AF&PA/ASCE 16-95

# ASCE STANDARD

American Forest & Paper Association
American Society of Civil Engineers

# Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction

ASCE 1852

AF&PA/ASCE 16-95

American Forest & Paper Association
American Society of Civil Engineers

# Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction

Published by the American Society of Civil Engineers
345 East 47th Street
New York, NY 10017-2398

**ABSTRACT:**

This first *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction* was prepared as a joint activity between the American Society of Civil Engineers and the American Forest & Paper Association (AF & PA) to provide design provisions based on reliability theory. This standard was the result of deliberations of a team of structural engineers and wood material scientists with wide experience and high professional standing. The team included professionals from private practice, industry, government, and universities. By reflecting this current state of knowledge, the *LRFD Standard* offers uniform practice in the design of engineered wood structures. It specifically covers such topics as: 1) Design requirements; 2) tension members; 3) compression members and bearing; 4) flexural members; 5) members with combined bending and axial loads; 6) mechanical connections; 7) structural-use panels; 8) shear walls and diaphragms; and 9) serviceability. In addition, the Appendices address issues such as: 1) Resistance of spaced columns; 2) glued laminated timber; 3) ponding; 4) qualification of fasteners and connectors; 5) resistance of shear plates; and 6) design of panel-based assemblies. Finally, a Glossary and a Commentary that provides additional background information are included.

Library of Congress Cataloging-in-Publication Data

American Society of Civil Engineers.
Standard for load and resistance factor design (LRFD) for engineered wood construction: AF&PA/ASCE-16-95 / American Society of Civil Engineers.
p. cm.
ISBN 0-7844-0041-5
1. Building, Wooden—Standards—United States. 2. Load and resistance factor design—Standards—United States. I. Title.
TA666.A52 1996 95-51202
624.1'84—dc20 CIP


Library of Congress Catalog Card No: 95-51202
ISBN 0-7844-0041-5
Manufactured in the United States of America.

# PREFACE

The design of wood structures has previously been governed by the general design provisions and recommended practice of the National Design Specification® for Wood Construction (NDS®). This specification was first adopted in 1944 and has been updated periodically to reflect new knowledge under the auspices of the American Forest & Paper Association (AF&PA) and its predecessor organizations, the National Lumber Manufacturers Association and the National Forest Products Association. In recognition of a new generation of standards based on reliability theory, this first Load and Resistance Factor Design (LRFD) Standard for Engineered Wood Construction was prepared as a joint activity between AF&PA and ASCE to provide alternate design provisions reflecting the current state of knowledge. The LRFD Standard was developed to provide uniform practice in the design of engineered wood structures.

Design criteria provide recommended practice for most applications but may not cover infrequently encountered designs for which additional judgment in applying data or recommendations must be exercised. It is intended that the LRFD Standard be used in conjunction with competent engineering design, accurate fabrication, and adequate supervision of construction. Particular attention is directed to the designer's responsibility to make adjustments for particular end use conditions.

The Appendices to this Standard are considered an integral part of the LRFD Standard. A Commentary has been prepared to provide additional background information. Users desiring further details leading to LRFD Standard provisions are requested to consult the Commentary and cited references.

Development of the LRFD Standard was a result of deliberations of a team of structural engineers and wood material scientists with wide experience and high professional standing. The team included professionals from private practice, industry, government, and universities. Review and trial use by practicing consulting engineers preceded publication.

The information contained herein is not intended as a representation or warranty, on the part of AF&PA or ASCE or any other person involved in its development, that this Standard is suitable for any general or particular use.

While every effort has been made to insure the accuracy of the data and information contained herein, neither AF&PA nor ASCE assumes responsibility for errors or omissions, nor for plans, designs, or construction prepared from this LRFD Standard.

Those using this LRFD Standard assume all liability arising from its use. The design of engineered structures is within the scope of expertise of licensed engineers, architects, or other licensed professionals for applications to a particular structure.

This Standard is not intended to preclude the use of any other materials, assemblies, or designs that can satisfactorily demonstrate adequate performance.

*This page intentionally left blank*

# STANDARDS

In April 1980, the Board of Direction approved ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Management Goup F (MGF), Codes and Standards. The consensus process includes balloting by the balanced standards committee made up of Society members and non-members, balloting by the membership of ASCE as a whole and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding five years.

The following standards have been issued:

ANSI/ASCE 1-82 N-725 Guidelines for Design and Analysis of Nuclear Safety Related Earth Structures
ANSI/ASCE 2-91 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-86 Seismic Analysis of Safety-Related Nuclear Structures
Building Code Requirements for Masonry Structures (ACI530-95/ASCE5-95/TMS402-95) and Specifications for Masonry Structures (ACI530.1-95/ASCE6-95/TMS602-95)
Specifications for Masonry Structures (ACI530-95/ASCE6-95/TMS602-95)
ANSI/ASCE 7-93 Minimum Design Loads for Building and Other Structures
ANSI/ASCE 8-90 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ANSI/ASCE 10-90 Design of Latticed Steel Transmission Structures
ANSI/ASCE 11-90 Guideline for Structural Condition Assessment of Existing Buildings
ANSI/ASCE 12-91 Guideline for the Design of Urban Subsurface Drainage
ASCE 13-93 Standard Guidelines for Installation of Urban Subsurface Drainage
ASCE 14-93 Standard Guidelines for Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-93 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)
ASCE 16-95 Standard for Load and Resistance Factor Design (LRFD) of Engineered Wood Construction

*This page intentionally left blank*

# ACKNOWLEDGMENTS

This standard was developed in two phases. The initial draft was funded by the wood products industry and was developed by Engineering Data Management, Inc., under the direction of Dr. James R. Goodman, P.E. The second phase of development was under the auspices of the ASCE Standards Committee for Design of Engineered Wood Construction, chaired by Mr. Thomas G. Williamson, P.E. Both phases were coordinated by the American Forest & Paper Association.

In addition to the participants listed below, the leadership of a Wood Industry Technical Committee, chaired by Dr. Kevin C.K. Cheung, Western Wood Products Association, and the Wood Industry Management Committee, chaired by Jeffrey M. Van Cott, and the vision of the funding organizations is acknowledged.

**Phase 1 funding organizations**
Alpine Engineered Products
Alpine Structures
American Forest & Paper Association
American Institute of Timber Construction
APA—The Engineered Wood Association
Boise Cascade
California Lumber Inspection Service
California Redwood Association/Redwood Inspection Service
Canadian Wood Council
Fabricated Wood Components
Fibreboard Technology
Georgia-Pacific
Jager Industries
Louisiana Pacific
MacMillan Bloedel
McCausey Lumber
Mitek
MSR Lumber Producers Council
National Timber Piling Council
Nordel
Northeast Lumber Manufacturers Association
Pacific Lumber Inspection Bureau
Southeast Lumber Manufacturers Association
Southern Forest Products Association
Southern Pine Inspection Bureau
Standard Structures
Superior Wood Products
Tecton Laminates
Timber Products Inspection
Trus Joist MacMillan
Truss Plate Institute
Truswal
Unit Structures
West Coast Lumber Inspection Bureau
Western Wood Preservers Institute
Western Wood Products Association
Weyerhaeuser
Willamette Industries

**Phase 2 Committee Members**
Albert H. Alexanian
Donald E. Breyer
James R. Brown
Linda S. Brown
R. Michael Caldwell, Vice Chair
Kevin C. Cheung
Kelly E. Cobeen
Marvin E. Criswell
Thomas P. Cunningham
Nancy H. Devine
Susan Dowty
Bruce R. Ellingwood
Kenneth J. Fridley
Charles B. Goehring
Allan H. Gold
James R. Goodman
James S. Graham
David S. Gromala, Secretary
Kirk H. Grundahl
Dominique P. Janssens
Stuart L. Lewis
John D. Lowood
Catherine M. Marx
Thomas McLain
Joseph F. Murphy
Michael O'Halloran
Michael P. O'Reardon
Marcia Paton-Mallory
Clarkson W. Pinkham
Robert M. Powell
Chandrasekhar Putcha
Don T. Pyle
Julie Ruth
Douglas L. Sarkkinen
Donald J. Sharp
Bradley E. Shelley
John H. Showalter, Jr.
Judith J. Stalnaker
Nader Tomasbi
Michael H. Triche
Thomas G. Williamson, Chair

*This page intentionally left blank*

# Contents

PAGE

# FOREWORD

The material presented in this publication has been prepared in accordance with recognized engineering principles. This Standard and Commentary should not be used without first securing competent advice with respect to their suitability for any given application. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Society of Civil Engineers, or of any other person named herein, that this information is suitable for any general or particular use or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

*This page intentionally left blank*

# Notation

| | |
|---|---|
| $A$ | Gross area |
| $A_n$ | Net area, net bearing area |
| $A_{min}$, $B_{min}$ | Minimum spacing permitted for shear plates and split rings, parallel and perpendicular to grain, respectively |
| $A_{opt}$, $B_{opt}$ | Required spacing of shear plates and split rings to achieve reference connection resistance, parallel and perpendicular to grain, respectively |
| $B_{bx}$, $B_{by}$ | Moment magnification factor for loads that result in no appreciable sidesway (strong and weak axes, respectively) |
| $B_{sx}$, $B_{sy}$ | Moment magnification factor for loads that result in sidesway (strong and weak axes, respectively) |
| $C_E$ | Composite action factor |
| $C_F$ | Size factor |
| $C_G$ | Grade/construction factor for structural panels |
| $C_H$ | Shear stress factor |
| $C_I$ | Stress interaction factor |
| $C_L$ | Beam stability factor |
| $C_M$ | Wet service factor |
| $C_P$ | Column stability factor |
| $C_T$ | Buckling stiffness factor for dimension lumber |
| $C_V$ | Volume effect factor for structural glued laminated timber |
| $C_b$ | Bearing area factor |
| $C_b$ | Bending coefficient dependent on moment gradient |
| $C_c$ | Curvature factor for structural glued laminated timber |
| $C_{cs}$ | Critical section factor for round timber piles |
| $C_d$ | Penetration depth factor for connections |
| $C_{di}$ | Diaphragm factor |
| $C_{eg}$ | End-grain factor for connections |
| $C_f$ | Form factor |
| $C_{fu}$ | Flat-use factor |
| $C_g$ | Group action factor for connections |
| $C_m$, $C_{mx}$, $C_{my}$ | Moment shape factor for biaxial bending (general, strong, and weak axes, respectively) |
| $C_{pt}$ | Preservative treatment factor |
| $C_r$ | Load-sharing factor |
| $C_{rt}$ | Fire-retardant treatment factor |
| $C_{sp}$ | Single pile factor |
| $C_{st}$ | Metal side plate factor for 4 in. shear plate connections |
| $C_t$ | Temperature factor |
| $C_{tn}$ | Toe-nail factor for nailed connections |
| $C_u$ | Untreated factor for round timber piles |
| $C_w$ | Width factor for structural panels |
| $C_\Delta$ | Geometry factor for connections |
| $D$ | Diameter |
| $D$ | Dead load |
| $D$, $D'$ | Reference and adjusted diaphragm shear resistance per unit length |
| $D_u$ | Diaphragm shear force per unit length due to factored loads |
| $D_1$, $D_2$ | Minimum and maximum diameters in round tapered members |
| $E$ | Earthquake load |
| $E$, $E'$ | Reference and adjusted mean modulus of elasticity |

| | |
|---|---|
| $E_{05}$, $E_{05}'$ | Reference and adjusted fifth percentile modulus of elasticity |
| EA | Axial stiffness |
| EI | Flexural stiffness |
| $F_b$, $F_b'$ | Reference and adjusted bending strength |
| $F_{bx}^*$ | Bending strength for strong (x-x) axis bending multiplied by all applicable adjustment factors except $C_{fu}$, $C_V$, and $C_L$ |
| $F_c$, $F_c'$ | Reference and adjusted compression strength parallel to grain |
| $F_c^*$ | Compression strength parallel to grain multiplied by all applicable adjustment factors except $C_P$ |
| $F_{c\perp}$, $F_{c\perp}'$ | Reference and adjusted compression strength perpendicular to grain |
| $F_e$ | Dowel bearing strength |
| $F_{em}$, $F_{es}$ | Dowel bearing strength of main and side members, respectively |
| $F_{e\parallel}$, $F_{e\perp}$, $F_{e\theta}$ | Dowel bearing strength parallel, perpendicular, and at an angle to the grain, respectively |
| $F_g$, $F_g'$ | Reference and adjusted bearing strength parallel to grain |
| $F_r'$, $F_{rc}'$, $F_{rt}'$ | Adjusted radial strength (general, compression, and tension, respectively) |
| $F_s$, $F_s'$ | Reference and adjusted rolling shear strength for structural panels |
| $F_t$, $F_t'$ | Reference and adjusted tensile strength parallel to grain |
| $F_{tv}'$ | Adjusted torsional shear strength |
| $F_v$, $F_v'$ | Reference and adjusted shear strength parallel to grain (horizontal shear) |
| $F_v$, $F_v'$ | Reference and adjusted through-thickness shear strength for structural panels |
| $F_{yb}$ | Bending yield strength of fastener |
| G | Specific gravity |
| G, G′ | Reference and adjusted shear modulus |
| $G_v$, $G_v'$ | Reference and adjusted shear modulus for structural panels |
| I | Moment of inertia |
| J | Torsional constant for a section |
| $K_M$ | Moisture content coefficient for sawn lumber truss compression chords |
| $K_T$ | Truss compression chord coefficient for sawn lumber |
| $K_e$ | Effective length factor for compression members |
| L | Design span of bending member or compression member |
| L | Live load caused by storage, occupancy, or impact |
| $L_r$ | Roof live load |
| M, M′ | Reference and adjusted moment resistance |
| $M_1$, $M_2$ | Smaller and larger end moment in a beam or segment |
| $M_{bx}$, $M_{by}$ | Factored moment from loads that result in no appreciable sidesway (strong and weak axes, respectively) |
| $M_e$ | Elastic lateral buckling moment |
| $M_{mx}$, $M_{my}$ | Factored moment, including magnification for second-order effects (strong and weak axes, respectively) |
| $M_s'$ | Adjusted moment resistance computed with $C_L = 1.0$ |
| $M_{sx}$, $M_{sy}$ | Factored moment from loads that result in sidesway (strong and weak axes, respectively) |
| $M_t$, $M_t'$ | Reference and adjusted torsion resistance |
| $M_{tu}$ | Torsion due to factored loads |
| $M_u$, $M_{ux}$, $M_{uy}$ | Moment due to factored loads (general, strong and weak axes, respectively) |
| $M_x'$, $M_y'$ | Adjusted moment resistance (strong and weak axes, respectively) |
| $M_x^*$ | Moment resistance for strong (x-x) axis bending multiplied by all applicable adjustment factors except $C_{fu}$, $C_V$, and $C_L$ |
| P, P′ | Reference and adjusted compression resistance parallel to grain |
| $P_0'$ | Adjusted member axial parallel to grain resistance of a zero length column (i.e., the limit obtained as length approaches zero) |
| $P_a$ | Assumed axial load acting on a side bracket |
| $P_e$ | Euler buckling resistance |
| $P_g$, $P_g'$ | Reference and adjusted bearing resistance |

