of bearing on the wall
end of the beam is 12,988/(3)(560) = 4.00 in. At the

(cont.)
Example C5.4-1 (cont.)

interior end, half of the purlin load is assumed to be transferred to the beam end. Required length in bearing = (2500+7500+488)/(5)(650) = 3.23 in. These required bearing lengths give a span length of 31.5 + (4.00/2)/12 + (3.23/2)/12 = 31.8 ft. This reduces the moment less than two percent, which is not enough to permit the use of the next smaller size beam. In some cases, however, the change in length may permit a change in size of the member.

Deflection

The Specification does not give specific deflection limitations for roofs. In some applications, deflection may be critical and the designer may wish to limit deflection. Usually the average $E$ in Tables 5A, 5B or 5C is used. However, the modulus of elasticity to the 5th or some other percentile, may be needed for some calculations. The customary engineering equations are used to determine bending deflection, but the designer may wish to include shear deflection as well. Ordinarily, the latter is small and is not considered.

Dead load deflection is usually calculated to determine the desired camber of the beam. The camber usually recommended is $1.5 \times$ dead load deflection.

Deflection for three 5000 lb concentrated loads at the 1/4 points plus the 30 lb/ft beam weight is

\[
\Delta_{total} = \frac{19PE^3}{384EI} + \frac{5wL^4}{384EI}
\]

\[
= \frac{(19)(5000)((32)(12))^3}{(384)(1,700,000)(4437)} + \frac{(5)(30/12)((32)(12))^4}{(384)(1,700,000)(4437)}
\]

\[
= 1.857 + 0.094 = 1.951 \text{ in.}
\]

Total Deflection = $\Delta /197$ of the span which is reasonable

\[
\Delta_{dead \ load} = (1000/5000)(1.857) + 0.094 = 0.465 \text{ in.}
\]

Camber = $1.5 \times \Delta_{dead \ load} = (1.5)(0.465) = 0.698$

Use Camber = 3/4 in.

Southern Pine 24F-V1 5x22 glued laminated member satisfies NDS design criteria
PART VI: ROUND TIMBER PILES

6.1-GENERAL

Background

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 (218).

Timber piles are commonly used in sand, clay, silt and other soils in which they are relatively easy to drive and which will provide significant support through skin friction (218). However, because of the difficulty of quantifying friction forces and confinement pressures, timber piles today are designed primarily on the basis of their end bearing load-carrying capacity.

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 (22). 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 since well before World War II (57). 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 (26). 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 (57). Standard design procedures were not available.

To meet the growing need for uniform design recommendations, the American Association of State Highway Officials 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 (218). 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, the American Association of State Highway Officials, 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 (218) (see Commentary for 4.2.3.2). Building codes also began to establish allowable pile stresses using basic stresses and other information given in ASTM D245 (196).

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 (16). This consensus standard provides 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 (20) are 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. Compression design values parallel to grain established under D2899 are of the same general magnitude as those previously specified earlier by user and code groups.

A table of design values for round timber piles made of Douglas fir, southern pine, red pine and red oak as recommended by the American Wood Preservers Institute was included in the 1971 edition of the Specification. A new timber piling section was introduced as Part X of the Specification in the 1973 edition which included a revised table of design values based on the methods of ASTM D2899. Covering the same species as were included in the 1971 edition, the 1973 design values were limited to piles conforming to
the size and quality provisions of ASTM D25 and to
the treating provisions of AWPA Standard C3.

In 1977, provisions for round timber piles in the
Specification were redesignated as Part VI and expand-
ed to reference AWPA Standard C18 (Marine Use) and
to include information on modification of design values
for size and other factors, including adjustment of
values for piles acting singly rather than in clusters.
Tabulated design values were not changed from the

Timber pile provisions of the 1977 edition, including
tabulated design values, have been carried forward to
the 1991 edition essentially unchanged.

6.1.1-Application

6.1.1.2 The provisions of Part VI of the Specifi-
cation relate solely to the properties of the piles
themselves. It is the responsibility of the designer to
determine soil loads, such as frictional forces from
subsiding soils and fills, the adequacy of the surround-
ing soil or water to provide adequate lateral bracing,
the method of pile placement that will preclude damage
to the pile, the bearing capacity of the strata at the
pile tip, and the effects of any other surrounding
environmental factors on pile loads or pile support.

6.1.2-Pile Specifications

6.1.2.1 In addition to setting standard pile sizes,
ASTM D25 (22) establishes minimum quality require-
ments, straightness criteria, and knot limitations. All
piles are required to have an average 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 tip cross sectional area of piles
conforming to ASTM D25 essentially meet lumber
requirements for dense material (18).

Knots in piles are limited by ASTM D25 to a
diameter of not more than one-sixth of the circumfer-
ence 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.

6.1.2.2 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.

6.1.3-Standard Sizes

Standard sizes (22) 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 circumfer-
ence 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.

6.1.4-Preservative Treatment

6.1.4.1 Green timber piles are generally condi-
tioned prior to pressure treatment (25). 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 (62,88). 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 (62,88).
The Boulton process also is used with hardwood
species.

Both the steaming and Boulton conditioning
processes affect pile strength properties (16,218). These
effects are accounted for in pile design values given in
Table 6A of the Specification. In the 1991 edition,
conditioning by kiln drying is classified with the
Boulton process for purposes of establishing design
values (196,218).

6.1.4.2 Decay does not occur in softwood species
and in most hardwoods that are completely saturated
and an air supply is not available (88,170). Perma-
nently submerged piles meet these conditions.

6.2-DESIGN VALUES

6.2.1-Tabulated Values

Design values for round timber piles given in Table
6A are based on ASTM D2899 (16). All values are
derived from the properties of small clear specimens of
the applicable species as given in ASTM D2555 (20)
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 (197).

Tabulated compression design values parallel to grain, $F_c$, include a 10 percent reduction for pile grade, a 10 percent reduction to adjust average small clear values for the whole tree to the tips of the piles, a conservative 10 percent reduction in standard deviation of small clear values to account for the reduced variability of tree size piles, a reduction for conditioning, and the standard adjustment of short term test values for the property to a normal load duration. The combined factor applied to the nominal 5th percent exclusion value for small clear wood specimens of the species is 1/1.88 exclusive of the conditioning adjustment (197).

Similar adjustments are used for tabulated bending design values, $F_b$: 10 percent reduction for grade, 12 percent reduction to adjust average tree values to tip values, a conservative 12 percent reduction in standard deviation to account for the reduced variability of pile bending strength values, the conditioning adjustment, and the load duration adjustment for the property. The combined factor applied to the 5th percentile small clear strength value is 1/2.04 exclusive of the conditioning adjustment (197).

Tabulated shear design values parallel to the grain, $F_s$, are based on the 5th percentile clear wood strength value reduced for load duration and stress concentrations using the factor applied to lumber for these effects (18), a 25 percent reduction for possible splits and checks and a conditioning adjustment. The combined factor on the clear wood 5th percentile value is 5.47 exclusive of the conditioning adjustment (197).

Tabulated compression design values perpendicular to grain, $F_{c_L}$, in Table 6A represent the average proportional limit stress for small clear specimens reduced 1/1.5 for ring orientation and an adjustment for conditioning. No adjustments are made to average clear wood modulus of elasticity values for application to piles.

Tabulated design values, except modulus of elasticity, for Pacific Coast Douglas fir, red oak and red pine in Table 6A contain a 10 percent reduction for conditioning treatment. This factor is based on the Boulton process adjustment in ASTM D2899. Comparable values for southern pine contain a 15 percent reduction for conditioning, the factor for steam conditioning in D2899.

The species designation Pacific Coast Douglas fir listed in Table 6A refers to Douglas fir growing west of the summit of the Cascade Mountains in Washington, Oregon and northern California and west of the summit of the Sierra Nevada Mountains in other areas of California (17). Values for red oak in Table 6A apply only to the species northern red oak, (Quercus rubra) and southern red oak (Quercus falcata).

6.2.2-Other Species or Grades

Where piles of species other than those listed in Table 6A are used, it is the designer's responsibility to assure that the methods of ASTM D2899 for establishing design values are properly applied, including appropriate adjustments for conditioning process.

6.3-ADJUSTMENT OF DESIGN VALUES

6.3.2-Load Duration Factor, $C_D$

As shown in Table 6.3.1, the load duration factor, $C_D$, is applicable to compression design values perpendicular to grain, $F_{c_L}$. These pile design values are based on proportional limit stresses and, in accordance with ASTM D245 (18), are subject to load duration adjustments.

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. These exclusions were introduced in the 1977 edition for preservative oxides and the 1982 edition for fire retardant chemicals.

6.3.5-Untreated Factor, $C_u$

Increases in design values tabulated in Table 6A for piles that are air-dried before treating or are used untreated (see Commentary for 6.1.4.2) represent removal of the conditioning adjustments that are incorporated in the values for all properties except modulus of elasticity.

Design values in Table 6A for Pacific Coast Douglas fir, red oak and red pine contain a 10 percent reduction (1/1.11) for conditioning, assumed to be the Boulton or boiling-under-vacuum process. These values also are applied to piles that have been kiln dried prior to treatment. Tabulated strength values for southern pine piles contain a 15 percent reduction (1/1.18) for
conditioning which is assumed to be by the steaming-
and-vacuum process.

6.3.6-Fire Retardant Treatment

(See Commentary for 2.3.6.)

6.3.7-Beam Stability Factor

A round member can be considered to have a \( \frac{d}{b} \) ratio of 1 and therefore, in accordance with 3.3.3.1, lateral support for beam buckling is not required.

6.3.8-Size Factor, \( C_F \)

Bending design values, \( F_b \), for round timber piles that are larger than 13.5 inches in diameter at the critical section in bending are adjusted for size using the same equation

\[
C_F = \left( \frac{12}{d} \right)^{1/9} \quad (C6.3-1)
\]

used to make size adjustments with sawn lumber Beams & Stringers and Posts & Timbers (see Commentary for 4.3.2.2). When applied to round timbers, equation C6.3-1 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 (see Commentary for 2.3.8).

6.3.9-Form Factor

Pile bending design values include an adjustment relating the results of strength tests of full-size piles to the results of test of small clear rectangular specimens selected from the same piles. Thus the effect of form is included in the tabulated values.

6.3.11-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, applicable to Douglas fir and southern pine, provides for an increase in the design value as the critical section moves from the pile tip toward the pile butt. The factor is limited to 10 percent as this is the adjustment for tip end location used in the establishment of compression design values parallel to grain, \( F_c \), for softwood species. As only limited data are available for red pine, the \( C_{cs} \) adjustment is not applied to this species.

The compression design value parallel to grain of red oak does not decrease with increase in height in the tree and the 10 percent tip end adjustment factor is not used in the establishment of \( F_c \) values for this species group (16).

6.3.13-Single Pile Factor, \( C_{sp} \)

Design values in Table 6A are considered applicable to piles used in clusters. Where piles are used such that each is expected to carry its full portion of the design load, multiplication of tabulated compression design values parallel to grain, \( F_c \), and bending design values, \( F_b \), by a \( C_{sp} \) factor of 0.80 \((1/1.25)\) and 0.77 \((1.30)\), respectively, may be appropriate.

It is the designer's responsibility to determine the applicability of the \( C_{sp} \) factors, designated as factors of safety in ASTM D2899, to the specific design. In making such evaluations, it is to be noted that the tabulated design values apply to the weakest material in the pile located in the pile tip; that the cross-sectional area of the pile at any location along its length may be larger than those associated with the minimum butt and tip diameters specified in ASTM D25; and that the full design load commonly does not reach the pile tip because of the support given by soil friction (16).
PART VII: MECHANICAL CONNECTIONS

7.1-GENERAL

7.1.1-Scope

7.1.1.1 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 7.3.6.

It is recognized that some designers have used different fastener types in the same connection where the addition of one 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 Commentary for 1.1.1.4). 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.

7.1.1.3 (See Commentary for 3.1.3, 3.1.4 and 3.1.5.)

7.1.1.4 The adequacy of alternate methods or procedures for designing and verifying the strength of connections which provide allowable loads that differ from those in the Specification is the responsibility of the designer. 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 Commentary for 1.1.1.4).

7.1.2-Stresses in Members at Connections

All connection designs should be checked for conformance of structural members to the net section area requirements of 3.1.2 and the shear design provisions of 3.4.5 (see Commentary for these sections). 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 3.1.3 provision). 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. Example C7.1-1 illustrates consideration of these provisions.

Example C7.1-1

Design a bolted connection to join two 2x4's to carry an axial tension live load of 2000 lb.

1. Try a lapped joint using No. 2 Southern Pine and 1/2 in. bolts

$F_b = 1500 \text{ psi } E = 1,600,000 \text{ psi}$  (Table 4B)
$F_t = 825 \text{ psi } C_D = 1.0 \text{ } C_F = 1.0 \text{ } C_M = 1.1$

Bolt Design

For 1/2-in. bolts in single shear,

$Z_{bol} = 530 \text{ lb/bolt}$  (Table 8.2A)

Number of bolts $= P/Z = 2000/530 = 3.8$ bolts

Try four 1/2-in. bolts in a single row at 2 in. on-center

With $a = 4$  $E_m = E_s = 1,600,000 \text{ psi } A_m = A_s = 5.25 \text{ in}^2$ and $s = 2 \text{ in.}$

$C_s = 0.985$  (Eq. 7.3-1)

$Z' = Z_{bol}C_DC_FC_s = (530)(1.0)(0.985)(1.0)$  (7.3.1)

$= 522 \text{ lb}$

$P = 2000 \text{ lb } nZ' = (4)(522) = 2088 \text{ lb } ok$

Net Section  (3.1.2)

Allow for an additional 1/16 in. per bolt hole  (8.1.2.1)

$A_{net} = (1.5)(3.5 - (1/2 + 1/16)) = 4.41 \text{ in}^2$

$S_{net} = (3.5 - (1/2 + 1/16))(1.5)^2/6 = 1.10 \text{ in}^3$

Tension  (3.8.1)

$F_t' = F_tC_DC_F = (825)(1.0)(1.0) = 825 \text{ psi}$  (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi } < F_t' = 825 \text{ psi } ok$

(cont.)
Example C7.1-1 (cont.)

Bending

Bending is induced about the weak axis due to the eccentricity of the lapped connection of 1.5 in.

\[ F_b^* = F_b'C_D'C_L'C_{fu} = (1500)(1.0)(1.1) = 1650 \text{ psi} \]

Since \( d < b \) (1.5 < 3.5 in.), \( C_L = 1.0 \)

\[ F_b' = F_b'^* = F_b'C_D'C_L'C_{fu} = (1500)(1.0)(1.1) = 1650 \text{ psi} \]

\[ f_b = M/S_{net} = P_e/S_{net} = (2000)(1.5)/(1.10) = 2727 \text{ psi} > F_b' = 1650 \text{ psi} \]

Connection is not adequate for bending, try another design.

2. Try a lapped joint using Select Structural Southern Pine and 1/2 in. bolts

\[ F_b = 2850 \text{ psi} \quad E = 1,800,000 \text{ psi} \quad (\text{Table 4B}) \]

\[ F_t = 1600 \text{ psi} \quad C_D = 1.0 \quad C_F = 1.0 \quad C_{fu} = 1.1 \]

Bolt Design

As before, \( Z_{11} = 530 \text{ lb/bolt} \) (Table 8.2A)

Try four 1/2-in. bolts in a single row at 2 in. on-center

With \( n = 4 \), \( E_m = E_s = 1,800,000 \text{ psi} \), \( A_m = A_s = 5.25 \text{ in}^2 \) and \( s = 2 \text{ in.} \)

\[ C_g = 0.987 \quad \text{(Eq. 7.3-1)} \]

\[ Z' = Z_{11}/C_D'C_g'C_{fu} = (530)(1.0)(0.987)(1.0) = 523 \text{ lb} \]

\( P = 2000 \text{ lb} < nZ' = (4)(523) = 2092 \text{ lb} \quad \text{ok} \)

Tension

\[ F_t' = F_tC_DC_F = (1600)(1.0)(1.0) = 1600 \text{ psi} \quad (2.3.1) \]

\[ f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi} < F_t' = 1600 \text{ psi} \quad \text{ok} \]

Bending

Eccentricity = 1.5 in.

\[ F_b^* = F_b'C_D'C_L'C_{fu} = (2850)(1.0)(1.1) = 3135 \text{ psi} \quad (3.9.1) \]

\[ F_b' = F_b'^* = F_b'C_D'C_L'C_{fu} = (2850)(1.0)(1.1) = 3135 \text{ psi} \]

\[ f_b = M/S_{net} = P_e/S_{net} = (2000)(1.5)/(1.10) = 2727 \text{ psi} < F_b' = 3135 \text{ psi} \quad \text{ok} \]

Combined Bending and Axial Tension

\[ \frac{f_t + f_b}{F_t'} = \frac{454 + 2727}{1600} = 1.15 > 1.0 \quad \text{ng} \]

\[ \frac{f_b - f_t}{F_b'^*} = \frac{2727 - 454}{3135} = 0.73 < 1.0 \quad \text{ok} \]

Connection is not adequate for combined bending and tension, try another design.

3. To reduce the eccentricity of the connection, try a spliced joint using Select Structural Southern Pine with a single 5/16-in. steel plate and 1/2 in. bolts

Bolt Design

For 1/2-in. bolts in single shear with 5/16-in. side plate, \( Z_{11} = 613 \text{ lb/bolt} \) (Eq. 8.2-3)

Number of bolts = \( P/Z = 2000/613 = 3.3 \) bolts

Try four 1/2-in. bolts in a row on each side of splice at 2 in. on-center

With \( n = 4 \), \( E_m = 1,800,000 \text{ psi} \), \( E_s = 30,000,000 \text{ psi} \),
\( A_m = 5.25 \text{ in}^2 \), \( A_s = 1.094 \text{ in}^2 \) and \( s = 2 \text{ in.} \)

\[ C_g = 0.949 \quad \text{(Eq. 7.3-1)} \]

\[ Z' = Z_{11}/C_D'C_g'C_{fu} = (613)(1.0)(0.949)(1.0) = 582 \text{ lb} \]

\( P = 2000 \text{ lb} < nZ' = (4)(582) = 2328 \text{ lb} \quad \text{ok} \)

Tension

\( F_t' = F_tC_DC_F = (1600)(1.0)(1.0) = 1600 \text{ psi} \quad (2.3.1) \]

\( f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi} < F_t' = 1600 \text{ psi} \quad \text{ok} \)

Bending

\( F_b^* = F_b'C_D'C_L'C_{fu} = (2850)(1.0)(1.1) = 3135 \text{ psi} \quad (3.9.1) \]

\[ F_b' = F_b'^* = F_b'C_D'C_L'C_{fu} = (2850)(1.0)(1.1) = 3135 \text{ psi} \]

\[ f_b = M/S_{net} = P_e/S_{net} = (2000)(0.906)/(1.10) = 1648 \text{ psi} < F_b' = 3135 \text{ psi} \quad \text{ok} \]

Combined Bending and Axial Tension

\[ \frac{f_t + f_b}{F_t'} = \frac{454 + 1648}{1600} = 0.81 < 1.0 \quad \text{ok} \]

\[ \frac{f_b - f_t}{F_b'^*} = \frac{1648 - 454}{3135} = 0.38 < 1.0 \quad \text{ok} \]

This connection is adequate, but requires 8 bolts. Try a double plate/double shear spliced connection to eliminate the eccentricity, reduce the number of bolts and allow for the use of a lower grade of lumber.

4. Try a spliced joint using No. 3 Southern Pine with two 1/8-in. steel plates and 1/2 in. bolts

\[ F_b = 850 \text{ psi} \quad E = 1,400,000 \text{ psi} \quad (\text{Table 4B}) \]

\[ F_t = 475 \text{ psi} \quad C_D = 1.0 \quad C_F = 1.0 \quad C_{fu} = 1.1 \]

(cont.)
Example C7.1-1 (cont.)

### Bolt Design

For 1/2-in. bolts in double shear with 1/8-in. side plates,
\[
Z_{ll} = 1153 \text{ lb/bolt} \quad \text{(Eq. 8.3-1)}
\]
Number of bolts = \( P/Z = 2000/1153 = 1.73 \) bolts
Try two 1/2-in. bolts in a row on each side of splice
With \( n = 2 \), \( E_m = 1,400,000 \text{ psi}, E_g = 30,000,000 \text{ psi} \)
\( A_m = 5.25 \text{ in}^2, A_g = 0.875 \text{ in}^2 \) and \( s = 2 \text{ in.} \)
\[
C_g = 0.991 \quad \text{(Eq. 7.3-1)}
\]
\[
Z' = Z_{ll}C_DC_gC_A = (1153)(1.0)(0.991)(1.0) \quad \text{(7.3.1)}
\]
\[
= 1143 \text{ lb}
\]
\[
P = 2000 \text{ lb} < nZ' = (2)(1143) = 2286 \text{ psi} \quad \text{ok}
\]

#### Tension

\[
F'_t = F_lC_DC_gC_F = (475)(1.0)(1.0) = 475 \text{ psi} \quad \text{(2.3.1)}
\]
\[
f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi} < F'_t = 475 \text{ psi} \quad \text{ok}
\]

Since the eccentricity of the connection has been eliminated there is no bending. The connection design satisfies NDS provisions.

5. Try a spliced joint using No. 3 Southern Pine with two Industrial 45 (No. 2 stresses) Southern Pine 1x4 stress rated boards as side plates and 1/2 in. bolts

No. 3: \( F_l = 475 \text{ psi} \quad E = 1,400,000 \text{ psi} \quad \text{(Table 4B)} \)
\[
C_{DP} = 1.0 \quad C_F = 1.0
\]
Ind. 45 (No. 2): \( F_l = 825 \text{ psi} \quad E = 1,600,000 \text{ psi} \)

(SPIB Standard Grading Rules for Southern Pine Lumber)

### Bolt Design

For 1/2-in. bolts in double shear with two 1x4 side plates,
\[
Z_{ll} = 1077 \text{ lb/bolt} \quad \text{(Eq. 8.3-3)}
\]
Number of bolts = \( P/Z = 2000/1077 = 1.86 \) bolts
Try two 1/2-in. bolts in a row on each side of splice
With \( n = 2 \), \( E_m = 1,400,000 \text{ psi}, E_g = 1,600,000 \text{ psi} \)
\( A_m = 5.25 \text{ in}^2, A_g = 5.25 \text{ in}^2 \) and \( s = 2 \text{ in.} \)
\[
C_g = 0.999 \quad \text{(Eq. 7.3-1)}
\]
\[
Z' = Z_{ll}C_DC_gC_A = (1077)(1.0)(0.999)(1.0) \quad \text{(7.3.1)}
\]
\[
= 1076 \text{ lb}
\]
\[
P = 2000 \text{ lb} < nZ' = (2)(1076) = 2152 \text{ lb} \quad \text{ok}
\]

#### Tension

\[
F'_t = F_lC_DC_gC_F = (475)(1.0)(1.0) = 475 \text{ psi} \quad \text{(2.3.1)}
\]
\[
f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi} < F'_t = 825 \text{ psi} \quad \text{ok}
\]

2x4's

\[
F'_t = F_lC_DC_gC_F = (475)(1.0)(1.0) = 475 \text{ psi} \quad \text{(2.3.1)}
\]
\[
f_t = P/A_{net} = 2000/4.41 = 454 \text{ psi} < F'_t = 475 \text{ psi} \quad \text{ok}
\]

No eccentricity and therefore, no bending. The connection design satisfies NDS provisions.

Connection designs 3, 4 and 5 all satisfy NDS provisions, with designs 4 or 5 probably being the most practical/economical.

### 7.1.3-Eccentric Connections

Avoidance of fastener eccentricity that induces tension perpendicular to grain stresses in the main wood member at the connection was first introduced as a cautionary note in the 1944 edition of the Specification. Where multiple fasteners occurred with eccentricity, fasteners were to be placed, insofar as possible, such that the wood between them was placed in compression rather than in tension (load coming into the joint through the right hand member and leaving the joint through the left hand member in Figure 7A of the Specification).

The cautionary provisions on tension perpendicular to grain stresses at eccentric connections were dropped from the Specification in 1948 when new provisions for shear design of bending members at connections were introduced. The present provision that eccentric connections that induce tension perpendicular to grain stresses are not to be used unless it has been shown by analysis or test that such joints can safely carry all applied loads has been a part of the Specification since the 1982 edition.

Because of building code requirements calling for design checks for uplift or other load reversals, avoidance of tension perpendicular to grain stresses in configurations such as that shown in Figure 7A often is not possible. An alternative to this detail is to lap both web members on the same fastener axis or, where monoplane webs are required, to use steel straps attached to the ends of the webs to carry the loads to and from the chord member through the same bolt or pin.

It is to be emphasized that no tension design values perpendicular to grain are given in the Specification.
NDS Commentary

This is because undetectable ring shake and checking and splitting that may occur as a result of drying in service make it impractical to establish reliable, generally applicable design values for the property.

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 can not be avoided is a common practice.

7.2-DESIGN VALUES

7.2.1-Single Fastener Connections

Previous Basis. Design values for connections in the 1986 and earlier editions of the Specification were based on generalized relationships established from tests of the various types of fasteners. These relationships used compression parallel or perpendicular to the grain strength, or specific gravity, which is relatively closely correlated with clear wood compression properties, as the measure of the influence of wood quality on connection load-carrying capacity. Adjustment of these basic wood properties for fastener diameter, length and placement was based on the results of joint tests.

1991 Edition. Lateral load 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 Appendix I of the Specification) 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 (93,167). Equations have been developed for each mode relating the joint load to the maximum stresses in the wood members and in the fastener (93,166). 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 load 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 (163). This nominal yield point is intermediate between the proportional limit and maximum loads for the material and for the connection. Figure C7.2-1 graphically illustrates a typical load-deformation curve from a fastener embedment test.

Lateral design loads 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 loads based on direct application of the yield limit equations have been reduced to the nominal average design load levels published in previous editions for connections made with equivalent species and member sizes. This was done by establishing average ratios of previous Specification loads to yield model loads for each mode of failure and direction of loading (parallel and perpendicular to grain). As noted under the commentaries for specific fastener types, this soft conversion procedure based on average design load levels results in some new design loads for each fastener type being higher and some lower than previous values depending upon the fastener diameter and the thickness of main and side member.

7.2.2-Multiple Fastener Connections

The allowable design value for a connection containing two or more fasteners is obtained by summing the allowable loads for each individual fastener. It is to be understood that this provision requires application of the group action factor of 7.3.6 to the individu-
al fastener design value wherever a row of two or more split ring connectors, shear plate connectors, bolts or lag screws are involved.

Summation of individual fastener design values to obtain a total design value for a connection containing two or more fasteners is limited to designs involving the same type and the same size of fastener (see Commentary for 7.1.1.1). 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.

7.2.3-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 fasteners in wood connections subject to lateral loads. These stresses are accounted for in this Specification under the provisions for the particular fastener type involved. In all cases where the design value for a connection involving metal fasteners is based on the provisions of the Specification, the adjustment factors of 7.3 are to be applied.

Where the capacity of the connection is controlled by the strength of the metal fastener or part, the adjustment factors of 7.3 are not to be applied. In these cases, the design for such metal fasteners and parts are not to be increased 1/3 for wind or earthquake loadings if the design load on the connection or part has been reduced for load combinations in accordance with the applicable building code or national standard (10). Load combinations in which the probability of simultaneous occurrence is reflected in a reduced total design load generally will include dead load plus live load plus wind load or dead load plus live load plus earthquake load (see Commentary 2.3.2.3).

7.3-ADJUSTMENT OF DESIGN VALUES

7.3.1-Applicability of Adjustment Factors

Table 7.3.1 indicates what adjustment factors apply to connector design values based on the type of load on the connection: $Z$, $P$ and $Q$ refer to lateral loads; $W$ refers to withdrawal loads.

Design values for all fastener types are adjusted for load duration, wet service and temperature except values for toe-nails loaded in withdrawal are not modified for wet service. Adjustments for multiple fasteners in a row, $C_a$, apply only to laterally loaded bolts, lag screws, shear plates, split rings, drift pins and drift bolts. The geometry factors, $C_y$, refer to end and edge distance and spacing requirements for these same fasteners and for laterally loaded spike grids. Diaphragm and toe-nail adjustment factors are limited to nail and spikes only.

Specific design provisions for drift bolts, drift pins and spike grids are not given in the Specification. Other authoritative sources for the design of connections with these fastener types should be consulted (see Part XIV of the Specification).

The metal side plate adjustment factor, $C_{sp}$, cited in the footnote of Table 7.3.1 refers to the modification of design values for shear plate connectors when metal rather than wood side plates are used (see 10.2.4 of Specification).

7.3.2-Load Duration Factor, $C_D$

The impact load duration factor of 2.0 is not to be applied to design loads for connections. This new limitation is a result of the use of yield model equations to establish the capacities of laterally loaded connections made with dowel type fasteners. These equations take into account the bending yield strength of the metal fasteners, a property which influences the design load of the connection in many configurations. As load duration adjustments for wood properties are not applicable to metal properties, increases in connector lateral design loads where impact loading conditions occur is not appropriate. Lateral connector loads based on the procedures of the Specification may be increased for other load durations, including the 1.6 modification for wind and earthquake loads, because the reduction factors used to adjust yield model values (10 minute duration) to normal load design levels include a 1/1.6 component.

Extension of the 1.6 maximum duration of load adjustment limit to connections made with non-dowel type fasteners and to those where the fastener is subject to withdrawal loads has been made for purposes of uniformity and in recognition that design loads for these other connections also are derived from the results of standard short-term tests (5-10 minute duration) rather than impact tests.
7.3.3-Wet Service Factor, $C_M$

Applications representing dry conditions of use are discussed in the Commentary for 2.3.3.

The wet service factors in Table 7.3.3 for bolts and lag screws, split ring and shear plate connectors, wood screws and common nails have been provisions of the Specification since the 1944 edition. The factor for threaded, hardened nails was added in 1962. These adjustments were recommended as part of the early research on wood connections (57,62).

The factors for metal connector plates were added in 1968. The 0.80 factor for plates installed in partially seasoned or wet lumber is based on the results of both truss and tension in-line joint tests (1,150,195). The factors for drift pins were added in 1977.

The factor of 0.40 in the footnote of Table 7.3.3 for multiple rows of bolts or lag screws installed in partially seasoned wood used in dry conditions of service has been a provision of the Specification since 1948. In earlier editions, this factor was 1/3. The adjustment is based on limited tests of connections fabricated with unseasoned members joined at right angles to each other and tested after drying (62).

7.3.4-Temperature Factor, $C_t$

The temperature adjustment factors for connections in Table 7.3.4 are equivalent to those for bending, compression and shear design values in 2.3.4 (see Commentary for this section). Bearing under metal fasteners is closely correlated with compression parallel to grain or compression perpendicular to grain properties.

7.3.5-Fire Retardant Treatment

(See Commentary for 2.3.6.)

7.3.6-Group Action Factor, $C_g$

Background

Modification factors for two or more split ring connectors, shear plate connectors, bolts or lag screws 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 (46,48,50,92,100).

The tables of factors included in the 1973 edition to account for the nonuniform 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 (103). 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 modulii 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 side and main members being joined. Other variables were assumed to have the following values (201):

Wood to wood connections:

- $E$ of side and main members: 1,800,000 psi
- Load/slip fastener modulus: 220,000 lb/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 lb/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 (201). 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.

1991 Edition. The group action factor equation given in 7.3.6 is a newly developed consolidated expression for the analytical procedure used to establish the modification factors given in previous editions (234). 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 modulii for different types of
connections (234). These new joint stiffness parameters are:

- 4-inch split ring or shear plate connectors: 500,000 lb/in.
- 2.5-inch split ring or 2.625-inch shear plate connectors: 400,000 lb/in.
- Bolts or lag screws:
  - wood to wood connection: 180,000 \( (D^{1.5}) \)
  - wood to metal connections: 270,000 \( (D^{1.5}) \)

where: \( D = \) diameter, inches

The foregoing moduli for 4-inch connectors and 1-inch diameter bolts or lag screws were used to develop the group action factors given in Tables 7.3.6A-7.3.6D. Factors for connections involving wood side plates (Tables A and B) were developed assuming an \( E \) of 1,400,000 psi for both main and side members and a spacing of 4 inches for bolts or lag screws (Table A) and 9 inches for connectors (Table B). The effects of assuming different properties and spacings than those used previously to develop tabulated group action factors for connections with wood side plates is illustrated by the selected comparisons shown in Table C7.3-1.