| | |
|---|---|
| $P_\perp$, $P_\perp'$ | Reference and adjusted compression resistance perpendicular to grain |
| $P_\theta$, $P_\theta'$ | Reference and adjusted compression resistance in bearing at angle θ |
| $P_s$ | Assumed horizontal side load placed at center of height of column |
| $P_u$ | Compressive or bearing force due to factored loads |
| Q | Statical moment of an area about the neutral axis |
| R | Load caused by initial rain water and/or ice |
| R, R′ | Reference and adjusted resistance |
| $R_B$ | Slenderness ratio of bending member |
| $R_{EA}$ | Ratio of minimum to maximum member axial stiffness in a connection |
| $R_e$ | Ratio of main to side member embedment strength in a connection |
| $R_t$ | Ratio of main to side member thickness in a connection |
| $R_f$, $R_m$ | Radius of curvature at the inside face and at mid-depth, respectively |
| $R_u$ | Force due to factored loads |
| S | Section modulus |
| S | Snow load |
| T, T′ | Reference and adjusted tension resistance parallel to grain |
| $T_u$ | Tensile force due to factored loads |
| V, V′ | Reference and adjusted shear resistance |
| $V_u$ | Shear force due to factored loads |
| W | Wind load |
| Z, Z′ | Reference and adjusted connection lateral resistance |
| $Z_u$ | Connection force due to factored loads |
| $Z_W$, $Z_W'$ | Reference and adjusted connection withdrawal resistance |
| $Z_\alpha'$ | Adjusted resistance of a fastener loaded at an angle to the surface of the wood member |
| $Z_\parallel'$, $Z_\perp'$, $Z_\theta'$ | Adjusted resistance of a fastener loaded parallel, perpendicular, and at an angle to the grain, respectively |
| a | End distance for a connection |
| $a_i$ | Effective number of fasteners for row i |
| $a_{min}$ | Minimum end distance permitted for connections |
| $a_{opt}$ | Required end distance to achieve reference connection resistance |
| b | Member width |
| b | Edge distance for connection |
| $b_{min}$ | Minimum edge distance permitted for connections |
| $b_{opt}$ | Required edge distance to achieve reference connection resistance |
| c | Coefficient in column stability factor equation |
| $c_b$ | Coefficient in beam stability factor equation |
| d | Member depth |
| $d_1$, $d_2$ | Minimum and maximum depth for a uniform width, linearly tapered member |
| $d_e$ | Effective depth of member at a connection |
| $d_n$ | Depth of member remaining at a notch |
| e | Eccentricity |
| h | Height |
| $\ell$ | Design span of bending member or compression member |
| $\ell$ | Distance between points of lateral support of a compression member |
| $\ell$ | Span length, clear span of arch between hinges |
| $\ell_b$ | Bearing length |
| $\ell_{br}$ | Distance from the bottom of the column or column segment to the top of the column bracket, in. |
| $\ell_c$ | Clear span |
| $\ell_e$ | Effective length |
| $\ell_m$ | Length of dowel-type fastener in main member |
| $\ell_P$ | Distance measured vertically from point of application of load on bracket to farther end of column |

| | |
|---|---|
| $\ell_u$ | Laterally unsupported span length of bending or compression member |
| $n_f$ | Total number of fasteners in a connection |
| $n_i$ | Number of equally spaced fasteners in row i |
| $n_r$ | Number of serial rows of fasteners in a connection |
| p | Depth of fastener penetration into wood member |
| r | Radius of gyration |
| s | Spacing of fasteners in a connection (also called pitch spacing) |
| $s_{min}$ | Minimum spacing for adjusted connection resistance |
| $s_{opt}$ | Required spacing for reference connection resistance |
| t | Thickness |
| $t_m$, $t_s$ | Thickness of main and side members, respectively, in a connection |
| w | Uniform load |
| $\alpha$ | Angle between applied force vector and the surface of the wood member |
| $\alpha$ | Angle of connector axis with respect to member longitudinal axis |
| $\alpha_b$ | Factor in design of flexural members |
| $\alpha_c$ | Factor in design of columns |
| $\gamma$ | Load/slip constant for a single fastener |
| $\Delta$ | Deflection |
| $\lambda$ | Time-effect factor |
| $\phi$ | Resistance factor |
| $\phi_b$ | Resistance factor for flexure |
| $\phi_c$ | Resistance factor for compression |
| $\phi_s$ | Resistance factor for stability |
| $\phi_t$ | Resistance factor for tension |
| $\phi_v$ | Resistance factor for shear/torsion |
| $\phi_z$ | Resistance factor for connections |
| $\theta$ | Angle of cut taper or cut notch from the grain direction |
| $\theta$ | Angle of force vector with respect to a direction parallel to grain |
| $\theta_b$ | Angle between bearing force and the direction of grain |

# CHAPTER 1

# General Provisions

### 1.1 Scope

This standard provides design criteria for structures constructed of structurally graded lumber, structural glued laminated timber, panel products, poles, piles, and other structural wood components, and their connections. This standard is for the design of buildings and similar structures. Wherever in this standard reference is made to the appendices, the provisions of the appendices shall apply. The derivation of design strengths and resistances is outside the scope of this standard.

Design strengths and resistances established for use with this standard shall be determined in accordance with ASTM Specification D5457-93.

**1.1.1 Units.** Where units are required in the provisions of this standard, they are provided both in metric (SI) and in U.S. customary units. Many of the checking equations do not require explicit statement of units; in these equations the designer shall use units for all quantities that are consistent.

### 1.2 Applicable Documents

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC.

American Society of Civil Engineers. 1993. *Minimum Design Loads for Buildings and Other Structures. ASCE 7-93.* New York, NY.

American National Standards Institute. 1981. *American National Standards for Wood Screws (inch series). ANSI B18.6.1-1981.* New York, NY.

American National Standards Institute. 1981. *American National Standard for Square and Hex Bolts and Screws (inch series). ANSI B18.2.1-1981.* New York, NY.

American National Standards Institute. 1992. *Specifications and Dimensions for Wood Poles. ANSI O5.1-1992.* New York, NY.

American Society for Testing and Materials. 1986. *Standard Method for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers and Rivets. ASTM F606-86.* Philadelphia, PA.

American Society for Testing and Materials. 1987. *Specification for Zinc Coating (Hot Dip) on Iron Steel Hardware. ASTM A153-87.* Philadelphia, PA.

American Society for Testing and Materials. 1993. *Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber. ASTM D245-93.* Philadelphia, PA.

American Society for Testing and Materials. 1988. *Establishing Clear Wood Strength Values. ASTM D2555-88.* Philadelphia, PA.

American Society for Testing and Materials. 1989. *Standard Specification for Ferritic Malleable Iron Castings. ASTM A47-89.* Philadelphia, PA.

American Society for Testing and Materials. 1994. *Standard Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists. ASTM D5055-94.* Philadelphia, PA.

American Society for Testing and Materials. 1994. *Standard Methods of Testing Small Clear Specimens of Timber. ASTM D143-94.* Philadelphia, PA.

American Society for Testing and Materials. 1991. *Round Timber Piles. ASTM D25-91.* Philadelphia, PA.

American Society for Testing and Materials. 1991. *Establishing Allowable Properties for Visually Graded Dimension Lumber from In-Grade Tests of Full-size Specimens. ASTM D1990-91.* Philadelphia, PA.

American Society for Testing and Materials. 1993. *Establishing Stresses for Structural Glued Laminated Timber (Glulam). ASTM D3737-93c.* Philadelphia, PA.

American Society for Testing and Materials. 1993. *Specification for Evaluation of Structural Composite Lumber Products. ASTM D5456-93.* Philadelphia, PA.

American Society for Testing and Materials. 1993. *Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design. ASTM D5457-93.* Philadelphia, PA.

Society of Automotive Engineers. 1985. *Mechanical and Material Requirements for Externally Threaded Fasteners. SAE J429.* Warrendale, PA.

Society of Automotive Engineers. 1990. Chemical Composition of SAE J403. In *SAE Handbook,* Vol. 1: Materials. Warrendale, PA.

Society of Automotive Engineers. 1989. *General Characteristics and Heat Treatment of Steels. SAE J412.* Warrendale, PA.

Truss Plate Institute. 1995. *National Design Standard for Metal Plate Connected Wood Truss Construction. ANSI/TPI 1-1995.* Madison, WI.

U.S. Department of Commerce. 1994. *Construction and Industrial Plywood. PS 1-94.* Washington, DC.

U.S. Department of Commerce. 1992. *Performance Standard for Wood-Based Structural-Use Panels. PS 2-92.* Washington, DC.

U.S. Department of Commerce. 1994. *American Softwood Lumber Standard. PS 20-94.* Washington, DC.

## 1.3 Loads and Load Combinations

Nominal loads shall be those required by the applicable building code. In the absence of a governing code, the nominal loads shall be those stipulated in ASCE 7-93.

**1.3.1 Nominal loads.** The following nominal loads shall be considered:

- D Dead load caused by the weight of permanent construction, including walls, floors, roofs, ceilings, fixed partitions, stairways, and fixed service equipment.
- L Live load produced by the use and occupancy of the building, including impact, but not including environmental loads such as snow, wind, rain, etc.
- $L_r$ Live load on the roof produced during maintenance by workers, equipment, and materials, or during ordinary use by movable objects and people.
- S Snow load caused by uniform deposition, drifting, and/or other unbalanced snow conditions.
- R Rain water or ice load exclusive of contributions caused by ponding.
- W Wind load.
- E Earthquake load, determined in accordance with ASCE 7-93.

**1.3.2 Load combinations.** Except where applicable codes require otherwise, structures, structural members, and their connections shall be designed using the following factored load combinations (subsets of the load combinations are further defined in Sec. 1.3.3).

$$1.4D \tag{1.3-1}$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \tag{1.3-2}$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \tag{1.3-3}$$

$$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \tag{1.3-4}$$

$$1.2D + 1.0E^1 + 0.5L + 0.2S \tag{1.3-5}$$

$$0.9D - (1.3W \text{ or } 1.0E^1) \tag{1.3-6}$$

Exception: The load factor on L in the combinations of Eqs. 1.3-3, 1.3-4, and 1.3-5 shall equal 1.0 for parking garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf (4.78 kPa). Refer to the governing building code for allowable reductions in live load magnitudes.

Each relevant limit state shall be investigated, including cases where some of the loads in a combination are equal to zero. Unbalanced load conditions shall be investigated in accordance with applicable building code provisions.

**1.3.3 Other loads.** Structural effects of other actions, including but not limited to weight and lateral pressure of soils, restrained dimensional changes caused by temperature differentials, shrinkage, moisture, creep, and differential settlement, shall be investigated in design where their effects are significant.

The structural effects of loads caused by fluid (F), soil (H), ponding (P), and temperature (T) shall be investigated in design as the following factored loads: 1.3F, 1.6H, 1.2P, and 1.2T.

**1.3.4 Counteracting loads.** When the effects of loads counteract one another in a structural member or connection, the design shall account for reversal of axial forces, shears, or moments.

## 1.4 Design Basis

**1.4.1 Limit states design.** Structural members and connections shall be proportioned so that no applicable limit state is exceeded when the structure is subjected to applicable design loads.

Strength limit states shall include each required resistance (force or stress) considered for each system, member, or connection.

Serviceability limit states shall be as set forth in Chap. 10.

**1.4.2 Structural analysis.** Load effects on individual components and connections shall be determined by elastic methods of structural analysis. The analysis shall take into account equilibrium, general stability, geometric compatibility, and both short- and long-term material properties. Alternatively, nonlinear or inelastic analysis shall be per-

[1]The factored load 1.0E is for use with earthquake loads determined in accordance with ASCE 7-93.

**TABLE 1.4-1.**
**Resistance factors, $\phi$.**

| Application | Symbol | Value |
|---|---|---|
| Compression | $\phi_c$ | 0.90 |
| Flexure | $\phi_b$ | 0.85 |
| Stability | $\phi_s$ | 0.85 |
| Tension | $\phi_t$ | 0.80 |
| Shear/Torsion | $\phi_v$ | 0.75 |
| Connections | $\phi_z$ | 0.65 |

mitted provided that substantiating data on behavior is available and approved by the authority having jurisdiction.