### Table C7.3-1 - Comparison of Group Action Factors for Connections made with Wood Side Plates

<table>
<thead>
<tr>
<th>Basis</th>
<th>Previous editions</th>
<th>1991 edition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_s, E_m )</td>
<td>1,800,000</td>
<td>1,400,000</td>
</tr>
<tr>
<td>Load/slip modulus:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws</td>
<td>220,000</td>
<td>180,000</td>
</tr>
<tr>
<td>4-inch connectors</td>
<td>220,000</td>
<td>500,000</td>
</tr>
<tr>
<td>Spacing:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws</td>
<td>6.5</td>
<td>4.0</td>
</tr>
<tr>
<td>4-inch connectors</td>
<td>6.5</td>
<td>9.0</td>
</tr>
<tr>
<td><strong>Group Action Factor</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_s = 5, A_m = 10: )</td>
<td>( n = 4 )</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.32</td>
</tr>
<tr>
<td>( A_s = 64, A_m = 64: )</td>
<td>( n = 4 )</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.82</td>
</tr>
<tr>
<td>4-inch connectors:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_s = 5, A_m = 10: )</td>
<td>( n = 4 )</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.32</td>
</tr>
<tr>
<td>( A_s = 64, A_m = 64: )</td>
<td>( n = 4 )</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.82</td>
</tr>
</tbody>
</table>

The foregoing comparisons show that the group action factors for wood side plate connections tabulated in the 1991 edition for 1-inch bolts and lag screws (Table 7.3.6A) are slightly larger than those applicable to these connections in previous editions. For smaller diameter bolts and smaller spacings, the differences would be larger. Alternatively, group action factors tabulated in the 1991 edition for 4-inch split ring or shear plate connectors (Table 7.3.6B) are significantly lower than those applicable to these connections in previous editions. This is primarily a result of the larger load/slip modulus assigned to this size connector relative to the average modulus assigned all connectors in previous editions. Differences between group action factors tabulated in the 1991 edition and previous editions would be less for connections made with smaller split ring or shear plate connectors and smaller spacings.

Differences between tabulated group action factors in the 1991 edition and previous editions for connections made with metal side plates are similar to those for connections made with wood side plates. This is illustrated by the comparisons shown in Table C7.3-2.

### Effect of Joint Properties.

As indicated in the footnotes to Tables 7.3.6A and 7.3.6B, group action factors tabulated in the 1991 edition and previous editions for connections made with metal side plates are similar to those for connections made with wood side plates. This is illustrated by the comparisons shown in Table C7.3-2.

### Table C7.3-2 - Comparison of Group Action Factors for Connections made with Steel Side Plates

<table>
<thead>
<tr>
<th>Basis</th>
<th>Previous editions</th>
<th>1991 edition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_m )</td>
<td>1,400,000</td>
<td>1,400,000</td>
</tr>
<tr>
<td>( E_s )</td>
<td>30,000,000</td>
<td>30,000,000</td>
</tr>
<tr>
<td>Load/slip modulus:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws</td>
<td>330,000</td>
<td>270,000</td>
</tr>
<tr>
<td>4-inch connectors</td>
<td>330,000</td>
<td>500,000</td>
</tr>
<tr>
<td>Spacing:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws</td>
<td>5.75</td>
<td>4.0</td>
</tr>
<tr>
<td>4-inch connectors</td>
<td>5.75</td>
<td>9.0</td>
</tr>
<tr>
<td><strong>Group Action Factor</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-inch bolts or lag screws:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_s = 2, A_m = 24: )</td>
<td>( n = 4 )</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.49</td>
</tr>
<tr>
<td>( A_s = 5, A_m = 120: )</td>
<td>( n = 4 )</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.81</td>
</tr>
<tr>
<td>4-inch connectors:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( A_s = 2, A_m = 24: )</td>
<td>( n = 4 )</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.49</td>
</tr>
<tr>
<td>( A_s = 5, A_m = 120: )</td>
<td>( n = 4 )</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>( n = 12 )</td>
<td>0.81</td>
</tr>
</tbody>
</table>

### Mechanical Connections

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factors for connections made with members having $E$ values greater than 1,400,000 psi, spacings less than 4 inches, fasteners less than 1-inch in diameter and connectors less than 4-inch in diameter will be higher than the tabulated factors. The sensitivity of the group action factor to changes in these variables is shown in Table C7.3-3.

As shown in the Table C7.3-3, changes in $E$ of the joint members, spacing and connector size result in changes to group action factors of less than 20 percent. These tables may be used to help determine when the general equation of 7.3.6.1 should be used rather than Tables 7.3.6A - 7.3.6L to assign group action factors for specific designs.

Table C7.3-3 - Effect of Joint Properties on Group Action Factors

<table>
<thead>
<tr>
<th>Wood Side Plates: $A_m = 10$, $A_s = 5$</th>
<th>Group Action Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of $E \times 10^6$</td>
<td>1-inch bolt load/slip = 180,000</td>
</tr>
<tr>
<td>bolts</td>
<td>main</td>
</tr>
<tr>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>5</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Steel Side Plates: $A_m = 24$, $A_s = 2$</th>
<th>Group Action Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of $E \times 10^6$</td>
<td>1-inch bolt load/slip = 270,000</td>
</tr>
<tr>
<td>bolts</td>
<td>main</td>
</tr>
<tr>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>5</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
</tbody>
</table>

It is to be noted that the variable $A_s$ in the group action equation (7.3-1) 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 8.4). 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.

The modulus of elasticity values to be used with equation 7.3-1, and which are the basis for the factors in Tables 7.3.6A - 7.3.6D, are the design $E$ values given in Tables 4 and 5 of the Supplement and Table 6A.

Perpendicular to Grain Loading. Connections involving perpendicular to grain loading, such as joints between web and chord members, generally do not involve rows of bolts containing large numbers of fasteners in a row perpendicular to grain. Similarly, large beams are generally supported on hangers rather than on stacked fasteners aligned perpendicular to grain in order to avoid splitting that can occur as a result of drying in low relative humidity service conditions.

Group action factors are limited by the maximum loads on the end fasteners in a row without any adjustment for the redistribution of load to other more lightly loaded fasteners in the row that is known to occur as a result of yielding under load. Such redistribution is considered to be significant where fasteners load the member perpendicular to the grain because of the relatively low stiffness of wood in this direction.

Based on the infrequent use of more than two bolts or other fasteners in a row perpendicular to grain, and the redistribution of load that occurs between fasteners in such connections, it is standard practice to use the same group action factor for rows of fasteners aligned perpendicular to grain as that for fasteners aligned parallel to grain. This procedure, which has been satisfactorily used since 1973 when the group action factor was first introduced, is continued in the current Specification.

7.3.6.2 The criteria for determining when staggered fasteners are considered to represent a single row have been part of the Specification since 1977.

7.3.6.3 The use of gross section areas and the definition of cross-sectional area for fastener groups loaded perpendicular to the grain were introduced in the 1973 edition at the time group action modifications were added to the Specification.
VIII: BOLTS

8.1-GENERAL

8.1.1-Quality of Bolts

In the previous three editions, the bolt quality standard referenced in the Specification was ASTM Standard A307, Low Carbon Steel Externally and Internally Threaded Fasteners. The current standard of this designation, A307-88a, is now titled, Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength. The scope of the present ASTM A307 is too narrow for it to be used as a reference standard as bolt provisions in the 1991 edition of the Specification provide for the use of bolts of any bending yield strength (see Appendix I). ANSI/ASME Standard B18.2.1-1981, Square and Hex Bolts and Screws (Inch Series), which is being used as the bolt quality reference in the 1991 edition, is the referenced standard for dimensions in A307.

Bolt design values given in previous editions of the Specification were based on a bending yield strength, , of 45,000 psi. This value is applicable to A36 steel having an ultimate tensile strength of 58,000 psi. Bolt design values tabulated in Tables 8.2A-D and 8.3A-D of the 1991 edition are based on bolts made of steel having these properties.

Maximum bolt diameter. Bolt design values for bolts up to 1-1/4 inches in diameter were tabulated in the 1968 and earlier editions of the Specification. Values for 1-1/2 inch bolts were added to the design load tables beginning with the 1971 edition. In the 1991 edition, bolt design provisions and tabulated bolt design loads apply only to bolts having diameters of 1 inch or less. This conservative change was made following several reported field problems with connections involving large diameter bolts in glued laminated timber members and the results of new research (40,173). The latter showed drying in service, workmanship variables and perpendicular to grain load of bolts bearing on wood members be kept to a practical minimum was added as a good practice recommendation in the 1986 edition. Allowance of some thread bearing on wood without modification of bolt design values is long standing practice supported by field experience. Note should be made that when threads occur in the shear plane, the effect of such threads on the design shear strength of the bolt itself is to be taken into account (see 7.2.3).

8.1.2-Fabrication and Assembly

8.1.2.1 The range of allowable bolt holes of 1/32-inch to 1/16-inch larger than the bolt diameter has been a provision of the Specification since 1948. Designation of these limits as minimum and maximum oversizes was added in the 1986 edition.

Generally, the smaller diameter bolts will use the smaller oversize holes and the larger bolts the larger oversize. The same target oversize is to be used for all holes in the same connection.

8.1.2.2 Centering of holes and avoidance of forcible driving have been good practice provisions of the Specification since the 1944 edition.

8.1.2.3 Use of washers or equivalent metal parts under the head and nut to prevent localized crushing of the wood at bolt holes has been a requirement of the Specification since the 1960 edition.

8.1.2.4 Design values for bolted joints have been applied to connections having both tight and loose nuts since the 1944 edition. This provision 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 (183). It is to be noted that 8.1.2.4 addresses only 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 bolt design values for these factors is required when connections are assembled with wet or partially seasoned wood (see 7.3.3).

8.1.2.5 The requirement that the threaded portion of bolts bearing on wood members be kept to a practical minimum was added as a good practice recommendation in the 1986 edition. Allowance of some thread bearing on wood without modification of bolt design values is long standing practice supported by field experience. Note should be made that when threads occur in the shear plane, the effect of such threads on the design shear strength of the bolt itself is to be taken into account (see 7.2.3).
### 8.2-DESIGN VALUES FOR SINGLE SHEAR CONNECTIONS

#### 8.2.1-Wood-to-Wood Connections

**Background**

Bolt design values tabulated in the Specification prior to the 1991 edition were based on early research (183) which related the bearing strength of wood under bolts to the compression parallel and perpendicular to grain properties of the wood members and the resistance of the bolt to bending as measured by the ratio of the length of bolt in the main member to its diameter. Except for adjustments made to reflect the change in lumber design values in 1970 resulting from the introduction of new clear wood strength properties and species grouping criteria in ASTM D2555, and the provisions of the new softwood lumber product standard PS 20, the general methodology for establishing bolt design values remained essentially unchanged from the 1944 through the 1986 editions. Under this methodology, allowable bolt design values were established in accordance with the equations shown below.

Bolt design values parallel to grain:

\[ Z = c_1 F_1 L d r_1 \]  

(C8.2-1)

where:

- \( Z \) = nominal bolt design value, pounds
- \( c_1 \) = adjustment for difference between proportional limit load of bolted joints and proportional limit under uniform compression load = 0.80
- \( F_1 \) = unseasoned clear wood maximum compression design value parallel to grain 5 percent exclusion value for each species or species group (based on ASTM D2555), reduced 1.0/1.9 for load duration and factor of safety based on ASTM D245, and increased 1.20 for seasoning based on early research (183), psi
- \( L \) = length of bolt in main member, inches
- \( d \) = bolt diameter, inches
- \( r_1 \) = adjustment depending on \( L/d \) ratio of bolt and \( F_1 \) (ranging from 1.00 at \( L/d \) of 5.00 to 0.310 at \( L/d \) of 13)

Although the proportional limit joint load factor \( c_{J} \) for hardwoods was found to be 1.00 (183), and the ASTM D245 load duration/factor of safety adjustment for hardwoods is 2.1, the softwood values for these terms were used for all species for simplicity.

Bolt design values perpendicular to grain:

\[ Z = c_2 F_2 L d r_2 \]  

(C8.2-2)

where:

- \( c_2 \) = width of bearing increase based on bolt diameter (183) (ranging from 2.50 for 1/4-inch bolt to 1.27 for 1-inch bolt)
- \( F_2 \) = clear wood unseasoned average proportional limit stress for species or species group based on ASTM D2555, reduced 1.0/1.5 for ring placement based on ASTM D245, increased 1.10 for normal loading, and increased 1.20 for seasoning based on early research (183)
- \( r_2 \) = adjustment depending on \( L/d \) ratio of bolt and \( F_2 \) (ranging from 1.00 at \( L/d \) of 5.00 to 0.375 at \( L/d \) of 13)

other symbols as previously defined

The bolt design values established by the foregoing equations were applicable to three member joints in which the side members were one-half the thickness of the main member. Allowable bolt design values for two member or four or more member joints were established as proportions of the three member bolt design values in accordance with specified rules.

#### 1991 Edition

In the 1991 edition, lateral design values for bolts are based on a yield limit model which considers the different ways the bolted connection can deform under load (see Commentary for 7.2.1 and Appendix I). The capacity of a specific joint is determined for each yield mode and the lowest design value calculated for the different modes is selected as the nominal bolt design value, \( Z \), for the joint. The yield mode equations are entered with the dowel bearing strengths and thicknesses of the wood members and the diameter and bending yield strength of the fastener.

Wood dowel bearing strengths used in the yield mode equations are based on a load representing a 5 percent diameter offset on the load-deformation curve obtained from a bolt embedment test. This load is intermediate between the proportional limit and ultimate loads obtained from such a test (see Commentary section 7.2.1).

Although the yield limit model represents significantly different methodology than that used previously to establish bolt design values, the relative effects of various joint variables shown by both procedures are generally similar (166). Short-term bolt design values
obtained from application of the yield model equations have been reduced to the average bolt design value levels published in previous editions of the Specifications for connections made with the same species and member sizes (see Commentary for 7.2.1). As noted above, these previous tabulated bolt design values are indexed to nominal proportional limit bolt design values rather than 5 percent offset bolt design values.

**Previous Methodology for Single Shear Bolted Connections.** Bolt design values tabulated in the 1986 and earlier editions applied to three member joints in which the side members were each one-half the thickness of the main member. For two member joints, those with a single shear plane, a proportion of the bolt design value for the three member connection was used. Prior to the 1977 edition, the two-member proportion was taken as one-half the tabulated three member bolt design value for a piece twice the thickness of the thinner piece. This early practice for establishing single shear bolt design values, introduced in 1935 (127), was considered appropriate application of the results of the original bolt research (58). In 1977, based on new research and reevaluation of original test results (82,183,200), the basis for bolt design values for two member connections was changed to the smaller of (i) one-half the tabulated three member bolt design value for a piece the thickness of the thickest member, or (ii) one-half the tabulated three member bolt design value for a piece twice the thickness of the thinner member. This change provided more conservative design values for all two member wood-to-wood single shear bolted connections in which the thicker member was less than twice the thickness of the thinner member, with reductions of 50 percent occurring in some joints in which the two members were of equal thickness. The new practice for establishing design values for two member bolted connections was continued through the 1986 edition.

Results from application of the yield model confirm the general relationship between design values for two member and three member bolted connections established in the 1977 edition (202).

**Joint Members Loaded in Different Directions.** The change in single shear joint provisions introduced in the 1977 edition resulted in inconsistent treatment of joints in which one member was loaded parallel to grain and the other at an angle to grain a bolt design value equal to the lesser of (i) one-half the tabulated bolt design value for a piece the thickness of the parallel to grain loaded member, or (ii) the bolt design value obtained from the equation for allowable bearing at an angle to grain (Appendix J of the Specification) using one half the tabulated parallel to grain and perpendicular to grain bolt design values for a piece the thickness of the angle-to-grain member as P and Q, respectively. This procedure assured that allowable loads on joints in which one member was loaded at an angle to grain would converge with those for joints in which the angled member was at 90° to the parallel loaded member.

In the 1991 edition, the condition where members of a single shear bolted connection are loaded at different angles to the grain is provided for by the factor, $K_\theta$, in the denominator of the yield mode equations which accounts for the maximum angle of load to grain for any member in the connection; and by Equation 8.2-7 of the Specification which uses the bearing angle to grain equation (Appendix J) to adjust the dowel bearing strength of each member loaded at an angle to grain.

**Yield Mode Equations**

The bolt design value equations (8.2-1 to 8.2-6) for single shear wood-to-wood connections were developed from European research (93,104) and have been confirmed by bolt tests on domestic species (116,163,166,167). The limiting yield modes covered by these equations are bearing in the main or side members (Mode I), bolt rotation without bending (Mode II), development of a plastic hinge in the bolt in main or side member (Mode III) and development of plastic hinges in the bolt in both main and side members (Mode IV) (see Appendix I of the Specification). The term $4K_\theta$, $3.6K_\theta$ or $3.2K_\theta$ in the denominator of equations 8.2-1 to 8.2-6 represents the average factor relating yield model design value for each mode based on 5 percent offset dowel bearing strength to the proportional limit based bolt design values tabulated in the 1986 edition (202). For bolts loaded parallel to grain, $K_\theta$ equals one. For perpendicular to grain loading, $K_\theta$ equals 1.25 for a connection with one member loaded parallel to grain and the other member loaded perpendicular to grain (202).

Dowel bearing strengths used in the yield mode equations are tabulated in Table 8A for all structurally graded lumber species. These values also apply to
main members of glued laminated timber. The values in Table 8A represent 5 percent diameter offset values determined in accordance with the following equations (203):

Parallel to grain:

\[ F_e = (11,200) \ \text{G} \quad \text{(C8.2-3)} \]

Perpendicular to grain:

\[ F_e = (6,100) \ \text{G}^{1.45} \ \text{D}^{-0.5} \quad \text{(C8.2-4)} \]

where:

- \( F_e \) = dowel bearing strength, psi
- \( G \) = specific gravity based on oven dry weight and volume
- \( D \) = bolt diameter, inches

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 (203).

The specific gravity values given in Table 8A for each species or species group are those used to establish the corresponding dowel bearing strength values, \( F_e \). These specific gravity values represent average values from in-grade lumber test programs (see Commentary for 4.2.3.2) or are based on information from ASTM D2555. No separate specific gravity values are available for dense Douglas-fir and dense southern pine; therefore, no dowel bearing strength values nor bolt design values are tabulated for these dense species combinations in the 1991 edition.

The bending yield strength, \( F_{yb} \), of the bolt used in the yield mode equations is taken as the average of the yield and ultimate tensile strengths of the metal (see Appendix I). For A36 and stronger steels, \( F_{yb} \) equal to 45,000 psi is a conservative value and is equivalent to the bolt strength reported in the original bolt test research (183).

For each particular joint configuration, the nominal bolt design value for each yield mode must be calculated to determine the limiting value for the connection. Such yield mode bolt design values for a number of different joint designs are shown in Example C8.2-1 to illustrate the results of applying equations 8.2-1 to 8.2-6.

### Example C8.2-1

Yield mode design values for wood-to-wood single shear bolted connections:

Hem-fir two member connections made with 1/2 and 1 inch bolts, side member thickness of 1-1/2 inches, main member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

<table>
<thead>
<tr>
<th>Thickness, in. &amp; Bolt Diam.</th>
<th>Main Yield Mode Design Value, lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2 // 1-1/2 // 1-1/2</td>
<td>900 900 414 550 550 663</td>
</tr>
<tr>
<td>1-1/2 // 1-1/2 // 1-1/2</td>
<td>720 382 250 380 324 442</td>
</tr>
<tr>
<td>1-1/2 // 1-1/2 // 1-1/2</td>
<td>382 720 250 324 380 442</td>
</tr>
<tr>
<td>1 // 1-1/2 // 1-1/2</td>
<td>1800 900 674 845 550 663</td>
</tr>
<tr>
<td>1 // 1-1/2 // 1-1/2</td>
<td>1440 382 472 592 224 442</td>
</tr>
<tr>
<td>1 // 1-1/2 // 1/2</td>
<td>765 720 346 433 380 442</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>3600 1800 1359 2200 1884 2652</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>2880 540 874 1378 1143 1567</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1080 1440 551 1117 1135 1567</td>
</tr>
<tr>
<td>5-1/2 // 1-1/2 // 1-1/2</td>
<td>6600 1800 2451 3161 1884 2652</td>
</tr>
<tr>
<td>5-1/2 // 1-1/2 // 1-1/2</td>
<td>5280 540 1678 2021 1143 1567</td>
</tr>
<tr>
<td>5-1/2 // 1-1/2 // 1-1/2</td>
<td>1980 1440 856 1311 1135 1567</td>
</tr>
</tbody>
</table>

**Member Loaded at Angle to Grain.** Equation 8.2-7 is used to calculate the dowel bearing strength for a main or side member loaded at an angle to grain. This equation, a form of the bearing angle to grain equation (Appendix J), has been used since the 1944 edition to determine allowable design values for bolts acting in a plane inclined to the direction of grain. In earlier editions, the equation was entered with allowable bolt design values parallel and perpendicular to grain. In the 1991 edition, the equation is entered with the parallel and perpendicular dowel bearing strengths for the member and the bolt design value is determined from the yield mode equations using \( F_{ce} \) as the dowel bearing strength for the main or side member. The bolt design value obtained from this procedure is similar to that obtained from using parallel to grain and perpendicular to grain \( Z \) values in the bearing interaction formula to obtain a \( Z \) design value for the
connection (202). Determining a \(Z_\theta\) design value using this latter approach is an acceptable alternative to calculating \(F_{ \theta \theta }\) for use in each yield mode equation and allows the use of tabulated \(Z\) values from the Specification.

Tabulated Two Member Wood-to-Wood Bolt Design Values. Bolt design values for lumber to lumber (Table 8.2A) and glued laminated timber to lumber (Table 8.2B) connections have not previously been tabulated in the Specification, being taken as a proportion of the tabulated three member bolt design value in earlier editions (see Commentary on Previous Methodology for Single Shear Bolted Connections). With the added refinement of the yield mode equations, two member bolt design values are not necessarily a fixed one-half the three member bolt design value. Separate two member bolted connection design values are therefore given to facilitate designer use. All tabular bolt design values are based on a bolt bending yield strength of 45,000 psi. Two bolt design values for perpendicular to grain loading are shown: one for a connection with the side member loaded perpendicular to grain and the main member loaded parallel to grain \((Z_{21})\); and one for a connection with main member loaded perpendicular to grain and the side member loaded parallel to grain \((Z_{m2})\).

The soft conversion procedure used to translate short term yield mode bolt design values to bolt design values previously tabulated in the Specification involved use of average adjustment factors for each mode based on all bolt and member sizes, all species combinations, and both wood and steel side members. For two member connections, the adjustment factor was based on joints in which the side member was one-half the thickness of the main member. Under this indexing procedure, new bolt design values are both higher and lower than previous bolt design values depending upon the bolt diameter, the thicknesses of the main and side member, and the particular species combination involved.

Comparison of 1991 and Earlier Edition Bolt Design Values. Differences between 1991 and earlier edition wood-to-wood single shear bolt design values for two species combinations are shown in Table C8.2-1.

For the joint configurations compared, bolt design values based on the 1991 edition for the parallel and perpendicular to grain loading cases averaged 11 percent and 54 percent higher, respectively, than those based on the 1986 edition. It is to be noted that the short-term yield mode bolt design values on which the 1991 bolt design values are based were reduced to the average level of 1986 and earlier tabulated bolt design values using average factors for joints in which the side member was one-half the thickness of the main member. This was the configuration used for the early tests from which the bolt design values in the 1986 and earlier editions were derived. Only one configuration (3-1/2 main and 1-1/2 side) in the table above meets this calibration condition. The average ratios of 1991 to 1986 bolt design values for the joints in the table which have this calibration geometry are 0.97 and 1.18 for parallel and perpendicular to grain loading, respectively.

<table>
<thead>
<tr>
<th>Thickness, in. Board</th>
<th>Bolt Design Value, lbs</th>
<th>Southern pine</th>
<th>Spruce-Pine-Fir</th>
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<tr>
<td>1-1/2</td>
<td>1/2</td>
<td>530</td>
<td>470</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>800</td>
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<td>1315</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1740</td>
<td>1875</td>
</tr>
<tr>
<td>3-1/2</td>
<td>1/2</td>
<td>750</td>
<td>635</td>
</tr>
<tr>
<td></td>
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<td>1690</td>
<td>1400</td>
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<td>2435</td>
</tr>
<tr>
<td>5-1/2</td>
<td>1/2</td>
<td>1690</td>
<td>1430</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>2870</td>
<td>2435</td>
</tr>
</tbody>
</table>

The yield mode equations on which the 1991 bolt design values are based provide a fully rationalized and consistent measure of the effects of main and side member thicknesses and bolt diameter. As such they
earlier editions which represent a conservative application of test results for only one main to side member configuration.

**Mixed Species Connections.** Design values for bolted connections made with a side member of different species than the main member can be calculated using the yield mode equations (Eqs. 8.2-1 to 8.2-6) and the appropriate dowel bearing strength for each species. Mixed species connections were provided for in the 1982 and 1986 editions by assigning the joint the lesser of the bolt design value applicable to a comparable joint made with members of the main member species or a comparable joint made with members of the side member species. In lieu of using the yield mode equations, bolt design values for connections made with different main and side member species may be based on bolt design values in Table 8.2A and 8.2B for the species with the lower dowel bearing strength, $F_c$.

### 8.2.2-Wood-to-Metal Connections

**Background**

In the 1977 and earlier editions of the Specification, bolted connections made with steel side members were assigned design values that were 25 percent larger than tabulated bolt design values for wood main members loaded parallel to grain. No increase for steel side plates was recognized when wood main members were loaded perpendicular to grain. These provisions were based on early bolt tests (183) involving two softwood and two hardwood species which showed that proportional limit bolt design values for joints made with steel side members and the main member loaded parallel to grain averaged 25 percent higher than proportional limit bolt design values for the comparable wood side member joint.

In the 1981 edition, following additional research on two and three member joints made with steel and wood side plates, the adjustment of tabulated parallel to grain bolt design values for use of steel side plates was increased from 25 to 75 percent. The new research (113) was conducted as a result of the impact of the 1977 change in the procedure for establishing bolt design values for two member connections (see Commentary for 8.2.1 - Previous Methodology for Single Shear Bolted Connections). The 1977 change reduced allowable design values for single shear bolted connections, including those made with steel side members, as much as 50 percent below bolt design values previously used successfully for many years. Applications particularly affected included shear wall tie-downs where the change in the bolt design provisions of the Specification required the number of bolts used in such hardware to be doubled for the same code specified loads.

Results from the additional parallel to grain bolt tests (113), which involved joints made with 1/2-inch bolts, 1/4-inch thick steel side plates and southern pine main and side members, showed that the average proportional limit bolt design values for joints made with steel side plates were 20 to 63 percent higher than those for the matching wood side plate joints; that the lowest proportional limit joint design value (wood-to-wood three member joint) was 32 percent lower than the applicable tabulated bolt design value; and that the slip of the metal-to-wood joints was less than that of the wood-to-wood joints. The design values of the metal-to-wood joints associated with the proportional limit slip of the matching wood-to-wood joints were 75 percent larger on the average than the proportional limit design values for the wood-to-wood joints. These average equivalent test slip design values for the metal side plate joints in turn averaged from 20 to 70 percent lower than the maximum average test loads for these joints.

The information obtained from the new testing on the difference in bolt design values on joints made with steel and wood side members at the proportional limit slip of the latter was consistent with that reported in the early bolt research which formed the basis of the Specifications bolt design provisions (183). In the earlier work, a slip of about 0.025 inches was reported associated with the proportional limit design values of joints made with steel side plates and 1/2 inch diameter bolts, and these bolt design values averaged about 25 percent larger than those for comparable joints made with wood side plates. However, the slip associated with the proportional limit load of the wood side member joints was 0.035 inches, or about 40 percent greater than the proportional limit slip of the steel side member joints. As a first approximation, these results indicated the load of the steel side member joints associated with the proportional limit slip of the wood side member joints was $[(1.25P_{w})/0.025] 	imes 0.035$, or about 75 percent larger than the proportional limit load of the wood-to-wood joint; the same as observed in the new tests.

On the basis of the 30 years of successful performance of two member bolted connections made with steel side plates at bolt design values up to twice those established in the 1977 edition, and recognizing that the structural serviceability of a joint is related to its stiffness or slip; a 75 percent adjustment factor for
joints loaded parallel to grain and made with steel side plates, as shown by the slip equivalent loads, was introduced in the 1982 edition. This revision had the effect of increasing design values for steel side member bolted connections loaded parallel to grain approximately 40 percent over those provided by 1977 provisions.

In the 1986 edition, more conservative provisions for establishing design values for connections made with steel side members and large diameter bolts were introduced. This further revision followed unsatisfactory field experience with long span truss design involving glued laminated timber tension chords and large diameter bolts designed in accordance with provisions of the 1977 edition. In response to questions raised about design procedures for all joints made with 1-inch and larger bolts, a special study (97) was conducted of double-shear connections made with glued laminated 4-1/2 by 4-1/2 inch main members, 1-1/4-inch bolts and 2-1/4-inch wood and 1/2-inch steel side members. Although the average ratio of ultimate test load to 1982 NDS bolt design value was 2.7 for the joints made with steel side plates, the ratio was lower than the comparable ratio of 3.9 obtained for connections made with 1/2-inch bolts and steel side members that were tested earlier (113). Wood-to-wood 1/2 inch and 1-1/4 inch bolted joints both had test to design load ratios exceeding 4.0.

Based on these results, it was considered appropriate to reduce the design values for connections made with steel side members and large diameter bolts such that the load ratio factor for the 1-1/4 inch bolt tests was comparable to that for the 1/2 inch bolt tests. This was accomplished in the 1986 edition by providing a variable adjustment for connections loaded parallel to grain and made with steel side members with limits of 75 percent for bolts 1/2 inch or less in diameter and 25 percent for bolts 1-1/2 inch in diameter, with proportionate adjustments for intermediate diameters.

1991 Edition. Bolt design provisions in the current edition are limited to bolts 1 inch or less in diameter. This new limitation reflects concern about the effects of workmanship variables and drying in service on the performance of large diameter bolted connections (see Commentary for 8.1.2). Further, all bolt design values are now based on the yield limit model which does not account for degree of deformation or slip (see Commentary for 8.2.1 - Background). Short-term bolt design values based on direct application of the yield model equations have been reduced to the nominal average bolt design value levels published in earlier editions for connections made with the same species and member sizes, with the exception that conversion factors for joints made with steel side members were referenced to 1977 edition bolt design values which utilized a 25 percent increase for joints loaded parallel to grain and made with metal side plates. This approach was used on the basis that bolt design values in the 1977 and earlier editions were related to proportional limit joint loads, yield model bolt design values are based on loads associated with an offset of 5 percent of fastener diameter, and both are independent of a specific slip level. In addition, the same conversion factors are applied to connections made with steel side members as to wood side members.

It is recognized that relating short-term yield model bolt design values for parallel to grain loaded connections made with steel side members to 1977 edition bolt design values for these connections results in a reduction in bolt design values from 1982 and 1986 edition levels. The advantages of consistency of treatment across all dowel fastener types, and the ability to determine by fully rationalized methodology the effects of member thickness, member strength, bolt size, bolt strength and number of members, both singly and in combination, were considered to outweigh the impact of the reduction in bolt design values for those connections employing steel side members. Differences between 1991 and 1986 design values for such bolted joints are illustrated in the Commentary for 8.2.2.1. In this regard, it is to be noted that design values for bolted connections involving proprietary tie-downs or similar hardware may be established by other procedures than those given in the 1991 edition (see 7.1.1.4 and 1.1.1.4 of the Specification). However, use of such alternate methodologies is the sole responsibility of the manufacturer and of the designer utilizing the design values so derived.

8.2.2.1 The same yield mode equations used for single shear wood-to-wood bolted connections are used for wood-to-metal bolted connections except equation 8.2-2 for mode \( I_4 \) which is for uniform bearing in the metal side member. This condition is checked independently in accordance with 8.2.2.2 and 7.2.3 (see Commentary for this latter section).

The yield mode equations of 8.2.1 may be entered with a dowel bearing strength, \( F_{y_b} \), of 58,000 psi for the side member when A36 or higher strength steel side plates are used. This value is equivalent to the ultimate tension strength of the steel (see Appendix I.2).

A nominal bolt design value, \( Z \), is calculated for each of the five applicable yield mode equations and the lowest value is selected as the design value for the bolted connection. The effects of different main
member thicknesses, bolt diameters and loading directions on values of \(Z\) that can be obtained from each yield mode equation, and on the limiting load for the connection, are illustrated by the example joint design calculations shown in Example C8.2-2.

Tabulated bolt design values for lumber and glued laminated timber single shear connections with 1/4 in. steel side plates given in Tables 8.2C and 8.2D assume A36 steel having a dowel bearing strength of 58,000 psi, and a bolt bending yield strength of 45,000 psi.

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences in bolt design values for lumber-to-steel single shear connections between the 1991 and the 1986 editions are illustrated in Table C8.2-2. For the species and joint configurations compared, the parallel to grain design values for single shear steel-to-wood bolted connections based on the provisions of the 1991 edition average 25 percent lower than those based on provisions in the 1986 edition. Perpendicular to grain bolt design values average 30 percent higher in the new edition than the perpendicular to grain bolt design values in previous editions which contained no increase for use of steel side plates.

The 1986 edition bolt design values in Table C8.2-2 represent one-half the bolt design value for a three member joint made with wood side members one-half the thickness of the main member and then, for the case of parallel to grain loading only, increased 75 percent, 62-1/2 percent and 50 percent for 1/2 inch, 3/4 inch and 1 inch bolts, respectively, when steel rather than wood side members were used. As previously discussed in the Background commentary to this section, the 1991 bolt design values are based on a conversion of short term yield mode bolt design values

### Example C8.2-2

Yield mode design values for wood-to-metal single shear bolted connections:

Hem-fir two member wood-to-metal connections made with 1/2 and 1 inch bolts, steel side member thickness of 1/4 inch, main member thicknesses of 1-1/2, 3, 3-1/2 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

\[
\begin{align*}
F_{\text{em}} &= 4800 \text{ psi parallel} \\
&= 2550 \text{ psi perpendicular, 1/2 inch bolt} \\
&= 1800 \text{ psi perpendicular, 1 inch bolt} \\
F_{\text{yb}} &= 58,000 \text{ psi} \\
F_{\text{yb}} &= 45,000 \text{ psi}
\end{align*}
\]

<table>
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<tr>
<th>Wood Thickness, in.</th>
<th>Load Direction</th>
<th>Bolt Diam., in.</th>
<th>Z_{II}</th>
<th>Z_{III}</th>
<th>Z_{IV}</th>
<th>Z_{V}</th>
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### Table C8.2-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Single Shear Bolt Design Values

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<td>1877</td>
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<td>520</td>
<td>410</td>
<td>1.27</td>
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<td>1880</td>
<td>2378</td>
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<td>0.75</td>
<td>900</td>
<td>775</td>
<td>1.16</td>
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</table>
to the average levels of previously tabulated bolt design values increased only 25 percent for steel side members. The fact that the 1991 bolt design values for the parallel to grain load cases in Table C8.2-2 average 25 percent lower than the 1986 bolt design values reflects the average effect of this difference ($1-1.25/1.625$ or 23 percent) in steel side member adjustment. (For additional discussion see Commentary for 8.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values).

8.2.2.2 (See Commentary for 7.2.3)

8.2.3-Wood-to-Concrete Connections

A specific provision for establishing design values for a single shear connection involving a wood member attached to concrete or masonry through an embedded bolt in the latter was introduced in the 1982 edition. Such connections were assigned a bolt design value equal to one-half the tabulated bolt design value for a piece twice the thickness of the wood member. This procedure was based on the conservative assumption that the concrete or masonry was providing bearing support and fixity at least equivalent to that provided by a wood member twice the thickness of the attached wood member. The 1982 provision for connections involving concrete and masonry was continued in the 1986 edition.

The 1991 edition continues the approach established in earlier editions of considering a wood-to-concrete bolted connection equivalent to a single shear wood-to-wood bolted connection made with a main member twice the thickness of the side member. To determine the allowable bolt design value for such a connection, the yield mode equations of 8.2.1 can be entered with $t_p$ equal to twice $t_s$ and $F_{cm}$ equal to $F_{cs}$; or the design value from Table 8.2A or 8.2B for the applicable main and side member relative thicknesses and bolt diameter can be used.

It is the designer's responsibility to assure that the concrete or masonry has sufficient embedment and dowel bearing strength to resist loads imposed through the embedded fastener.

8.2.4-Load at Angle to Bolt Axis

Two member connections in which the load acts at an angle to the axis of the bolt are checked using the component of the load acting at 90° to the axis and member thicknesses equal to the length of the bolt in each member measured at the centerline of the bolt (see Specification Figure 8B). This methodology has been a provision of the Specification since the 1944 edition. Prior to the 1977 edition, the allowable bolt design values for such joints were taken as one-half the tabulated bolt design value for a member whose thickness was twice the bolt length in the thinner piece. This was changed in the 1977 and subsequent editions to the lesser of one-half the tabulated bolt design value of the thicker member or one-half the tabulated bolt design value for a piece twice the thickness of the thinner member (see Commentary for 8.2.1 - Previous Methodology for Single Shear Bolted Connections), where the length of the bolt in each member was used as the thickness of that member. The centerline of the bolt was made the reference for measuring the bolt length in each member in the 1986 edition.

In the 1991 edition, allowable bolt design values for connections in which the load acts at an angle to the bolt axis are checked using tabulated compression design values perpendicular to grain, $F_g$, adjusted as appropriate by the bearing area factor, $C_b$, (see 2.3.10); and bearing on the angled member should be evaluated using tabulated and allowable bearing design values parallel to grain, $F_g$ and $C_b F_{cs}$, in the interaction equation of 3.10.3.

8.3-DESIGN VALUES FOR DOUBLE SHEAR CONNECTIONS

8.3.1-Wood-to-Wood Connections

Background (See Commentary for 8.2.1)

Yield Mode Equations

The yield mode equations for three member, double shear bolted connections parallel those for two member, single shear bolted connections in 8.2.1 except that two of the modes for the latter configuration are not applicable: bolt rotation without bending, Mode II; and development of a plastic hinge in one of the side members, Mode III. The equations for the remaining modes ($I_m$, $I_s$, $I_m$ and IV) are the same as those for the single shear configuration except for three of the conversion factors, $n K_g$, used to relate short-term yield model bolt design values to average nominal proportional limit bolt design values tabulated in previous editions of the Specification, as shown below.
The angle factor $K_\theta$ has the same values for the double shear as for the single shear case: 1.00 for parallel to the grain loading and 1.25 for perpendicular to grain loading of either the main or side member.

The four yield mode equations for double shear bolted connections are solved for $Z$ and the lowest value obtained is the allowable design value for the joint. The effects of member thickness, bolt diameter and direction of loading on the $Z$ values for each mode and the limiting value are illustrated in Example C8.3-1.

**Example C8.3-1**

Yield mode design values for wood-to-wood double shear bolted connections:

Hem-fir three member connections made with 1/2 and 1 inch bolts, side member thickness of 1-1/2 inches, main member thickness of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

\[
F_{em}F_{Es} = 4800 \text{ psi parallel} \\
= 2550 \text{ psi perpendicular, 1/2 inch bolt} \\
= 1800 \text{ psi perpendicular, 1 inch bolt} \\
F_{yb} = 45,000 \text{ psi}
\]

**Table C8.3-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Double Shear Bolt Design Values**

<table>
<thead>
<tr>
<th>Thickness, in. &amp; grain direction</th>
<th>Bolt Diam. in.</th>
<th>Yield Mode Design Value, lbs</th>
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<td><strong>Z_{III}</strong></td>
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<tr>
<td>1-1/2 // 1/2</td>
<td>1/2</td>
<td>900</td>
</tr>
<tr>
<td>1-1/2 // 1-1/2</td>
<td>1/2</td>
<td>720</td>
</tr>
<tr>
<td>1-1/2 // 1/2</td>
<td>1/2</td>
<td>382</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1/2</td>
<td>1/2</td>
<td>1800</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1/2</td>
<td>1440</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1/2</td>
<td>1/2</td>
<td>765</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1</td>
<td>3600</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1</td>
<td>2880</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1/2</td>
<td>1/2</td>
<td>1080</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1/2</td>
<td>1820</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1/2</td>
<td>2160</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
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<td>2160</td>
</tr>
<tr>
<td>3 // 1-1/2 // 1-1/2</td>
<td>1/2</td>
<td>2160</td>
</tr>
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</table>

Allowable bolt design values, $Z$, for sawn lumber and glued laminated timber double shear connections are tabulated in Tables 8.3A and 8.3B. The latter table is applicable to glued laminated timber main members and sawn lumber side members.

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences in design values for double shear bolted connections in the 1991 and 1986 editions are illustrated for two species in Table C8.3-1. For the joint configurations compared in this table, bolt design values in the 1991 edition average 16 percent and 35 percent higher for parallel and perpendicular to grain loading, respectively, than bolt design values based on 1986 provisions. Average differences for the one configuration in which the thickness of the
side members is one-half the thickness of the main members (3 inch main and 1-1/2 inch side) are 8 percent and 22 percent for parallel and perpendicular to grain loading, respectively. This main to side member thickness ratio was used to establish the conversion factors between short term yield mode bolt design values and bolt design values tabulated in previous editions of the Specification (see Commentary for 8.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values).

Mixed Species Connections. Where the side member species differs from the main member species, bolt design values in Tables 8.3A and 8.3B for the species with the lowest dowel bearing strength may be used (see Commentary for 8.2.1 - Mixed Species Connections).

Loads at Angle to Grain. When the main or side members are loaded at an angle to grain, the yield mode equations of 8.3.1 may be entered with dowel bearing strengths, $F_{cd}$, determined in accordance with Equation 8.2.7.

8.3.2-Wood-to-Metal Connections

Background (See Commentary for 8.2.2 - Background)

8.3.2.1 Yield mode equations used for double shear wood-to-wood bolted connections are used for double shear bolted connections made with wood main members and metal side plates except equation 8.3-2 for mode $I_y$, which is for uniform bearing in the metal side members, is not applied. This condition is checked independently in accordance with 8.3.2.3 and 7.2.3.

A nominal bolt design value, $Z$, is calculated for each of the three applicable yield mode equations and the lowest value is selected as the design value for the bolted connection. Values of $Z$ and the limiting mode bolt design value for example joint configurations are illustrated in Example C8.3-2.

Where the wood member of a double shear wood-to-metal bolted connection is loaded at an angle to grain, a dowel bearing strength, $F_{cd}$, based on Equation 8.2.7 may be used.

Tabulated bolt design values for lumber and glued laminated timber double shear bolted connections made with 1/4 in. steel side plates given in Tables 8.3C and 8.3D are based on A36 steel having a dowel bearing strength of 58,000 psi, and a bolt bending yield strength of 45,000 psi.

Example C8.3-2

Yield mode design values for wood-to-metal double shear bolted connections:

Hem-fir double shear bolted connections made with metal side plates with 1/2 and 1 inch bolts, steel side member thickness of 1/4 inch, main member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

\[
F_{em} = 4800 \text{ psi parallel} \\
F_{em} = 2550 \text{ psi perpendicular}, 1/2 \text{ inch bolt} \\
F_{em} = 1800 \text{ psi perpendicular}, 1 \text{ inch bolt} \\
F_{ps} = 58,000 \text{ psi} \\
F_{yb} = 45,000 \text{ psi}
\]

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<tr>
<th>Wood Thickness, in.</th>
<th>Load Direction</th>
<th>Bolt Diam. in.</th>
<th>Yield Mode Design Value, lbs</th>
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<td>1</td>
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</tr>
<tr>
<td></td>
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<td>540</td>
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</tr>
<tr>
<td>3</td>
<td>// 1/2</td>
<td>1</td>
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<td>2621</td>
</tr>
<tr>
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<td>// 1/2</td>
<td>1</td>
<td>$Z_{II}$ $Z_{III}$ $Z_{IV}$</td>
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<tr>
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</table>

Comparison of 1991 and Earlier Edition Bolt Design Values. Differences in bolt design values for lumber-to-steel double shear connections between the 1991 and the 1986 editions are illustrated in Table C8.3-2. For the joint configurations compared, the 1991 parallel to grain bolt design values for double shear joints made with steel side plates average 23 percent lower than those based on provisions in the 1986 edition. This difference is a result of the procedure used to convert short-term yield mode bolt design values to the level of previous edition bolt design values wherein a metal side plate adjustment of 1.25 rather than from 1.75 to 1.50 as provided in the 1986 edition (see Commentary for 8.2.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values). Also, for the joint configurations compared, the 1991 perpen-
### Table C8.3-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Double Shear Bolt Design Values

<table>
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<td><strong>Z</strong></td>
<td><strong>Z</strong></td>
<td><strong>Z</strong></td>
<td><strong>Z</strong></td>
<td><strong>Z</strong></td>
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<td><strong>Steel</strong></td>
<td><strong>in.</strong></td>
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<tr>
<td>5-1/2</td>
<td>1/4</td>
<td>3/4</td>
<td>5380</td>
<td>7605</td>
<td>5380</td>
<td>7605</td>
<td>0.84</td>
</tr>
<tr>
<td>3/4</td>
<td>3300</td>
<td>4648</td>
<td>3300</td>
<td>4648</td>
<td>0.71</td>
<td>0.71</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>5750</td>
<td>7605</td>
<td>5750</td>
<td>7605</td>
<td>0.76</td>
<td>0.76</td>
<td></td>
</tr>
</tbody>
</table>

Bolt design values determined from the applicable yield mode equations for several different configurations of steel main member joints are shown in Example C8.3-3.

#### Example C8.3-3

Yield mode design values for steel main member double shear bolted connections:

Hem-fir three member connections made with 1/2 and 1 inch bolts, 1/4 inch steel main member, wood side member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel to the grain of the side members

- $F_{sm} = 58,000 \text{ psi}$
- $F_{sz} = 4800 \text{ psi}$
- $F_{yb} = 45,000 \text{ psi}$

#### 8.4-DESIGN VALUES FOR MULTIPLE SHEAR CONNECTIONS

**Background**

Evaluating bolted connections made with four or more members of equal thickness on the basis of the sum of the allowable bolt design value for each shear plane has been a provision of the Specification since 1944. In the 1977 edition, provisions were added to cover allowable bolt design values for joints of four or more members that were not of equal thickness. Such joints were resolved into the maximum number of contiguous three-member joints and one-half the bolt design value applicable to each such joint was assigned to each shear plane in the connection. For shear planes assigned two different bolt design values, the lowest value was assigned the plane. Where the loads on each member were known, the bolt design value for any member in the joint was the sum of the bolt design values for each shear plane acting on that member. Where members in the joint carried equal
loads or the loads carried by each were unknown, the design value for the bolted connection was taken as the lowest bolt design value for any shear plane times the number of shear planes. These new provisions were carried forward unchanged to the 1982 and 1986 editions.

1991 edition. The procedures of the current edition require evaluation of each individual shear plane using the yield mode equations of 8.2.1 or 8.2.2 and then assigning the connection a bolt design value equal to the lowest value for any single plane times the number of planes in the joint. This methodology, which encourages use of symmetrical member thicknesses, presumes that the connection load is shared in proportion to member thickness and that members are loaded in no more than two different angles to grain. Where more complex connection configurations occur, evaluation of the adequacy of the bolt design value for each individual shear plane may be required (179).

Where a multiple member connection consists of members loaded at three or more different angles to grain, the following procedures may be used to determine allowable bolt design values on individual shear planes:

(1) Determine the loads in each member or pair of members entering the connection;

(2) Number consecutively the "i" members in the connection from outside toward the center as 1, 2, 3...i;

(3) Enter the yield mode equations of 8.2.1 or 8.2.2 with \( F_{e\theta} \) based on the angular difference (\( \theta \)) between the direction of the resultant force in the shear plane and the grain direction of the member, and with \( \theta_{\text{max}} \) equal to the largest \( \theta \) for the two members adjacent to the shear plane being considered. Calculate the applicable \( \theta \) for each member adjacent to each shear plane and the allowable bolt design value for that plane by the steps below:

(i) Determine the allowable bolt design value for the plane between members 1 and 2 based on a load \( (P_i) \) acting in the direction of the stress in member 1, where \( \theta_1 = 0^\circ \) or \( 90^\circ \) and \( \theta_2 \) is the difference between the grain orientations of members 1 and 2;

(ii) Determine the resultant \( (P_{i,2}) \) of the forces in members 1 and 2 and the direction \( (\theta_{i,2}) \) this resultant is acting. Determine the allowable bolt design value for the plane between members 2 and 3 based on a load acting at \( \theta_{2} \), where \( \theta_2 \) is the difference between \( \theta_{i,2} \) and the grain orientation of member 2 and \( \theta_3 \) is the difference between \( \theta_{i,2} \) and the grain orientation of member 3;

(iii) Determine the resultant \( (P_{i,2,3}) \) of the forces in members 1, 2 and 3 and the direction \( (\theta_{i,2,3}) \) this resultant is acting. Determine the allowable bolt design value for the plane between members 3 and 4 based on a load acting at \( \theta_{i,2,3} \), where \( \theta_3 \) is the difference between \( \theta_{i,2,3} \) and the grain orientation of member 3 and \( \theta_4 \) is the difference between \( \theta_{i,2,3} \) and the grain orientation of member 4;

(iv) Continue as in (iii) until the allowable bolt design value in the plane between members 1 and i has been determined;

(v) For symmetrical joints in which members on either side of the center member are oriented in the same direction and carry equal load to the connection, only bolt design values for the shear planes between the outer member and the center member on one side of the connection need to be evaluated.

(3) Determine the adequacy of the connection design by checking the resultant loads in each shear plane \( (P_1, P_{i,2}, P_{i,2,3}, \ldots, P_{i,2,3,i}) \) against the allowable bolt design value determined for that plane.

The foregoing procedure is illustrated in Example C8.4-1, which is adapted from an earlier reference (179).

8.5-Placement of Bolts

8.5.1-Terminology

8.5.1.2 For a joint in which one member is loaded at an angle to the bolt axis (see Specification Figure 8B), end distance requirements are expressed in terms of shear area. 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 bolt (see Figure 8B). 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 bolt 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 bolt in the member.

The use of equivalent shear areas to check end distance requirements in members loaded at an angle to the bolt axis was introduced in the 1986 edition. The
The methodology is used to check end distances for joints loaded parallel to grain in both tension (bolt bearing toward member end) and compression (bolt bearing away from member end) (see Specification and Commentary for 8.5.4.3 and Example C8.5-1).

8.5.2-Geometry Factor, $C_A$

The geometry factor expresses the provisions in the previous edition for proportionate reduction of bolt design value for less than full end distance or less than full spacing distance given in the equation format of the 1991 edition. It should be noted that the lowest geometry factor for any bolt in a joint applies to all other bolts in that same connection, not just to the end bolt or a pair of bolts in a row. This is a continuation of the provision in the 1982 and 1986 editions which required reduction of the full bolt design value on a joint when less than full end or spacing distances were used.

The requirement of 8.5.2 that bolt 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 bolts in the joint presumes that members are loaded in only one or two angles to grain and that total joint capacity is proportional to the number of shear planes. For

**Example C8.4-1**

Procedures to determine allowable design values for a multiple shear bolted connection:

Assume a seven member connection consisting of two compression diagonal web members (members 1 and 7), two main bottom chord members (2 and 6), two tension diagonal web members (3 and 5), and a single vertical web member (4) with total loads and orientations as shown in the member force and configuration diagram of Figure A. The magnitude and direction of the resultant total shear force on each pair of shear planes (1-2 and 6-7, 2-3 and 5-6, and 3-4 and 4-5) are shown in the load vector analysis of Figure B.

Assume all members are Southern pine 1-1/2 inch thick and that a 3/4 inch diameter bolt is being used. The angle the resultant load in each shear plane is acting relative to the grain direction of the members (from Figure B), the dowel bearing strengths applicable to each member for each shear plane (Table 8A and Equation 8.2-7 of the Specification), the allowable bolt design value for each shear plane (Equations 8.2-1 to 8.2-6) and the applied load on each plane (one-half the resultant vector loads from Figure B) are tabulated below.

<table>
<thead>
<tr>
<th>Dowel Bearing Strength</th>
<th>Bolt Design Value, lbs</th>
<th>Load, lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Plane Member</td>
<td>Load to Strength, psi</td>
<td>$\theta_{max}$</td>
</tr>
<tr>
<td>1-2 $\theta_1 = 0^\circ$</td>
<td>6150</td>
<td>39$^\circ$</td>
</tr>
<tr>
<td>$\theta_2 = 39^\circ$</td>
<td>4202</td>
<td></td>
</tr>
<tr>
<td>2-3 $\theta_2 = 29^\circ$</td>
<td>4900</td>
<td>29$^\circ$</td>
</tr>
<tr>
<td>$\theta_3 = 7^\circ$</td>
<td>6052</td>
<td></td>
</tr>
<tr>
<td>3-4 $\theta_3 = 54^\circ$</td>
<td>3597</td>
<td>54$^\circ$</td>
</tr>
<tr>
<td>$\theta_4 = 0^\circ$</td>
<td>6150</td>
<td></td>
</tr>
</tbody>
</table>

Assuming a $C_B$ of 1.15 applies, a 3/4 inch bolt is adequate for all shear planes in the connection.
those connections in which members are loaded in three or more different angles to grain and allowable bolt design values for each shear plane are evaluated (see Commentary for 8.4), the geometry factor for each member adjoining a shear plane may be used to determine the allowable design value on that plane for an individual bolt. Although shear planes may be assigned different geometry factors, all bolts intersecting the same planes are assigned the lowest factor applicable to any bolt in that plane.

8.5.3-Edge Distance

8.5.3.1 Minimum edge distance requirements in Table 8.5.3 for parallel to grain loading of 1.5\(D\) or the greater of 1.5\(D\) or 1/2 the spacing between rows for \(\ell/\ell\) greater than 6, and for loaded edge - perpendicular to grain loading of 4\(D\) are based on early research (183) and have been provisions of the Specification since the 1944 edition. The unloaded edge - perpendicular to grain minimum of 1.5\(D\) was introduced in the 1971 edition as a good practice recommendation.

Section 8.5.3.1 does not provide specific guidance on edge distance requirements for loads applied at angles other than 0° and 90°, and on edge distance requirements for loaded edge - perpendicular to grain under less than full allowable bolt design value. Where these conditions are encountered, the following procedures, based on long standing split ring and shear plate connector design provisions (see Part 10 of the Specification), may be used.

1) For angles of loading between 0° and 45°, minimum edge distances (loaded edge) shall be based on linear interpolation between 1.5\(D\) and 4\(D\), or

\[
ED_{\ell-\ell} = 1.5D + (A)(2.5D/45) \quad \text{(C8.5-1)}
\]

where:

- \(ED_{\ell-\ell}\) = loaded edge - minimum edge distance
- \(A\) = angle of load
- \(D\) = bolt diameter

For angles of loading greater than 45°, a loaded edge - minimum edge distance of 4\(D\) shall apply.

2) For angles of loading greater than 15°, a reduced loaded edge - minimum edge distance not less than 2\(D\) may be used if the bolt design value is reduced proportionately to the reduction in edge distance.

For perpendicular to grain (90°) loading, (2) provides for a loaded edge - minimum edge distance of not less than 2\(D\) when the full bolt design value is reduced 50 percent or more. For a load acting 30° to the grain, the required edge distance for full bolt design value from (1) is 3.2\(D\). This edge distance may be reduced to not less than 2\(D\) if the full bolt design value is reduced 37-1/2 percent or more.

8.5.3.2 The \(\ell/D\) equations for determining minimum edge distance requirements for parallel to grain loading have been added to the Specification to clarify the specific ratio being referenced. It is to be noted that the ratio of the length of bolt in side member material to bolt diameter, \(L_c/D\), 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 \(\ell/D\) ratio for the wood members are considered for determination of edge distance requirements.

8.5.3.3 Avoidance of heavy or medium suspended loads below the neutral axis of a beam was introduced as a good practice recommendation in the 1982 edition. This recommendation was added to the Specification 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.

It should be noted that any bolted connection which transmits a transverse load to a bending member is required to be checked for shear in accordance with 3.4.5 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 bolt. When a connection is within 5\(d\) from the end of the member, the actual shear stress based on \(d_{c}\) is further increased by the ratio \(d/d_{c}\).

8.5.4-End Distance

8.5.4.1 End distance requirements in Table 8.5.4 for full bolt design value parallel to grain are based on early recommendations (183) and have been provisions of the Specification since the 1944 edition. For tension loads (bolts bearing toward member end), the minimum end distances for full bolt design value of 7\(D\) for softwoods and 5\(D\) for hardwoods were established by test. For compression loads (bolt bearing away from member end), the minimum end distance for full bolt design value of 4\(D\) was based on the minimum spacing for full design value for bolts in a row (183). The limit for full bolt design value for perpendicular to grain loading of 4\(D\) was introduced in 1962. Earlier editions depended on the requirements for checking...
shear at connections (3.4.5) to provide for appropriate joint designs for perpendicular loading. Minimum end distances for reduced bolt design values, limited to one-half those for full bolt design values, were introduced in the 1982 edition to provide for design flexibility.

Special note should be made of the requirements of 3.4.5 when bolted connections produce perpendicular to grain loading in bending members.

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

8.5.4.2 The provisions introduced in 1982 for use of reduced end distances for bolted connections when proportionate reductions (geometry factors) are made in design values are supported by early research (57,62,-183) 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 (162). Other recent research further substantiates the adequacy of the end distance requirements for bolted joints loaded in both compression and tension parallel to grain (144,151).

Reduced end distances less than 50 percent of those required for full bolt design value (geometry factors less than 0.50) are not allowed. The reduced end distance provisions for bolted connections are similar to those that have been used with timber connectors since 1944.

8.5.4.3 For members loaded at an angle to the bolt axis, end distance requirements are expressed in terms of equivalent shear areas (see Commentary for 8.5.1.2). As with end distance requirements for parallel member connections, reduced shear areas less than 50 percent of those required for full bolt design value are not allowed. It is recommended as good practice that the distance between the bolt axis and the inside juncture of the angled side member and the main member (see Figure 8B of the Specification) be at least 1.5D. Example C8.5-1 illustrates the use of end and edge distance requirements.

8.5.5-Spacing Between Bolts in a Row

8.5.5.1 The minimum spacing requirement of 4D for bolts in a row loaded at full design value parallel to grain has been a requirement of the Specification since 1944. This requirement is sufficient to cover the effects of nonuniform distribution of shear stresses through the thickness of the member (concentrated at the edges) that occur as the bolt bends (183). The practice of basing spacing requirements for joint members loaded perpendicular to grain on the requirements for the other attached members also has been a provision since 1944.

Use of reduced spacing between bolts in a row in proportion to the ratio of the applied load to the bolt design value for the attached members has been recognized since 1944. The lower spacing limit for reduced design value of 3D for both parallel and perpendicular to grain loading was introduced in 1982.

8.5.5.2 Reduced spacings less than 75 percent of those required for the full design value (geometry factors less than 0.75) are not permitted.

8.5.6-Spacing Between Rows of Bolts

8.5.6.1 Minimum distances between rows of bolts for perpendicular to grain loading in Table 8.5.6 are based on early research (183) and have been provisions of the Specification since 1944. These requirements relate the tendency of the bolt to bend and cause nonuniform bearing stresses and the resistance of the wood between rows to resist splitting. It is for this reason that staggering of bolts loaded perpendicular to grain is desirable (see 8.5.7.2 of Specification).

The minimum spacing between bolt rows for parallel to grain loading of 1.5D was added to the Specification in 1971 as a good practice recommendation. Prior to the revision, this distance was considered sufficiently controlled by net section requirements.

8.5.6.2 In computing the $\ell/D$ ratio for determining the appropriate minimum spacing between rows for perpendicular to grain loading, the ratio for side members is based on the combined thickness ($\ell$) of both side members where three or more wood member joints are involved (see Commentary for 8.5.3.2).

8.5.6.3 In the 1960 edition, a maximum limit of 5 inches between rows of bolts paralleling the member was introduced. The provision was made more restrictive in the 1991 edition by limiting the maximum distance between outer rows of bolts on the same splice plate to 5 inches. Although connections with greater distances between the outer rows of bolts have been used successfully in the past, the new criterion has been added as a good practice recommendation to avoid splitting that could occur in members at connections as a result of restraint of shrinkage associated with drying in service.

The limitation on row spacing applies to metal as well as wood side plates, to members loaded perpendicular...
Example C8.5-1

A No. 3 Hem-Fir 2x4 tension web intersects a No. 2 Hem-Fir 2x4 chord at a 30° angle, connected by a single 1/2-in. bolt as shown. Check edge and end distance requirements in the web and check the adequacy of the web to resist the resultant tension force based on the allowable bolt design value. Assume $C_D = 1.0$. Also assume that the bolt is centered on the members 1.5" dimension.

Check Edge Distance Requirements (8.5.3)

For parallel to grain loading
minimum edge distance: 

$$= 1.5D = 1.5(0.5)$$

$$= 0.75 \text{ in.} = 0.75 \text{ in.} \text{ ok}$$

Check End Distance Requirements (8.5.4)

For the loaded end of the web use an equivalent shear area for a parallel member with $t_m = 3.5$ in.

minimum end distance (tension):

for full design value 

$$ = 7D$$

minimum shear area:

for full design value 

$$ = (7D)(l_s)$$

$$ = (7)(0.5)(3.5) = 12.25 \text{ in}^2$$

for reduced design value 

$$ = (1/2)(12.25) = 6.125 \text{ in}^2$$

actual shear area: 

$$ = (1/2)(x_{end})(l_s)$$

where $x_{end} = \ell_s / \tan \alpha$

$$ = (1/2)(3.5 / \tan 30^\circ)(3.5)$$

$$ = 10.61 \text{ in}^2 < 12.25 \text{ in}^2 \text{ ng}$$

$$ > 6.125 \text{ in}^2 \text{ ok}$$

Since the actual shear area is between the minimums for reduced and full design values, the geometry factor, $C_g$, must be calculated:

$$C_g = \frac{\text{actual shear area}}{\text{minimum shear area for full design}}$$

$$ = 10.61 / 12.25 = 0.866$$

Allowable Bolt Design Value

For a 1/2-in. bolt in single shear with $t_m = t_s = 3.5$ in. and Hem-Fir lumber:

$$Z_{ll} = 660 \text{ lb/bolt}$$

(Table 8.2A)

$$Z' = Z_{ll}C_DC_gC_f = (660)(1.0)(1.0)(0.866)$$

$$ = 572 \text{ lb}$$

(load acts perpendicular to bolt)

Tension in Web Based on Allowable Bolt Design Value

resultant tension force 

$$ = Z' / \cos \alpha$$

$$ = (572) / (\cos 30^\circ) = 660 \text{ lb}$$

Check net section at bolt (critical)

$$A_{net} = (3.5)(1.5 - (1/2 + 1/16)) = 3.28 \text{ in}^2$$

For No. 3 Hem-Fir 2x4:

$$F_t = 300 \text{ psi} \quad C_P = 1.5$$

$$F'_t = F_tC_DC_F = (300)(1.0)(1.5) = 450 \text{ psi}$$

$$f_t = P/A_{net} = 660 / 3.28 = 201 \text{ psi} < F'_t = 450 \text{ psi} \text{ ok}$$

Bolted web connection satisfies NDS edge and end distance and strength criteria.
effects of the resultant eccentric loading on both the load carrying capacity of the members and the capacity of the connection (see Specification and Commentary for 3.1.3).
PART IX: LAG SCREWS

9.1-GENERAL

9.1.1—Quality of Lag Screws

Methodology given in the Specification for the establishment of design values for lag screws prior to the 1957 edition was referenced to lag screws having a yield strength of 45,000 psi and a tensile strength of 77,000 psi. Where lag screws having other strength properties were used, withdrawal design values and lateral design values were to be adjusted in proportion to the ratio of tensile strength and in proportion to the ratio of the square root of the yield strength, respectively. In the 1957 edition, these adjustment provisions, based on early lag screw research (134), were continued but the reference to specific strength metals was replaced with reference to common lag screws.