*1.4.2.1 Modulus of elasticity*. For determination of load effects in indeterminate structures and for calculation of deflections and other serviceability conditions, the adjusted mean value, $E'$, shall be used.

The adjusted modulus of elasticity, $E'$, that must be used in design, depends on the application. In design cases in which structural strength or stability are computed, the adjusted fifth percentile value, $E_{05}'$, shall be used. The value of $E_{05}'$ shall be computed as:

$$E_{05}' = 1.03E'(1 - 1.645(COV_E))$$

where 1.03 is the adjustment from tabulated E to shear-free E; and $COV_E$ is the coefficient of variation of E.

*Exception*: For glued-laminated timber, the adjustment shall be 1.05, rather than 1.03.

Modulus of elasticity shall not be adjusted by the time effect factor, $\lambda$.

*1.4.2.2 End restraints*. The design of connections shall be consistent with assumptions in the structural analysis and with the type of construction called for on the design drawings. Simple framing, in which the rotational restraint is ignored, shall be assumed unless the capacity of the connection for a specified degree of restraint can be demonstrated by experimental or analytical means. Connections must have sufficient rotation capacity to avoid overloading connecting elements under design loads.

*1.4.2.3 Long-term loading*. Structures and members that accumulate residual deformations under service loads shall have the added deformations expected to occur during their service life included in their analysis when such deformations affect strength or serviceability.

**1.4.3 Strength limit states.** The design of structural systems, members, and connections shall ensure that the design resistance at all sections for each system, member, and connection equals or exceeds the force due to factored loads, $R_u$.

*1.4.3.1 Force due to factored loads*. Member and connection forces, $R_u$, shall be determined from the factored load combinations in Sec. 1.3.

*1.4.3.2 Design resistance*. The design resistance shall be calculated for each applicable limit state as the product of an adjusted resistance, $R'$, a resistance factor, $\phi$, and a time effect factor, $\lambda$. The design resistance shall equal or exceed the force due to factored loads, $R_u$:

$$R_u \leq \lambda\phi R' \qquad (1.4\text{-}1)$$

where $R'$ indicates the adjusted resistance of a member, component, or connection, such as adjusted flexural resistance, $M'$, adjusted shear re-

**TABLE 1.4-2.**
**Time effect factors.**

| Load Combination | | Time Effect Factor ($\lambda$) |
|---|---|---|
| 1.4D | *(1.3-1)* | 0.6 |
| 1.2D + 1.6L + 0.5($L_r$ or S or R) | *(1.3-2)* | 0.7 when L is from storage<br>0.8 when L is from occupancy<br>1.25 when L is from impact[2] |
| 1.2D + 1.6($L_r$ or S or R) + (0.5L or 0.8W) | *(1.3-3)* | 0.8 |
| 1.2D + 1.3W + 0.5L + 0.5($L_r$ or S or R) | *(1.3-4)* | 1.0 |
| 1.2D + 1.0E + 0.5L + 0.2S | *(1.3-5)* | 1.0 |
| 0.9D − (1.3W or 1.0E) | *(1.3-6)* | 1.0 |

[2]For connections, $\lambda$ = 1.0 when L is from impact.

sistance, V′, etc. $R_u$ is similarly replaced with $M_u$, $V_u$, etc. for specific member and/or connection forces.

The adjusted resistance, R′, shall include the effects of any and all applicable adjustment factors resulting from end use or other modifying factors.

The resistance factors, $\phi$, provided in the appropriate chapters of this standard are summarized in Table 1.4-1.

Except where noted otherwise, the time-effect factor that shall be used with the load combinations of Sec. 1.3.2 is defined in Table 1.4-2.

**1.4.4 Serviceability limit states.** Structural systems and components shall be designed to limit deflections, lateral drift, vibrations, creep, or other deformations that will adversely affect the serviceability of the building or structure. Serviceability limit states are addressed in Chap.10.

**1.4.5 Existing structures.** The design provisions of this standard shall be applicable to the evaluation of existing structures. When an existing building or other structure is altered, consideration shall be given to the possible effects of deterioration and physical damage.

# CHAPTER 2

# Design Requirements

## 2.1 Scope

This chapter contains provisions that are common to the standard as a whole.

## 2.2 Gross and Net Areas

**2.2.1 Gross area.** The gross area, A, of a wood member at any point is the sum of the areas of each element comprising the member, measured normal to the axis of the member.

**2.2.2 Net area.** The net area of a wood member, $A_n$, shall be obtained by deducting the area of all material removed by boring, grooving, dapping, notching, or other means from the gross area, unless otherwise specified. See Chap. 7 for provisions on net areas associated with connections.

## 2.3 Stability

Stability shall be provided for the structure as a whole and for each component within the structure. The design shall take into account load effects resulting from the deflected shape of the structure or of individual components of the lateral load resisting system.

At points of support for beams, trusses, and other wood structural members and/or components, restraint against rotation about their longitudinal axis shall be provided unless restraint against rotation is otherwise assured or analysis or test results demonstrate such restraint is not required. See Chap. 5 for conditions of lateral support for flexural members.

## 2.5 Reference Conditions

The reference resistance, R, and the reference connection resistance, Z, shall be determined based on the following reference conditions.

(a) Dry use conditions for which the maximum equilibrium moisture content does not exceed 19% for solid wood and 16% for glued wood products and the lower limit of average yearly equilibrium moisture content is 6%.

(b) Reference resistance values shall apply at continuous exposure to temperatures up to 100°F (32°C); temperatures occasionally reaching 150°F (65°C) for members and connections; or temperatures briefly exceeding 200° F (93°C) for structural-use panels. Wood members and connections shall not be permanently exposed to temperatures in excess of 150°F (65°C). Structural-use panels shall not be exposed to temperatures above 200°F (93°C) for more than very brief periods. For sustained temperature conditions above 100°F (32°C) the temperature adjustment shall apply.

(c) Wood products that are untreated, except as noted for poles and piles, see Sec. 2.6.6.

(d) New product, as opposed to wood materials removed for reuse.

(e) Single members or connections without load sharing or composite action.

The adjusted resistance value, R′ (or Z′), for other conditions shall be determined in accordance with Sec. 2.6.

## 2.6 Adjusted Resistance and Adjusted Strength

**2.6.1 General.** The adjustment factors set forth in Tables 2.6-1 and 7.1-1 shall be applied as required by this section, when noted as applicable by these tables. The adjusted resistance shall be calculated as:

$$R' = R\, C_1\, C_2 \ldots C_n \qquad (2.5\text{-}1)$$

where R′ is adjusted resistance, R is reference resistance, and $C_i$ are the applicable adjustment factors.

Where this section does not reference a specific section for calculation of the adjustment factor, adjustment factors shall be determined as follows.

(a) For solid sawn lumber, structural glued laminated timber, timber piles and connections, adjustment factors shall be taken from the *National Design Specification® for Wood Construction*.
(b) For other wood-based products and connections, adjustment factors shall be determined in accordance with approved standards.

**2.6.2 Adjustment factors for end-use.** For end-use conditions that differ from the reference conditions set forth in Sec. 2.5, the following adjustment factors apply.

$C_M$ = wet service factor to account for in-service moisture content higher than 19% for solid wood and 16% for glued wood products.
$C_t$ = temperature factor to account for in-service temperature higher than a sustained temperature of 100°F.
$C_{pt}$ = preservative treatment factor to account for preservative treating of wood products and connections. Appropriate adjustment factors shall be obtained from suppliers of preservatively treated products and/or from applicable codes and standards.
$C_{rt}$ = fire-retardant treatment factor to account for fire-retardant treating of wood products and connections. Appropriate adjustment factors shall be obtained from suppliers of fire-retardant treated products and/or suppliers of treatment chemicals.

**2.6.3 Adjustment factors for member configuration.** In addition to the factors set forth in Sec. 2.6.2, the following shall apply to all members and products, with limitations as specified.

$C_E$ = composite action factor for members in wood floors, walls, and roofs to account for the increase in resistance when the sheathing acts in composite action with framing members as specified in Chap. 5.
$C_r$ = load sharing factor for built-up beams or members in wood floors, walls, and roofs to account for the increase in resistance within the assembly as specified in Chap. 5 or as specified in the applicable product standard for other applications.
$C_F$ = size factor to account for the effect of member size as specified by the applicable product standard.
$C_L$ = beam stability factor to account for the effect of partial lateral support as specified in Chap. 5.
$C_P$ = column stability factor to account for the effect of partial lateral support as specified in Chap. 4.
$C_b$ = bearing area factor to account for the increase in effective bearing area of beams as specified in Chap. 4.
$C_f$ = form factor to account for the effect of nonrectangular cross section on computed bending resistance as specified in Chap. 5 and applicable product standards.

**2.6.4 Additional adjustments for structural lumber and glued laminated timber.** In addition to the factors set forth in Sec. 2.6.2 and 2.6.3, the following shall apply to structural lumber and glued laminated timber.

$C_H$ = shear stress factor to account for increased shear strength in sawn lumber members with limited splits, checks, or shakes.
$C_I$ = interaction stress factor to account for the stress increase at a cut tapered surface of glued-laminated timber.
$C_T$ = buckling stiffness factor to account for increased stiffness of sheathed dimension lumber truss chords.
$C_V$ = volume effect factor for structural glued laminated timber loaded perpendicular to the wide face of laminations to account for the effect of member volume on moment resistance.
$C_c$ = curvature factor for structural glued laminated timber to account for the effect of curvature on moment resistance.
$C_{fu}$ = flat-use factor to account for increased moment resistance of lumber members used in a flatwise orientation.

**TABLE 2.6-1.**
**Applicability of adjustment factors for LRFD[1].**

Adjusted Property = Reference Property x Applicable Adjustment Factors

**Adjustment factors for end-use**

| Adjusted Property = | Reference Property x | Wet service | Temperature | Preservative treatment | Fire-retartant treatment |
|---|---|---|---|---|---|
| all | all | $C_M$[2] | $C_t$ | $C_{pt}$ | $C_{rt}$ |

**Adjustment factors for member configuration**

| Adjusted Property = | Reference Property x | Composite action | Load sharing | Size | Beam stability[3] | Column stability | Bearing area | Form |
|---|---|---|---|---|---|---|---|---|
| $F_b' =$ | $F_b$ | | $C_r$ | $C_F$ | $C_L$ | | | $C_f$ |
| $F_t' =$ | $F_t$ | | | $C_F$ | | | | |
| $F_v' =$ | $F_v$ | | | $C_F$ | | | | |
| $F_c' =$ | $F_c$ | | | $C_F$ | | $C_P$ | | |
| $F_{c\perp}' =$ | $F_{c\perp}$ | | | | | | $C_b$ | |
| $E' =$ | $E$ | $C_E$ | | | | | | |

**Additional Adjustments for Structural lumber and/or Glued-laminated timber**

| Adjusted Property = | Reference Property x | Shear stress | Buckling stiffness | Volume | Curvature | Flat-use |
|---|---|---|---|---|---|---|
| $F_b' =$ | $F_b$ | | | $C_V$ | $C_c$ | $C_{fu}$ |
| $F_v' =$ | $F_v$ | $C_H$ | | | | |
| $E' =$ | $E$ | | $C_T$ | | | |

**Additional Adjustments for Structural Panels**

| Adjusted Property = | Reference Property x | Width | Grade/ construction |
|---|---|---|---|
| $F_b' =$ | $F_b$ | $C_w$ | $C_G$ |
| $F_t' =$ | $F_t$ | $C_w$ | $C_G$ |
| $F_v' =$ | $F_v$ | | $C_G$ |
| $F_c' =$ | $F_c$ | | $C_G$ |
| $F_{c\perp}' =$ | $F_{c\perp}$ | | $C_G$ |
| $E' =$ | $E$ | | $C_G$ |

**Additional Adjustments for Timber Poles and Piles**

| Adjusted Property = | Reference Property x | Critical section | Single-pile | Untreated |
|---|---|---|---|---|
| $F_c' =$ | $F_c$ | $C_{cs}$ | $C_{sp}$ | $C_u$ |
| $F_b' =$ | $F_b$ | | $C_{sp}$ | $C_u$ |
| $F_v' =$ | $F_v$ | | | $C_u$ |
| $F_c' =$ | $F_c$ | | | $C_u$ |
| $F_g' =$ | $F_g$ | | | $C_u$ |

**Additional Adjustments for Structural Connections**

| Adjusted Property = | Reference Property x | Diaphragm | Group action | Geometry | Penetration depth | End grain | Metal side plate | Toe-nail |
|---|---|---|---|---|---|---|---|---|
| | | | | ***Nails, spikes*** | | | | |
| $Z' =$ | $Z$ | $C_{di}$ | | | $C_d$ | $C_{eg}$ | | $C_{tn}$ |
| $Z_W' =$ | $Z_W$ | | | | | $C_{eg}$ | | $C_{tn}$ |
| | | | | ***Wood screws*** | | | | |
| $Z' =$ | $Z$ | | | | $C_d$ | $C_{eg}$ | | |
| $Z_W' =$ | $Z_W$ | | | | | | | |
| | | | | ***Bolts*** | | | | |
| $Z' =$ | $Z$ | | $C_g$ | $C_\Delta$ | | | | |
| | | | | ***Lag screws, drift pins*** | | | | |
| $Z' =$ | $Z$ | | $C_g$ | $C_\Delta$ | $C_d$ | $C_{eg}$ | | |
| $Z_W' =$ | $Z_W$ | | | | | $C_{eg}$ | | |
| | | | | ***Shear plates, split rings*** | | | | |
| $Z_\parallel' =$ | $Z_\parallel$ | | $C_g$ | $C_\Delta$ | $C_d$ | | $C_{st}$ | |
| $Z_\perp' =$ | $Z_\perp$ | | $C_g$ | $C_\Delta$ | $C_d$ | | | |

[1]Numerical values for adjustment factors, where not provided in this standard, shall be taken from applicable product standards.

[2]Exception: The wet service factor is not applicable to the bearing strength, $F_g$.

[3]The beam stability factor, $C_L$, shall not be applied simultaneously with the volume factor, $C_V$, for glued laminated timber bending members—the lessor of these adjustments shall apply.

**2.6.5 Additional adjustments for structural panels.** In addition to the factors set forth in Sec. 2.6.2 and 2.6.3, the following shall apply to structural panels.

$C_w$ = width factor to account for increased panel resistance for members of narrow widths.
$C_G$ = grade/construction factor for a panel that has different physical properties than the reference grade for which resistance values are available. This factor is also used for panel materials with layups for which reference resistance values are not published.

**2.6.6 Additional adjustments for timber poles and piles.** In addition to the factors set forth in Sec. 2.6.2 and 2.6.3, the following shall apply to timber poles and piles.

$C_{cs}$ = critical section factor for round timber piles.
$C_{sp}$ = single pile factor for round timber piles.
$C_u$ = untreated factor for round timber piles.

**2.6.7 Additional adjustments for structural connections.** In addition to the factors set forth in Sec. 2.6.2 and 2.6.3, the following shall apply to connections.

$C_{di}$ = diaphragm factor to account for the increased resistance of nails used in diaphragm construction as specified in Chap. 9.
$C_{eg}$ = end grain factor.
$C_g$ = group action factor to account for unequal loading of multiple fasteners in a row as specified in Chap. 7.
$C_\Delta$ = geometry factor to account for connection geometry that does not conform to a standard as specified in Chap. 7.
$C_d$ = penetration depth factor to account for reduced fastener penetration as specified in Chap. 7.
$C_{eg}$ = end-grain factor to account for reduced strength of fasteners inserted into end-grain, as specified in Chap. 7.
$C_{st}$ = metal side plate factor for 4-inch (102 mm) shear plate connections as specified in Chap.7.
$C_{tn}$ = toe-nail factor for nailed connections as specified in Chap. 7.

# CHAPTER 3

# Tension Members

## 3.1 General

**3.1.1 Scope.** This chapter applies to members subjected to concentric axial tension and portions of members subjected to significant local tension because of connection details. Members loaded in combined bending and axial tension shall meet the requirements of Sec. 6.2. Additional requirements concerning tension in connector regions are contained in Chap. 7.

**3.1.2 Member design.** Tension members shall be designed such that:

$$T_u \leq \lambda \phi_t T' \qquad (3.1\text{-}1)$$

where $T_u$ is the tension force due to factored loads, $\lambda$ is the applicable time effect factor given in Table 1.4-2, $\phi_t$ is the resistance factor for tension parallel to grain = 0.80 and $T'$ is the adjusted tension resistance.

The adjusted resistance shall be computed by multiplying the reference resistance by the applicable adjustments in Sec.2.6.

**3.1.3 Special considerations.** Tension members shall not be notched.

## 3.2 Tension Resistance Parallel to Grain

**3.2.1 Tension resistance.** The adjusted tension resistance of a member loaded in concentric tension, $T'$, shall be evaluated at the critical net area:

$$T' = F_t' A_n \qquad (3.2\text{-}1)$$

where $F_t'$ is adjusted parallel to grain tension strength and $A_n$ is net area.

**3.2.2 Special considerations for unsymmetrical net areas.** When the centroid of an unsymmetrical net area of a group of 3 or more connectors differs from the centroid of the gross area by 5% or more of the member width, the local eccentricity shall be analyzed in accordance with established principles of engineering mechanics and the procedures specified in Sec. 6.2.

## 3.3 Tension Resistance Perpendicular to Grain

When tension forces perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist the tension force shall be provided. Radial tension arising in curved members and in pitched and tapered members shall be limited by the provisions of Sec. 5.6.

### 3.4 Resistance of Built-up and Composite Members

**3.4.1 Built-up members with components of similar materials.** Built-up members include chords of multiple member roof trusses, diaphragm chords, drag struts, and similar members consisting of two or more parallel components of similar material strength and stiffness connected together.

The resistance of such built-up members shall be taken as the sum of the resistances of the individual components provided the connections are adequate to insure a distribution of the axial tension among the individual components in proportion to their area. The effects of splices on reducing the member strength shall be accounted for in the design.

**3.4.2 Composite members with components of dissimilar materials.** The design of tension members assembled from sawn, glued-laminated, or other wood-based components of differing stiffnesses acting in parallel or acting in combination with metal plates or bars shall be based on transformed section concepts. Components shall be connected so that they act as a unit with forces distributed in proportion to the component stiffnesses. For a composite member so connected, the member resistance shall be determined by summing the forces acting in the components at the axial deformation at which the first component reaches its individual resistance.

# CHAPTER 4

# Compression Members and Bearing

### 4.1 General

**4.1.1 Scope.** The provisions of this chapter apply to members subjected to concentric axial compression and to localized compression in locations of bearing. Members loaded in combined bending and axial compression, including members with eccentric axial loads, shall meet the requirements of Sec. 6.3.

**4.1.2 Member design.** Compression members shall be designed such that:

$$P_u \leq \lambda \phi_c P' \tag{4.1-1}$$

where $P_u$ is the compression force due to factored loads, $\lambda$ is the applicable time effect factor given in Table 1.4-2, $\phi_c$ is the resistance factor for compression parallel to grain = 0.90, and $P'$ is the adjusted compression resistance.

The adjusted resistance shall be computed by multiplying the reference resistance by the applicable adjustments in Sec. 2.6.

Members with concentrated applied axial loads shall have sufficient local design resistance and stability in the affected end or connection regions to support these loads. Similarly, members shall have sufficient local design resistance and web stability at beam supports and at the locations of any concentrated transverse loads.

### 4.2 Slenderness and Effective Length Considerations

**4.2.1 Effective column length.** The actual unbraced length of a column or column segment, $\ell$, shall be taken as the center-to-center distance between lateral supports. The unbraced length shall be determined for both the strong and weak axes of a column.

The effective column length, $\ell_e$, for the direction considered shall be taken as $K_e\ell$, where $K_e$ is the buckling length coefficient for compression members. $K_e$ depends on the column-end restraint conditions and the presence or absence of sidesway.

For a compression member braced against sidesway in the direction being considered, the buckling length coefficient, $K_e$, shall be taken as unity unless a rational analysis shows that the end restraint conditions justify use of a smaller factor.

For a compression member unbraced against sidesway in the direction being considered, the buckling length coefficient, $K_e$, shall be greater than one and shall be determined by a rational analysis which accounts for the end restraint conditions.

**4.2.2 Column slenderness ratio.** The column slenderness ratio is given by the ratio of the effective length in the direction considered to the radius of gyration corresponding to that direction:

$$\text{slenderness ratio} = K_e\ell/r. \qquad (4.2\text{-}1)$$

The radius of gyration shall be based on the gross area, using a transformed section when all components are not of the same material stiffness. For notched and tapered members, the radius of gyration shall be determined in accordance with Sec. 4.3.3 and 4.3.4, respectively.

The slenderness ratio, $K_e\ell/r$, of columns shall not exceed 175.

## 4.3 Resistance of Solid Columns Concentrically Loaded in Compression

**4.3.1 Design material values and design factors.** The modulus of elasticity used in the equations of this section shall be the adjusted value at the fifth percentile as specified for use in resistance equations, $E_{05}'$.

**4.3.2 Resistance of prismatic columns.** The column resistance shall be determined based on the most critical column direction and member slenderness ratio. The adjusted column resistance shall be computed as:

$$P' = C_P AF_c^* = C_P P_0' \qquad (4.3\text{-}1)$$

The column stabiliy factor, $C_P$, shall be computed as:

$$C_P = \frac{1+\alpha_c}{2_c} - \sqrt{\left(\frac{1+\alpha_c}{2c}\right)^2 - \frac{\alpha_c}{c}} \qquad (4.3\text{-}2)$$

where:

$$\alpha_c = \frac{\phi_s P_e}{\lambda\phi_c P_0'} \qquad (4.3\text{-}3)$$

$$P_e = \frac{\pi^2 E_{05}' I}{(K_e\ell)^2} = \frac{\pi^2 E_{05}' A}{\left(K_e\frac{\ell}{r}\right)^2} \qquad (4.3\text{-}4)$$

and:

$A$ = gross area;
$F_c^*$ = parallel to grain compression strength multiplied by all applicable adjustment factors except $C_P$;
$E_{05}'$ = adjusted modulus of elasticity at the fifth percentile;
$P_e$ = critical (Euler) buckling resistance about the axis being considered;
$P_0'$ = adjusted member axial parallel to grain resistance of a zero length column; (i.e., the limit obtained as length approaches zero);
$c$ = 0.80 for solid sawn members;
$c$ = 0.85 for round poles and piles;
$c$ = 0.90 for glued laminated members and structural composite lumber;
$\phi_c$ = resistance factor for compression = 0.90;
$\phi_s$ = resistance factor for stability = 0.85.

The moment of inertia, I, the $E_{05}'$ values, and effective length, $K_e\ell$, shall be for the direction being considered. The value of c for members other than glued laminated members, poles, and piles shall be 0.80 unless a larger value has been justified by tests.

**4.3.3 Resistance of notched or bored prismatic columns.** In addition to the provisions of Sec. 4.3.2, the adjusted compression resistance of a notched or bored prismatic column shall be evaluated as follows.

*4.3.3.1 Notch in critical location.*

$$P' = C_P A_n F_c^* \qquad (4.3\text{-}5)$$

where $C_P$ shall be computed by using properties of the net area when notches or holes are located in the middle half of a length between inflection points of the buckled shape and:

(a) the net moment of inertia at such locations is less than 80% of the gross moment of inertia; or
(b) the longitudinal dimension of the notch or hole is greater than the larger cross-sectional dimension of the column.

*4.3.3.2 Notch in noncritical location.* For cases other than noted in Sec. 4.3.3.1, the adjusted compression resistance shall be computed as the lesser of Eq. 4.3-6 and 4.3-7:

$$P' = C_P A F_c^* \qquad (4.3\text{-}6)$$

where $C_P$ shall be computed by using properties of the gross area:

$$P' = A_n F_c^* \qquad (4.3\text{-}7)$$

**4.3.4 Resistance of tapered columns.**

*4.3.4.1 Tapered circular columns.* The adjusted compression resistance of uniformly tapered circular columns shall be determined using the

**TABLE 4.3-1.**
**Design diameter ($D$) of a tapered circular column, defined as $D = D_1 + X(D_1 - D_2)$.**

| Case | Condition | $X=$ |
|---|---|---|
| 1 | Cantilever "flagpole," large end fixed | $0.52 + 0.18\,D_1/D_2$ |
| 2 | Inverted "flagpole" pile, small end fixed | $0.12 + 0.18\,D_1/D_2$ |
| 3 | Singly tapered member, both ends simply supported | $0.32 + 0.18\,D_1/D_2$ |
| 4 | Doubly tapered member, both ends simply supported | $0.52 + 0.18\,D_1/D_2$ |

For all other support conditions, $X = 1/3$.

equations in Sec. 4.3.2. The design diameter shall be the small-end diameter, or when the small-end diameter, $D_1$, is at least one-third of the large-end diameter, $D_2$, the design diameter shall be that specified in Table 4.3-1.

*4.3.4.2 Tapered rectangular columns.* The adjusted compression resistance of rectangular columns with constant width and uniformly tapered depth shall be determined using the equations in Sec. 4.3.2. The design depth shall be the small-end depth, or when the small-end depth, $d_1$, is at least one-third of the large-end depth, $d_2$, the design depth shall be that specified in Table 4.3-2.

*4.3.4.3* In addition to meeting the provisions of Sec. 4.3.4.1 or Sec. 4.3.4.2, the adjusted compression resistance of a tapered member shall be evaluated at the critical net area occuring at the small end:

$$P' = A_n F_c^* \qquad (4.3\text{-}8)$$

## 4.4 Resistance of Spaced, Built-up, and Composite Columns

**4.4.1 Spaced columns.** Spaced columns shall be designed according to the provisions of App.A1.

**4.4.2 Built-up columns.** The resistance of built-up columns shall be based on an analysis that takes into account the effectiveness of the fasteners connecting the components and the geometry of the components. Alternatively, the resistance of the built-up column shall be permitted to be taken as the sum of the individual component resistances assuming each acts independently.

**4.4.3 Composite columns.** The resistance of composite columns shall be determined using transformed section concepts. The components of the composite member shall be connected so that the assembly acts as a unit. When the connection is by other than rigid glues, the analysis shall consider the resulting finite deformations of the fasteners or the resistance shall be limited to the sum of the resistances of the individual components assuming each acts independently.

## 4.5 Resistance in Bearing

**4.5.1 Resistance in end bearing.** The design end bearing resistance shall be computed such that:

$$P_u \leq \lambda \phi_c P_g' \qquad (4.5\text{-}1)$$

where $P_u$ is the compression force due to factored loads, $\lambda$ is the applicable time effect factor given in Table1.4-2, $\phi_c$ is the resistance factor for compression parallel to grain = 0.90, and $P_g'$ is the adjusted member resistance in parallel to grain (end grain) bearing and equals:

$$P_g' = A_n F_g' \qquad (4.5\text{-}2)$$

**TABLE 4.3-2.**
**Design depth ($d$) of a tapered rectangular column of constant width, defined as $d = d_1 + X(d_1 - d_2)$.**

| | | $X=$ | |
|---|---|---|---|
| Case | Condition | Buckling motion in depth direction | Buckling motion in width direction |
| 1 | Cantilever "flagpole," large end fixed | $0.55 + 0.15\,d_1/d_2$ | $0.63 + 0.07\,d_1/d_2$ |
| 2 | Inverted "flagpole" (or pile), small end fixed | $0.15 + 0.15\,d_1/d_2$ | $0.23 + 0.07\,d_1/d_2$ |
| 3 | Singly tapered member, both ends simply supported; singly or doubly tapered member, both ends fixed or one end fixed and one end simply supported (use case 2 when small end is fixed) | $0.35 + 0.15\,d_1/d_2$ | $0.43 + 0.07\,d_1/d_2$ |
| 4 | Doubly tapered member, both ends simply supported | $0.55 + 0.15\,d_1/d_2$ | $0.63 + 0.07\,d_1/d_2$ |

For all other support conditions, $X = 1/3$.

where $A_n$ is net bearing area and $F_g'$ is adjusted end grain bearing strength.