A new provision was added in the 1977 edition indexing lag screw design provisions and design values to metal conforming to ASTM Standard A307 (see Commentary for 8.1.1). In addition, a requirement was added that the tensile strength of the lag screw at root section was not to be exceeded in withdrawal loads. Use of the ratio of the square roots of the yield strengths to adjust tabulated lateral design values for use of metal of other properties was continued. These 1977 provisions were continued through the 1986 edition.

The 1991 edition references ANSI/ASME Standard B18.2.1-1981 as the quality basis for lag screws in place of ASTM A307. The latter standard is applicable only to bolts and studs and now is specific to steel having a strength of 60,000 psi. Standard B18.2.1 provides standard lag screw dimensions (see Appendix L of the Specification) but does not specify metal of particular strength properties. In the 1991 edition, 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 is a required input variable to the lateral design value yield mode equations of 9.3.1. Additionally, the actual tensile stress in the lag screw at the root diameter must be checked when designing lag screw connections for withdrawal (see 9.2.3 of Specification).

9.1.2—Fabrication and Assembly

9.1.2.1 Provisions relating to the clearance hole of the shank and the lead hole size for the threaded portion of the lag screw have been part of the Specification since the 1944 edition. The lead hole requirements for the 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 (134).

9.1.2.2 The provision for allowing 3/8 inch and smaller diameter lag screws loaded primarily in withdrawal to be inserted in wood of medium to low specific gravity was added to the Specification in the 1982 edition. Consideration of this limited exception to the lead hole requirement was initiated by field inquiries concerning the acceptability of using power driven tools to insert small lag screws to join members in factory built housing construction. On the basis of early lag screw research (134), available information on the withdrawal resistance of tapping screws inserted with different size lead holes (204), and field experience, use of small lag screws without lead holes was judged acceptable when the following conditions were met: the screws were being designed primarily for withdrawal loading, woods classified in fastener Groups III and IV for specific gravity (G) were the foundation members, and placement of screws was such that excessive splitting was avoided. The maximum specific gravity value for those species classified in fastener Groups III and IV in the 1982 and 1986 editions was 0.49.

The exception and related qualifying conditions to the lead hole requirement for 3/8 inch and smaller diameter lag screws introduced in 1982 have been continued in the 1991 edition except that the maximum specific gravity limitation is now expressed in terms of a specific gravity value (0.50) rather than in terms of eligible fastener species groups. The latter are no longer being used in the 1991 edition.

A lag screw subjected to both combined withdrawal and lateral loading may be considered loaded primarily in withdrawal when the axis of the screw is at angle of 75° 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.

9.1.2.3 The provision that lag screws be inserted by turning with a wrench and not by driving with a
hammer has been a good practice requirement of the Specification since the 1944 edition.

9.1.2.4 Use of a lubricant to facilitate lag screw insertion also has been a good practice requirement since 1944. This requirement is not waived when small diameter screws are inserted without the use of lead holes.

9.2-WITHDRAWAL DESIGN VALUES

9.2.1-Withdrawal from Side Grain

The methodology used to establish the lag screw withdrawal design values given in Table 9.2A is the same as that incorporated in the 1944 edition of the Specification. Tabulated design values are computed from the following equation based on the results of early research (62,134):

\[ W = K_W G^{3/2} D^{3/4} \quad (C9.2-1) \]

where:

- \( W \) = withdrawal design value per inch of penetration into main member, lbs
- \( K_W = 1800 \)
- \( G \) = specific gravity based on oven dry weight and volume
- \( D \) = lag screw shank diameter, in.

The value of \( K_W \) represents one-fifth 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 (134), increased by 20 percent; or

\[ K_W = (7500/5)1.2 = 1800 \quad (C9.2-2) \]

The twenty percent adjustment was introduced as part of the World War II emergency increase in wood design values, and then subsequently codified for lag screws as 10 percent for the change from permanent to normal loading and 10 percent for experience (see Commentary for 2.3.2).

The withdrawal design value equation above was included in the early editions of the Specification. Beginning with the 1960 edition, the equation was replaced by a table of lag screw withdrawal design values for the full range of species specific gravity values.

It is to be noted that when the total allowable withdrawal design value on a lag screw is determined by multiplying the tabulated 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 screw was not considered as part of the effective penetration depth in the original joint tests (134). In addition, the thickness of any washer used between the 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 thread length and length of tapered tip, are given in Appendix L of the Specification.

9.2.2-End Grain Factor, \( C_{eg} \)

Tabulated 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). This adjustment, based on lag screw joint tests (134), has been a provision of the Specification since the 1944 edition. Because of the greater possibility of splitting when subject to lateral load, it has long been recommended that insertion of lag screws in end grain surfaces be avoided (62,128).

9.2.3-Tensile Strength of Lag Screw

In the original lag screw joint tests (134), penetration of the threaded portion of the screw into the main member ranging from 7 diameters for the highest density wood and 11 diameters for the lowest density wood tested was found to develop lag screw strengths approximately equal to the tensile strength of the lag screw. However, because tabulated withdrawal design values represent only about 25 percent of average ultimate test loads, the allowable tensile strength of the screw associated with the root diameter will not generally limit the withdrawal design value. This will be the case even when tabulated design values are increased 1.6 for wind or earthquake load. (See also Commentary for 7.2.3 and 9.1.1.)

9.3-LATERAL DESIGN VALUES

9.3.1-Wood-to-Wood Connections

Background

Lag screws perform the same general function as bolts but do not require a nut on the joint face opposite the fastener head. The threaded portion of the screw replaces the nut in providing resistance to withdrawal. Lag screws typically range from 0.19 to 1-1/4 inch in diameter and lengths of less than 1 to more than 12 inches. The threaded portion of the
screw is generally one-half the screw length plus 1/2 inch or 6 inches, whichever is shorter (see Appendix L).

Lag screw lateral design values provided in the 1986 and all earlier editions of the Specification were established using the same procedures. This methodology was developed from the results of early lag screw research (134) which showed that proportional limit design values in lag screw connections loaded parallel to grain were a function of the specific gravity of the members being joined, the Shank diameter of the screw, the ratio of the cleat or side member thickness to the Shank diameter, the location of the Shank/thread boundary relative to the edge of the main or foundation member, and the depth of penetration of the screw in the main member. For perpendicular to grain loading, an additional adjustment was introduced for area of bearing as related to the diameter of the screw. This adjustment for grain direction was adapted from a similar relationship developed for bolted connections (62,117,183). The specific formulas used to establish lag screw values in previous editions are shown below:

Parallel to grain loading:

\[ P_1 = K_L D^2 F_1 F_2 F_3 \]  \hspace{1cm} (C9.3-1)

where:

- \( P_1 \) = design load, lbs
- \( K_L \) = species constant based on specific gravity \((G)\) of wood members
  - 2640, Group I, \( G \leq 0.75 \)
  - 2280, Group II, \( 0.75 < G < 0.55 \)
  - 2040, Group III, \( 0.55 < G < 0.49 \)
  - 1800, Group IV, \( G < 0.49 \)
- \( D \) = Shank diameter, in.
- \( F_1 \) = adjustment for ratio of cleat thickness \((t)\) to Shank diameter (ranging from 0.62 at ratio of 2 to 1.00 at ratio of 3.5 to 1.22 at ratio of 6.5)
- \( F_2 \) = adjustment for location of Shank/thread boundary
- \( SP = S - (t+w) \)

where:

- \( S \) = length of Shank
- \( w \) = washer thickness
  - 1/8 in. for \( D \geq 1/2 \) in.
  - 0 in. for \( D < 1/2 \) in.

For \( SP/D < 0 \):

\[ F_2 = 1.0 - 0.2(t+w-S)/t \]

For \( SP/D \geq 0 \):

\[ F_2 \text{ function of } SP/D \text{ (ranging from 1.08 for} \]

\( ratio \) of 1 to 1.17 for ratio of 2 to 1.26 for ratio of 3 to 1.39 for ratio of 7)

\[ F_3 = \text{adjustment for depth of penetration in main member} \]
\[ = DP/RP \leq 1.0 \]

where:

\[ DP = T - E + SP \]
\[ T = \text{length of thread} \]
\[ E = \text{length of tapered tip} \]
\[ RP = RSP F_1 F_2 \text{ when } F_1 F_2 < 1.0 \]
\[ = RSP(F_1 F_2)^{1/2} \text{ when } F_1 F_2 \geq 1.0 \]
\[ RSP = \text{required standard penetration} \]
\[ = 7D \text{ for Group I} \]
\[ = 8D \text{ for Group II} \]
\[ = 10D \text{ for Group III} \]
\[ = 11D \text{ for Group IV} \]

Perpendicular to grain load:

\[ P_2 = P_1 F_4 \]  \hspace{1cm} (C9.3-2)

where:

- \( P_2 \) = design load perpendicular to grain
- \( P_1 \) = design load parallel to grain
- \( F_4 \) = adjustment for Shank diameter (ranging from 1.00 for 3/16 in. to 0.85 for 5/16 in. to 0.70 for 7/16 in. to 0.50 for 1 in.)

The complexity of the foregoing methodology reflects the number of variables that affect lag screw connection performance. The equations are based on parallel to grain tests of northern white pine, Douglas-fir, southern pine and white oak connections made with dry material (15 percent moisture content) and lag screws having an average tensile strength of 77,000 psi (134). The values of \( K_L \) in the equation for \( P_1 \) represent average proportional limit joint design values divided by 2.25 and then increased 20 percent for normal loading and experience (see Commentary for 9.2.1 related to the latter adjustment).

In the earlier editions, not all species were assigned to fastener groups based on their specific gravity alone. Some softwood species having the same specific gravities as species classified in Group III were classified in Group IV (62). These assignments paralleled early group assignments for wood screws and nails. Beginning with the 1971 edition, classification of species into fastener groups for the purpose of assignment of \( K_L \) factors was based solely on specific gravity. The specific gravity classes for \( K_L \) assignments shown in the

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NDS Commentary
The foregoing procedures were included as part of the Specification until the 1960 edition when they were replaced with a table of design values for a range of lag screw sizes for each of the four lag screw groups. This change in presentation, which was made to facilitate design and use of lag screw connections, was continued through the 1986 edition. In accordance with recommendations based on the original research (134), tabulated design values in the 1960 through the 1986 editions were considered directly applicable to lag screws having a yield strength of 45,000 psi. For lag screws having other yield strengths, adjustment of tabulated values in proportion to the square root of the ratio of the yield strength of the metal to 45,000 was allowed.

1991 Edition. As with bolts and other dowel type fasteners, lateral design values for lag screw connections in the 1991 edition are based on application of the yield limit model (see Commentary for 7.2.1 and 8.2.1). In the case of lag screws, three general modes of yielding can occur: bearing in the side member or cleat (Mode I), development of a plastic hinge in the screw in the main member (Mode III) and development of plastic hinges in the screw in both main and side members (Mode IV). However, three possible conditions may exist within both Modes III and IV depending upon whether the maximum bending occurs in the shank or threaded portion of the screw or bending occurs at a location other than at the point of maximum moment (114,117).

Behavioral equations for each of the seven possible conditions were developed (114,117) and used to predict the joint design values of all configurations tested in the original research (134). Values of 5 percent diameter offset dowel bearing strength \( F_{os} \) required for these equations were estimated from the specific gravity values of the test material using equations approximately equivalent to those used for bolts (see Commentary for 8.2.1 - Yield Mode Equations). Yield mode design values for the test configurations were compared to those obtained from the original methodology (equations C9.3-1 and C9.3-2) to verify there was a reasonably stable relationship between the two for the range of joint configurations available (114).

Reexamination of the original lag screw research (134) showed that although the \( F_s \) factor (the adjustment for length of lag screw in the main member) in the original lag screw equations was related to ultimate load or strength of the joint, the proportional limit load (approximately one-fourth the ultimate) was unaffected as long as the total length of penetration (shank as well as threaded depth) of the screw in the main member was at least five times the shank diameter, or \( 5D \) (114,117). In view of the fact that previous design values were keyed to proportional limit rather than ultimate joint loads, the position was taken that the yield mode equations would be implemented in the 1991 edition and compared with previously developed design values assuming that the full design value is developed if the length of the screw (shank plus threaded portion less length of tapered tip) in the main member is at least \( 8D \), regardless of species specific gravity; and that proportionate design values are achieved for penetrations between a minimum of \( 4D \) and \( 8D \). These criteria are in agreement with the penetration requirement in the 1988 draft standard of Eurocode No. 5 (45,114).

To establish adjustment factors which would reduce yield mode equation values to the design value levels used in previous editions of the Specification, the ratios of yield mode design value to design value based on previous methodology (modified by the new penetration criteria) were determined for a wide range of lag screw joint configurations made with both wood and steel side members. An average ratio was developed for each of the seven yield mode conditions, one for Mode I\(_s\) and three each for Modes III and IV. Rather than include three equations for each of the latter two modes in the Specification, it was assumed that Mode III consisted only of yielding of the threaded portion of the screw in the main member and that Mode IV consisted only of yielding of the shank portion of the screw in the side member and of the threaded portion of the screw in the main member (117). It was found that this simplification could be accomplished, while obtaining approximately the same overall average ratio of yield mode design value to previous design value as that resulting from use of three equations for each Mode, by assuming a constant ratio of yield moment of the threaded portion to yield moment of the shank of 0.75 (114,117). The yield moment ratio \( R_m \) for the screw appears in the selected behavioral equations for Modes III\(_s\) and IV (114,117) as shown below.

Mode I

\[
P = D t_s F_{es}
\]

(C9.3-3)

Mode III\(_s\)

\[
P = \frac{k D t_s F_{em}}{(2 + R_e)}
\]

(C9.3-4)
where:

- \( R_e \) = \( \frac{F_{em}}{F_{es}} \)
- \( t_s \) = thickness of side member
- \( D \) = unthreaded shank diameter of lag screw
- \( F_{es} \) = dowel bearing strength of main member
- \( F_{em} \) = dowel bearing strength of side member
- \( R_m \) = ratio of yield moment of threaded portion to yield moment of shank

**Yield Mode Design Equations**

Substituting 0.75 for \( R_m \) in the foregoing behavioral equations for Modes II, IIIa, and IV and adding the conversion factors relating yield mode design values to previously published design values gives the lag screw parallel to grain lateral design value \( \bar{Z} \) equations given in 9.3.2 of the Specification. The factors \( 4K_\theta \), \( 2.8K_\theta \), and \( 3K_\theta \) in the denominators of Equations 9.2-1, 9.2-2, and 9.2-3, respectively, are the adjustments that convert yield mode design values based on 5 percent diameter offset dowel bearing strength to the level of proportional limit based design values tabulated in the 1986 and earlier editions of the specification. For parallel to grain loading, \( K_\theta \) equals one. For perpendicular to grain loading, \( K_\theta \) equals 1.25 for a connection with one member loaded parallel to grain and one member loaded perpendicular to grain.

Dowel bearing strength values \( F_e \) used in the lag screw yield mode equations are given in Table 9A for all structurally graded lumber species. The values also apply to glued laminated timber. The values in Table 9A have been established using the same dowel bearing strength equations used for bolts (see Commentary for 8.2.1 - Yield Mode Equations). For perpendicular to grain loading, the dowel strength equation is entered with the shank diameter of the lag screw.

A bending yield strength, \( F_{yb} \), of 45,000 psi may be used for common lag screws having a shank diameter of 3/8 inch or larger. For smaller diameter screws, use of higher yield strengths may be appropriate (see Appendix I and Tables 9.3A and 9.3B). The yield mode equations are applicable only to connections in which the total penetration of the lag screw (shank and threaded portion) in the main member is at least four times the shank diameter \( (4D) \), excluding the length of the tapered tip, and the minimum thread length is that specified for the diameter in Appendix L. This Appendix provides standard shank and root diameters and total, thread and tapered-tip lengths for different screw sizes. Tapered-tip lengths are calculated as 0.866 the root diameter.

It is to noted that when the length of the threaded portion of the lag screw is greater than the standard threaded portion length given in Appendix L of the Specification, the root diameter rather than the shank diameter should be used as \( D \) in the yield mode equations if the boundary between the shank and threaded portion of the screw falls within the cleat or side member.

For each joint configuration, the nominal design value, \( Z \), for each yield mode is calculated to determine the limiting value for the connection. In most cases, Mode IIIa or IV will be the limiting case. For a member loaded at an angle to grain, the lag screw yield mode equations are entered with a dowel bearing strength, \( F_{es} \), calculated in accordance with Equation 9.3-4, a form of the standard bearing angle to grain formula (see Appendix J). Design values for lag screws acting at an angle to grain have been based on this equation using allowable lag screw parallel and perpendicular to grain design values as the reference design value levels since the 1944 edition (see Commentary for 8.2.1 - Member Loaded at an Angle to Grain).

**Tabulated Wood-to-Wood Design Values.** Lag screw design values given in Table 9.3A assume the threaded portion of the screw is located completely in the main member, the total penetration of the screw in the main member is a minimum of \( 8D \) (excluding the length of the tapered tip), and screws of standard diameters, threaded lengths and other dimensions (see Appendix L) are used. Values also are based on lag screw bending yield strength values of 45,000 psi for screws of 3/8 inch and larger diameter, 60,000 psi for 5/16 inch diameter and 70,000 psi for 1/4 inch diameter. The values assumed for the screw sizes less than 3/8 inch diameter are those estimated from tests of common wire nails of the same diameter (see Appendix I). It is the responsibility of the designer to assure that the lag screws specified and used qualify for tabulated design values.

Two lateral design values for perpendicular to grain loading are shown in Table 9.3A: one for lag screw connections with the side member loaded perpendicular to grain and the main member loaded parallel to grain.
(Z_{m\perp})}; and one for a connection with main member loaded perpendicular to grain and the side member loaded parallel to grain (Z_{m\parallel}).

**Comparison of 1991 and Earlier Edition Values.** Lag screw design values given in the 1991 edition cover a greater range of side member geometries than those given in earlier editions. Because of the simplifications made in the yield mode equations and the averaging procedures used to adjust yield mode lag screw design values to design values previously tabulated, new design values are both higher and lower than those given for the same joint configurations in the earlier editions. This is shown by the comparisons in Table C9.3-1. The penetration depth factor, \(C_d\), shown in this table, applicable to the 1991 tabulated values, is the adjustment for less than full design value penetration (8D) of the screw in the main member (see 9.3.3 of Specification and foregoing Commentary). Washer thickness of 1/8 inch for 1/2 inch and larger diameters screws and 0 inch for smaller diameters screws were assumed in calculating values of \(C_d\). Adjustments for penetration depth are embedded in the values tabulated in previous editions (see Equations C9.3-1 and C9.3-2).

In terms of the design value ratios for the joint configurations compared, lag screw design values for wood-to-wood connections based on 1991 edition provisions average 22 percent and 25 percent higher for parallel to grain loading and perpendicular to grain loading, respectively, than those tabulated in the 1986 edition. The overall higher design values for parallel to grain loading (Z_{m\perp}) are a result of the procedure used to translate yield limit design values to the level of proportional limit based design values tabulated in previous editions and to the equations used to establish dowel bearing strength values. This procedure included considering ratios of yield limit and previous design values for both joints with wood and with steel side members when establishing a uniform conversion factor for each yield mode equation (see Commentary for 9.3.1 - Yield Mode Equations). In the case of perpendicular to grain loading (Z_{m\parallel}), the overall higher level of design values in the 1991 edition reflects the effect of the equation used to establish the dowel bearing strength values for perpendicular to grain loading in Table 9A. It is to be noted that lag screw perpendicular to grain design values tabulated in earlier editions of the Specification were based on application of procedures originally developed for bolts rather than on tests of lag screw joints under perpendicular to grain loading. The yield mode equations provide a fully rationalized basis for evaluating the interactions of member bearing strength, member thicknesses, fastener diameter and fastener strength.

For joints made with two different species, tabulated design values for the species with the lower dowel bearing strength may be used in lieu of using the yield mode equations of 9.3.1 with the appropriate dowel bearing strength for each species.

**9.3.2-Wood-to-Metal Connections**

**9.3.2.1** In previous editions, design values for lag screw joints made with metal side plates were established as 1.25 times the value for a wood-to-wood joint of equivalent configuration as determined from Equation C9.3-1 or C9.3-2 (134). Metal side plates were assumed to be 1/2 inch thick and resultant values were considered applicable to side plates of lesser thickness.
For thicker side plates, tabulated design values were required to be reduced for the lesser penetration of the lag screw.

Under the methodology of the 1991 edition, lag screw design values for joints made with metal side plates are determined using the Mode III and Mode IV equations for wood-to-wood joints in 9.3.1 using a dowel bearing strength, $F_{ds}$, applicable to the metal used in the side plates (see Appendix I) and the actual thickness of the plate as the side member thickness. As previously noted (see Commentary for 9.3.1 - Background), the factors used to relate yield mode design values to proportional limit based design values tabulated in previous editions were based on lag screw joints made with both wood and metal side plates. Thus Equations 9.3-2 and 9.3-3 apply to joints with metal as well as wood side plate when the appropriate input variables are used.

The lag screw design values for joints made with steel side plates given in Table 9.3B assume the same strength lag screws as those used for Table 9.3A, a dowel bearing strength of 58,000 psi for 1/4 inch steel (ASTM A36) side plates, and a dowel bearing strength of 45,000 psi for steel plates less than 1/4 inch. The latter value is the tensile strength for ASTM A446, Grade A galvanized sheet steel. It is to be noted from Table 9.3B that, for constant screw diameter and steel strength, design values based on the yield mode equations decrease as steel plate thickness decreases. Previous methodology recognized no effect of plate thickness below 1/2 inch.

**Comparison of 1991 and Earlier Edition Values.** Differences in 1991 and earlier edition design values for wood-to-metal lag screw joints are illustrated in Table C9.3-2. The 1991 design values are for joints made with 1/4 inch side plates. The 1986 values apply to joints made with 1/2 inch or thinner side plates. The values of $C_d$ less than 1.000 in Table C9.3-2 are the adjustments applied to 1991 tabulated design values when there is less than full design value penetration ($8D$) of the screw in the main member (see 9.3.3 of Specification and foregoing Commentary). Joints were assumed made without washers.

The average design value ratios for the configurations compared show 1991 design values for metal-to-wood connections were 2 percent lower and 16 percent higher for the parallel and perpendicular to grain loading cases, respectively, than the comparable 1986 design values. The higher overall design values for the perpendicular to grain comparisons reflect the effects of the equation used to establish dowel bearing strengths for perpendicular to grain loads in Table 9A (see

### Table C9.3-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Single Shear Lag Screw Lateral Design Values

<table>
<thead>
<tr>
<th>Steel Side Member Thickness in.</th>
<th>Penetration Depth Factor $C_d$</th>
<th>Lag Screw Lateral Design Value, lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$L$</td>
<td>$D$</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Southern Pines:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>4</td>
<td>1/4</td>
</tr>
<tr>
<td>3/8</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>1/2</td>
<td>0.859</td>
<td>697</td>
</tr>
<tr>
<td>6/8</td>
<td>1.000</td>
<td>510</td>
</tr>
<tr>
<td>1/2</td>
<td>1.000</td>
<td>810</td>
</tr>
<tr>
<td>3/4</td>
<td>0.875</td>
<td>1452</td>
</tr>
<tr>
<td>8/1</td>
<td>1.000</td>
<td>810</td>
</tr>
<tr>
<td>3/4</td>
<td>1.000</td>
<td>1660</td>
</tr>
<tr>
<td>7/8</td>
<td>1.000</td>
<td>2220</td>
</tr>
<tr>
<td>10/5</td>
<td>1.000</td>
<td>1190</td>
</tr>
<tr>
<td>3/4</td>
<td>1.000</td>
<td>1660</td>
</tr>
<tr>
<td>1</td>
<td>1.000</td>
<td>2870</td>
</tr>
<tr>
<td>Spruce-Pine-Fir:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>4</td>
<td>1/4</td>
</tr>
<tr>
<td>3/8</td>
<td>1.000</td>
<td>450</td>
</tr>
<tr>
<td>1/2</td>
<td>0.859</td>
<td>619</td>
</tr>
<tr>
<td>6/8</td>
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<td>450</td>
</tr>
<tr>
<td>1/2</td>
<td>1.000</td>
<td>720</td>
</tr>
<tr>
<td>3/4</td>
<td>0.875</td>
<td>1295</td>
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<tr>
<td>8/1</td>
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<td>720</td>
</tr>
<tr>
<td>3/4</td>
<td>1.000</td>
<td>1480</td>
</tr>
<tr>
<td>7/8</td>
<td>1.000</td>
<td>1980</td>
</tr>
<tr>
<td>10/5</td>
<td>1.000</td>
<td>1060</td>
</tr>
<tr>
<td>3/4</td>
<td>1.000</td>
<td>1480</td>
</tr>
<tr>
<td>1</td>
<td>1.000</td>
<td>2550</td>
</tr>
</tbody>
</table>

Commentary for 9.3.1 - Yield Mode Equations, Comparison.

9.3.2.2 (See Commentary for 7.2.3)

9.3.3-Penetration Depth Factor, $C_d$

The penetration depth factor provides for reduced design values when the length of penetration of the lag screw in the main member, both shank and threaded portion, is less than eight times the shank diameter ($8D$). The full proportional limit design value of lag screw joints of any species is considered developed when this penetration depth occurs (see Commentary for 9.3.1 - Background, 1991 Edition). Penetrations down to 50 percent or $4D$ of the full design value
penetration depth are allowed if tabulated design values or yield mode equation design values are reduced proportionately.

The penetration depth of the lag screw in the main member is calculated as the length of the lag screw \( (P) \) minus the sum of the thickness of the side member \( (t - w + E) \),

\[
P - (t - w + E)
\]

Lag screw dimensions, including thread and tapered tip lengths, are given in Appendix L.

Consistent with the basis of lag screw design values published in previous editions, a washer thickness of 1/8 inch for screw diameters of 1/2 inch and larger and 0 inch for smaller diameter screws may be assumed.

Examples of the use of the \( C_d \) factor to adjust design values for less than full penetration are shown in Tables C9.3-1 and C9.3-2 and related Commentary.

9.3.4-End Grain Factor, \( C_g \)

Use of two-thirds the perpendicular to grain lag screw lateral design value for screws inserted in the end grain of the main member (62) has been a provision of the Specification since the 1944 edition. When design values for this case are based on the yield mode equations of 9.3.1, the dowel bearing strength of the main member is assumed equal to the perpendicular to grain bearing strength of the species from Table 9A.

Because of the tendency of the member to split under lateral loading, structural lag screw connections in end grain surfaces should be avoided (62, 66). Lag screws in end grain surfaces particularly should not be subjected to combined withdrawal and lateral loading.

9.3.5-Combined Lateral and Withdrawal Loads

In the 1977 through the 1986 editions, the Specification provided that lag screws subjected to combined lateral and withdrawal loads be analyzed separately for the resistance of the screw to each load. The results of recent lag screw tests (115) showed that withdrawal load components did not reduce lateral load capacity when maximum joint loads are considered. However, when joint resistance was evaluated at the design load level by expressing the strength of the joint loaded at any angle to the surface as the lesser of the proportional limit load divided by 1.875 (2.25/1.2) or the maximum load divided by 4.167 (5/1.2), an interaction of the load components was observed with larger diameter screws at load angles less than 45° (115). Use of these alternative design load bases is required because lag screw lateral design values are based on proportional limit joint loads whereas withdrawal design values are based on maximum joint loads. The factors of 1.875 and 4.167 are the adjustments used to convert the two types of test values to allowable levels (see Commentary for 9.2.1 and 9.3.1 - Background).

To account for the interaction observed in the new lag screw tests, Equation 9.3-6 has been introduced in the 1991 edition for determining the allowable design value of lag screws subject to combined lateral and withdrawal loads. This equation, a form of the bearing angle to grain equation (see Appendix J), is

\[
Z_a' = \frac{(W' p) Z'}{(W' p) \cos^2 a + Z' \sin^2 a}
\]

where:

- \( Z_a' \) = allowable design value for lag screw loaded at angle to the surface of main member
- \( Z' \) = lateral design value for lag screw connection
- \( W' \) = withdrawal design value for lag screw connection per inch of penetration
- \( p \) = length of thread penetration in the main member
- \( a \) = angle between wood surface and direction of applied load

The length of penetration of the threaded portion of the screw in the main member excludes the length of the tapered tip and includes the reduction in penetration resulting from the use of a washer under the screw head.

Equation C9.3-6 will give generally conservative design values for load angles greater than 45°. Equation C9.3-6 can also be used to determine the allowable design value of lag screws embedded at an angle to grain in the wood member and loaded in a direction normal to the wood member. For this condition \( a \) would be defined as the angle between the wood surface and the lag screw as shown in Figure C9.3-1.
9.4-PLACEMENT OF LAG SCREWS

9.4.1-Geometry Factor, Edge Distance, End Distance, Spacing

Application of the same positioning requirements for lag screws as those for bolts has been a provision of the Specification since the 1944 edition. The similarity of the performance characteristics of the two types of fasteners was recognized in the original lag screw research (134) and the use of edge distance, end distance and spacing criteria for bolts with lag screws was specifically recommended (62). Lag screw tests conducted on Douglas-fir joints in 1963 confirmed that a $4D$ spacing between lag screws in a row was more than sufficient to develop the full proportional limit load capacity of the joint (102).

9.4.2-Multiple Lag Screws

The group action factor, $C_g$, has been applied to lag screw joints containing two or more screws in a row (screws aligned in the direction of the load) since the factor was first introduced in 1973.
PART X: SPLIT RING AND SHEAR PLATE CONNECTORS

10.1-GENERAL

Background

Split ring and shear plate connectors act like dowels or keys in distributing loads from one member to another in a joint (57). 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, the most efficient of timber fastenings, 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 (179).

The design provisions for split ring and shear plate connectors in the Specification are based on early research (146,159) and have remained essentially unchanged since the 1944 edition. In addition to these connectors, the first edition included allowable design values for claw plates and toothed rings (128). Claw plates, which are similar to shear plates, were dropped in the 1948 edition. Toothed rings, alternates to split rings but made of thinner metal, requiring no grooving and having about one-half the allowable design values, were dropped in the 1973 edition.

Also since the first edition, connector design values were reduced approximately 9 percent in 1962 to increase the level of conservatism in the allowable design values for this type of fastening. The adjustment had the effect of offsetting that portion of the World War II emergency increase of 20 percent autorized for all wood design values that was not subsequently covered by the conversion of all wood design values from a permanent to a normal loading basis (see Commentary for 2.3.2). New in the 1991 edition is the inclusion of a number of additional species and broad species groups in connector Group D (Table 10A of the Specification). The broad species groups (Western Woods, Eastern Species and Northern Species) may include any species produced in the applicable region (see Design Value Supplement).

10.1.1-Terminology

A connector unit is expressed 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 Tables 10.2A and 10.2B).

10.1.2-Quality of Split Ring and Shear Plate Connectors

10.1.2.1 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 (159,179). The position of the tongue-slot joint in the ring relative to the direction of loading is not significant (159).

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, which have been part of the pressed shear plate description since the 1944 edition, do not affect plate load-carrying performance.

10.1.2.2 Dimensions for split rings and pressed steel and malleable iron shear plates have been included in the Specification since the 1944 edition. Dimensions for light gage shear plates were introduced in 1960.

In addition to connector diameter, the depth of the connector in the member and its thickness affect joint load-carrying capacity. It is to be understood that only those split rings that have equivalent or larger inside diameter, metal depth and metal thickness than those given in Appendix K of the Specification qualify for the connector design values provided in Table 10.2A. Similarly, only those shear plates that have equivalent or larger plate diameter, plate depth and plate thickness than those given in Appendix K qualify for the connector design values provided in Table 10.2B.

The projected areas given in Appendix K for split rings are calculated as the sum of the inside groove diameter and twice the groove width times the groove depth. These projected areas are 1.10 and 2.25 square inches for 2-1/2 and 4 inch rings, respectively.