The adjusted resistance shall be computed by multiplying the reference resistance by the applicable adjustments in Sec.2.6.

When the factored compression load exceeds $0.75\ \lambda\phi_c P_g'$, the bearing shall be on a steel plate or strap of not less than 20-gage thickness or on other durable, homogeneous material of similar strength.

Ends of compression members in end-to-end bearing shall be cut accurately and parallel so that a snug fit exists between member ends (including any required bearing plates). Such ends shall also be laterally supported in both directions.

**4.5.2 Resistance in side bearing.** The design side bearing resistance shall be computed such that:

$$P_u \leq \lambda\phi_c P_\perp' \tag{4.5-3}$$

where $P_u$ is the compression force due to factored loads, $\lambda$ is the applicable time effect factor given in Table1.4-2, $\phi_c$ is the resistance factor for compression = 0.90, and $P_\perp'$ is the adjusted member resistance in perpendicular to grain (side grain) bearing and equals:

$$P_\perp' = A_n F_{c\perp}' \tag{4.5-4}$$

where $A_n$ is net (side grain) bearing area and $F_{c\perp}'$ is adjusted perpendicular to grain compression strength.

The adjusted resistance shall be computed by multiplying the reference resistance by the applicable adjustments in Sec.2.6.

When the length in bearing, $\ell_b$, is no more than 6 in. (150 mm) along the member length and the full bearing length is at least 3 in. (75 mm) from the member end, $P_\perp'$ from Eq. 4.5-4 shall be permitted to be multiplied by $C_b$:

$$C_b = (\ell_b + 0.375)/\ell_b \tag{4.5-5}$$

where $\ell_b$ is in inches or by:

$$C_b = (\ell_b + 9.5)/\ell_b \tag{4.5-5-M}$$

where $\ell_b$ is in mm.

The time effect factor, $\lambda$, applies for all bearing resistances, including side bearing. See Sec. 4.1.2.

**4.5.3 Bearing at an angle to grain.** The design bearing resistance at an angle to the grain shall be computed such that:

$$P_u \leq \lambda\phi_c P_\theta' \tag{4.5-6}$$

where $P_u$ is the compression force due to factored loads, $\lambda$ is the applicable time effect factor given in Table1.4-2, $\phi_c$ is the resistance factor for compression = 0.90, and $P_\theta'$ is the adjusted member resistance in bearing at an angle, $\theta_b$, to the grain and equals:

$$P_\theta' = A_n \frac{F_g' F_{c\perp}'}{F_g' \sin^2\theta_b + F_{C\perp}' \cos^2\theta_b} \tag{4.5-7}$$

where $A_n$ is net bearing area, $F_g'$ is the adjusted end-grain bearing strength, $F_{c\perp}'$ is the adjusted perpendicular to grain compression strength, and $\theta_b$ is the angle between bearing force and the direction of grain, 0° for bearing parallel to grain, and 90° for bearing perpendicular to grain.

When $\theta_b$ is 80° or larger, it shall be permitted to assume, in lieu of meeting Eq. 4.5-7, that the bearing force acts perpendicular to the grain and that the bearing lengths dependent on provisions of Sec. 4.5.2 apply.

### 4.6 Radial Compression in Curved Members

The provisions of Sec. 5.6 apply for curved members of constant cross-section. Pitched and tapered beams shall be designed in accordance with App. A2.

# CHAPTER 5

# Flexural Members, Bending and Shear

## 5.1 General

**5.1.1 Scope.** This chapter applies to members and components loaded as flexural members. Provisions are given for both flexural bending and flexural shear. Members loaded in biaxial bending and/or combined bending and axial tension or compression shall meet the requirements of Chap. 6. Strength concerns are addressed in this chapter; serviceability criteria for design of flexural members are contained in Chap. 10. Ponding is addressed in App. A3.

**5.1.2 Member design.** Flexural members shall be designed as follows.

For flexural bending:

$$M_u \leq \lambda \phi_b M' \tag{5.1-1}$$

where $M_u$ is the moment due to factored loads, $\lambda$ is the applicable time effect factor given in Table 1.4-2, $\phi_b$ is the resistance factor for flexure = 0.85, and $M'$ is the adjusted moment resistance.

For flexural shear:

$$V_u \leq \lambda \phi_v V' \tag{5.1-2}$$

where $V_u$ is the shear force due to factored loads, $\lambda$ is the applicable time effect factor given in Table 1.4-2, $\phi_v$ is the resistance factor for shear = 0.75, and $V'$ is the adjusted shear resistance.

For torsion:

$$M_{tu} \leq \lambda \phi_v M_t' \tag{5.1-3}$$

where $M_{tu}$ is the torsion due to factored loads, $\lambda$ is the applicable time effect factor given in Table 1.4-2, $\phi_v$ is the resistance factor for torsion = 0.75, and $M_t'$ is the adjusted torsion resistance.

The adjusted resistance shall be computed by multiplying the reference resistance by the applicable adjustments in Sec. 2.6.

Members shall have adequate local design resistance and stability at points of concentrated applied loads.

**5.1.3 Design span.** The design span shall be used to calculate member shears, moments, and deflections. For simple span members not built integrally with supports, the design span is equal to the clear span plus one-half the required bearing length at each end of the member.

**5.1.4 Notching of flexural members.** Notching of flexural members, especially away from the ends of the member and on the tension face, shall be avoided. Stress concentrations caused by notches shall be reduced by using a gradual tapered notch configuration rather than a square-cornered notch.

Notches at the ends of flexural members shall not exceed one-fourth the beam depth for solid sawn members and one-tenth the beam depth for structural glued laminated members. Members shall not be notched at locations other than at simple supports at the member ends. Notches shall not be placed in the vicinity of interior supports of continuous beams nor at supports adjacent to cantilever spans.

Exception: For solid sawn members of less than 4 in. (100 mm) nominal thickness, notches that do not exceed one-sixth of the member depth shall be permitted if they are located outside the middle third of the span.

The member flexural resistance at any notched section shall not exceed that for the net section at that notched location when the notch is on the compression side. When a notch occurs on the tensile face and the moment acting anywhere along the notch length exceeds one-half the member flexural resistance based on the minimum net section at the notch, the flexural resistance of the entire beam shall be based on the net section at the tension-face notch.

The effects of notches on shear strength shall be accounted for using the provisions of Sec. 5.4.3.

Notching or other modifications of the cross-section of wood I-joists, I-beams, and structural composite lumber are not within the scope of this standard and require special investigation.

**5.1.5 Member orientation and support conditions.** Members graded or manufactured for single-span applications shall not be used for continuous or cantilever construction unless a detailed analysis and/or regrading shows that the member is adequate for the proposed configuration. Glued laminated timbers manufactured for single-span use shall not be used for continuous or cantilevered beams unless the reduced flexural resistance which results when the normal compression side of the member is stressed in tension is accounted for in the design.

Similarly, members graded or manufactured for any specified loading and/or member orientation shall be analyzed in detail if the member is not used in its standard configuration.

**5.1.6 Partial composite action of parallel member assemblies.** The design resistance of sheathed and other partially composite parallel member assemblies shall be determined using:

(a) Sec. 5.1.2;
(b) Sec. 5.1.2 modified by assembly adjustment factors in Sec. 5.3; or
(c) a structural analysis that accounts for the partial composite action and load sharing.

**5.1.7 Moment resistance of square and circular prismatic members.** The adjusted member bending resistance given by Eq. 5.2-2 shall be multiplied by a form factor, $C_f$, of 1.15 for circular members other than poles and piles, and by 1.40 for square members bent about the diagonal.

**5.1.8 Moment resistance of box beams and I-beams.** The moment resistance of beams having box or I shapes and assembled from connected components shall be determined using the transformed section properties unless tests show that a higher resistance is substantiated.

Beams having box or I shapes that incorporate panel products shall meet the provisions of Chap. 8 and App. A6.

**5.1.9 Moment resistance of nonprismatic members.** For nonprismatic members, including poles and piles, the location of maximum moment for the member geometry and loading under consideration shall be determined by analysis.

When the nonprismatic shape is the result of a taper cut on the member, the provisions of Sec. 5.1.10 and 5.1.11 shall be satisfied.

**5.1.10 Tapering of members.** When a straight glued laminated beam is tapered by cutting on the compression face, the interaction of the compressive stresses parallel to grain, compressive stresses perpendicular to grain, and shear acting along the grain adjacent to the sloping cut shall be considered at the critical location of flexural stress using the provisions of Sec. 5.1.11.

The tension side of glued laminated members shall not be tapered by cutting.

The effects of a taper cut removing higher quality outer laminations in the compression zone of a glued laminated member shall be investigated and, when required, a reduction shall be taken in the reference bending strength, $F_b$.

**5.1.11 Stress interaction at a cut face of a member.** When a cut tapered surface at an angle $\theta$ from the grain direction exists along the compression face of a glued laminated beam, a stress interaction factor, $C_I$, shall be computed for the critical stress location using the following equation:

$$C_I = \sqrt{\frac{1}{1+\left(\frac{\phi_b F_b' \tan\theta}{\phi_v F_v'}\right)^2+\left(\frac{\phi_b F_b' \tan\theta}{\phi_c F_{c\perp}'}\right)^2}} \qquad (5.1\text{-}4)$$

The adjusted bending strength for strong axis bending, $F_{bx}'$, for use in Eq. 5.1-4 shall be modified by multiplication by only the smaller of $C_I$ and the volume factor, $C_V$, because these two factors are not cumulative.

**5.1.12 Moment resistance of composite members.** Composite members, including lumber-sheathing, wood-steel, wood-concrete, and other material combinations, shall be designed using transformed section concepts and principles of engineering mechanics for the type of composite member being considered. The components of the composite member shall be connected so that the assembly acts as a unit.

The moment resistance of a composite bending member consisting of elements partially connected together shall be taken as the lesser of the resistance controlled by connections between components or by the resisting capacity of the critical member. Alternatively, a higher moment resistance shall be permitted when it is based on either an analysis explicitly recognizing the partial composite action or tests which demonstrate that this higher strength is obtained.

**5.1.13 Moment resistance of built-up members.** The adjusted moment resistance of built-up beams of three or more vertical plies of similar depth and with the applied load distributed among all plies shall be computed as the sum of the individual member moment resistances adjusted by the factors in Sec. 5.3.2.

When the individual plies are of varying stiffness, the applied load shall be apportioned to the plies based on their relative stiffnesses.

When the load is applied unequally among the plies of a built-up beam, the connections between plies shall be designed to distribute the applied load among the plies. When the applied load acts only over a portion of the beam width or is framed into one or both sides of a built-up beam with a width to depth ratio of two or more, the load sharing increase shall not apply.

The design shall include consideration of torsional moments when such moments are created on built-up members by eccentric loading conditions.

## 5.2 Conditions of Lateral Support

### 5.2.1 General

*5.2.1.1 Consideration of lateral support conditions.* The reduction in moment resistance of members bent about their strong axis because of lack of fully effective lateral stability shall be accounted for according to the provisions of this section and Sec. 5.2.3.

Lateral stability of members during construction shall conform with approved engineering methods.

*5.2.1.2 General requirements for lateral bracing.* Flexural members with depths exceeding twice their widths and loaded about the axis with the larger moment of inertia, $I_x$, shall be provided with restraint at points of support to prevent rotation and lateral displacement.

Lateral bracing is not required for members of circular or square cross-section or for rectangular members bent about only the weak axis.

Lateral bracing shall prevent lateral motion of the flexural compression face and rotational displacement of the member at the braced locations.

Alternatively for solid sawn members, restraint to prevent rotation or lateral displacement shall be provided as follows, based on the ratio of depth to width, d/b, based on nominal dimensions:

(a) $d/b \leq 2$: no lateral support shall be required;

(b) $2 < d/b < 5$: the ends shall be held in position by full-depth solid blocking, bridging, hanger, nailing, or bolting to other framing members, or other acceptable means;

(c) $5 \leq d/b < 6$: the compression edge shall have continuous lateral support for its entire length;

(d) $6 \leq d/b < 7$: bridging, full-depth solid blocking, or cross bracing shall be installed at intervals not exceeding 8 ft. (2.4 m) unless both edges are held in line or unless the compression edge of the member is supported throughout its length to prevent lateral displacement by adequate sheathing or subflooring, and the ends at points of bearing have lateral support to prevent rotation;

(e) $d/b \geq 7$: both edges shall be held in line for their entire length.

*5.2.1.3 Effective laterally unsupported length.* The effective unbraced length of a prismatic bending member or member segment to be analyzed by the general lateral torsional buckling equation, Eq. 5.2-8, shall be taken as the actual length between bracing points of the compression side of the beam, $\ell_u$.

Alternatively, for solid rectangular members of width b and depth d, the simplified critical buckling equation given by Eq. 5.2-7 may be used. For this equation, the actual length between lateral supports, $\ell_u$, shall be replaced by the effective length, $\ell_e$, which accounts for both the lateral motion and torsional phenomena. If the equivalent unbraced length approach is used, $\ell_e$ shall be determined as shown in Table 5.2-1.