The projected areas for shear plates given in Appendix K are based on the groove diameter times the groove depth for the nominal shear plate dimensions shown. These groove diameters are 2.63 inches and 4.03 inches for 2-5/8 and 4 inch plates respectively. The groove depths assumed to correspond to the plate tabulated flange depths are 0.45, 0.38 and 0.64 inches for 2-5/8 inch pressed steel, 2-5/8 inch light gage and 4 inch malleable iron plates, respectively; giving the tabulated projected area values for these connectors of 1.18, 1.00 and 2.58 square inches. Prior to the 1960 edition, slightly smaller projected areas for the pressed steel and malleable iron plates, based on the actual cross-sectional dimensions of the material cut from the member to accommodate the flanged plate and integral hub and enclosed central bolt hole, were used.

Tabulated projected areas 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 inches 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 tabulated values.

When connectors are installed in unseasoned or partially seasoned wood intended for use in dry conditions of service, tabulated design values are to be adjusted in accordance with the factors in Table 7.3.3. 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 (62,159). 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 Section 3.1.2.3 and Appendix A.12 of the Specification).
10.2-DESIGN VALUES

10.2.1-Tabulated Nominal Design Values

Background

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 (57,62,146,159). This species effect was accounted for by classifying species into four connector load groups based on their specific gravity. These four groupings, originally established in the 1944 edition, are shown in Table 10A of the Specification. The present listing includes species and species groups that have been added in various subsequent editions since 1944. The 1991 edition includes the addition of several new individual species and three broad regional species groups in Group D, as well as the deletion or reclassification of other species or groups based on changes in commercial importance or specific gravity values. The broad species groups (Western Woods, Eastern Species and Northern Species) may include any species produced in the applicable region (see Design Value Supplement).

The specific gravity ranges of the species in the connector groups of Table 10A are

Group A: 0.67 to 0.73
Group B: 0.49 to 0.58
Group C: 0.42 to 0.47
Group D: 0.31 to 0.41

Specific gravity values for the individual species listed in Table 10A are given in Table 8A or the other dowel bearing strength tables in the Specification. Intermediate specific gravity values for species not listed may conservatively be placed in a lower group.

Connector design values given in the 1944 edition of the Specification included a 20 percent increase authorized for all wood design values as part of the national war emergency program of World War II. Subsequently, one-half of this emergency increase was codified as part of the conversion of wood design values from a permanent to a normal loading basis (see Commentary for 2.3.2). Based on field experience, the remaining one-half of the war time increase was retained for all wood connection design values, including those for split ring and shear plate connections. In the 1962 edition, in response to changing construction and workmanship practices, design values for joints made with split rings and shear plates were reduced approximately 9 percent. This adjustment had the effect of removing the experience portion of the WW II emergency increase. The 1962 design values have been carried forward unchanged to the 1991 edition.

Design values in Table 10.2A and 10.2B represent maximum joint test loads reduced by a factor of 3.6 that includes adjustments for variability and load duration (57,62,159). These design values, applicable to normal loading conditions, are considered to be less than 70 percent of proportional limit test loads (62,159). Tabulated design values apply only to those joint designs which meet the thickness, end distance, edge distance and spacing requirements for full design value given in Tables 10.2A, 10.2B and 10.3. Net thickness requirements refer to the actual thickness of the member before grooving.

10.2.1.1 Design values for split ring connections in Table 10.2A and for shear plate connections in Table 10.2B 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 design value for the two members being joined is the design value for the shear plane. Example C10.2-1 illustrates this provision.

Example C10.2-1

Determine the tabulated allowable design value for a single 2-1/2 inch split ring connector in each shear plane of a three member joint made of 2 1 inch thick Group B side members and a 1-1/2 inch thick Group B main member.

Both side and main members loaded parallel to grain:

Side members: Design value from Table 10.2A, row 1, column 6 for connector on one face and 1" thickness = 2270
Main member: Design value from Table 10.2A, row 3, column 6 for connector on two faces and 1-1/2" thickness = 2100

Limiting design value, each shear plane = 2100 lbs

Side members loaded perpendicular to grain and main member loaded parallel to grain:

Side members: Design value from Table 10.2A, row 1, column 10 = 1620
Main member: Design value from Table 10.2A, row 3, column 6 = 2100

Limiting design value, each shear plane = 1620 lbs
10.2.1.2 The limiting design values given in footnote 2 of Table 10.2B are the same as those given in the 1986 edition. The 2900 pound value for the 2-5/8 inch shear plate is the maximum allowable bearing load for a pressed steel plate without a reenforcing hub about the bolt hole. The 4400 and 6000 pound values for the 4 inch plates used with 3/4 and 7/8 inch bolts respectively are the maximum allowable shear design values for A307 bolts of these diameters. The 4 inch plates have integral reinforcing hubs about the central bolt hole. Somewhat higher maximum design values were permitted for 4 inch plates in the 1982 and earlier editions based on higher allowable bolt shear strength.

Because the limiting design values specified in footnote 2 of Table 10.2B are based on metal strength, they are not to be increased by load duration or other adjustment factors given in 7.3.2 for values controlled by wood members.

10.2.2-Thickness of Wood Members

10.2.2.1 The minimum member thicknesses required for use of the split ring and shear plate connector values in Tables 10.2A and 10.2B have been established from the results of joint tests (159).

10.2.2.2 The provision for use of linear interpolation between minimum thicknesses and those required from maximum design values is based on the original connector research (159).

10.2.3-Penetration Depth Factor, Cd

10.2.2.3 The lag screw penetration depth requirements and adjustments for penetrations less than required for full design value given in Table 10.2.3 have been provisions of the Specification since the 1960 edition. Editions prior to 1960 did not have separate penetration requirements for different connector groups, expressed penetration requirements in terms of the anchorage provided by the threaded portion of the screw only, assumed a nonlinear relationship between reduced design value and penetration depth, and did not provide for use of full design value with 2-5/8 inch shear plates at minimum specified penetration when metal side plates were used.

10.2.4-Metal Side Plate Factor, Csp

Increases for metal side plates used with 4 inch shear plate connectors given in Table 10.2.4 have been recognized in the Specification since the 1944 edition. The adjustments are based on original connector research involving claw plates (159). Both claw plate and shear plate connectors fit into prebored recesses in wood members and depend upon the bolt to transmit shear between the members being joined. The claw plate differs from the shear plate in having short teeth extending from one side of the plate that are forced into the wood as the bolt nut is tightened. Special note is to be made that the increased values for 4 inch shear plates loaded parallel to grain are permitted only when the maximum metal part design values of footnote 2 of Table 10.2B are not exceeded.

10.2.5-Load at Angle to Grain

Use of the standard bearing angle to grain equation (Equation 10.2-1 and Appendix J) to determine allowable 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° has been a provision in the Specification since the 1944 edition. The original research showed that maximum design values on claw plate connectors loaded at different angles to grain varied in accordance with the standard angle to grain equation (159). The tests of split ring connectors in this same study showed the relationship between maximum design value and grain angle could be described by a linear relationship without appreciable error. For purposes of conservatism and consistency with the provisions for other fasteners, the standard angle to grain equation is used in the Specification to adjust both split ring and shear plate connector design values for grain angle.

10.2.6-Split Ring and Shear Plate Connectors in End Grain

Prior to the 1977 edition, the Specification contained no provisions for the design of connectors in end grain surfaces. Such provisions were introduced in 1977 to cover joint configurations frequently encountered in practice, such as those at the peak of A-frames or similar arches.

Design values for split ring and shear plate connectors in end grain surfaces are keyed to use of a design value for connectors in square-cut end surfaces equal to sixty percent of the tabulated design value for connectors in side grain surfaces loaded perpendicular to grain, or

\[ Q'_{90} = 0.60 Q' \]  
(C10.2-1)

where:

\[ Q'_{90} = \text{allowable design value for a split ring or shear plate connector unit in a square-cut end surface, loaded in any direction in the plane of the surface } (\alpha=90^\circ, 0^\circ \leq \varphi \leq 90^\circ) \text{ calculated from (C10.2-1)} \]
NDS Commentary

\[ Q' = \text{allowable design value for a split ring or shear plate connector unit in a side grain surface, loaded perpendicular to grain (} \alpha=0^\circ, \phi=90^\circ). \]

The \( Q'_{90} \) condition is illustrated in Figure 10F of the Specification. The use of 0.60 \( Q' \) as the allowable design value for a square-cut end surface was originally based on experience with connector design with glued laminated timber prior to 1977 (4). Available data from a comprehensive study of the capacity of shear plates in sloping grain end surfaces in Douglas fir (107) 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 (41,101). The end grain connector design provisions in the 1977 edition, which have continued to provide connections of satisfactory field performance, have been carried forward unchanged to the 1991 edition.

Use of the standard bearing angle to grain equation (Appendix J) to establish allowable design values for connectors in sloping end-grain surfaces also has been a basic provision of the Specification since the 1977 edition. When the load on the sloping surface is acting parallel to the axis of cut of the surface (\( \phi=0^\circ \)), as illustrated in Figure 10G of the Specification, the standard angle to grain equation is entered with the tabulated allowable connector design value for side grain parallel to grain loading and the calculated allowable design value for a square-cut end surface. Thus, for the case of \( \phi=0^\circ \), the equation for any slope of surface cut, \( \alpha \), is

\[ P'_{a} = \frac{P' \cdot Q'_{90}}{P' \cdot \sin^2 \alpha + Q'_{90} \cdot \cos^2 \alpha} \]  

(C10.2-2)

where:

\[ P'_{a} = \text{allowable design value for a split ring or shear plate connector unit in a sloping surface, loaded in a direction parallel to the axis of cut (} 0^\circ < \alpha < 90^\circ, \phi = 0^\circ \) \]

\[ P' = \text{allowable design value for a split ring or shear plate connector unit in a side grain surface, loaded parallel to grain (} \alpha=0^\circ, \phi=0^\circ \) \]

\[ Q'_{90} = \text{allowable design value for a split ring or shear plate connector unit in a square-cut end surface, loaded in any direction in the plane of the surface (} \alpha=90^\circ, 0^\circ \leq \phi \leq 90^\circ \) \]

\[ \alpha = \text{the least angle formed between a sloping surface and the general direction of the wood fibers, from } 0^\circ \text{ to } 90^\circ \]

When the load on the sloping surface is acting perpendicular to the axis of cut of the surface (\( \phi=90^\circ \)), as illustrated in Figure 10H of the Specification, the value of \( P' \) in equation C10.2-2 is replaced with the tabulated allowable connector design value for side grain perpendicular loading. For this case of \( \phi=90^\circ \), the equation for any slope of surface cut, \( \alpha \), is

\[ Q'_{a} = \frac{Q' \cdot Q'_{90}}{Q' \cdot \sin^2 \alpha + Q'_{90} \cdot \cos^2 \alpha} \]  

(C10.2-3)

where:

\[ Q'_{a} = \text{allowable design value for a split ring or shear plate connector unit in a sloping surface, loaded in a direction perpendicular to the axis of cut (} 0^\circ < \alpha < 90^\circ, \phi = 90^\circ \) \]

\[ Q' = \text{allowable design value for a split ring or shear plate connector unit in a side grain surface, loaded perpendicular to grain (} \alpha=0^\circ, \phi=90^\circ \) \]

When the load on the sloping surface is acting at an angle between \( 0^\circ \) and \( 90^\circ \) to the axis of cut of the surface (\( 0^\circ \leq \phi \leq 90^\circ \)), as illustrated in Figure 10I, the values of \( P' \) and \( Q'_{90} \) in the equation C10.2-2 are replaced with \( P'_{a} \) and \( Q'_{a} \), respectively, or

\[ N'_{a} = \frac{P'_{a} \cdot Q'_{a}}{P'_{a} \cdot \sin^2 \phi + Q'_{a} \cdot \cos^2 \phi} \]  

(C10.2-4)

where:

\[ N'_{a} = \text{allowable design value for split ring or shear plate connector unit in a sloping surface, when direction of load is at an angle } \phi \text{ from the axis of cut.} \]

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 (see Example C10.2-2). Where the end grain surface is square cut, the thickness of the member may be taken as the length of the lag screw in the member. Example C10.2-2 illustrates the use of provisions for shear plates.
Example C10.2-2

Determine the allowable vertical force with $C_D$ of 1.15 on a 2-5/8 inch shear plate connector unit joining two Douglas fir-larch 4x10 rafters at the ridge where the roof slope is 12 in 8 producing an angle of 33.7° from the vertical at the peak. Use a 3/4-inch bolt with 2 inch diameter wrought iron washers recessed into each rafter (see Figure C10.2-1).

The angle between the direction of load and the axis of cut of the end grain surface, $\varphi$, is 0°. The slope of the surface cut, $\alpha$, or angle between the axis of cut and the direction of grain in the members, is 33.7°.

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Assume connectors will be installed in unseasoned wood: from Table 7.3.3, $C_M = 0.80$

Locate the connector center 8-5/16 inches from the apex. Effective thickness of each member (see Figure C10.2-1)

$$t = \tan 33.7° \times (8.312 - 2.625/2) = 4.669 \text{ in.}$$

Minimum end distance for full design value from 10.3.6.1 and Table 10.3 = 4 in.; minimum for reduced design value = 2-1/2 in. Actual end distance = 8-5/16 in.

Full design value applies, $C_a = 1.0$

Minimum edge distance for full design value from 10.3.6.1 and Table 10.3 = 1-3/4 in.; minimum for reduced design value = 1-3/4 in.

Actual edge distance = 3.5/2 = 1-3/4 in.

Full design value applies, $C_a = 1.0$

From Table 10.2B for 2-5/8 in. diameter plate with 3/4 in. bolt, 1 face containing connector, minimum member thickness of 1-1/2 in., Group B species:

$$P = 2670, P' = (2670 C_D C_M C_a) = 2456$$

$$Q = 1860, Q' = (1860 C_D C_M C_a) = 1711$$

From Equation C10.2-1: $Q'_{90} = 1027$

From Equation C10.2-2 with $\alpha = 33.7°$: $P'_{90} = 1719$

The maximum allowable vertical load that can be transferred from one rafter to the other through the connector unit is 1719 lbs.

10.3-PLACEMENT OF SPLIT RING AND SHEAR PLATE CONNECTORS

10.3.1-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.

10.3.2-Geometry Factor, $C_a$

The geometry factor has been introduced in the 1991 edition as part of the general conversion of the provisions of the Specification to an equation format. The factor adjusts tabulated design values for use of end distances, edge distances and/or spacings which are less than those required to achieve the full design value capacity of the specified connector unit. The use of reduced connector design values to accommodate imposed geometric design limitations has been a provision of the Specification since the 1944 edition.

It is to be noted that 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. This provision for allowing less than full design connector distances to be used only when the total load on the joint or shear plane is reduced in proportion to the reduction in distances was added in the 1982 edition to clarify the intent of requirements for end distance and spacing. The 1991 edition specifically extends the clarification to edge distance requirements.

10.3.3-Edge Distance

10.3.3.1 Connector edge distance requirements and related geometry factors in Table 10.3 are unchanged since the 1944 edition. Specific values are based on the original connector research (159).
10.3.3.2 Determination of edge distance requirements for members loaded at angles to grain between 0° and 90° is simplified by the fact that the edge distance requirement for a given size connector in Table 10.3 is the same for both minimum and full design value for unloaded and loaded edge parallel to the grain and for unloaded edge perpendicular to the grain. Only in the loaded edge perpendicular to grain case is a larger edge distance required to achieve full design value. In this instance, the edge distance required for all other conditions is associated with a reduced minimum design value having a geometry factor, \( C_a \), of 0.83. A one inch larger edge distance is required for \( C_a = 1.00 \). Example C10.3-1 illustrates these provisions.

Example C10.3-1

A 2-1/2 inch split ring connector with 1-3/4 inch unloaded and loaded edge distance. For the unloaded edge, \( C_a = 1.00 \) under any angle of loading. For the loaded edge, \( C_a = 0.83 \) under any angle of loading. To obtain a \( C_a = 1.00 \) for the connector, the loaded edge distance must be increased to 2-3/4 inches when the angle of loading is 45° or greater and proportionally less for angles between 0° and < 45°. For an angle of loading of 22.5°, the loaded edge distance required for \( C_a = 1.00 \) is 2.75 - (22.5/45)(2.75 - 1.75) or 2-1/4 inches.

The edge distance for the loaded edge establishes the geometry factor for edge distance that must be applied, if limiting, to the tabulated values of \( P \) and \( Q \) to obtain the values of \( P' \) and \( Q' \) required by the standard angle to grain equation in 10.2.5.1. Example C10.3-2 illustrates these provisions.

Example C10.3-2

For a 2-1/2 inch split ring connector loaded at an angle to grain of 22.5°, the required loaded edge distance for full design value is 2.25 inches, and for minimum reduced design value is 1.75 inches. For an actual loaded edge distance of 2 inches, the geometry factor for the connector based on 10.3.3.1 is by linear interpolation

\[
C_a = 0.83 + \frac{(2.00 - 1.75)(1.00 - 0.83)}{(2.25 - 1.75)} = 0.915
\]

10.3.4-End Distance

10.3.4.1 The end distance requirements for full and reduced design values in Table 10.3 are based on the original connector research and have been part of the Specification since the 1944 edition. These requirements vary depending upon whether the member is being loaded in tension or compression, with the latter also differing depending upon whether loading is parallel or perpendicular to grain.

10.3.4.2 The use of linear interpolation between tabulated end distances for parallel and perpendicular to grain loading to determine end distance requirements for members loaded at angles to grain between 0° and 90° has been a provision of the Specification since the 1944 edition. Example C10.3-3 illustrates these provisions.

Example C10.3-3

A 2-1/2 inch split ring connector is loaded in compression at an angle to grain of 30° and has an end distance of 3.5 inches. For minimum reduced design value acting at an angle of 30°, the required end distance from 10.3.4.2 is \([2.50 + (30/90)(2.75 - 2.50)], or 2.58 inches. For full design value acting at an angle of 30°, the required end distance is \([4.0 + (30/90)(5.50 - 4.0)], or 4.5 inches. From 10.3.4.1, the end distance geometry factor for the connection is

\[
C_a = 0.625 + (3.5 - 2.58)(1.00 - 0.625)/(4.5 - 2.58) = 0.804
\]

10.3.5-Spacing

10.3.5.1 Spacing requirements in Table 10.3 are based on the original connector research (159) and have remained unchanged since the 1944 edition with two minor exceptions. In 1962, the minimum spacings for reduced design values for parallel to grain loading and parallel to grain spacing were increased from 3-3/8 to the present 3-1/2 inches for the 2-1/2 - 2-5/8 inch connectors and from 4-7/8 to the present 5 inches for the 4 inch connectors.

However, the reduced design value percentage or geometry factor associated with the minimum spacings in the 1991 edition is more conservative than the percentage used in the earliest editions of the Specification. From 1944 through the 1960 edition, the geometry factor associated with the minimum allowed spacing for parallel to grain loading - parallel to grain spacing was 0.75, and for perpendicular to grain loading - perpendicular to grain spacing was 0.83, the latter factor being that associated with the minimum loaded edge distance for this case. These early factors were based on parallel to grain test results in the original research (159,179). In 1962, the 0.83 factor for the
perpendicular loading and spacing case was dropped to 0.75 for purposes of uniformity.

In the 1971 edition, the geometry factor for minimum allowed spacings was reduced from 0.75 to the present 0.50. This change reflected recommendations made previously for simplifying adjustment of connector design values for end distances and longitudinal spacing (62).

The original connector research indicated that the load-carrying capacity of a joint made with two or more connectors aligned parallel to grain and loaded perpendicular to grain was less than the sum of the maximum design values for the same connectors acting singly (62,159). Staggering or offsetting of connectors so that they do not act along the same line along the grain of the transverse loaded member was found to give somewhat higher design values (62). When such staggering or offsetting is used, the line connecting the centers of two or more connectors located in the same contact face, the connector axis \( \phi \), may not be oriented parallel or perpendicular to the grain of the member or to the direction of load, \( \theta \). Spacings intermediate to those given in Table 10.3 for reduced and full design values are applicable to such cases. Because the variables involved are not linearly related, a graphical method has been developed for determining spacing requirements for designs for these cases where the connector axis is at an angle to the grain of the member (4,179). This graphical method is based on numerical procedures given in the 1968 and earlier editions of the Specification. The numerical procedures are given in the Commentary for 10.3.5.2.

Use of linear interpolation to establish geometry factors for spacings intermediate to those associated with tabulated minimum and full design values has been a provision of the Specification since the 1944 edition.

10.3.5.2 The graphical method for determining minimum spacing requirements for members loaded at an angle to grain (4,179) is based on numerical procedures given in the Specification prior to 1971. These procedures, which combine the effects of both variable connector axis angle, \( \varphi \), and variable angle to grain loading, \( \theta \), are given below.

Minimum spacing \( R \) required for full design value for any connector axis angle \( \varphi \) between 0° and 90° and for any angle of load to grain \( \theta \) between 0° and 90° is determined from the equation

\[
R = \frac{AB}{\sqrt{A^2 \sin^2 \varphi + B^2 \cos^2 \varphi}}
\]

(C10.3-1)

where:

\( A \) and \( B \) are spacing values selected from Table C10.3-1 for the applicable connector type and size and angle of load to grain.

### Table C10.3-1 - Connector Spacing Values

<table>
<thead>
<tr>
<th>Connector type and size</th>
<th>Angle of load to grain, ( \theta )</th>
<th>( A ), in.</th>
<th>( B ), in.</th>
<th>( C ) (min. for ( C_a = 0.5 )), in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1/2 in. split ring</td>
<td>0°</td>
<td>6-3/4</td>
<td>3-1/2</td>
<td>3-1/2</td>
</tr>
<tr>
<td>or</td>
<td>15°</td>
<td>6</td>
<td>3-3/4</td>
<td>3-1/2</td>
</tr>
<tr>
<td>or</td>
<td>2-5/8 in. shear plate</td>
<td>30°</td>
<td>5-1/8</td>
<td>3-7/8</td>
</tr>
<tr>
<td></td>
<td>45°</td>
<td>4-1/4</td>
<td>4-1/8</td>
<td>3-1/2</td>
</tr>
<tr>
<td></td>
<td>60-90°</td>
<td>3-1/2</td>
<td>4-1/4</td>
<td>3-1/2</td>
</tr>
<tr>
<td>or</td>
<td>4 in. split ring</td>
<td>0°</td>
<td>9</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>15°</td>
<td>8</td>
<td>5-1/4</td>
<td>5</td>
</tr>
<tr>
<td>or</td>
<td>4 in. shear plate</td>
<td>30°</td>
<td>7</td>
<td>5-1/2</td>
</tr>
<tr>
<td></td>
<td>45°</td>
<td>6</td>
<td>5-3/4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>60-90°</td>
<td>5</td>
<td>6</td>
<td>5</td>
</tr>
</tbody>
</table>

The value of \( R \) determined from Equation C10.3-1 is the required spacing for full design value, \( C_a = 1.0 \). The value of \( C \) from Table C10.3-1 is the minimum allowed spacing which is associated with the reduced design value, \( C_a = 0.50 \). For a load angle of 0°, values of \( A \) and \( B \) are the spacings from Table 10.3 of the Specification for full design and reduced design value, respectively, for the parallel spacing - parallel loading case. For a load angle of 90°, the values of \( A \) and \( B \) are the spacings from Table 10.3 of the Specification for reduced design and full design value, respectively for the perpendicular spacing - perpendicular loading case.

For angles of load to grain, \( \theta \), intermediate to those tabulated, values of \( A \) and \( B \) may be obtained by linear interpolation. For actual spacing, \( S \), between \( R \) and \( C \), the geometry factor, \( C_a \), is determined by linear interpolation or

\[
C_a = 0.50 + \frac{(S - C)(1.0 - 0.50)}{(R - C)}
\]

(C10.3-2)

Example C10.3-4 illustrates the use of these provisions.

10.3.6-Split Ring and Shear Plate connectors in End Grain

10.3.6.1 Procedures for establishing minimum and full design value spacing and edge and end distances
Example C10.3-4

Two 2-1/2 inch split ring connectors spaced 4.0 inches apart on a connector axis angle, $\phi$, of 30° are loaded at an angle to grain, $\theta$, of 22.5°. Determine the spacing geometry factor, $C_g$.

From Equation C10.3-1 and Table C10.3-1:

$$A = 6 - (6 - 5.125)/2 = 5.562$$
$$B = 3.75 + (3.875 - 3.75)/2 = 3.812$$
$$R = 4.541 \text{ for } C_{A} = 1.00$$
$$C = 3.5 \text{ for } C_{A} = 0.50$$

$$C_{g}(s=4.0) = 0.50 + (4 - 3.5)(1.0 - 0.50)/(4.541 - 3.5) = 0.76$$

for connectors in end grain surfaces follow the same logic as that employed to establish allowable design values for such configurations in 10.2.6. The procedures have been part of the Specification since provisions for connectors in end grain were introduced in the 1977 edition.

10.3.6.2 Special attention should be given to the requirement for checking the shear capacity of members supported by connectors in end grain surfaces. Where the slope of the surface cut, $\alpha$, 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, $V_s$, in Equation 3.4-5 of the Specification. 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 (see Figure C10.2-1). Example C10.3-5 illustrates these provisions.

10.3.7-Multiple Split Ring or Shear Plate Connectors

10.3.7.1 The inclusion of same bolt axis connectors with connectors in a row in 10.3.7.1 of the 1991 edition is an inadvertent carryover of language introduced in the 1977 edition when the adjustment for fasteners in a row was first added to the Specification.

The group action factor, $C_{g}$, of 7.3.6 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 Commentary for 7.3.6). 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.

It is to be noted that the definitive criterion for application of the group action factor is alignment of two or more connectors in the direction of load on the shear plane. Two or more connectors which are aligned parallel or perpendicular to the grain of a member are not subject to the group action factor if such connectors are not also aligned with the resultant force on the shear plane. Design values for the connectors in the joint illustrated in Figure 10J of the Specification are not adjusted for group action.

10.3.7.2 The provision for handling two sizes of split ring grooves cut concentrically on the same wood surface has been part of the Specification since the 1944 edition. When this occurs and rings are installed in both grooves as required, the total load on the joint is limited to the allowable design value for the larger ring only.

Example C10.3-5

Check the shear at the end of the rafter in the end grain connector from Example C10.2-2. The joint consists of a 2-5/8 inch shear plate connector unit joining two Douglas fir-Larch 4x10 rafters at the ridge where the roof slope is 12 in 8 producing an angle of 33.7° from the vertical at the peak. The allowable connector design value for $C_D$ of 1.15 on the end grain surface is 1719 pounds.

$$V = V' \sin \alpha = (1719) \sin 33.7° = 954 \text{ lbs}$$
$$d_e = (8.312 + 2.625/2) \sin 33.7° = 5.34 \text{ in.}$$

From Equation 3.4-5:

$$F'_{\nu} = [3(954)(9.25)/\{2(3.5)(5.34)^2\}] = 133 \text{ psi}$$

Allowable shear for Douglas fir-Larch 4x10:

$$F'_{\nu} = 95 \text{ of } C_D = 95(1.15) = 109 < 133$$

The design does not check for shear at the connection. Either limit $V'$ to

$$V' = (109(2)(3.5)(5.34)^2)\{3(9.25)\} = 784 \text{ lbs}$$
$$V' = (784)/\sin 33.7° = 1413 \text{ lbs}$$

or increase the distance, $d_e$, from the apex to the center of the connector to

$$d_e = [(3(954)(9.25))/\{2(3.5)(109)\}]^{1/2} = 5.89 \text{ in.}$$
$$d' = (5.89)/\sin 33.7° = 10.615 \text{ in.}$$
$$d_e = (10.615 - 2.625)/2 = 9.30 \text{ in.}$$
PART XI: WOOD SCREWS

11.1-GENERAL

11.1.1-Quality of Wood Screws

In the 1986 and earlier editions, wood screws were not required to meet particular dimensional and thread type standards to qualify for the design values given in the Specification. The general performance requirement in these editions was that the wood screws be of sufficient strength to cause failure in the wood rather than the metal.

The 1991 edition requires wood screws to conform to the dimensional data of ANSI/ASME Standard B18.6.1-1981 in order to qualify for tabulated design values and related design provisions. Standard B18.6.1, which covers both cut and rolled thread screw types, does not specify minimum metal strength properties. However, bending yield strength of the screw is a required input to the lateral design value yield mode equations of 11.3.1. Additionally, the actual tensile stress in the screw at the root diameter is required to be checked when designing the connection for withdrawal loads (see 11.2.3 of the Specification).

Tabulated lateral design values for wood screws in Tables 11.3A and 11.3B apply to screws with specified strength properties. Irrespective of whether the lateral design values in these tables are used or lateral design values are developed directly from the yield mode equations of 11.3.1, it is the designer’s responsibility to specify the metal strength properties of the wood screws that are to be employed for the job.

11.1.2-Fabrication and Assembly

11.1.2.1 Prior to the 1962 edition, lead hole requirements for wood screws were 90 percent of the screw root diameter for hardwoods and 70 percent of the root diameter for softwood species. These provisions were 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 (55). In the 1962 edition, the lead hole requirements were referenced to new fastener species groups established on the basis of specific gravity. The 90 percent of root diameter requirement was applied to Group I species, those with a specific gravity greater than 0.60; and the 70 percent of root diameter requirement was applied to species of lower specific gravity in Groups II, III and IV.

In the 1982 edition, the provision allowing the insertion of wood screws in Group III and IV species without a lead hole when the screw was subject to withdrawal loads only was introduced. The specific gravity of species in these groups was less than 0.51. This provision for use of screws without a lead hole paralleled that made for 3/8 inch and smaller diameter lag screws in the same edition and was supported by both research results and field experience (see Commentary for 9.1.2.2).

The lead hole provisions for screws in withdrawal in the 1991 edition continue the requirements of the 1982 and earlier editions except that the species group designations have been replaced by specific gravity ranges.

11.1.2.2 In the 1960 and earlier editions, wood screws resisting lateral loads were required to have shank and threaded portion lead holes in hardwoods equal to the shank and thread root diameters, respectively; and in softwood species equal to seven-eighths the shank and thread root diameters, respectively. These provisions were based on early lateral load tests of wood screws (57, 62, 98). The hardwoods and softwoods designations were replaced in the 1962 edition with species Group I and Groups II, III and IV, respectively, based on specific gravity. Species classified in Group I were those with specific gravities greater than 0.60 while those with lower specific gravities were classified in the higher numbered groups.

In the 1991 edition, lateral design values for wood screws are no longer tabulated in terms of species groups but are given for each individual species combination. Thus the lead hole requirements for screws resisting lateral loads that were given in previous editions are applied in terms of whether the specific gravity of the species is greater than 0.60 or equal to or less than 0.60. It is to be noted that lead holes are required for all wood screws subject to lateral loads regardless of wood specific gravity.

11.1.2.3 Insertion of wood screws by turning rather than driving has been a requirement since the 1944 edition. All tests on which wood screw provisions in the Specification are based involved joints made with this method of assembly (55, 62, 98).

11.1.2.4 Use of lubrication to facilitate screw insertion and avoid screw damage, a recommendation since the 1944 edition, has been made mandatory in the
1991 edition. Early tests show the lubricant has no significant effect on design values (55,57,98).

11.2-WITHDRAWAL DESIGN VALUES

11.2.1-Withdrawal from Side Grain

Background

Withdrawal design values for wood screws are based on the equation

\[ W = K_W G^2 D \]  \hspace{1cm} \text{(C11.2-1)}

where:

- \( W \) = nominal withdrawal design value per inch of screw length or per inch of penetration of the threaded portion, lbs
- \( K_W \) = constant based on ultimate load tests and screw length basis
- \( G \) = specific gravity, oven dry weight and volume
- \( D \) = shank diameter of the screw, in.

This equation was based on early extensive testing with cut thread wood screws and seven wood species (55). In the 1960 and earlier editions, the equation was published in the Specification in lieu of tabulated withdrawal design values. In the 1944 edition, a value of 2040 was used for the constant \( K_W \). This value gave a withdrawal design value per inch of total screw length assuming the depth of penetration into the piece receiving the point was at least two-thirds the length of the screw; and a withdrawal design value which was approximately one-fifth the ultimate load determined from screw withdrawal tests (55). The one-fifth factor represented an originally recommended one-sixth factor on ultimate load (57) increased by twenty percent as part of the World War II emergency adjustment in design values. Following the war, this 1.2 factor was codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see Commentary for 2.3.2).

In the 1950 edition, the basis for \( W \) was changed from design value per inch of total screw length to design value per inch of penetration of the threaded part of the screw. The value of the constant \( K_W \) was changed to adjust the equation for this new index point (57,62). The equation became

\[ W = 2850 G^2 D \]  \hspace{1cm} \text{(C11.2-2)}

where:

\( W \) = nominal withdrawal design value per inch of penetration of the threaded portion of the screw in the piece receiving the point, lbs

A table of allowable screw withdrawal design values based on Equation C11.2-2 also was introduced in the 1950 edition.

In the 1960 edition, the table of screw gages and lengths, which had been part of the Specification since the 1944 edition, was dropped. In the 1962 edition, Equation C11.2-2 also was dropped from the Specification in favor of the tabulated withdrawal design values alone.

1991 Edition

The wood screw withdrawal design values in Table 11.2A of the 1991 edition are based on Equation C11.2-2 and remain unchanged from those given in earlier editions. The specific application of the tabulated withdrawal design values to rolled thread wood screws as well as cut thread screws is a new provision. Previous editions did not specify thread type, although the early research on which wood screw withdrawal design values are based was conducted on cut thread 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 gage and nominal diameter of screw, both screw thread types have the same threads per inch, the same outside diameter of thread and the same thread depth. If the tensile strength of the screw is adequate and the lead hole provisions based on root diameter are employed, the withdrawal resistance of rolled thread screws is considered equivalent to that of cut thread screws. This is supported by comparative tests of one type of tapping screw and cut thread wood screws. Although, the thread depth of tapping and rolled thread screws are not the same, both types have outside thread diameters that are larger than their body diameters. The comparative tests showed that, for comparable diameters and penetrations of the threaded portion of the screws, the withdrawal design values of the tapping screw were slightly higher than those of the cut thread fastener (65,204).

Screw length as well as screw gage or nominal diameter must be specified. The ANSI/ASME B18.6.1 standard requires thread length to be equivalent to at least two-thirds of the nominal screw length. The screw diameters associated with the screw gages given in Table 11.2A are shown in Tables 11.3A and 11.3B.
Table 11.2A is entered with the specific gravity of the lumber species combination receiving the threaded portion of the screw. Specific gravity values are given for current commercial lumber species combinations in Table 11A. Species specific gravity values tabulated in the Specification have changed in various editions as a result of new property information, changes in the commercial importance of some species, and the introduction of new species groupings. Specific gravity values tabulated in the 1991 edition reflect a number of such changes from values given in previous editions.

11.2.2-Withdrawal from End Grain

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 (55). 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 (55). Because of this variability, structural loading of wood screws in withdrawal from end grain has been prohibited since the 1944 edition. Where splitting is avoided, use of an end grain to side grain withdrawal design value ratio of 75 percent has been suggested (57,66).

11.2.3-Tensile Strength of Wood Screw

(See Commentary for 11.1.1)

11.3-LATERAL DESIGN VALUES

11.3.1-Wood-to-Wood Connections

Background

From the 1944 through the 1986 editions, lateral design values for wood screws loaded at any angle to grain were based on the equation

\[ Z = K_L D^2 \]  

(C11.3-1)

where:

\[ Z = \text{nominal wood screw lateral design value, lbs} \]

\[ K_L = \text{species group constant based on specific gravity (G) of wood members} \]

\[ \begin{align*}
4800 & \quad \text{Group I} \\
3960 & \quad \text{Group II} \\
3240 & \quad \text{Group III} \\
2520 & \quad \text{Group IV}
\end{align*} \]

\[ G = 0.62 - 0.75 \quad \text{or} \quad 0.51 - 0.55 \quad \text{or} \quad 0.42 - 0.49 \quad \text{or} \quad 0.31 - 0.41 \]

\[ D = \text{shank diameter, in.} \]

The equation was based on early lateral load tests of wood screws in southern pine, cypress and oak in which the depth of penetration of the screw into the piece receiving the block was at least 7 times the shank diameter (57,98). The values shown for the constant \( K_L \) provided lateral design values which were about 75 percent of those associated with test proportional limit values. The \( K_L \) values included the originally recommended adjustment of 1.6 (57) increased 20 percent as part of the World War II emergency increase in wood design values. The latter adjustment was subsequently codified as a 10 percent adjustment for the change from permanent to normal loading and 10 percent for experience (see Commentary for 2.3.2). Lateral design values of 75 percent of proportional limit values are about one-fifth maximum test loads (57).

Beginning with the 1950 edition, tabulated wood screw lateral design values based on Equation C11.3-1 were included in the Specification. Presentation of the equations for each group was subsequently discontinued beginning with the 1962 edition. In the 1960 and earlier editions, a wood screw fastener group between Groups II and III was provided in the Specification. This group for intermediate density hardwood species also was dropped beginning with the 1962 edition.

Prior to 1971, grain type and other features than specific gravity were considered in classifying certain lower density softwood species into \( K_L \) factor groups. This was evidenced by some species having the same specific gravities as those in Group III being classified in Group IV (57,62). Beginning with the 1971 edition, specific gravity was used as the sole criterion for assignment of species for wood screw lateral design value constants. The specific gravity class limits for \( K_L \) values shown in the legend for Equation C11.3-1 were used to classify species from 1971 through the 1986 edition.

1991 Edition

Similar to the treatment of bolts and lag screws, lateral design values for wood screws in the 1991 edition are based on application of the yield limit model (see Commentary for 7.2.1 and 8.2.1). The same general equations used to describe different modes of yielding of lag screws are also used with wood screws (see Commentary for 9.3.1). Three modes of yielding are provided for: bearing in the side member or cleat (Mode I_s), development of a plastic hinge in the screw in the main member (Mode III_s), and development of plastic hinges in both main and side members (Mode IV).

The term \( K_L \) in the denominator of the yield mode equations of 11.3.1 relates yield mode equation design values based on a 5 percent diameter offset.
dowel bearing strength to the general level of proportional limit based lateral design values previously given in the Specification. For small diameter dowel type fasteners (wood screws and nails), this conversion factor was set at 2.2 when the fastener penetration in the member receiving the point is sufficient to develop the full lateral load capacity of the joint.

In the 1986 and earlier editions of the Specification, lateral design values for small diameter lag screws were lower than those for wood screws of comparable diameter. This difference was a result of the different methodologies used to establish lateral design values for the two fastener types, primarily the reduction factor applied to proportional limit test values. In the 1991 edition, this difference in lateral design values between small diameter lag screws loaded parallel to grain and wood screws of similar diameter has been minimized by using a conversion factor, $K_D$, of 3.0 for the largest diameter wood screws. This value relates to the 2.8, 4.0 and 3.0 conversion factors associated with the Mode I, IIIa and IV yield mode equations, respectively, for lag screws loaded parallel to grain (see Commentary for 9.3.1 - Yield Mode Equations). To provide for a gradual transition of $K_D$ values between small and large diameter wood screws, a variable value of $K_D$ is used for intermediate diameter screws as shown below.

Unthreaded shank diameter, inches = $D$
Diameter coefficient for wood screws = $K_D$

\[
\begin{align*}
D \leq 0.17 & \quad K_D = 2.2 \\
0.17 < D < 0.25 & \quad K_D = 10(D) + 0.5 \\
D \geq 0.25 & \quad K_D = 3.0
\end{align*}
\]

No adjustment of wood screw yield mode equation design values are made for varying angles of load to grain. This is a continuation of procedures in previous editions of the Specification which assigned the same wood screw lateral design values to both parallel and perpendicular to grain loading conditions.

The lowest value of $Z$ obtained from the three yield mode equations of 11.3.1 is selected as the nominal lateral design value for the connector. The equations are entered with unthreaded screw shank diameter, side member thickness and dowel bearing strengths, $F_{es}$ and $F_{am}$, for the species of wood being used for the side or main member. Such dowel bearing strength values are given in Table 11A and are based on the equation

\[
F_e = (16,600) \cdot G^{1.84} \quad \text{(C11.3-2)}
\]

where:

$G =$ specific gravity based on oven dry weight and volume

The research on which Equation C11.3-2 is based involved nail bearing tests on five species (203). Tests of three nail diameters (0.148 - 0.225 in.) with one species showed diameter to have no significant effect on small-diameter dowel bearing strength (203).

The bending yield strength value, $F_{yb}$, for the wood screw being used also is an input to the yield mode equations. For screws having a diameter equal to or larger than 3/8 inch, $F_{yb}$ may be taken as 45,000 psi (see Appendix I). Bending tests of nails of various diameters show bending yield strength tends to increase as diameter decreases (106). Wood screw bending yield strength values based on the relationships found in this nail research have been used to develop wood screw lateral design values tabulated in the Specification for screws less than 3/8 inch diameter (see Appendix I).

**Tabulated Wood-to-Wood Lateral Design Values.** Lateral design values given in Table 11.3A apply to single shear connections made only with cut thread wood screws having the unthreaded shank diameters tabulated. The table provides lateral design values for screw gages from 6 to 24 and for major individual species combinations. In earlier editions, lateral design values were tabulated in terms of four fastener groups based on specific gravity classes.

Tabulated lateral design values apply to connections made with side member and main member of the same species. Where different species are used, tabulated lateral design values for the species with the lowest specific gravity may be applied or lateral design values may be determined from the yield mode equation directly. For connections involving species not listed in the table, lateral design values given for a species of lower specific gravity may be used.

Lateral design values in Table 11.3A apply to screws having the bending yield strengths, $F_{yb}$, given in footnote 2 of the table. For screw gages 6 to 20, the strengths shown are based on Equation C12.3-2 (see Commentary for 12.3.1) developed from the tests of common nails. For screws, this equation is entered with the unthreaded shank diameter as $D$.

**Comparison of 1991 and Earlier Edition Lateral Design Values.** The lateral design values in the 1991 edition are based on side member thickness, screw shank diameter, dowel bearing strengths of the side and main members, and the bending yield strength of the wood screw. Lateral design values in earlier
editions were based on screw shank diameter and fastener group or specific gravity class. Both the 1991 and earlier editions assume wood screw penetration into the main member of at least 7 times the shank diameter. The effect of these different bases and the conversion factors, $K_D$, used to relate yield mode lateral design values to the level of previous tabulated lateral design values are shown by the comparisons of 1991 and 1986 lateral design values in Table C11.3-1.

Table C11.3-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Wood Screw Lateral Design Values

<table>
<thead>
<tr>
<th>Side Member Thickness, in.</th>
<th>Wood Screw Gage</th>
<th>Wood Screw Diameter, in.</th>
<th>Wood Screw Lateral Design Value, lbs</th>
<th>1991</th>
<th>1986</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern pine:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2 8g</td>
<td>0.164</td>
<td>108-106</td>
<td>1.02</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g 8g</td>
<td>0.216</td>
<td>138-185</td>
<td>0.75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>190-342</td>
<td>0.56</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>548</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4 8g</td>
<td>0.164</td>
<td>132-106</td>
<td>1.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g 8g</td>
<td>0.216</td>
<td>159-185</td>
<td>0.86</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>210-342</td>
<td>0.61</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>283-548</td>
<td>0.52</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-1/2 8g</td>
<td>0.164</td>
<td>148-106</td>
<td>1.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>0.216</td>
<td>200-185</td>
<td>1.08</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>284-342</td>
<td>0.83</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>394-548</td>
<td>0.72</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spruce-Pine-Fir:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2 8g</td>
<td>0.164</td>
<td>79-87</td>
<td>0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g 8g</td>
<td>0.216</td>
<td>103-151</td>
<td>0.68</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>145-280</td>
<td>0.52</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>-448</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4 8g</td>
<td>0.164</td>
<td>90-87</td>
<td>1.03</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>0.216</td>
<td>112-151</td>
<td>0.74</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>152-280</td>
<td>0.54</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>207-448</td>
<td>0.46</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-1/2 8g</td>
<td>0.164</td>
<td>115-87</td>
<td>1.32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g 8g</td>
<td>0.216</td>
<td>155-151</td>
<td>1.03</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18g 8g</td>
<td>0.294</td>
<td>204-280</td>
<td>0.73</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>268-448</td>
<td>0.60</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The lower 1991/1986 lateral design value ratios for the larger gage wood screws relative to the lateral design value ratios for the 8g screw reflect the use of larger $K_D$ factors in the yield mode equations for the larger gages in order to bring wood screw lateral design values in line with lateral design values for lag screws of similar diameter. The differences between the 1991/1986 lateral design value ratio for the joint with 8g screw and 1/2 inch side members and that with 8g screw and 1-1/2 inch side members (1.02 vs. 1.40 and 0.91 vs. 1.32 for southern pine and spruce-pine-fir respectively) indicates the significant effect of side member thickness. The generally higher lateral design value ratios for southern pine compared to spruce-pine-fir is due to the fact southern pine was at the upper limit of its 1986 fastener group (specific gravity ) class whereas spruce-pine-fir was near the lower limit of its class. In the 1991 edition, use of the specific gravity value for each species combination to establish dowel bearing strength values ties wood screw lateral design values for each combination more closely to its own properties.

11.3.2-Wood-to-Metal Connections

11.3.2.1 In the 1986 and earlier editions of the Specification, design values for wood screws in lateral resistance were increased 25 percent when metal side plates were used (57). This adjustment was similar to that used for bolted joints with metal side plates prior to 1982 (see Commentary for 8.2.2).

Under the provisions of the 1991 edition, wood screw lateral design values for joints made with metal side members are determined from the Mode III and Mode IV equations of 11.3.1. The Mode I equation is not used as bearing in the metal side members is considered separately in the design of metal parts (see 11.3.2.2). The yield mode equations are entered with the Dowel bearing strength of the metal as $F_{cs}$ and the thickness of the metal as the side member thickness. Earlier editions did not account for the effect of the metal gage used.

Table 11.3B gives lateral design values for cut thread wood screw joints made with steel side members from 0.048 inches (18 gage) to 0.239 inches (3 gage) thick. A Dowel bearing strength of 45,000 psi, applicable to ASTM A446 Grade A galvanized steel, was assumed for all thicknesses. The same wood screw bending yield strengths, $F_{yB}$, used to develop the wood-to-wood joint lateral design values in Table 11.3A were used to develop the Table 11.3B lateral design values.

Comparison of 1991 and Earlier Edition Lateral Design Values. Differences in 1991 and earlier edition wood screw lateral design values are illustrated in Table C11.3-2. Lateral design values shown for both editions are based on a penetration of the threaded portion of the screw in the main member of seven times the shank diameter. The 1986 lateral design values are applicable to any metal side plate thickness that permits this screw penetration requirement to be met.
### Table C11.3-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Wood Screw Lateral Design Values

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern pine:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>8g</td>
<td>0.164</td>
<td>132</td>
<td>132</td>
<td>1.00</td>
</tr>
<tr>
<td>(14g)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g</td>
<td>0.216</td>
<td>176</td>
<td>231</td>
<td>0.76</td>
<td></td>
</tr>
<tr>
<td>18g</td>
<td>0.294</td>
<td>249</td>
<td>428</td>
<td>0.58</td>
<td></td>
</tr>
<tr>
<td>0.134</td>
<td>8g</td>
<td>0.164</td>
<td>147</td>
<td>132</td>
<td>1.11</td>
</tr>
<tr>
<td>(10g)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g</td>
<td>0.216</td>
<td>188</td>
<td>231</td>
<td>0.81</td>
<td></td>
</tr>
<tr>
<td>18g</td>
<td>0.294</td>
<td>260</td>
<td>428</td>
<td>0.61</td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>356</td>
<td>685</td>
<td>0.52</td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>8g</td>
<td>0.164</td>
<td>106</td>
<td>109</td>
<td>0.97</td>
</tr>
<tr>
<td>(14g)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g</td>
<td>0.216</td>
<td>141</td>
<td>189</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>18g</td>
<td>0.294</td>
<td>199</td>
<td>350</td>
<td>0.57</td>
<td></td>
</tr>
<tr>
<td>0.134</td>
<td>8g</td>
<td>0.164</td>
<td>119</td>
<td>109</td>
<td>1.09</td>
</tr>
<tr>
<td>(10g)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12g</td>
<td>0.216</td>
<td>152</td>
<td>189</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>18g</td>
<td>0.294</td>
<td>209</td>
<td>350</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>24g</td>
<td>0.372</td>
<td>286</td>
<td>560</td>
<td>0.51</td>
<td></td>
</tr>
</tbody>
</table>

The lower lateral design value ratios in Table C11.3-2 for the larger screw gages are a result of the procedures introduced in the 1991 edition to bring lateral design values for the larger wood screws into line with those for lag screws of comparable diameter (see Commentary for 11.3.1 - Comparison). The comparisons made show that, for equivalent screw sizes, an approximate 80 percent increase in steel side member thickness is associated with an increased lateral design value of 11 percent or less.

#### 11.3.2.2 (See Commentary for 7.2.3)

### 11.3.3-Penetration Depth Factor, \( C_d \)

Use of reduced lateral design values for penetrations of the threaded portion of the screw in the main member of less than 7 times the shank diameter \( D \) has been a provision of the Specification since the 1944 edition. The minimum penetration requirement of \( 4D \) and the use of the ratio of actual penetration to that required for full lateral design value \( (p/7D) \) as the factor for adjusting lateral design values for penetration are based on early wood screw research (57,98).

### 11.3.4-End Grain Factor, \( C_{eg} \)

Use of two-thirds the lateral design value for wood screws inserted in side grain as the design value for wood screws inserted in the end grain of the main member has been a provision of the Specification since the 1944 edition. This is the same end grain factor as that used for lag screws.

#### 11.3.5-Combined Lateral and Withdrawal Loads

Earlier editions of the Specification assumed the capacity of wood screws subject to withdrawal and lateral loads at the same time was the same as the capacity of the screw under each load acting separately. In the 1991 edition, the interaction equation introduced for combined withdrawal and lateral loading on lag screws (see Commentary for 9.3.5) has been applied to wood screws. Although available lag screw test data indicate withdrawal and lateral load components interact only at total load angles less than \( 45^\circ \) and only with larger diameter screws (115), the lag screw interaction equation has been applied to wood screws for purposes of uniformity and conservatism. The equation, which is of similar form to the bearing angle to grain equation (see Appendix J), is

\[
Z'_a = \frac{Z' (W' p)}{Z' \cos^2 \alpha + (W' p) \sin^2 \alpha} \quad (C11.3-3)
\]

where:

- \( Z'_a \) = allowable design value for wood screw loaded at angle to the surface of main member
- \( Z' \) = lateral design value for wood screw joint
- \( W' \) = withdrawal design value for wood screw joint per inch of thread penetration in main member
- \( p \) = length of thread penetration of the wood screw in the main member
- \( \alpha \) = angle between wood surface and direction of applied load

Equation C11.3-3 can also be used to determine the allowable design value of wood screws embedded at an angle to grain in the wood member and loaded in a direction normal to the wood member. For this condition \( \alpha \) would be defined as the angle between the wood surface and the lag screw as shown in Figure C11.3-1.

### 11.4-PLACEMENT OF WOOD SCREWS

#### 11.4.1-Edge Distance, End Distance, Spacing

The absence of splitting has been used as a performance criterion to determine the adequacy of end and edge distances and spacing for wood screws since the 1944 edition.
In lieu of specific code requirements for end and edge distance for wood screws, Table C11.4-1 may be used to establish wood screw patterns. Designers should note that specie type, moisture content and grain orientation will affect spacing (pitch) between fasteners in a row.

### 11.4.2-Multiple Wood Screws

Lateral design values for wood screws are not subject to the group action factor, $C_g$, for fasteners aligned in the direction of load (see 7.3.6). The design value for a connection involving more than one wood screw is the sum of the design values for each individual wood screw when all wood screws in the connection are of the same type, diameter and length, join the same members and resist load in the same shear plane.

### Table C11.4-1 Wood Screw Minimum Spacing Tables

<table>
<thead>
<tr>
<th></th>
<th>Wood Side Members</th>
<th>Steel Side Members</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Not Prebored</td>
<td>Prebored</td>
</tr>
<tr>
<td>Edge distance</td>
<td>2.5$d$</td>
<td>2.5$d$</td>
</tr>
<tr>
<td>End distance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- tension load parallel to grain</td>
<td>15$d$</td>
<td>10$d$</td>
</tr>
<tr>
<td>- compression load parallel to grain</td>
<td>10$d$</td>
<td>5$d$</td>
</tr>
<tr>
<td>Spacing (pitch) between fasteners in a row</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>15$d$</td>
<td>10$d$</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>10$d$</td>
<td>5$d$</td>
</tr>
<tr>
<td>Spacing (gage) between rows of fasteners</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- in-line</td>
<td>5$d$</td>
<td>3$d$</td>
</tr>
<tr>
<td>- staggered</td>
<td>2.5$d$</td>
<td>2.5$d$</td>
</tr>
</tbody>
</table>

Wood Screws

139
PART XII: NAILS AND SPIKES

12.1-GENERAL

12.1.1-General Provisions

Provisions for common steel wire nails and spikes have been part of the Specification since the 1944 edition. Threaded hardened-steel nails and box nails were added in the 1962 edition and the 1977 edition, respectively.

Differences in Nail Types. For the same pennyweight classification and length, round spikes have a larger shank diameter than common nails and may have either a chisel point and countersunk oval head or a diamond point with a flat head. Spikes over 60d are generally specified by length (57,70).

Threaded or deformed shank hardened-steel nails include both ring or annularly threaded and helically threaded nails. For equivalent pennyweight and length, hardened-steel nails have a smaller shank diameter than common nails for pennyweights of 8d and larger. The head diameter of 20d and smaller hardened-steel nails is larger than that of the corresponding common nail but is smaller for larger sizes (those over 4 inches in length) (70).

Box nails have a smaller shank diameter than common nails of equivalent pennyweight. The head diameter of box nails is larger than that of common nails for sizes less than 10d, is the same for the 10d to 16d sizes and is smaller for sizes 20d and larger (70).

Nail Specifications. The requirement that nails intended for use in engineering construction be specified by length and diameter was introduced in the 1986 edition. The specification also should include head diameter if the nail is produced in more than one head diameter for the same length and shank diameter; and thread type or head and point type if other than common or box nails are to be used (70).

General Construction. Most nailed joints in light frame wood construction are not engineered but are made in accordance with standard practices that have been established from many years of field experience (126). Such practices are expressed in terms of nailing schedules which give the number, size and type of nail, and the direction of driving (e.g. face nailing, toenailing) to be used for different connections. For example, use of three 8d toenails to attach joists to a sill plate; and use of four 8d toenails or two 16d face or end nails to attach studs to bottom and top plates are standard practices accepted by most building codes.

The provision recognizing the use of standard nailing schedules (126) in lieu of designing joints for specific loads was first introduced in the 1982 edition.

12.1.2-Quality of Nails and Spikes

12.1.2.1 Standard diameters for the various types of nails and spikes were tabulated in previous editions of the Specification. In the 1991 edition, Federal Specification FF-N-105B (70) is referenced as the basic dimensional standard. Paragraphs 3.6.5, 3.6.11.2, 3.16.9 and 3.9-Style 3 of this Federal Specification cover steel wire box nails, steel wire common nails, pallet or threaded hardened steel nails, and round steel wire spikes, respectively. Nails or spikes outside the diameter and length classes covered in the Federal Specification for each type may be available. Provisions of the 1991 edition may be applied to such nails or spikes when the applicable diameters and lengths are specified and used to determine design values.

The nail provisions of the 1991 edition also apply to common wire nails made of copper or aluminum alloy conforming to the sizes given for such metals in Federal Specification FF-N-105B. It is the designer's responsibility to use appropriate bending yield strengths for such metals when determining lateral design values in accordance with 12.3 and to assure the tensile, bearing and shear strengths of the fastener are adequate to resist loads being transferred through the fastener to the wood members in the joint (see 7.2.3 of Specification).

12.1.2.3 The requirements for threaded hardened-steel nails have remain unchanged since the 1962 edition.

12.1.3-Fabrication and Assembly

12.1.3.1 A limitation on the size of prebored holes for nails and spikes was first introduced in the 1952 edition. Such holes were limited to 75 percent of the diameter of the fastener. In the 1962 edition, the limitation was changed to 90 percent for Group I species, those having a specific gravity of 0.62 or larger; and 75 percent for Group II, III and IV species, those having a specific gravity of 0.59 or less. These provisions were carried forward unchanged through the 1986 edition. The limitation on prebored nail and spike holes remains the same in the 1991 edition except that
the separation between the most dense species and other species is defined in terms of specific gravity rather than fastener species groups. The latter are no longer used for connection design in the 1991 edition.

12.1.3.2 Toenailing 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 have been part of the Specification since the 1952 edition. Based on lateral and withdrawal tests of nailed joints in frame wall construction (62,160), the toenail factors of 12.2.3 and 12.3.7 presume use of these driving procedures and the absence of excessive splitting. If such splitting does occur, a smaller nail should be used.

12.2-WITHDRAWAL DESIGN VALUES

12.2.1-Withdrawal from Side Grain

Background

Withdrawal design values for nails and spikes are based on the equation

\[ W = 1380 G^{5/2} D \]  

(C12.2-1)

where:

\[ W = \text{nail or spike withdrawal design value per inch of penetration in member holding point, lbs} \]

\[ G = \text{specific gravity of member holding point based on oven dry weight and volume} \]

\[ D = \text{shank diameter of the nail or spike, in.} \]

Equation C12.2-1 was based on early research (56,64) and has been used to establish nail and spike withdrawal design values since the 1944 edition. Withdrawal design values obtained from the equation represented about one-fifth average ultimate test values expressed on the same unit penetration basis. The one-fifth factor included the originally recommended one-sixth factor on ultimate load (57) increased 20 percent as part of the World War II emergency adjustment in wood design values. The 1.2 factor was subsequently codified as 10 percent for the change from permanent to normal loading and 10 percent for experience (see Commentary for 2.3.2).

Both Equation C12.2-1 and resultant tabulated withdrawal design values for common wire nails and spikes by species were presented in the 1944 and 1948 editions. In 1950, tabulated withdrawal design values began to be shown in terms of specific gravity rather than species. In the 1962 edition, Equation C12.2-1 was dropped from the Specification in favor of the withdrawal design value table alone.

Also in the 1962 edition, provisions for assigning withdrawal design values to threaded hardened nails were introduced. Such nails were assigned the same withdrawal design values as those for common wire nails of the same pennyweight class. However, the procedure was modified in the 1968 edition to account for the fact that although the diameter of common wire nails increases as pennyweight class increases from 20d to 60d, the diameters of threaded hardened nails in this range and larger do not. The diameter of 30d, 40d, 50d and 60d threaded hardened nails is the same as that (0.177 inches) for the 20d nail of this type, and the diameter for 80d and 90d threaded hardened nails was the same as that (0.207 inches) for the 70d size. To adjust for these differences, the 20d to 60d threaded hardened nails were assigned the same withdrawal design value as that for 20d common nails having a diameter of 0.192 inches. The 70d to 90d threaded hardened nails were assigned the same withdrawal design value as that for a common nail having a diameter which was in the same ratio to the 70d threaded hardened nail diameter of 0.207 as the common to threaded nail diameter ratio for 20d nails, or 0.192/0.177. This equivalent diameter is 0.225 inches, the diameter of a 40d common nail. For 20d and smaller pennyweights of threaded hardened nails, the same withdrawal design values as those for the equivalent size common nails were used as in the previous edition.

Withdrawal design values for box nails based on Equation C12.2-1 were introduced in the 1977 edition. The foregoing procedures for establishing withdrawal design values for nails and spikes were carried forward unchanged through the 1986 edition.

1991 Edition

Withdrawal design values tabulated in the 1991 edition are based on the same methodology used in the previous edition. However, withdrawal design values in Table 12.2A of the 1991 edition for common nails, spikes and box nails are consolidated in one part of the table under a common set of shank diameters, rather than being listed separately. Reference should be made to Tables 12.3A, 12.3B, and 12.3D to determine which diameters apply to box nails, common nails and spikes. Withdrawal design values for threaded hardened nails are given in a separate portion of Table 12.2A because of their different basis.

Special note is to be made of the wet service factor \( C_M \) of 0.25 that is applicable to withdrawal design
Clinching. It is to be noted that the withdrawal resistance of smooth-shank nails can be significantly increased by clinching (35). Increases in withdrawal resistance of 45 to 170 percent due to clinching have been reported when nails are tested soon after driving. When installed in unseasoned or partially seasoned wood and tested after seasoning, increases of 250 to 460 percent as a result of clinching have been observed. Clinching across the grain was found to give 20 percent higher withdrawal design values than clinching along the grain. When a greater assurance of a given level of withdrawal resistance is needed with smooth-shank nails, clinching should be considered.

12.2.2-Withdrawal from End Grain

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 (57,160). 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 in the Specification since the 1944 edition.

12.2.3-Toe-Nail Factor, $C_{la}$

The 0.67 adjustment of withdrawal design values for toenailing has been a provision of the Specification since the 1951 edition. The adjustment is based on the results of joint tests comparing slant driving and straight driving (57) and of typical toenailed and end nailed joints used in frame wall construction (160) 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 (57,66,160). When properly driven (see Commentary for 12.1.3.2), 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 (62,160).

Where toenailing is employed, the depth of penetration 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 C12.3-1, or (see 12.1.3.2)

\[ \rho_w = \frac{L_a - \frac{L_a}{3}}{\cos 30^\circ} = 0.615 \, L_a \]  \hspace{1cm} (C12.2-2)

where:

- $\rho_w$ = penetration of nail in member holding point, in.
- $L_a$ = length of nail, in.

12.3-LATERAL DESIGN VALUES

12.3.1-Wood-to-Wood Connections

Background

From the 1944 through the 1986 editions, lateral design values for nails and spikes loaded at any angle to grain were based on the equation

\[ Z = K_L D^{3/2} \]  \hspace{1cm} (C12.3-1)

where:

- $Z$ = nominal nail or spike lateral design value, lbs
- $K_L$ = species group constant based on specific gravity ($G$) of wood members
- $D$ = nail or spike shank diameter, in.

Equation C12.3-1 was based on early nail tests (57,59) and assumed certain minimum penetrations of the fastener in the main member (see Commentary for 12.3.4). Values assigned to the constant $K_L$ gave lateral design values that were approximately 75 percent of proportional limit test values. These values included the originally recommended adjustment of 1.6 (57) increased 20 percent as part of the World War II emergency increase in wood design values. The latter adjustment was subsequently codified as a 10 percent increase for the change from permanent to normal loading and 10 percent for experience (see Commentary for 2.3.2). Nail lateral design values of 75 percent of proportional limit values are about one-fifth maximum test values for softwoods and about one-ninth maximum test values for hardwoods (57).

From the 1944 through the 1960 editions, the individual group equations were presented in the Specification along with the list of species to which each applied. Beginning with the 1950 edition, tabulated nail and spike lateral design values based on the
equations for each group were included in the Specification. Presentation of the lateral design value equations was then discontinued beginning with the 1962 edition. In the 1960 and earlier editions, a nail fastener group between Groups II and III was included in the specification for intermediate density hardwood species. This intermediate group also was dropped beginning with the 1962 edition as most of the included species were of little commercial importance.

Prior to the 1971 edition, grain type and other features than specific gravity were taken into account in classifying species into fastener groups. This was evidenced by some species with the same specific gravities being classified in Group III and others in Group IV (57,62). Beginning with the 1971 edition, specific gravity was used as the sole criterion for assignment of species for nail lateral design value constants. The specific gravity class limits shown in the legend for Equation C12.3-1 were used to classify species from 1971 through the 1986 edition.