Lateral bracing shall be provided to all solid, rectangular beams such that the beam slenderness ratio, $R_B$, does not exceed 50, where:

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} \tag{5.2-1}$$

**TABLE 5.2-1.**
**Factors for determination of $\ell_e$ for solid rectangular members using equivalent unbraced length approach.**

| Span Condition | Loading Condition | Bracing Condition | $\ell_e$ | | |
|---|---|---|---|---|---|
| | | | $\ell_u/d < 7$ | $7 \leq \ell_u/d \leq 14.3$ | $\ell_u/d > 14.3$ |
| | Any condition not listed below | | $2.06\,\ell_u$ | $1.84\,\ell_u$ | $1.63\,\ell_u + 3d$ |
| Single span | concentrated load at midspan | braced at ends only | $1.80\,\ell_u$ | | $1.37\,\ell_u + 3d$ |
| | uniformly distributed load | braced at ends only | $2.06\,\ell_u$ | | $1.63\,\ell_u + 3d$ |
| Cantilever | concentrated load at unsupported end | — | $1.87\,\ell_u$ | | $1.44\,\ell_u + 3d$ |
| | uniformly distributed load | — | $1.33\,\ell_u$ | | $0.90\,\ell_u + 3d$ |
| Span of length, L | uniformly spaced concentrated loads: | braced at each concentrated load: | | $\ell_e$ | |
| | one load | $\ell_u = L/2$ | | $1.11\,\ell_u$ | |
| | two loads | $\ell_u = L/3$ | | $1.68\,\ell_u$ | |
| | three loads | $\ell_u = L/4$ | | $1.54\,\ell_u$ | |
| | four loads | $\ell_u = L/5$ | | $1.68\,\ell_u$ | |
| | five loads | $\ell_u = L/6$ | | $1.73\,\ell_u$ | |
| | six loads | $\ell_u = L/7$ | | $1.84\,\ell_u$ | |
| | seven or more loads | — | | $1.84\,\ell_u$ | |
| Span with equal end moments | — | — | | $1.84\,\ell_u$ | |

**5.2.2 Moment resistance of laterally supported beams.** The provisions of this section shall be limited to bending members of circular or square cross-section, to rectangular members bent about their weak axis, to members with continuous lateral support of the compression face, and members braced in accordance with the alternative requirements of Sec. 5.2.1.2. The adjusted moment resistance of a prismatic flexural member bent about the strong (x-x) axis is:

$$M' = M_x' = S_x F_{bx}' \tag{5.2-2}$$

where $M' = M_x'$ is adjusted strong (x-x) axis moment resistance; $S_x$ is section modulus for strong (x-x) axis bending; $F_{bx}'$ is adjusted bending strength for strong (x-x) axis bending; and $C_L$, beam stability factor, is 1.0.

The adjusted moment resistance of a prismatic flexural member about its weak (y-y) axis is:

$$M' = M_y' = S_y F_{by}' \tag{5.2-3}$$

where $M' = M_y'$ is member weak (y-y) axis moment resistance; $S_y$ is section modulus for weak (y-y) axis bending; $F_{by}'$ is adjusted bending strength for weak axis bending; and $C_L$, beam stability factor, is 1.0.

**5.2.3 Moment resistance of members without full lateral support.** The provisions of this section shall apply to prismatic bending members not meeting the limitations of Sec. 5.2.2.

*5.2.3.1 Strength and stiffness.* The modulus of elasticity values used in the equations of this section are the adjusted value at the fifth percentile, $E_{05}'$. When the strong axis and weak axis values differ, the weak axis values, $E_{y05}'$, shall be used.

The volume effect factor, $C_V$, for glued laminated timbers shall be taken as unity in the computation of $F_{bx}'$ for use in Eq. 5.2-2.

*5.2.3.2 Prismatic beams.* The adjusted moment resistance about the strong (x-x) axis of a laterally unbraced rectangular prismatic beam or an unbraced portion thereof is:

$$M' = C_L\, S_x\, F_{bx}^* \tag{5.2-4}$$

The beam stability factor, $C_L$, shall be computed as:

$$C_L = \frac{1+\alpha_b}{2c_b} - \sqrt{\left(\frac{1+\alpha_b}{2c_b}\right)^2 - \frac{\alpha_b}{c_b}} \tag{5.2-5}$$

where:

$$\alpha_b = \frac{\phi_s M_e}{\lambda \phi_b M_x^*} \tag{5.2-6}$$

and $S_x$ is section modulus for strong (x-x) axis bending; $M_x^*$ is moment resistance for strong (x-x) axis bending multiplied by all applicable adjustment factors except $C_{fu}$, $C_V$, and $C_L$; $c_b$ is 0.95; $\phi_s$ is resistance factor for stability = 0.85; and $M_e$ is elastic lateral buckling moment.

When the provisions of Sec. 5.2.1.3 for rectangular sections are used to determine the effective laterally unbraced length, $\ell_e$, the elastic lateral buckling moment shall be computed as:

$$M_e = 2.40 E_{y05}' \frac{I_y}{\ell_e} \tag{5.2-7}$$

*5.2.3.3 Nonrectangular members.* For nonrectangular members excluding I-joists, and when the general provisions for determining the elastic lateral torsional buckling moment are used, the buckling moment shall be computed as:

$$M_e = \frac{\pi C_b}{1.15\, \ell_u} \sqrt{E_{y05}' I_y G' J} \tag{5.2-8}$$

where $\ell_u$ is unbraced length; $E_{y05}'$ is adjusted modulus of elasticity for bending about the weak (y-y) axis at the fifth percentile; $I_y$ is moment of inertia about the weak axis; $G'$ is adjusted shear modulus (shall be taken as $E_{y05}'/16$ for solid sawn and glued laminated members); and J is torsional constant. For a rectangular member with a narrow face dimension, b, and a wide face dimension, d, J shall be taken as:

$$J = \frac{db^3}{3}\left(1 - 0.63\frac{b}{d}\right) \tag{5.2-9}$$

$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2$ but $C_b \le 2.3$ when the largest moment in the beam or beam segment being considered is at the end of the unbraced segment and where $M_1/M_2$ is the ratio of the end moment smaller in magnitude, $M_1$, to the larger end moment, $M_2$. $M_1/M_2$ is negative when the end moments produce single curvature.

$C_b$ = 1.0 for unbraced cantilevers and for unbraced members and member segments where the largest moment is not at an end of the unbraced segment.

When the volume effect factor, $C_V$, is other than unity, the adjusted moment resistance of a laterally unbraced beam shall be taken as the smaller of the values from Eqs. 5.2-2 and 5.2-4.

The moment resistance of laterally unbraced tapered beams shall be determined using a rational analysis.

*5.2.3.4 Wood I-joists.* The lateral stability of wood I-joists shall be computed in accordance with Sec. 5.2.3.2 considering the section properties of the compression flange only. The compression flange shall be evaluated as a column continuously restrained in the direction of the web.

## 5.3 Moment Resistance of Assemblies

**5.3.1 Scope.** The provisions of this section shall be used to determine the moment resistance of sheathed structural assemblies unless a complete structural analysis including load sharing and partial composite action is carried out or the benefits of assembly effects are neglected. Such assemblies include light-frame floors, walls, and roofs, and other structural configurations with parallel flexural members joined by sheathing.

If a structural analysis based on load sharing is used, the loads in the analysis shall be distributed to each member based on that member's stiffness, relative to the stiffness of the entire assembly.

Provisions for joined parallel assemblies (built-up members) are given in Sec. 5.1.13. Special provisions for sheathed truss chords are contained in Sec. 6.6.

**5.3.2 Adjustment factors for uniformly-loaded assemblies.** This section includes resistance adjustment factors, which shall be applied in conjunction with tributary area loading assumptions to recognize the increased assembly performance over single member stiffness and resistance.

*5.3.2.1 Composite action factor.* When computing deflections, use of the following composite action factor, $C_E$, shall be permitted in determining the stiffness of solid sawn members, provided that the assemblies consist of members 12 in. (305 mm) or less in depth, spaced not more than 24 in. (610 mm) on center, and connected by structural-use panels 15/32 in. (12 mm) or more in thickness:

$C_E$ = 1.00 for nailed assemblies,
$C_E$ = 1.10 for nailed-glued assemblies,
$C_E$ = 1.15 for joint-glued assemblies.

A nailed-glued assembly includes structural-use panels fastened to framing using both nails spaced no more than 8 in. (205 mm) on center and an elastomeric adhesive. If the nailed-glued assembly has no gaps, or if sheathing elements are connected by glued tongue-and-groove joints, the use of the factor for joint-glued assemblies shall be permitted. The value of $C_E$ shall be taken as unity for parallel members sheathed by boards, decking, or similar nonpanel products. Stiffness increases for wood I-joists used in assemblies shall be based on principles of engineering mechanics.

*5.3.2.2 Load-sharing factor.* The moment resistance of assemblies consisting of three or more framing members spaced not more than 24 in. (610 mm) on center, and connected by load-distributing elements, such as sheathing, which are adequate to support the applied uniform load shall be permitted to be multiplied by the following load-sharing factor, $C_r$:

$C_r$ = 1.15 for sawn lumber framing members;
$C_r$ = 1.05 for glued-laminated timber, I-beams, and structural composite lumber (SCL);
$C_r$ = 1.15 for prefabricated I-joists with visually graded lumber flanges;
$C_r$ = 1.07 for prefabricated I-joists with machine stress-rated (MSR) lumber flanges; and
$C_r$ = 1.04 for prefabricated I-joists with SCL flanges.

The load-sharing factor, $C_r$, applies only to moment resistance. For trusses spaced not more than 24 in. (610 mm) on center and fabricated using solid sawn lumber, the $C_r$ adjustment factor of 1.15 shall be permitted in the adjusted moment resistance, $M'$, of all truss members.

## 5.4 Resistance of Members in Shear

**5.4.1 Calculation of design shear force.** When the loads causing bending are placed on the beam face opposite to the bearing area of a support, all loads located within a distance equal to the member depth, d, from the face of the member support are not required to be included in the calculation of the required shear force except in the design of I-joists. For other loading conditions and for I-joists, the design shear force shall be taken as that at the face of the support.

For I-joists and I-beams used as simple span members, the design shear shall be computed considering all loads on the clear span plus one-half of the minimum required bearing length. When these members are continuous over a support, the design shear shall be computed at the center of the support.

When the loading includes a single moving load, this load shall be placed at a distance, d, from the face of the member support and included in the design shear force. When the loading includes two or more moving loads, the pattern of loads shall be placed so the shear at the distance, d, from the face of the member support is at a maximum.

**5.4.2 Flexural shear resistance.** The adjusted shear resistance of a flexural member, V′, shall be computed from the following equation:

$$V' = \frac{F_v' I b}{Q} \tag{5.4-1}$$

where $F_v'$ is adjusted horizontal shear strength; I is member moment of inertia for bending corresponding to the shear direction of interest; b is member width; and Q is statical moment of an area about the neutral axis.

For a rectangular section of width, b, and depth, d, Eq. 5.4-1 simplifies to:

$$V' = \frac{2}{3} F_v' bd \tag{5.4-2}$$

Alternatively, for continuous or cantilevered bending members of sawn lumber, the adjusted shear resistance at locations at least three times the member depth from the member end shall be permitted to be determined using Eq. 5.4-1 or the following:

$$V' = \left( \begin{matrix} V' \text{ from Eq.} \\ 5.4-1 \text{ or } 5.4-2 \end{matrix} \right) \left( 1 + \frac{(x-3d)}{3d} \right)$$
$$but \le 2 \left( \begin{matrix} V' \text{ from Eq.} \\ 5.4-1 \text{ or } 5.4-2 \end{matrix} \right) \tag{5.4-3}$$

where x is the distance from the end of the member.

For rigidly connected composite members, the I and Q values in Eq. 5.4-1 shall be based on transformed section properties and the resistance limited to the value at which the first component reaches its adjusted resistance in shear.

**5.4.3 Shear resistance in the vicinity of notches.** At sections within the length of a notch in a rectangular member of depth, d, the adjusted notched section shear resistance shall be computed as:

$$V' = \left( \frac{2}{3} F_v' \, b \, d_n \right) \left( \frac{d_n}{d} \right) \tag{5.4-4}$$

where d is the depth of the unnotched member and $d_n$ is the depth of the member remaining at the notch.

Alternatively, if a gradual tapered cut at an angle θ from the grain direction is provided at the notch end(s) to reduce the stress concentrations, the adjusted notched section shear resistance shall be permitted to be computed as:

$$V' = \left( \frac{2}{3} F_v' \, b \, d_n \right) \left( 1 - \frac{(d - d_n) \sin\theta}{d} \right) \tag{5.4-5}$$

**5.4.4 Shear resistance in the vicinity of connections.** Where a connection to a rectangular flexural member transfers a force large enough to generate more than one-half the member shear force on either side of the connection, the adjusted resistance in horizontal shear shall be computed as:

$$V' = \left( \frac{2}{3} F_v' \, b \, d_e \right) \left( \frac{d_e}{d} \right) \tag{5.4-6}$$

where $d_e$ is the effective depth of member at a connection, measured as the depth of the member minus the distance from the unloaded edge of the member to the nearest edge of the nearest connector for split rings and shear plates, or to the center of the fastener nearest the unloaded edge for other fasteners.

Alternatively, when the entire connection is more than 3d from the member ends, the adjusted resistance in horizontal shear shall be permitted to be computed as:

$$V' = \left( \frac{2}{3} F_v' b \, d_e \right) \left[ 1 + \left( \frac{x - 3d}{6d} \right) \right] \tag{5.4-7}$$
$$but \le F_v' b \, d_e$$

where x is the distance from the end of the member.