Provisions for establishing lateral design values for threaded hardened nails were introduced in the 1962 edition. These nails were assigned the same lateral design values as those for common nails of the same pennyweight class. The smaller diameter of the threaded hardened nails compared to the common nails was considered to be offset by the higher bending strength of the former. In the 1968 edition, assignments for 30d and larger threaded hardened nails were reduced to account for the fact that the diameters of the 30d to 60d pennyweight sizes were the same as the diameter for the 20d nail of this type, and the diameters of the 80d and 90d sizes were the same as the 70d size. Under this change, the 20d to 60d threaded hardened nails were assigned the same lateral design value as that for the 20d common nail and the 70d to 90d threaded hardened nails were assigned the same lateral design value as that for the 40d common nail. The ratio of the diameter of the 70d threaded hardened nail to that of the 40d common nail is the same as the ratio of the 20d threaded hardened nail to that of the 20d common nail (see Commentary for 12.2.1 - Background). These procedures for establishing lateral design values for threaded hardened nails were continued through the 1986 edition.

Lateral design values for box nails based on Equation C12.3-1 were first introduced in the 1977 edition.

It is to be noted that lateral design values tabulated in the 1986 and earlier editions of the Specification were not considered associated with a specific deformation level. However, it was reported that the proportional limit of a nailed joint could be assumed to be associated with a slip of approximately 0.015 inches (65,160). As Equation C12.3-1 is considered to give average lateral design values that are about 75 percent of average proportional limit test values, the lateral design values tabulated in previous editions would index to an average initial slip of 0.011 inches. This deformation level excludes the effects of any creep occurring under design loads.

1991 Edition

Nail and spike lateral design values in the 1991 edition are based on application of the yield limit model (see Commentary for 7.2.1 and 8.2.1) which also has been used to establish the lateral design values for bolts, lag screws and wood screws. The yield mode equations given in 12.3.1 for nails and spikes in single shear have been developed and verified in recent research (27,28). The equations provide for four modes of yielding: bearing in the side member being attached (Mode I\(_S\)), development of a plastic hinge in the side member (Mode III\(_S\)), development of a plastic hinge in the main member (Mode III\(_M\)), and development of plastic hinges in both main and side members (Mode IV). The lowest lateral design value, \(Z\), obtained from the four equations is taken as the basic lateral design value for the particular connection being evaluated. It is to be noted that the Mode III\(_S\) equation, unlike the other nail yield mode equations and those for other dowel type fasteners, includes a term to account for the length or penetration of the nail in the main member. All equations, however, require a minimum penetration of six diameters in the main member.

The \(K_D\) term in the denominator of each yield mode equation represent factors to reduce yield mode equation values based on a 5 percent diameter offset dowel bearing strength to the general level of the proportional limit based lateral design values tabulated in previous editions of the Specification. Values of \(K_D\) vary depending upon the shank diameter, \(D\), of the nail or spike, as shown below.

\[
\begin{align*}
0.17 < D < 0.25 & \quad K_D^D = 10(D) + 0.5 \\
D \geq 0.25 & \quad K_D = 3.0
\end{align*}
\]

The 2.2 factor for fasteners 0.17 inches or less in diameter is based on a comparison of yield mode lateral design values with nail lateral design values published in the 1986 edition for joints made with 8d and 16d nails in each of two species for a range of side member thicknesses (117). The 0.17 diameter limit covers all standard box nail sizes (up to 40d) and common and threaded hardened nails up to 16d. Most...
spikes fall in the larger diameter classes. Because side member thickness is a variable in the yield mode equations but was not a factor in developing lateral design values given in previous editions, the $K_D$ value of 2.2 gives lower lateral design values for joints made with the thinnest side members (5/16 inch) than those previously tabulated for equivalent species and nail size, but larger lateral design values for joints made with thicker side members (1-1/2 inch). In addition to having substantially higher lateral design values than those previously given in the Specification, nailed and spiked joints made with the thicker side members also had significantly higher lateral design values than lag screw and wood screw joints made with the same diameter fastener and side member thickness when the $K_D$ value of 2.2 was used in the nail yield equations for these combinations. As the larger diameter nails and spikes are used with the thicker side members, the foregoing inconsistencies were addressed by increasing the $K_D$ factor for nail and spike diameters of 0.25 inches or more from 2.2 to 3.0, limiting the 2.2 factor to 0.17 diameters and less, and use of a linear transition ($K_D = 10d + 0.5$) for intermediate diameters. These $K_D$ factor assignments are the same as those used with the yield mode equations for wood screws (see 11.3.1).

No adjustment of nail or spike yield mode equation values are made for varying angles of load to grain. This is a continuation of provisions in previous editions of the Specification which assigned the same nail or spike lateral design values to both parallel and perpendicular to grain loading conditions.

The nail and spike yield mode equations are entered with the fastener diameter, the side member thickness, the dowel bearing strengths of the main and side members ($F_{em}$ and $F_{es}$), and the bending yield strength of the fastener ($F_{yb}$). Dowel bearing strengths for all species combinations are listed in Table 12A. These dowel bearing strength values are based on the same specific gravity equation used to establish dowel bearing strengths for wood screws (see Commentary for 11.3.1 - 1991 Edition and Equation C11.3-2). The equation is based on research which involved bearing tests of nails 12d to 40d in size and which showed that diameter was not a significant independent variable with specific gravity (203).

The bending yield strength of the nail or spike, $F_{yb}$, may be specified by the designer or the values given in the footnotes in Tables 12.3A, 12.3B, 12.3C or 12.3D may be used (see Appendix I). The footnote values are based on the results of tests of common wire nails which showed that bending yield strength tends to increase as diameter decreases (106). The regression of test values from which nail bending yield strength values were estimated is given below.

$$F_{yb} = 130.4 - 214D$$  \hspace{1cm} (C12.3-2)

where:

- $F_{yb}$ = bending yield strength of steel based on 5 percent diameter offset, 1000 psi
- $D$ = nail diameter, in.

The increase in yield strength associated with the decrease in diameter is attributed to the work hardening of the steel as it is formed into progressively smaller diameters (106).

Bending yield strength values for hardened steel nails given in the footnote of Tables 12.3C are based on bending yield strength values for common nails of equivalent diameter increased approximately 30 percent (see Appendix I).

Tabulated Wood-to-Wood Lateral Design Values. Lateral design values for single shear connections made with 1/2 to 1-1/2 inch thick side members for each of the major individual species combinations are given in Tables 12.3A to 12.3D. Tables A, B, C and D give lateral design values for 6d to 40d box nails, 6d to 60d common nails, 6d to 90d threaded hardened nails and 10d to 3/8d spikes, respectively. Tabulated lateral design values apply to joints made with side and main members of the same thickness. Where different species are used, lateral design values for the species with the lowest specific gravity may be applied. However, in certain designs, use of the tabulated lateral design values for the lowest specific gravity wood in the joint may be considered overly conservative. In such cases, determining allowable joint lateral design values directly from the yield mode equations may prove beneficial. The difference between tabulated lateral design values and yield mode lateral design values for mixed species joints are illustrated in the nail examples presented in Example C12.3-1.

The effect of side member thickness on nail lateral design values is not linear. Tabular lateral design values should not be extrapolated to obtain lateral design values for side member thicknesses larger than the maximum thickness listed in the tables (see Example C12.3-1). For joints made of species other than those tabulated, lateral design values for a listed species combination with a lower specific gravity than that for the species being used, as determined from Table 12A, may be applied. With the highest density hardwoods,
Example C12.1-1
Yield mode lateral design values for wood-to-wood single shear nailed connections:

Single and mixed species joints of southern pine and spruce-pine-fir made with 8d common nails in side member thicknesses of 3/8, 1/2 and 5/8 inches and with 60d common nails in side member thicknesses of 1/2, 1-1/2, and 2-1/2 inches

\[ F_{w}, F_{s} = 5550 \text{ psi southern pine (SP)} \]
\[ = 3350 \text{ psi spruce-pine-fir (SPF)} \]
\[ F_s = 100,000 \text{ psi 8d} \]
\[ = 70,000 \text{ psi 60d} \]
\[ D = 0.131 \text{ 8d} \]
\[ = 0.263 \text{ 60d} \]
\[ K_D = 2.2 \text{ 8d} \]
\[ = 3.0 \text{ 60d} \]
\[ p = \text{nail length - side member thickness} \]

The consistently higher 1991/1986 lateral design value ratios for the 3/4 and 1-1/2 inch side member thicknesses relative to the ratios for the 1/2 inch side member thickness reflect the use of side member thickness as a variable in the yield mode equations whereas previous lateral design values were independent of member thickness. The generally lower lateral design value ratios for the fasteners with diameters over 0.25 inches relative to those with diameters less than 0.17 inches represents the effect of the larger \( K_D \) factor (3.0) used in the yield mode equations for the former as compared to the factor (2.2) used with the latter.

**Table C12.3-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Single Shear Nail Lateral Design Values**

<table>
<thead>
<tr>
<th>Side Member</th>
<th>Nail Thickness</th>
<th>Nail Speed</th>
<th>Nail Length</th>
<th>Yield Mode Design Values, lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penny-Weight in.</td>
<td>SP</td>
<td>SP</td>
<td>SP</td>
<td>Yield Mode Design Values, lbs</td>
</tr>
<tr>
<td>3/8</td>
<td>8d</td>
<td>78</td>
<td>0.113</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.148</td>
<td>78</td>
<td>0.192</td>
</tr>
<tr>
<td></td>
<td>5/8</td>
<td>8d</td>
<td>79</td>
<td>0.113</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.148</td>
<td>78</td>
<td>0.192</td>
</tr>
<tr>
<td>1/2</td>
<td>8d</td>
<td>78</td>
<td>0.113</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.148</td>
<td>78</td>
<td>0.192</td>
</tr>
<tr>
<td>1-1/2</td>
<td>8d</td>
<td>78</td>
<td>0.113</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.148</td>
<td>78</td>
<td>0.192</td>
</tr>
<tr>
<td>3/8</td>
<td>8d</td>
<td>78</td>
<td>0.113</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>0.148</td>
<td>78</td>
<td>0.192</td>
</tr>
</tbody>
</table>

Comparison of 1991 and Earlier Edition Lateral Design Values. Differences in lateral design values for single shear nailed connections in the 1991 and 1986 editions resulting from the change to the yield limit model and the use of individual species rather than fastener group values is illustrated in Table C12.3-1.

The consistiently higher 1991/1986 lateral design value ratios for the 3/4 and 1-1/2 inch side member thicknesses relative to the ratios for the 1/2 inch side member thickness reflect the use of side member thickness as a variable in the yield mode equations whereas previous lateral design values were independent of member thickness. The generally lower lateral design value ratios for the fasteners with diameters over 0.25 inches relative to those with diameters less than 0.17 inches represents the effect of the larger \( K_D \) factor (3.0) used in the yield mode equations for the former as compared to the factor (2.2) used with the latter.
**NDS Commentary**

For the joint configurations compared, the averages of the 1991/1986 lateral design value ratios were 1.13 for southern pine and 1.02 for spruce-pine-fir. This difference reflects the fact that southern pine was at the upper limit of its 1986 fastener group (specific gravity) class whereas spruce-pine-fir was near the lower limit of its class. By basing the dowel bearing strength used in the yield mode equations on the specific gravity of each species combination rather than basing lateral design values on specific gravity groups, the 1991 provisions tie nail lateral design values more closely to the performance capabilities of each species combination than was previously the case.

**12.3.2-Wood-to-Metal Connections**

12.3.2.1 From the 1944 through the 1986 edition, lateral design values for nails and spikes were increased 25 percent when metal rather than wood side plates were used (57). This is the same metal side plate adjustment previously used with wood screws over the same period and with bolts prior to the 1982 edition. Recent test results indicated the 25 percent increase for nailed wood-to-metal joints was conservative (165). In the 1991 edition, the effect of the metal side plates is accounted for directly by entering the Dowel bearing strength, \( F_{zb} \), and the thickness of the metal side plate in the yield mode equations of 12.3.1. Only yield Modes \( III_m \), \( III_s \) and IV are considered. The Mode \( I_s \) equation for bearing in the side member is not used as this property is considered separately in the design of metal parts (see 12.3.2.2 and Commentary for 7.2.3).

Tables 12.3E, 12.3F, 12.3G and 12.3H give lateral design values for joints made with steel box nails, common nails, threaded hardened nails and spikes, respectively, and steel side members ranging from 0.036 inches (20 gage) to 0.239 inches (3 gage) in thickness. Tabulated lateral design values for all side plate thicknesses are based on a dowel bearing strength of 45,000 psi applicable to ASTM A446 Grade A galvanized steel. The same nail and spike bending yield strengths, \( F_{yb} \), used to develop the wood-to-wood joint lateral design values in Tables 12.3A - 12.3D were used to develop the tabulated lateral design values for joints made with steel side plates.

**Comparison of 1991 and Earlier Edition Lateral Design Values.** Differences between 1991 and 1986 lateral design values for joints made with steel side plates and common nails are illustrated in Table C12.3-2.

The average of the 1991/1986 lateral design value ratios for the two plate thicknesses and three nail sizes compared in the table are 1.00 and 0.98 for the southern pine and spruce-pine-fir joints, respectively. Because the variation in thickness of typical steel side plates is small (0.036 to 0.239) relative to the variation in thickness of wood side members (1/2 to 1-1/2), plate thickness has a smaller effect on 1991 joint lateral design values than does wood side member thickness when both are compared to 1986 lateral design values. As with the wood-to-wood joints, the lower 1991/1986 lateral design value ratios for the larger diameter nails reflect the larger adjustment factor, \( K_D \), used with these fasteners compared to that used for the smaller diameter fasteners.

**12.3.2.2 (See Commentary for 7.2.3)**

12.3.2.3 Design values for joist hangers, tie downs and other similar products that involve wood-to-metal nailed joints often are established by testing connections made with the installed product rather than by use of the provisions of the Specification. As noted in the Commentary for 12.3.1, lateral design values tabulated in the 1991 and earlier editions are derived from nominal proportional limit level values and are about one-fifth short term ultimate test values. Design values for proprietary products such as hangers and other

<p>| Plate Thick- Nail Nail Nail Lateral Design Value, lbs |</p>
<table>
<thead>
<tr>
<th>Steel</th>
<th>Wood-to-Metal Single Shear Nail</th>
<th>in.</th>
<th>Penny- Diam.</th>
<th>Southern Pine</th>
<th>Spruce-Pine-Fir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nails:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Box Nails:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>8d</td>
<td>0.113</td>
<td>78 79 0.99</td>
<td>63 64 0.98</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20d</td>
<td>0.148</td>
<td>124 118 1.05</td>
<td>99 96 1.03</td>
<td></td>
</tr>
<tr>
<td>0.134</td>
<td>8d</td>
<td>0.113</td>
<td>90 79 1.14</td>
<td>74 64 1.16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20d</td>
<td>0.148</td>
<td>137 118 1.16</td>
<td>111 96 1.16</td>
<td></td>
</tr>
<tr>
<td>Common Nails:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>8d</td>
<td>0.131</td>
<td>103 98 1.05</td>
<td>83 80 1.04</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20d</td>
<td>0.192</td>
<td>176 174 1.01</td>
<td>141 142 0.99</td>
<td></td>
</tr>
<tr>
<td>0.134</td>
<td>8d</td>
<td>0.131</td>
<td>115 98 1.17</td>
<td>94 80 1.18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20d</td>
<td>0.192</td>
<td>188 174 1.08</td>
<td>152 142 1.07</td>
<td></td>
</tr>
<tr>
<td>0.60d</td>
<td>263</td>
<td>256 279 0.92</td>
<td>206 228 0.90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spikes:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>16d</td>
<td>0.207</td>
<td>192 194 0.99</td>
<td>154 159 0.97</td>
<td></td>
</tr>
<tr>
<td></td>
<td>60d</td>
<td>0.283</td>
<td>266 310 0.86</td>
<td>212 254 0.83</td>
<td></td>
</tr>
<tr>
<td>0.134</td>
<td>16d</td>
<td>0.207</td>
<td>203 194 1.05</td>
<td>163 159 1.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>60d</td>
<td>0.283</td>
<td>274 310 0.88</td>
<td>221 254 0.87</td>
<td></td>
</tr>
<tr>
<td>0.383</td>
<td>0.375</td>
<td>404 474 0.85</td>
<td>321 388 0.83</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.383</td>
<td>0.375</td>
<td>411 474 0.87</td>
<td>330 388 0.85</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

146  **Nails and Spikes**
connecting devices may be based on proportional limit test values or on maximum test values. It is the responsibility of the designer to determine the appropriateness of the procedures used to establish such design values, including the adequacy of reductions for load duration and variability, and the appropriateness of applying other adjustments given in the Specification to those design values (see Commentary for 1.1.1.4 and 7.1.1.4).

12.3.3-Double Shear Wood-to-Wood Connections

An increase in lateral design values for connections in double shear where the nail fully penetrated all members of a three-member joint was introduced in the 1960 edition of the Specification. An increase of one-third was allowed when each side member was not less than one-third the thickness of the main or center member and two-thirds when the thicknesses of the side members was equal to the thickness of the main member. Interpolation for intermediate side member thicknesses subsequently was provided for in the 1962 edition. In the 1982 edition, a clarifying provision was added to the Specification which applied the penetration requirements for single shear joints to the center member of three member joints. These provisions covering the design of nailed joints in double shear were carried forward unchanged to the 1986 edition.

In the 1968 edition, a separate adjustment for three member joints made with clinched nails was introduced. This new provision allowed a doubling of the single shear lateral design value for joints made with 12d or smaller size nails if the thickness of the side members was 3/8 inch or larger and the nails extended beyond the side member by at least three diameters and were clinched. This addition was based on tests of single shear and clinched and un clinched double shear joints made with plywood side members (108). In the 1971 edition, the clinching requirement for three member joints made with threaded hardened nails was waived if the side member, nail size and nail length requirements for doubling of single shear lateral design values were met. This change, which reflected the difficulty of clinching this type of fastener, was based on tests of single and double shear joints made with un clinched threaded hardened nails (171). The provisions allowing the doubling of single shear lateral design values for certain nailed joints in double shear were also carried forward through the 1986 edition.

In the 1991 edition, a doubling of the lateral design value for single shear joints is recognized for any three member wood-to-wood connection in which the thickness of the center or main member is greater than six times the nail or spike diameter. The provision is based on recent research which shows that the total yield mode load capacity of a three member joint is equivalent to twice that of a comparable two member or single shear joint (28). The six times fastener diameter requirement on the thickness of the center member in a three member joint is related to the requirement of 12.3.4 that the minimum nail or spike penetration into the main member be at least six times the fastener diameter.

Unlike previous editions, the 1991 provisions for three member wood-to-wood joints have no requirements on side member thickness to qualify for an increase in the single shear lateral design value if the third member meets the penetration requirements of 12.3.4. This reflects the fact that side member thickness is a variable in the yield mode equations that are used to establish lateral design values in the 1991 edition. Application of the $12D$ penetration requirement of 12.3.4 to the third member of a three member joint would exclude use of certain panel side member materials which qualified for the three member joint increase in previous editions. To provide for such applications, the clinched nail provisions of earlier editions are used as an exception to the 12.3.4 penetration requirements. Under this exception, three member joints made with 12d or smaller size nails and 3/8 inch or thicker side members qualify for a doubling of the applicable single shear lateral design value when the nail extends at least three diameters beyond the side member and are clinched. The waiver of the clinching requirement for threaded hardened nails that was in previous editions has been dropped as the increased strength of these fasteners relative to common steel wire nails has been accounted for directly in the yield mode equations. Clinching of both common and hardened nails is considered necessary for both to qualify for twice their respective single shear lateral design values.

12.3.4-Penetration Depth Factor, $C_d$

In the 1960 and earlier editions of the Specification, nails and spikes were required to penetrate the main member a minimum of two-thirds the fastener length for softwoods and one-half the fastener length for hardwoods to qualify for specified lateral design values (57,59). In the 1951 edition, provision was made for use of lesser penetrations if the lateral design value was reduced proportionately; but the minimum penetration was required to be at least one-half the nail length for softwoods and two-fifths the nail length for hardwoods. In the 1960 edition, the minimum penetration allowed for reduced lateral design values was changed to two-fifths the nail length for softwoods and one-third the nail length for hardwoods.
Beginning in the 1962 edition, penetration requirements were changed from a fastener length to a fastener diameter basis and the softwood-hardwood classes were dropped in favor of fastener groups based on specific gravity (see Commentary for 12.3.1 - Background). For full lateral design value, penetration of the fastener in the main member of 10 diameters for Group I species, 11 diameters for Group II species, 13 diameters for Group III species and 14 diameters for Group IV species was required. The minimum penetration allowed for reduced lateral design values was set at one-third the penetrations required for full lateral design values. Based on new recommendations (62), the 1962 penetration provisions represented a relaxation of previous requirements. Expressing penetration requirements in terms of nail diameter rather than nail length made it possible to take into account the different diameters of fasteners that are available for the same pennyweight and length. The 1962 penetration provisions were carried forward unchanged to the 1986 edition.

In the 1991 edition, lateral design values are given by individual species combinations rather than by fastener groups. With this change, the penetration requirement for full lateral design value has been simplified to twelve fastener diameters for all species. The minimum allowed penetration for reduced lateral design value has been changed from 3.3 to 4.7 fastener diameters to 6 diameters for all species. This increase in the minimum penetration requirement, based on consideration of new information available on how nails perform in joints (27), serves to account for the consolidation of the previous four separate group requirements into one and for the higher lateral design values obtained for some configurations from the new yield mode models.

### 12.3.5-End Grain Factor, \( C_{eg} \)

The use of a 0.67 adjustment factor on lateral design values for nails or spikes driven in the end grain has been a provision of the Specification since the 1944 edition. The adjustment is based on early research on joints made with softwood species (57).

### 12.3.6-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 or glued laminated timber members acting as the flanges. Such assemblies distribute horizontal forces acting on the flanges to vertical resisting elements (145). 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 (182). The diaphragm factor, \( C_{di} \), applies to both horizontal and vertical diaphragms.

Beginning with the 1960 edition of the Specification, an increase in normal load lateral design values for nails and spikes of 30 percent was recognized when these fasteners were used in diaphragm construction. The increase, which applied in addition to wind and earthquake load duration increases, was based on experience with wood diaphragms on the west coast designed using code approved nail shear values approximately 30 percent larger than those given in the Specification (180,181). The increase also was considered appropriate in view of the fact that the lateral design values provided in the Specification represented approximately 75 percent of joint proportional limit test values (one-fifth of maximum test values) (see Commentary for 12.3.1 - Background) and that structural diaphragms involve use of many nails acting together which was viewed as reducing variability effects.

In the 1977 edition, the provision for increasing lateral design values for nails and spikes used in diaphragm construction by 30 percent was revised to clarify that the adjustment applied only to lateral design values and not to withdrawal design values. The diaphragm adjustment provision was carried forward to the 1986 edition without change.

In the 1991 edition the adjustment for diaphragm use has been reduced to 10 percent in recognition of the change in the load duration adjustments for wind and earthquake loads from 1.33 to 1.6. The 10 percent factor provides for approximately the same effective wind and earthquake design load for diaphragms when used with the new load duration factor (1.6 x 1.1) as did the previous 30 percent factor when used with the previous load duration factor (1.33 x 1.3). If a 1.33 adjustment for wind or earthquake load continues to be used in diaphragm design, the 1.30 \( C_{di} \) factor should be applied to nail lateral design values.

### 12.3.7-Toe-Nail Factor, \( C_{tn} \)

The toe nail factor of 0.83 has been an adjustment to nail lateral design values since the 1951 edition. This factor is between the full lateral design value applicable to nails driven perpendicular to grain (side grain) surfaces and the two-thirds of full lateral design value applicable to nails driven in parallel to grain (end grain) surfaces.
For toe nails subject to lateral loads, the depth of penetration of the nail in the member holding the point may be taken as the vertically projected length of nail in the member as shown in Figure C12.3-1, or (see 12.1.3.2)

\[ p_L = L_n \cos 30° - \frac{L_n}{3} \]  

(C12.3-3)

where:

- \( p_L \) = vertical projection of penetration of nail in main member, in.
- \( L_n \) = length of nail, in.

For purposes of establishing the single shear lateral design value applicable to a toe nailed joint, the side member thickness shall be taken as the length of the nail in the side member (see Figure C12.3-1) or

\[ ts = \frac{L_n}{3} \]  

(C12.3-4)

where:

- \( ts \) = effective side member thickness when toe-nailing is used, in.

Equation C12.3-4 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. The effective side member thickness for nails driven at any angle to the face of the member being attached should not exceed the actual thickness of that member.

![Figure C12.3-1 Effective penetration and side member thickness for toe nails subject to lateral loads.](image)

12.4-PLACEMENT OF NAILS AND SPIKES

12.4.1-Edge Distance, End Distance, Spacing

Absence of splitting has been the performance criterion for placement of nails and spikes since the 1944 edition. Smaller nails can be placed closer to member ends and edges than larger nails as they are less likely to cause splitting. Where splitting can not be avoided, preboring of nail holes should be used (see 12.1.3.1).

In lieu of specific code requirements for end and edge distance for nails, Table C12.4-1 may be used to establish nailing patterns. Designers should note that species, moisture content and grain orientation will affect spacing (pitch) between fasteners in a row.

<table>
<thead>
<tr>
<th>Table C12.4-1 Nail Minimum Spacing Tables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Side Members</td>
</tr>
<tr>
<td>Not</td>
</tr>
<tr>
<td>Prebored</td>
</tr>
<tr>
<td>Prebored</td>
</tr>
<tr>
<td>Edge distance</td>
</tr>
<tr>
<td>End distance</td>
</tr>
<tr>
<td>- tension load parallel to grain</td>
</tr>
<tr>
<td>- compression load parallel to grain</td>
</tr>
<tr>
<td>Spacing (pitch) between fasteners in a row</td>
</tr>
<tr>
<td>- parallel to grain</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
</tr>
<tr>
<td>Spacing (gage) between rows of fasteners</td>
</tr>
<tr>
<td>- in-line</td>
</tr>
<tr>
<td>- staggered</td>
</tr>
<tr>
<td>Steel Side Members</td>
</tr>
<tr>
<td>Not</td>
</tr>
<tr>
<td>Prebored</td>
</tr>
<tr>
<td>Prebored</td>
</tr>
<tr>
<td>Edge distance</td>
</tr>
<tr>
<td>End distance</td>
</tr>
<tr>
<td>- tension load parallel to grain</td>
</tr>
<tr>
<td>- compression load parallel to grain</td>
</tr>
<tr>
<td>Spacing (pitch) between fasteners in a row</td>
</tr>
<tr>
<td>- parallel to grain</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
</tr>
<tr>
<td>Spacing (gage) between rows of fasteners</td>
</tr>
<tr>
<td>- in-line</td>
</tr>
<tr>
<td>- staggered</td>
</tr>
</tbody>
</table>

12.4.2-Multiple Nails or Spikes

Since the 1944 edition, the total design value for a connection made with more than one nail or spike has been determined as the sum of the allowable design values for the individual fasteners. In the 1992 edition, this summation provision is limited to only those nails or spikes in the joint which are of the same size and type (see Commentary for 7.2.2 and 7.1.1.1).
PART XIII: METAL CONNECTOR PLATES

13.1-GENERAL

13.1.1-General Provisions

Provisions for metal connector plates have been part of the Specification since the 1968 edition. Originally, plates which depended entirely on nails for lateral load transfer were required to be designed in accordance with the nail provisions of the Specification. The present procedure of allowing loads for such nailed plates to be based on either nail provisions or metal connector plate provisions was introduced in the 1977 edition.

13.1.2-Quality of Metal Connector Plates

ASTM Standard A446 has been the basis for the property requirements of galvanized sheet steel used in metal connector plates since provisions for this type of joint were introduced in 1968.

13.2-DESIGN VALUES FOR METAL CONNECTOR PLATES

13.2.1-Tests for Design Values

Basing the design value for a metal connector plate on the smaller of 1/1.6 the test load at slip of 0.03 inches and 1/3 the ultimate test load has been a provision since the 1968 edition. The ASTM D1761 test method includes provisions for metal connector plates which is a tooth holding lateral resistance test where two end-butted wood members are joined by metal connector plates on both sides. The 0.03 inch slip of such a joint is equivalent to a slip of 0.015 inches in a joint made of a single wood center member and metal side members. If a linear load-slip relationship is assumed below the 0.03 inch slip level, the wood-to-metal joint slip at a load equivalent to 1/1.6 of the load at 0.03 inch slip in the standard test is approximately 0.009 inches. However, 1/3 the ultimate load often will represent a much lower load than that associated with 1/1.6 the load at 0.03 inch slip and usually controls (69,174).

An alternate more comprehensive test method for tooth holding lateral resistance is given by the Truss Plate Institute (187). This national standard, in contrast to ASTM D1761, provides for testing perpendicular to grain, differentiates between gross versus net area testing, provides for testing metal connector plates embedded in the narrow face of the wood members, provides for the largest metal connector plate to be tested which will induce lateral tooth withdrawal failure, provides for a 7-14 day wait period to allow for fiber relaxation, and provides for matched specimen tests for partial plate embedment.

13.2.2-Different Species of Wood

In the 1986 and earlier editions, the results of tests on one species of lumber could be applied to all other species in the same fastener group. These fastener groups were defined in terms of specific gravity ranges (see Commentary for 12.3.1). The 1991 edition allows metal connector plate design values determined from test results for one species to be applied only to species having the same or higher specific gravity.
PART XIV: MISCELLANEOUS FASTENERS

14.1-DRIFT BOLTS AND DRIFT PINS

14.1.1-Withdrawal Design Values

Drift bolts and pins are unthreaded rods used to join large structural members where a smooth surface without protruding metal parts is desired.

Provisions for establishing withdrawal design values for round drift bolts or pins were included in the 1973 and earlier editions of the Specification. Such design values were determined from the equation

\[ W = 1200 G^2 D \]  \hspace{1cm} \text{ (C14.1-1)}

where:

- \( W \) = allowable withdrawal design value per inch of penetration, lbs
- \( G \) = specific gravity based on oven dry weight and volume
- \( D \) = bolt or pin diameter, in.

Equation C14.1-1 assumes the fastener is driven into a prebored hole having a diameter 1/8 inch less than the fastener diameter (57). The equation, which is generally applicable to all species (57), gives design values that are approximately one-fifth average ultimate test values (57,62).

Beginning in the 1977 edition, specific provisions for calculating withdrawal resistance of drift bolts or pins were dropped from the Specification. Other fasteners, such as spiral dowels (twisted rods with spirally grooved ridges) (4) which are not as dependent upon friction and workmanship for providing withdrawal resistance, should be used where withdrawal loads are a significant element in the design. However, where drift bolts or pins must be designed for withdrawal resistance, current good practice for such design is based on Equation C14.1-1 and the use of a predrilled hole that is 1/8 inch smaller in diameter than the fastener (4,66).

14.1.2-Lateral Design Values

In the 1944 through the 1982 editions of the Specification, lateral design values for drift bolts or pins were required not to exceed, and to be generally taken as less than, those for common bolts of comparable diameter (62). Use of additional penetration of the fastener into the members was recommended to compensate for the fact that the head, nut and washers contributing to the lateral resistance of common bolts are not present with drift bolts and pins. In the 1986 edition of the Specification, the general lateral design value provision for drift bolts and pins was replaced with a specific provision limiting lateral design values for such fasteners to 75 percent of the design value for common bolts of the same diameter. This specific requirement has been continued in the 1991 edition with the additional provision that use of increased penetrations to compensate for the absence of head, nut and washer is now mandatory. To implement these provisions, the length of the drift bolt or pin in the side member and in the main member, reduced by either the thickness of the bolt head and washer or the thickness of the nut and washer, may be taken as the thickness of these respective members when entering the yield mode equations of 8.2 or 8.3, or when entering Tables 8.2A, 8.2C, 8.3A or 8.3C of the Specification.

It is to be noted that end distance, edge distance and spacing requirements, and group action adjustments that are applicable to common bolts are also applicable to drift bolts and drift pins.

14.2-SPIKE GRIDS

Spike grids were first referenced in the Specification in the 1977 edition. These connectors consist of malleable iron grids with blunt teeth or spikes protruding outward from both sides at grid intersection points (179). Square grids can be flat on both sides or flat on one side and curved on the other; the latter for attachment to a pole or pile. Circular grids have teeth protruding around the perimeter of the connector. Grids are approximately 4-1/8 inches square or 3-1/4 inches in diameter and have an overall depth of less than 1-1/4 inch. Grids, which behave similarly to toothed-rings (see Commentary for 10.1 - Background) are embedded in the members being joined by tightening 3/4 or 1 inch center bolts passing through the center of the grid through a prebored hole in the wood members.