## 5.5 Resistance of Members in Torsion

The adjusted torsion resistance, $M_t'$, of a solid rectangular beam shall be computed as:

$$M_t' = \frac{b^2\, d^2\, F_{tv}'}{3d + 1.8b} \tag{5.5-1}$$

where b is section width, the smaller side dimension; d is section depth, the larger side dimension; and $F_{tv}'$ is adjusted torsional shear strength.

For section shapes other than rectangular, the adjusted member torsion resistance shall be based on linear elastic torsion analysis with $F_{tv}'$ used as the maximum torsional shearing strength.

For solid sawn lumber, $F_{tv}'$ shall be taken as two-thirds of the adjusted horizontal shear strength, $F_v'$. For glued laminated members, $F_{tv}'$ shall be limited to $F_{rt}'$, the adjusted radial tension strength.

The torsional resistance of structural composite lumber members is not within the scope of this standard and requires special investigation.

## 5.6 Curved or Pitched/Tapered Curved Glued Laminated Beams

**5.6.1 Adjustment of moment resistance for curvature.** The adjusted moment resistance of a curved laterally supported glued laminated member of a constant cross-section shall be multiplied by the curvature factor $C_c$:

$$C_c = 1 - 2000\left(\frac{t}{R_f}\right)^2 \tag{5.6-1}$$

where t is thickness of lamination, and $R_f$ is radius of curvature at the inside face of a lamination in a curved member.

Curved members shall have an $R_f$ of at least 100t for hard woods and southern pine, and at least 125t for other soft woods.

**5.6.2 Radial tension and compression in curved members.** The design of curved beams, pitched and tapered curved beams, and arches shall consider the radial tension and compression that occurs in these members. Radial tension is induced when the applied moments act to decrease the curvature (increase the radius), and radial compression is induced when the applied moments act to increase the curvature (decrease the radius). The provisions of Sec. 5.6.2.1 and 5.6.2.2 apply to these conditions.

*5.6.2.1 Curved members of constant cross-section.* The adjusted moment resistance of a curved member of a constant rectangular cross-section is limited to the following value by radial stress requirements:

$$M' = \frac{2}{3} R_m b d F_r' \tag{5.6-2}$$

where $R_m$ is radius of curvature at the middepth of a member; b is member width; d is member depth; and $F_r'$ is the adjusted strength in radial stress loading.

The value of $F_r'$ depends on whether the stress is applied in tension or compression and whether radial reinforcement is provided, as follows.

$F_r'$ = $F_{rt}'$, adjusted radial tension strength, when the radial stress is tension and no radial reinforcement is provided.

$F_r'$ = $F_v'/3$ when the radial stress is tension, the species is Douglas-fir-Larch, Douglas-fir South, Hem-Fir, Western Woods, or Canadian soft wood species, and either the design load is wind or earthquake, or reinforcement is provided to carry the entire radial force.

$F_r'$ = $F_{rc}'$, adjusted radial compression strength, when the radial stress is compression. Unless otherwise specified, $F_{rc}'$ may be taken as $F_{c\perp}'$, adjusted perpendicular to grain compression strength.

For use in computing M', radial strengths shall be adjusted for moisture and temperature only.

*5.6.2.2 Pitched and tapered glued laminated beams.* The design of pitched and tapered glued laminated beams, including radial stresses, shall satisfy the provisions contained in App. A2.

## 5.7 Ponding

Roof systems shall be investigated by structural analysis in accordance with the provisions of App. A3.

# CHAPTER 6

# Members With Combined Bending and Axial Loads

## 6.1 General

**6.1.1 Scope.** The provisions of this chapter apply to members subjected to: (a) bending about both principal axes and/or bending combined with axial loads, whether tensile or compressive; and (b) eccentrically loaded columns.

**6.1.2 Member Design.** The adjusted member resistances, M′, P′, and T′, contained in the interaction equations of this chapter, shall be computed using the equations contained in Chap. 3, 4, and 5. Many of the parameters in the interaction equations vary along the length of the member. For such cases, the member design shall be based on the calculation at the most critical location along the member.

The following resistance factors, $\phi$, shall apply for use in the equations of this chapter:

| | |
|---|---|
| Bending | $\phi_b = 0.85$ |
| Tension parallel to grain | $\phi_t = 0.80$ |
| Compression parallel to grain | $\phi_c = 0.90$ |

The time-effect factor, $\lambda$, from Sec. 1.4.3 shall be applied as required in Chap. 3, 4, and 5 and where included in the equations of this chapter. A single time-effect factor value, as given by Table 1.4-2 for the loading combination under consideration, shall be used for all terms in the interaction equations.

## 6.2 Resistance in Combined Bending and Axial Tension

The resistance of a member subjected to combined bending and tension loading shall be controlled by either the tension face, for which lateral stability is not a concern, or by the compression face when the axial tension is not adequate to preclude lateral torsional buckling. The following equations shall be satisfied.

(a) Tension face (interaction with lateral stability assumed):

$$\frac{T_u}{\lambda\phi_t T'} + \frac{M_{ux}}{\lambda\phi_b M_s'} + \frac{M_{uy}}{\lambda\phi_b M_y'} \le 1.0 \qquad (6.2\text{-}1)$$

(b) Compression face (interaction with axial tension reducing the conditions for lateral torsional buckling):

$$\frac{\left(M_{ux} - \frac{d}{6}T_u\right)}{\lambda\phi_b M_x'} + \frac{M_{uy}}{\lambda\phi_b M_y'\left(1 - \frac{M_{ux}}{\phi_b M_e}\right)^2} \le 1.0 \qquad (6.2\text{-}2)$$

When the member is nonrectangular, the d/6 quantity in the first term, where d is the member depth, shall be replaced by $S_x/A$, the ratio of strong-axis section modulus to the gross cross-sectional area.

(c) Interaction at compression face with axial tension absent.

When the tension force does not act simultaneously with the bending moments, Eq. 6.2-2 with the axial load, $T_u$, set equal to zero shall be satisfied.

In Eqs. 6.2-1 and 6.2-2:

$T_u$ = tensile force due to factored loads.
$M_{ux}$, $M_{uy}$ = moment due to factored loads about the strong and weak axis, respectively.
$M_x'$, $M_y'$ = adjusted moment resistance about the strong and weak axis, respectively, for the existing lateral bracing conditions.
$M_e$ = elastic lateral buckling moment from Sec. 5.2.3.
$M_s'$ = $M_x'$ computed with the beam stability factor, $C_L$, equal to unity and with any volume factor, $C_V$, included.

## 6.3 Member Resistance in Biaxial Bending and in Combined Bending and Axial Compression

**6.3.1 Beams, columns, and frame members.** For a prismatic member loaded in biaxial bending, or in axial compression plus bending about one or both principal axes, the following condition shall be satisfied:

$$\left(\frac{P_u}{\lambda\phi_c P'}\right)^2 + \frac{M_{mx}}{\lambda\phi_b M_x'} + \frac{M_{my}}{\lambda\phi_b M_y'} \le 1.0 \qquad (6.3\text{-}1)$$

where:

$P_u$ = axial compressive force due to factored loads.
$P'$ = adjusted resistance for axial compression acting alone (without moments) for the axis of buckling providing the lower value of $P'$.
$M_{mx}$, $M_{my}$ = factored moment, including any magnification for second-order effects, for strong and weak axes, respectively.
$M_x'$, $M_y'$ = adjusted moment resistances for strong and weak axes, respectively, from the equations in Chap. 5 with $C_b = 1.00$.

All terms in Eq. 6.3-1 shall be taken as positive.

If a second-order analysis is not used, the magnified moments, $M_{mx}$ and $M_{my}$, shall be determined using the following magnified moment equations, which include separate multipliers for the first order moments from loads that result in no appreciable sidesway, $M_{bx}$ and $M_{by}$, and the first order moments from any loads acting on a rigid frame or cantilever member which result in appreciable sidesway, $M_{sx}$ and $M_{sy}$:

$$M_{mx} = B_{bx}M_{bx} + B_{sx}M_{sx} \qquad (6.3\text{-}2)$$

$$M_{my} = B_{by}M_{by} + B_{sy}M_{sy} \qquad (6.3\text{-}3)$$

where:

$M_{bx}$, $M_{by}$ = factored moment from loads that result in no appreciable structural sidesway (no lateral translation), calculated by a conventional first-order analysis, strong (x-x) and weak (y-y) axes, respectively.
$M_{sx}$, $M_{sy}$ = factored moment from loads that result in appreciable sidesway (lateral translation), calculated by a conventional first-order analysis, strong and weak axes, respectively.

For members not braced against sidesway, both $B_{bx}$, $B_{by}$ and $B_{sx}$, $B_{sy}$ shall be computed. For members braced against appreciable sidesway, $B_{sx}$, $B_{sy}$ are permitted to be taken as zero.

$$B_{bx} = \frac{C_{mx}}{\left(1 - \dfrac{P_u}{\phi_c P_{ex}}\right)} \geq 1.0 \qquad (6.3\text{-}4)$$

$$B_{by} = \frac{C_{my}}{\left(1 - \dfrac{P_u}{\phi_c P_{ey}} - \left(\dfrac{M_{ux}}{\phi_b M_e}\right)^2\right)} \geq 1.0 \qquad (6.3\text{-}5)$$

$$B_{sx} = \frac{1}{\left(1 - \dfrac{\Sigma P_u}{\phi_c \Sigma P_{ex}}\right)} \geq 1.0 \qquad (6.3\text{-}6)$$

$$B_{sy} = \frac{1}{\left(1 - \dfrac{\Sigma P_u}{\phi_c \Sigma P_{ey}}\right)} \geq 1.0 \qquad (6.3\text{-}7)$$

where:

$P_{ex}$, $P_{ey}$ = critical buckling resistance about strong (x-x) and weak (y-y) axes, respectively.
$\Sigma P_u$ = sum of axial compressive forces due to factored loads for columns involved in the sidesway mode being considered.
$\Sigma P_{ex}$, $\Sigma P_{ey}$ = sum of the critical buckling resistance for columns involved in the sidesway mode being considered, with all columns moving in the sidesway motion which bends the member being investigated about its strong axis for $\Sigma P_{ex}$ or its weak axis for $\Sigma P_{ey}$, respectively.

For a single cantilever member, only the axial quantities for the single member shall be included in these sums.

The factors relating actual moment diagram shape to an equivalent uniform moment diagram, $C_{mx}$ and $C_{my}$, for the strong and weak axes, respectively, shall be taken as follows:

(a) For compression members braced against lateral joint translation with member ends restrained against rotation and not subject to transverse loading between their supports and in the plane of bending being considered:

$$C_m = 0.60 - 0.40\left(\frac{M_1}{M_2}\right) \qquad (6.3\text{-}8)$$

where $M_1/M_2$ is the ratio of the smaller magnitude end moment to the larger moment at ends of that portion of the member unbraced in the plane of bending under consideration, with $M_1/M_2$ negative for single curvature.

(b) For compression members braced against joint translation in the plane of loading under consideration and subjected to transverse loadings between these joints, $C_m$ shall be determined by rational analysis. As an alternative to such analysis, the following values shall be used:
- for members whose ends are restrained against rotation, $C_m = 0.85$;
- for members whose ends are unrestrained against rotation, $C_m = 1.00$.

**6.3.2 Truss members.** The requirements of Sec. 6.3.1 also apply to members of trusses. However, members or portions of members extending between joints of a truss are permitted to be analyzed as braced (no translation) in the plane of the truss. For the other member axis, the lateral bracing of the truss joints shall be considered in determining the laterally unbraced length if the member is not continuously braced by roof, floor, or similar sheathing.

## 6.4 Columns Loaded on Side Brackets

Columns loaded by a side bracket located within the upper quarter of the unbraced column segment shall be designed for the following two equivalent loads.

(a) Move the axial load acting on the bracket, $P_a$, so that it acts as a concentric load added to any other axial loads acting over the full column length.

(b) Add a side (lateral) load, $P_s$, at the mid-height of the unbraced column or column segment and in the direction that causes moment in the same direction as caused by the actual eccentric load on the bracket:

$$P_s = \frac{3e_b \ell_{br} P_a}{\ell_u^2} \tag{6.4-1}$$

where:

$\ell_{br}$ = distance from the bottom of the column or column segment to the top of the column bracket.

$e_b$ = eccentricity of the load placed on the bracket, i.e., the horizontal distance from the load to the centroid of the column section.

$\ell_u$ = unbraced column length for buckling corresponding to the bracket moment.

The column then shall be designed as a beam column with these loads using the provisions of Sec. 6.3.

When the bracket is not in the upper quarter of the unbraced column segment length, either a rational analysis shall be made or Eq. 6.4-1 with $\ell_{br} = 0.75\ell_u$ shall be used.

## 6.5 Arches

The design of structural glued laminated timber arches loaded in combined bending and axial compression shall satisfy the provisions contained in App. A2.

## 6.6 Trusses

Trusses covered by this chapter include (a) triangulated assemblies of components and (b) pre-engineered prefabricated proprietary connected trusses with wood components.

**6.6.1 Sheathed truss compression chords.** The strong axis moment of inertia, $I_x$, of a sheathed truss compression chord is permitted to be multiplied by the buckling stiffness factor, $C_T$, when the following conditions are satisfied:

- chord size is nominal 2×4 or smaller,
- chord material is sawn lumber,
- chord orientation is wide face vertical,
- chord effective buckling length is $\leq$ 96 in. (2.4 m),
- sheathing is $\geq$ 3/8 in. (9.5 mm) thick structural-use panel, and
- sheathing is attached in accordance with approved nailing practice.