Spike grids are used in wood-to-wood connections in railway and highway trestle construction and similar uses where more than bolt strength is required and where resistance to loosening from vibration, impact and load reversals is needed (179). Grids have the advantage over split ring and shear plate connectors in not requiring precut grooving. Design values in excess of 3000 pounds for one spike grid and bolt in single shear can be obtained (178).
PART XV: SPECIAL LOADING CONDITIONS

15.1-LATERAL DISTRIBUTION OF A CONCENTRATED LOAD

15.1.1-Lateral Distribution of a Concentrated Load for Moment

The lateral distribution factors for moment in Table 15.1.1 have been part of the Specification since the 1944 edition. These factors, keyed to the nominal thickness of the flooring or decking involved (two to six inches thick) and the spacing of the stringers or beams in feet, S, are based on recommendations of the American Association of State Highway and Transportation Officials (2). The factors for moment are:

<table>
<thead>
<tr>
<th>Material</th>
<th>S/N</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 inch plank</td>
<td>S/4.0</td>
</tr>
<tr>
<td>4 inch nail-laminated</td>
<td>S/4.5</td>
</tr>
<tr>
<td>6 inch nail-laminated</td>
<td>S/5.0</td>
</tr>
<tr>
<td>Structural concrete</td>
<td>S/6.0</td>
</tr>
</tbody>
</table>

(For all cases when S > N, where N is the denominator of the moment factors, load on adjacent stringers is based on the reactions of the load assuming flooring acts as simple beam)

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 (2). Nails typically penetrate into two adjacent pieces, are staggered and are alternated on the top and bottom edges (53). Flooring is typically attached to stringers by toe nailing.

The factors obtained from the foregoing S/N ratios 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 midspan of the beam or stringer closest to the wheel load (2,53).

When the average spacing of the beams or stringers in a one traffic lane bridge exceeds the denominator (N) of the ratio (S/N), the concentrated live load on the two beams or stringers adjacent to the applied load is taken as the reactions of the load assuming the flooring or decking between the beams or stringers is acting as a simple beam. Examples C15.1-1 and C15.1-2 illustrate these provisions.

---

Example C15.1-1

Single lane bridge with 2 by 6 in. laminated decking edge bearing on 10 by 24 in. stringers spaced 23 in. apart. Total load on front and rear trailer axles is 65,000 lb. What is the concentrated load to be used for calculation of the stringer bending moment associated with total load on the trailer axles?

\[ S = 1.92 \text{ ft} \]

From Table 15.1.1:

<table>
<thead>
<tr>
<th>Lateral distribution factor</th>
<th>= S/N</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.92/5.0</td>
</tr>
<tr>
<td></td>
<td>0.384</td>
</tr>
</tbody>
</table>

Applied wheel load = front and rear

<table>
<thead>
<tr>
<th>axle, (one side)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2 (65,000)</td>
</tr>
<tr>
<td>32,500 lb</td>
</tr>
</tbody>
</table>

Concentrated load, \( P = 0.384 (32,500) \)

(for bending moment calculation) = 12,480 lb acting at midspan of stringer

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 simple beam (2). This procedure is comparable to that used where \( S > N \).

Lateral distribution factors determined in accordance with 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 (53). These tests indicate the factors are somewhat conservative, particularly at \( S/N \) ratios greater than 0.60.

For bridges of two or more traffic lanes, the American Association of State Highway and Transportation Officials (2) provides other lateral distribution factors.
Example C15.1-2
Consider the previous example but with stringers spaced 72 in. apart. What is the concentrated load to be used for calculation of the stringer bending moment associated with the 65,000 lb load on the trailer axles when the wheel load is 6 in. from the nearest stringer? 2 ft from the nearest stringer?

6 in. from stringer:

<table>
<thead>
<tr>
<th>S/N</th>
<th>Reaction</th>
<th>Applied wheel load</th>
<th>Concentrated load, P (for bending moment)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculation</td>
</tr>
<tr>
<td></td>
<td>= P (5.5/6.0) = 0.917 P</td>
<td>front and rear axle, (one side)</td>
<td>0.917 (32,500)</td>
</tr>
<tr>
<td></td>
<td>= 1/2 (65,000) = 32,500 lb</td>
<td>29,792 lb acting at midspan of stringer</td>
<td></td>
</tr>
</tbody>
</table>

2 ft from stringer:

<table>
<thead>
<tr>
<th>S/N</th>
<th>Reaction</th>
<th>Applied wheel load</th>
<th>Concentrated load, P (for bending moment)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Calculation</td>
</tr>
<tr>
<td></td>
<td>= P (4.0/6.0) = 0.667 P</td>
<td></td>
<td>0.667 (32,500)</td>
</tr>
<tr>
<td></td>
<td>= 32,500 lb</td>
<td>21,678 lb acting at midspan of stringer</td>
<td></td>
</tr>
</tbody>
</table>

15.1.2-Lateral Distribution of a Concentrated Load for Shear

The lateral distribution factors for shear in Table 15.1.2 have been provisions of the Specification since the 1944 edition. These factors relate the lateral distribution of concentrated load at the center of the beam or stringer span as determined under 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 midspan is closely described by the following relation:

\[ P_{1/4} = -1.807 + 1.405 \log (P_m) \]  \hfill (C15.1-1)

where:

- \( P_{1/4} \) = percentage of load at 1/4 point of center beam
- \( P_m \) = percentage of load at midspan of center beam
- \( (S/N) \) from Table 15.1.1 or other basis

Values of \( P_{1/4} \) from 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 Table 15.1.2 values and indicate they are conservative at S/N ratios above 0.50 (53).

15.2-SPACED COLUMNS

15.2.1-General

Background.

As used in the Specification, spaced columns refer to two or more individual members oriented with their longitudinal axis parallel, separated at the ends and in the middle portion of their length by blocking and joined at the ends by split ring or shear plate connectors capable of developing required shear resistance. The end fixity developed by the connectors and end blocks increases the load carrying capacity in compression parallel to grain of the individual members only in the direction perpendicular to their wide faces (parallel to the \( d_f \) dimension in Figure 15A of the Specification).

Design provisions for spaced columns (62) have remained essentially unchanged since the 1944 edition of the Specification except for changes in the general column equations applicable to all wood columns (see Commentary for 3.7.1.5). In this regard, the original 1944 end fixity factors, the load capacity criteria for connectors in end spacer blocks, and the slenderness ratio limits for individual members (\( \ell_f / d_f \)) and for the middle spacer block (\( \ell_f / d_f \)) have been continued through to the 1991 edition.

15.2.1.1 In the design of spaced columns, the allowable capacity for an individual member is determined in accordance with the provisions of 15.2 and other applicable provisions of the Specification and then the associated design load for each member is summed to obtain the allowable load for the column. It is to be understood that the actual compression stress parallel to grain, \( f_c' \), on the members of the spaced column is not to exceed the allowable compression design value parallel to grain, \( F_c' \), for these members based on all provisions of 3.6 and 3.7 except as modified or extended by the provisions of 15.2. The net section requirements of 3.6.3 are to be applied to the members of spaced columns.
15.2.1.2 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 simple 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 simple solid column because of the 50 percent reduction in effective column length (178).

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 allowable 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 3.7.

15.2.2 Spacer and End Block Provisions

15.2.2.1 Where more than one spacer block is used, the distance \(l_2\) (see Figure 15A) is the distance from the center of one spacer block to the centroid of the connectors in the nearest end block.

15.2.2.2 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 through spiking, bolting or other means to assure the compression members maintain their spacing under load (62). A web member joined by connectors to two chords making up a 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 \(l_2/d_2\) ratio limit of 40 (see 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. Use of connectors to join multiple spacer blocks to compression members has been a continuous requirement since the 1944 edition.

15.2.2.3 Spaced columns are used as compression chords in bowstring and other large span trusses (178). 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.

Until the 1962 edition, spaced-column web members were specifically provided for in the Specification by considering joints at the tension chord to be stayed laterally by the tautness of the tension chord and by the lateral bracing customarily used between trusses. Under present provisions, spaced-column web members may be designed using the procedures of 15.2 if the joints at both ends of the web member are laterally supported.

15.2.2.4 Prior to the 1962 edition, end and spacer blocks were permitted to have a thickness down to one-half that of the individual compression members if the length of the blocks was made inversely proportional to the thickness in relation to the required length of a full-thickness block. Since 1962, the thickness of end and spacer blocks is required to be equal to or larger than the thickness of the compression members (62).

It is to be noted that 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 Part X of the Specification. 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.

15.2.2.5 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 (62).

The equations for end spacer block constants in 15.2.2.5 are based on \(K_S\) of zero when \(l_1/d_1 \leq 11\) and a \(K_S\) equal to one-fourth a clear wood or basic compression design value parallel to grain for the species group when \(l_1/d_1 > 11\) (62). The equations give \(K_S\) values for intermediate slenderness ratios based on linear interpolation between these limits.

Except for modifications in load duration adjustment after World War II and conversion to a normal loading basis (see Commentary for 2.3.2), the limiting \(K_S\) values of 468, 399, 330 and 261 for species groups
A, B, C and D (defined in Table 10A of the specification), respectively, have remained unchanged since the 1944 edition. These values represent one-fourth the normal load, unseasoned basic compression design value parallel to grain applicable to representative species in each group in 1955 (62). Index basic compression design values parallel to grain used 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 firs-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, \( K_S \).

15.2.3-Column Stability Factor, \( C_p \)

15.2.3.1 Effective column length for spaced columns is determined in accordance with Figure 15A and adjusted by any applicable buckling length coefficient, \( K_e \), greater than one as specified in 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.

15.2.3.2 The slenderness ratio limits for spaced columns have been part of the Specification since the 1944 edition. The limit of 80 on the slenderness ratio \( l_1/d_1 \) for the individual members is a conservative good practice recommendation that effectively provides a safety factor on the spaced column fixity coefficient, \( K_x \). The limit of 50 on the slenderness ratio \( l_2/d_2 \) is the limit applied to simple solid columns (see 3.7.1.4). The limit of 40 on the \( l_1/d_1 \) 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.

15.2.3.3 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 simple solid columns (see 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 \( (F_{ce}') \) of the column stability factor \( (C_p) \), the tabulated compression design value parallel to grain \( (F_c) \) and all other applicable adjustment factors (see 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 3.1.2) when comparing with the \( C_p \) adjusted allowable compression design value parallel to grain, \( F_{ce}' \).

In all 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 tabulated compression design value parallel to grain and all applicable adjustment factors except the column stability factor (see 3.6.3).

15.2.3.4 Use of the lesser allowable compression design value parallel to grain, \( F_{ce}' \), for a spaced column having members of different species or grades to all members is a conservative good practice recommendation introduced in the 1977 edition. Where the design involves the use of compression members of different thicknesses, the \( F_{ce}' \) value for the thinnest member is to be applied to all other members.

15.2.3.5 The actual compression stress parallel to grain, \( f_c \), in spaced columns also is to be checked in all cases against the allowable compression design value parallel to grain, \( F_{ce}' \), based on the slenderness ratio \( l_2/d_2 \) and a \( C_p \) factor calculated in accordance with the provisions of 3.7 without use of the spaced column fixity coefficient, \( K_x \). Use of connectors to join individual compression members through end blocks increases the load carrying capacity of spaced columns only 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 allowable compression stress parallel to grain, \( F_{ce}' \), based on the slenderness ratio \( l_2/d_2 \) will control.

15.2.3.6 (See Commentary for 3.7.1.6)

15.2.3.7 Design provisions for spaced beams joined by end blocks and connectors are not included in the Specification. The beam-column equations of 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.
15.3-BUILT-UP COLUMNS

Background

As with spaced columns, built-up columns obtain their efficiency by increasing the buckling resistance of the individual laminations. The closer the laminations of a mechanically fastened built-up column deform together (the smaller the amount of slip occurring between laminations) under compressive load, the greater the relative capacity of that column compared to a simple solid column of the same slenderness ratio made with the same quality of material.

Prior to the 1991 edition, no specific provisions for the design of nailed or bolted layered (all laminations of same face width) columns were included in the Specification. However, from 1944 through the 1986 editions, the Specification referenced guidelines (57,62) for the design of built-up columns made with pieces connected by nails, bolts or other mechanical fastenings (57,62). The column configurations covered by these guidelines were rectangular sections consisting of either laminations boxed around a solid core or parallel laminations with edge cover plates.

Based on tests of columns of various lengths (156,157), the referenced guidelines expressed the capacity of the two equivalent column types as a percentage of the strength of one-piece columns made with material of the same grade and species. Such efficiencies ranged from a value of 82 percent at an \( \ell/d \) ratio of 6, decreasing to a low of 65 percent at an \( \ell/d \) of 18 and then increasing to 82 percent at an \( \ell/d \) of 26. Although the test columns were made with certain specific fastening schedules (156), the guidelines provided no information on the spacing and number of fastenings required to achieve the indicated efficiencies.

In 1977, new methodology for the design of layered, spaced and braced columns made with various types of mechanical fasteners was developed in Canada (111). The methodology enables the determination of 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 describe column buckling behavior using the tangent modulus theory expression proposed by a Finnish researcher in 1956 and which is now incorporated in the Specification as the general design procedure for all types of wood columns (see Commentary for 3.7.1.5). The formulas are entered with fastener load-slip values based on beam-on-elastic-foundation principles (99).

The comprehensive design methodology for built-up layered columns was simplified in 1989 to a form that permitted codification of the procedures for these members in design specifications (110). The provisions for built-up columns in 15.3 of the Specification are based on this simplified methodology.

15.3.1-General

The provisions of 15.3 apply only to layered columns in which the laminations are of the same width and are unjointed. The limitations on number of laminations are based on the range of columns that were tested in the original research (111) that met the connection requirements of 15.3.3 and 15.3.4. The minimum lamination thickness requirement assures use of lumber for which approved design values are available in the Specification.

15.3.2-Column Stability Factor, \( C_p \)

Provisions in 15.3.2 are the same as those applicable to simple solid columns in 3.7.1 except for the addition of the column stability coefficients, \( K_f \), in equation 15.3-1.

When nailed in accordance with the provisions of 15.3.3, the capacity of built-up columns has been shown to be more than 60 percent of that of an equivalent simple solid column at all \( \ell/d \) ratios (110). Efficiencies are higher for conforming 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 15.3.4 is more than 75 percent for all \( \ell/d \) ratios (110). 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 the former.

In accordance with 3.7.1.3, Equation 15.3-1 is entered with a value of \( F_{ce}^E \) based on the larger of \( \ell_{e1}/d_1 \) or \( \ell_{e2}/d_2 \), 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 (110) has shown that buckling about the weak axis of the individual laminations is a function of the amount of slip and load transfer that occur at fasteners between laminations. When the controlling slenderness ratio is the strong axis of the individual laminations, \( \ell_{e1}/d_p \) then \( K_f = 1.0 \). It is also necessary to compare \( C_p \) based on \( \ell_{e1}/d_1 \) and \( K_f = 1.0 \) with \( C_p \) based on \( \ell_{e2}/d_2 \) and \( K_f = 0.6 \) or 0.75
to determine the allowable compression design value parallel to grain, \( F_c' \).

15.3.3-Nailed Built-up Columns

15.3.3.1 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, \( K_f \), of 60 percent was established (110). 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 (41) 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.

In the nailing requirements of 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 off set 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.

15.3.4-Bolted Built-up Columns

15.3.4.1 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, \( K_f \), was established (110). Maximum end distance limits in (c) are good practice recommendations (41) 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 8.5.

As with nailed columns, a bolt row refers to those nails 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.

15.4-WOOD COLUMNS WITH SIDE LOADS AND ECCENTRICITY

15.4.1-General Equations

Equations for wood columns with side loads and eccentricity have been included in the Specification since the 1944 edition. Based on theoretical analyses (223), these equations remained essentially unchanged through the 1986 edition except for changes made in column design provisions and the interaction equations over this period (see Commentary for 3.7.1.5 and 3.9.2).

The equation in 15.4.1 for combined bending and eccentric axial compression loads reflects the introduction of the new continuous column equation and the new beam-column equation in the 1991 edition. The equation is an expansion of the interaction equation given in 3.9.2 to the general case of any combination of side loads, end loads and eccentric end loads (229).

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 15.4-1 reduces to

\[
\left( \frac{f_c}{F_c'} \right)^2 + \frac{f_{b1} + f_c(6e_t/d_t)[1 + 0.234(f_c/F_{cE1})]}{F_{b1}' [1 - (f_c/F_{cE1})]} \leq 1.0 \]

(C15.4-1)

or

\[
\left( \frac{f_c}{F_c'} \right)^2 + \frac{f_{b1} + f_c(6e_t/d_t)[1.234 - 0.234 C_{ml}]}{C_{ml} F_{b1}'} \leq 1.0 \]

(C15.4-2)

where:

\( e_t = \) eccentricity

\( C_{ml} = \) moment magnification factor = \( 1 - f_c/F_{cE1} \)

and other symbols as defined in the Commentary for 3.9.2.

For a long column (\( J = 1 \)), the comparable equation in the 1986 and earlier editions was
Example C15.4-1

A No. 1 Douglas Fir-Larch 2x10 is used as the upper chord of a roof truss. The truss is designed such that the flexural stress in the member from roof loads (DL+SL) is twice the axial compressive stress in the member from the truss reactions from roof and ceiling loads (DL+SL). At the panel points, the axial force acts eccentrically at a point 1.5 in. above the center of the width of the member. The top edge of the chord is laterally supported with $Q_e = 94$ in. and $d_l = 0$ in. Determine the allowable axial force in the member based on the interaction of bending and compression.

$F_b = 1000$ psi  $C_F = 1.1$  $C_D = 1.15$  
$F_c = 1450$ psi  $C_F = 1.0$  $E = 1,700,000$ psi  
$A = 13.88$ in$^2$  $S = 21.39$ in$^3$

**Allowable Compression Design Value**

Parallel to Grain (3.6, 3.7)

$F_c' = F_c C_D C_F = (1450)(1.15)(1.0) = 1668$ psi

$\ell e_2 / d_2 = 0$ (fully supported)

$\ell e / d = \ell e_2 / d_2 = (94) / (9.25) = 10.16 < 50$

$K_{CE} = 0.3$

$F_{CE} = K_{CE} E' = (0.3)(1,700,000) / (10.16)^2 = 4940$ psi

$F_c = 1 + \frac{(F_{CE} / F_c')}{2c} \sqrt{\frac{1 + (F_{CE} / F_c')}{2c} - \frac{F_{CE} / F_c'}{c}}$

$= \frac{1 + 4940 / 1668}{2(0.8)} \sqrt{\frac{1 + 4940 / 1668}{2(0.8)} - \frac{4940 / 1668}{0.8}} = 0.918$

$F_c' = F_c C_D C_F P = (1450)(1.15)(1.0)(0.918) = 1530$ psi

**Allowable Bending Design Value** (3.3.3)

Full lateral support along narrow face, $C_L = 1.0$ (3.3.3.3)

$F_b' = F_b C_D C_F = (1000)(1.0)(1.15) = 1265$ psi

**Combined Bending and Axial Compression** (15.4.1)

No minor axis bending

$F_c' = \frac{f_c}{F_c'} + \frac{f_{b1} + f_c (6 e_2 / d_2)(1 + 0.234 (f_c / F_{CE1}))}{F_{b1}'} \leq 1.0$

For $f_{b1} = 2 f_c$, $e = 1.5$ in., $d_l = 9.25$ in.

$\frac{f_c}{F_c'} + \frac{2 f_c + f_c (6(1.5)(9.25)(1 + 0.234 (f_c / 4940))}{(1265)(1 - (f_c / 4940))} \leq 1.0$

Solving for $f_c$ by iteration

$f_c = 368.75$ psi

$f_b = 2 f_c = 737.5$ psi

$P_{allowable} = f_c A = (368.75)(13.88) = 5118$ lb

**Maximum axial compression force in 2x10 chord member based on interaction of bending and compression = 5118 lb**

For the case of a bending load on the narrow face and an eccentric ($e_2$) axial load producing a moment perpendicular to the plane of bending due to the edgewise load, the applicable interaction equation is

$\frac{f_c}{F_c'} + \frac{f_{b1} + f_c (6 e_2 / d_2)(1.234 - 0.234 C_{m1})}{C_{m1} F_{b1}'} \leq 1.0$

where:

$C_{m1} = moment$ magnification factor $= 1 - f_c / F_{CE1}$

$C_{m2} = moment$ magnification factor $= 1 - f_c / F_{CE2} - (f_{b1} / F_{b1})^2$
or in expanded form

\[
\left( \frac{f_c}{F_e} \right)^2 + \frac{f_{bl}}{F_{bl} \left[ 1 - \left( \frac{f_c}{F_e} \right) \left( \frac{F_{bs}}{F_{bs}} \right) \right]} + \frac{f_{cl} \left( 6e_2/d_2 \right) \left[ 1 + 0.234 \left( \frac{f_c}{F_e} \right) + 0.234 \left( \frac{f_{bl}}{F_{bl}} \right) \right]}{F_{bs} \left[ 1 - \left( \frac{f_c}{F_e} \right) \left( \frac{F_{bs}}{F_{bs}} \right) \right] - \left( \frac{f_{bl}}{F_{bl}} \right)^2} \leq 1.0
\]  

(C15.4-6)

A comparable equation for this loading case was not provided in earlier editions.

### 15.4.2-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 and has been a provision of the Specification since the 1944 edition. The calculated side load, \( P_s \), acting at midspan is considered to produce a moment at this location equal to the moment produced by the load on the bracket, \( P_a \), times three-fourths the distance from the top of the bracket to the base of the column (\( \ell_p \)) divided by the column length (\( l \)).

When the bracket is at the top of the column, results obtained by entering Specification Equation 15.4-1 (or 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, Specification 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 section 15.4.2 assumes the moment due to the bracket load decreases linearly from the point of application to zero at the column base. The procedure of section 15.4.2 may be used for those columns where the point of application of the eccentric axial load is outside the column cross-section (see Example C15.4-2).

### Example C15.4-2

A 4x10 roof rafter is supported on an L2x2x1/4 steel side bracket (3.5 in. long), connected to a 10 ft long 4x4 column using two 5/8 in. bolts. There is a 2000 lb reaction at the bracket from the rafter due to dead load plus snow loads on the roof. Both members are No. 2 Douglas Fir-Larch. Check the adequacy of the connection and joined members.

![Diagram of 4x10 rafter with L2x2x1/4 bracket](image)

\( F_c = 875 \text{ psi} \)  \( C_p = 1.5 \text{ (4x4)} \)  \( C_D = 1.15 \)  \( (\text{Table 4A}) \)

\( F_e = 1300 \text{ psi} \)  \( C_p = 1.15 \text{ (4x4)} \)  \( E = 1,600,000 \) psi

\( F_v = 95 \text{ psi} \)  \( F_{es} = 625 \text{ psi} \)

**Check Rafter at Member End**

Assume member is adequate for bending and deflection

**Shear**  
\( F_{ve} = F_e C_p C_D C_t = (95)(1.15)(1.0)(1.0) = 109 \text{ psi} \)

\( V_{recession} = 2000 \text{ lb} \)

\( F_v = \frac{3V}{2bd} = \frac{(3)(2000)}{(2)(3.5)(9.25)} = 93 \text{ psi} < F_{ve} = 109 \text{ psi ok} \)

**Bearing**  
\( F_{es} = F_{es} C_p C_D C_t = (625)(1.0)(1.0) = 625 \text{ psi} \)

**Reaction** = 2000 lb

**Bearing area of bracket** = (2.0)(3.5) = 7.0 in\(^2\)

\( f_{es} = \frac{(2000)}{(7.0)} = 286 \text{ psi} < F_{es} = 625 \text{ psi ok} \)

**Rafter is adequate at member end**

**Check Bracket Bolts in Column**

For 5/8-in. bolts (side by side) in single shear with \( l_a = 3.5 \text{ in.} \) and \( l_v = 1/4 \text{ in.} \)

\( Z_{ll} = 1130 \text{ lb/bolt} \)  

\( Z' = Z_{ll} C_D C_p C_t = (1130)(1.15)(1.0) = 1300 \text{ lb} \)  

\( P = 2000 \text{ lb} < n Z' = (2)(1300) = 2600 \text{ lb ok} \)

**Check Column by Two Methods**

1. Assume eccentric load acts at the top end of the column

**Compression**  
\( F_c' = F_c C_p C_D = (1300)(1.15)(1.0) = 1719 \text{ psi} \)

**Check net section at bracket (no buckling)**

\( (3.7.1.5) \)  

(cont.)
Example C15.4-2 (cont.)

\[ A_{gros} = (3.5)(3.5) = 12.25 \text{ in}^2 \]
\[ A_{net} = (3.5)(3.5 - 2(5/8 + 1/16)) = 7.44 \text{ in}^2 \]
\[ f_c' = P/A_{net} = 2000/7.44 \]
\[ = 269 \text{ psi} < F_{ct} = 1719 \text{ psi} \quad \text{ok} \]

Check gross section (potential buckling) \((3.6.3, 3.7.1)\)

\[ \frac{A_{c}}{c} = \frac{(3.5)(3.5)}{(3.5 - 2(5/8 + 1/16))} = 34.3 < 50 \]
\[ K_{ct} = 0.3 \]

\[ F_{ct} = \frac{K_{ct}E'}{(l'/d)^2} = \frac{(0.3)(1,600,000)}{(34.3)^2} = 408 \text{ psi} \]

\[ C_p = \frac{1+(F_{ct}/F_{ct}^*)}{2c} \left[ \frac{1+(F_{ct}/F_{ct}^*)}{2c} \right]^2 - \frac{F_{ct}/F_{ct}^*}{c} \]
\[ = \frac{1+408/1719}{2(0.8)} \left[ \frac{1+408/1719}{2(0.8)} \right]^2 - \frac{408/1719}{0.8} \]
\[ = 0.2244 \]

\[ F_{ct} = F_{ct}C_pC_lC_p = (1300)(1.15)(1.0)(0.2244) = 386 \text{ psi} \]

\[ f_c' = P/A_{ct} = 2000/12.25 = 163 \text{ psi} < F_{ct} = 386 \text{ psi} \quad \text{ok} \]

**Bending** \((3.3)\)

Since \(d = b = 3.5 \text{ in.}, C_e = 1.0 \)

\[ F_{bt} = F_{ct}C_pC_lC_p = (875)(1.15)(1.0)(1.5) = 1509 \text{ psi} \quad \text{(2.3.1)} \]

Check net section at bracket \((3.2.1)\)

\[ S_{net} = (3.5 - 2(5/8 + 1/16))(3.5)^2/6 = 4.339 \text{ in}^3 \]


\[ f_b = M_{bracket}/S_{net} = (5076)/(4.339) = 1170 \text{ psi} < F_{bt} = 1509 \text{ psi} \quad \text{ok} \]

Check gross section (at end)

\[ S_{gross} = 7.146 \text{ in}^3 \]

\[ M_{net} = P \ell = (2000)(2.75) = 5500 \text{ in-lb} \]

\[ f_b = M_{net}/S_{gross} = (5500)/(7.146) = 770 \text{ psi} < F_{bt} = 1509 \text{ psi} \quad \text{ok} \]

Combined Bending and Axial Compression \((15.4.1)\)

\[ \left( \frac{f_c'}{F_{ct}'} \right)^2 + \frac{f_\ell}{F_{ct}'} \left( 1 - \frac{f_c'}{F_{ct}'} \right) \leq 1.0 \]

\[ \left( \frac{163}{386} \right)^2 + \frac{533}{1509} \left( 1 - \frac{163}{386} \right) = 0.77 < 1.0 \quad \text{ok} \]

Column satisfies NDS provisions by Method 2
REFERENCES


References

57. Forest Products Laboratory. Wood handbook. Washington, DC: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory; 1935.


63. Forest Products Laboratory. Forest Products Laboratory policy on basic stresses and working stresses. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory; 1960.


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ADDENDUM to the

1991 COMMENTARY

on the

NATIONAL DESIGN SPECIFICATION® (NDS®) FOR WOOD CONSTRUCTION

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.

February 1999

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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 $E$, $F_p$, $F_i$ 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:

\[ K_{be} = 0.745 - 1.225(COV_e) \]  

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

\[ K_{be} = 0.745 - 1.225(COV_e) \]  

(See 1991 Commentary 3.3.3 and 3.7.1.5)
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 COVE of 0.25, has been changed from 0.438 to 0.439, and that for COVE 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 COVE 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

\[
f_v = 3V/[2(A-(A-A_n)e/d_n)]
\]

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 \( (K_{eE}) \) 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 \( K_{eE} \) 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
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
\[ K_{cE} = 0.510 - 0.839(COV_e) \] (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, $F_{c\perp}$

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

$$F_{c\perp,0.02} = 5.6 + 0.73F_{c\perp}$$

(CA4.2-1)

to

$$F_{c\perp,0.02} = 0.73F_{c\perp}$$

(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 breadth 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

$$E_{0.05} = K_T E_{table}$$

(CA4.4-1)

where:

$$K_T = [1 - 1.645(COV_e)]$$

(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\textsubscript{E} of 0.25) and 0.82 for products with a COV\textsubscript{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_{bx} 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_{by} 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_{y}, is less than that for beams loaded perpendicular to the wide face of the laminations, E_{x}. 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, $F_{yfb}$, 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%

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

In the current edition, the wet and partially seasoned conditions have been combined and the moisture conditions redesignated as "$\leq 19\%" and "$>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 \(\leq 19\%\) 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 (\(\leq 19\%\)) 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 $F_{2e}$, 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, $Z_9$, 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, $F_{cem}$, 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 |
|---------------------------------|---------------------------------|-------|--------|
| 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           | 1000 | 1140 | 1.14  | 450 | 450  | 1.00  |
|                                | 3/4         | 1330 | 1760 | 1.32  | 530 | 520  | 0.98  |
|                                | 1           | 540 | 570  | 1.06  | 320 | 330  | 1.03  |

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 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, 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, and Mode III, 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 toenailed 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

\[
Z'_a = \frac{[(1 + K \sin^2 \alpha)(W'pZ')]}{[(W'p)\cos \alpha + (Z') \sin \alpha]} \quad (CA12.3-1)
\]

where:
- \( Z'_a \) = 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)
- \( \alpha \) = 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:

\[
\frac{W}{W'p} + \frac{Z'}{Z'} \leq 1 \quad (CA12.3-2)
\]

where \( W \) is the connection withdrawal force and \( Z \) is the connection lateral force. In equation 12.3-6 and CA12.3-2, \( Z'_a, W'p \) 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

\[
Z'_a = \frac{(W'p)Z'\sin^2 \alpha}{(W'p)\cos \alpha + (Z') \sin \alpha} \quad (CA12.3-3)
\]

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.
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:

\[ P_r = 280p^{0.32}n_Rn_C \]  

\[ \text{(CA13.2-1)} \]

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). 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 Pr values and Pw 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 Pr or applicable tabular Pw 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

\[ Q_r = 160p^{0.32}n_Rn_C \]  

(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<sub>s</sub>

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<sub>r</sub> and Q<sub>r</sub>) 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, $K_{el}$, 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 $\ell_{c1}/d_1$ or $\ell_{c2}/d_2$ 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 $\ell/d$ 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 ($K_{el}$) 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 ($\ell_{c1}/d_1$), 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 ($\ell_{c2}/d_2$ 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 $F_{el}$ is based on the weak axis slenderness ratio ($\ell_{c2}/d_2$). A new $K_f$ of 1.0 for both nailed and bolted columns is established for calculating $C_p$ values when $F_{el}$ is based on the strong axis slenderness ratio ($\ell_{c1}/d_1$). 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.
REFERENCES


Only one primary building material comes from a renewable resource; cleans the air and water, providing habitat, scenic beauty and recreation as it grows; utilizes nearly 100% of its resource for products; is the lowest of all in energy requirements for its manufacturing; creates fewer air and water emissions than any of its alternatives; and is totally reusable, recyclable and 100% biodegradable: wood. And it has been increasing in US net reserves since 1952, with growth exceeding harvest in the US by more than 30%.

Wood Works™, the argument is growing every day.