The buckling stiffness factor for this condition is:

$$C_T = 1 + \frac{K_M \ell_e}{E'_{05}} \tag{6.6-1}$$

where:

$\ell_e$ = effective unbraced length used in design of the compression chord for axial compression load.

$K_M$ = 2.3 for wood seasoned to 19% moisture content or less at the time of plywood attachment when $\ell_e$ is in inches and $E'_{05}$ is in ksi. ($K_M = 0.624$ when $\ell_e$ is in mm and $E'_{05}$ is in kPa.)

$K_M$ = 1.2 for unseasoned or partially seasoned wood at the time of plywood attachment when $\ell_e$ is in inches and $E'_{05}$ is in ksi. ($K_M = 0.326$ when $\ell_e$ is in mm and $E'_{05}$ is in kPa.)

$E'_{05}$ = adjusted modulus of elasticity at the fifth percentile.

In computing P′ as controlled by the strong axis for use as the $P_0'$ value in Eq. 6.3-1, $E_{05}'$ shall be replaced by the product of $E_{05}'$ and $C_T$ in the second form of Eq. 4.3-4. $C_T$ shall not be used as a multiplier of the gross area, A, in Eq. 4.3-1.

For trusses used under wet service conditions, $C_T$ shall be assumed equal to 1.

# CHAPTER 7

# Mechanical Connections

## 7.1 General

**7.1.1 Scope.** This chapter applies to connections for wood and wood-based members and components. Reference to wood members in this chapter shall include both solid wood members and members manufactured from wood-based materials. Reference to bolts or dowels in this chapter shall be defined as applying only to bolts or dowels in the range of diameters from 1/4 in. (6.3 mm) up to and including 1 in (25.4 mm).

Connections between and to wood members consist of connecting elements (e.g., gussets, splice plates, straps, angles, and brackets) and connectors (e.g., split rings, shear plates) or fasteners (e.g., nails, staples, spikes, wood screws, bolts, lag screws, and proprietary fastening systems).

The notation for lateral resistance, Z, Z′, is used to refer to the resistance of the entire connection, rather than to the resistance of an individual connector. In addition, the notation for withdrawal resistance, $Z_W$, $Z_W'$, refers to the total withdrawal resistance rather than to the resistance per unit of penetration. Note that these notations are different than those used in the National Design Specification.

**7.1.2 Connection design.** Connections shall be designed such that:

$$Z_u \leq \lambda \phi_z Z' \tag{7.1-1}$$

where $Z_u$ is the connection force due to factored loads, λ is the applicable time-effect factor given in Table 1.4-2, $\phi_z$ is the resistance factor for connections = 0.65, and Z′ is the adjusted connection resistance.

The adjusted connection resistance shall be determined by multiplying the reference resistance by applicable adjustments in Sec. 2.6 and in this chapter. The applicability of adjustment factors for each type of connection shall be as specified in Table 7.1-1.

**7.1.3 Adjustment factor issues for connections.** When applied to connections, the wet service factor, $C_M$, is not only based on the conditions of use, but also based on the conditions of fabrication. The reference condition of dry use refers to connections that are both fabricated from dry materials and used in dry service conditions, as defined in Sec. 2.6.

The wet service factor does not account for the effects of corrosion. Where connections will be exposed to a corrosive environment, the connection resistance shall take into account the effect of corrosion of metal connectors or connecting elements. Fasteners for use with chemically treated wood material shall be protected in accordance with App. A4 and the applicable documents in Sec. 1.2.

The diaphragm nailing factor, $C_{di}$, covered in Chap. 9 of this standard, is only applicable to shear wall and diaphragm design.

**7.1.4 Time effect factor for connections.** The time effect factor, λ, is not permitted to exceed 1.0 for connections. In addition, if failure of a non-wood connecting element or fastener controls the connection design, then λ = 1.0.

## 7.2 Material Property Basis

Connection resistances computed by the provisions of this chapter are based on specific assumptions related to material properties as discussed in this section.

**7.2.1 Fasteners, connectors, and connecting elements.** All fasteners and connectors and their nominal properties shall meet the minimum requirements of App. A4 and the applicable documents in Sec. 1.2. Metal plates, hangers, fasteners, and other metal parts shall be designed to resist applicable failure modes (e.g., tension, bending, buckling, bearing of metal on metal, and fastener shear).

**7.2.2 Specific gravity.** The design specific gravity, G, to be used for computing the dowel bearing strength and for other connection design requirements shall be based on established values for the species, species group, or grade specified in the design. The design specific gravity shall be based on oven-dry weight and volume.

Manufacture of structural glued laminated timber permits the use of different grades of lumber as well as species in the top, core, and the bottom of the member. This shall be taken into account in design of connections in the various zones of the member.

**7.2.3 Dowel bearing strength.** For connections containing bolts, lag screws, drift pins, or dowels, the dowel bearing strength, $F_{e\theta}$, of a wood member loaded at an angle to grain, θ, shall be:

$$F_{e\theta} = \frac{F_{e\|}\, F_{e\perp}}{F_{e\|} \sin^2\theta + F_{e\perp} \cos^2\theta} \qquad [7.1\text{-}4]$$

where:

$F_{e\|}$, $F_{e\perp}$ = dowel bearing strength parallel to grain and perpendicular to grain, respectively.

θ = angle of force vector with respect to a direction parallel to grain, degrees.

## 7.3 Connection Configuration Basis

Connection resistances computed by the provisions of this chapter are based on specific assumptions related to connection configuration as discussed in this section. Spacings between fasteners are defined in this chapter in terms of *pitch*, the spacing within a row and *gage*, the spacing between rows.

**7.3.1 Simple connections.** Connection resistances in this chapter are based on the assumptions of end restraint provided in Sec. 1.4.2.2.

**7.3.2 Bearing.** Design for bearing shall be in accordance with Sec. 4.5. At a bearing connection, sufficient fastening shall be provided to hold all members in line.

**7.3.3 Member stress at connection.** The presence of a connection influences the resistance of a structural member. At a minimum, the following shall be accounted for.

**Net area**: See Sec. 2.1.2 and 4.3.3. For parallel to grain loading with staggered bolts, lag screws, drift pins, or dowels, adjacent fasteners shall be considered as being placed at the critical section if the pitch spacing between fasteners in adjacent rows is less than 4D, where D is the fastener diameter. Where shear plates or split rings are staggered, adjacent connectors shall be considered as being placed at the same critical section if the parallel to grain spacing between connectors in adjacent rows is equal to or less than one connector diameter.

**Shear stress**: See Sec. 5.4.4.

**Eccentric joints**: Groups of fasteners designed to transmit axial forces into a member shall be sized and located so that the axis of each connected member intersects the effective center of resistance of the group of fasteners, unless provision is made for eccentricity, where groups of fasteners transmit eccentric forces (moments). The effects of these eccentric forces on fastener loads and member stresses shall be analyzed in accordance with established principles of engineering mechanics.

**Tension perpendicular to grain**: Designs with loads placed below the neutral axis (i.e., tension side) of bending members shall be avoided. When this loading condition cannot be avoided, mechanical reinforcement shall be provided to resist separation of the grain.

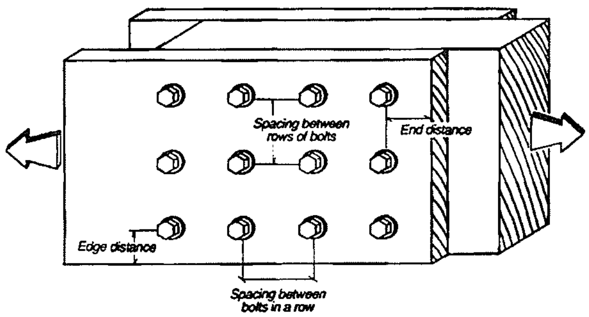
**7.3.4 Mixed fastener connections.** Design resistance of connections with more than one fastener type or size shall be based on tests or shall be justified by analysis. When adhesives and mechanical fasteners are used in combination, the differences in stiffness properties shall be accounted for in the determination of the connection design resistance.

**7.3.5 Placement of Fasteners**

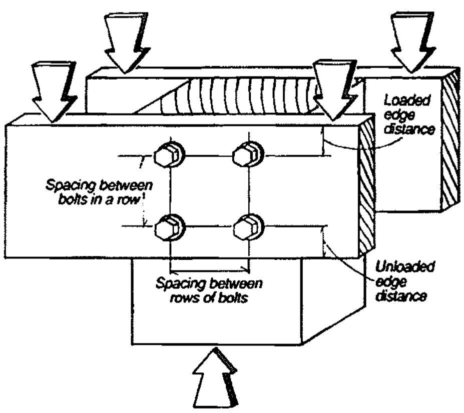
*7.3.5.1* "Edge distance" is the distance from the edge of a member to the center of the nearest fastener measured perpendicular to grain. When a member is loaded perpendicular to grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting. The unloaded edge shall be defined as the edge opposite the loaded edge (Fig. 7.3-1).

*7.3.5.2* "End distance" is the distance measured parallel to grain from the square-cut end of a member to the center of the nearest fastener (Fig. 7.3-1).

*7.3.5.3* "Spacing" is the distance between centers of fasteners measured along a line joining their centers (Fig. 7.3-1).




Parallel to grain loading




Perpendicular to grain loading

**Figure 7.3-1. Bolted connection geometry.**

*7.3.5.4* A "row of fasteners" is defined as two or more fasteners aligned with the direction of load (Fig. 7.3-1).

*7.3.5.5* "Pitch" is the spacing of fasteners within a row, and "gage" is the spacing between rows of fasteners.

**7.3.6 Multiple fasteners.** Connection resistances provided in this chapter assume that each fastener in a multiple-fastener connection is equally loaded, except as modified by $C_g$, which accounts for unequal load sharing for bolts, lag screws, split rings, shear plates, and similar devices. $C_g$ shall not apply for nails or wood screws. See Sec. 7.3.3 for eccentric joints.

*7.3.6.1 Group action factor.* If a connection contains one or more rows of bolts, lag screws, drift pins, dowels, shear plates, split rings, or similar devices, the reference connection resistance shall be multiplied by $C_g$ where:

$$C_g = \frac{1}{n_f}\sum_{i=1}^{n_r} a_i \qquad [7.3\text{-}1]$$

in which $n_f$ is the total number of fasteners in the connection; $n_r$ is number of rows in the connection; $a_i$ is the effective number of fasteners for row i due to unequal load sharing in a row, a value between 1 and $n_i$; and $n_i$ is number of equally spaced fasteners in row i, where:

$$a_i = \left[\frac{m(1 - m^{2n_i})}{(1 + R_{EA}m^{n_i})(1 + m) - 1 + m^{2n_i}}\right]\left[\frac{1 + R_{EA}}{1 - m}\right]$$

$$m = u - \sqrt{u^2 - 1}$$

$$u = 1 + \gamma\frac{s}{2}\left(\frac{1}{(EA)_m} + \frac{1}{(EA)_s}\right) \qquad [7.3\text{-}2]$$

$\gamma$ = load/slip modulus for a single fastener. Unless other supporting data are available, $\gamma$ shall be taken as:
= 500 kip/in (87.6 kN/mm) for 4 in. (102 mm) split ring or shear plate connectors,
= 400 kip/in (70.1 kN/mm) for 2-1/2 in. (64 mm) split ring or 2-5/8 in. (67 mm) shear plate connectors,
= $(180)(D^{1.5})$ kip/in. ($0.246\ D^{1.5}$ kN/mm) for bolts, lag screws, drift pins, or dowels in wood-to-wood connections,
= $(270)(D^{1.5})$ kip/in. ($0.369\ D^{1.5}$ kN/mm) for bolts, lag screws, drift pins, or dowels in wood-to-metal connections,

s = pitch spacing; measured center-to-center between fasteners in a row,

$(EA)_m$ = axial stiffness; mean modulus of elasticity of main member, psi, times gross cross-sectional area before boring or grooving,

$(EA)_s$ = axial stiffness; mean modulus of elasticity of side member(s), times sum of gross cross-sectional areas of side members before boring or grooving,

$$R_{EA} = \frac{(EA)_{min}}{(EA)_{max}}$$

$(EA)_{min}$ = smaller of $(EA)_m$ and $(EA)_s$,

$(EA)_{max}$ = larger of $(EA)_m$ and $(EA)_s$.

When fasteners in adjacent rows are staggered, $C_g$ shall be calculated based on the pitch (center-to-center) spacing between fasteners in adjacent rows and on the gage spacing (between rows). The following shall apply:

(a) If the gage spacing is less than or equal to one-quarter of the pitch spacing between fasteners in adjacent rows, adjacent rows are considered as one row with $n_i$ corresponding to the total number of fasteners in both rows. For a group of fasteners having an even number of rows, this principle shall apply to each pair of rows; for an odd number of rows, the least value calculated using combinations of pairs of rows shall apply.

(b) If the gage spacing is greater than one-quarter of the pitch spacing, $n_i$ for each row corresponds to the number of fasteners in that row.

## 7.4 Nails, Spikes, and Wood Screws

### 7.4.1 General

*7.4.1.1 Scope.* The following provisions apply to the design of connections using round wire nails and spikes having smooth or deformed shanks and wood screws. These provisions shall be used for the design of individual fasteners and connections. Alternatively, fastening for assemblies using structural use panels shall be in accordance with Chap. 8 and 9.

*7.4.1.2 Fastener properties and dimensions.* Fasteners shall meet the requirements of App. A4 and the applicable documents in Sec. 1.2.

Wood screws shall meet the requirements of ANSI/ASME B18.6.1. The length of the threaded portion of wood screws shall be at least two-thirds of the length of the shaft.

Connection resistance of nail and spike joints shall be determined based on fastener shank diameter, D, and tensile yield strength or bending yield strength, as required by this chapter.

*7.4.1.3 Installation.* Wood screws shall be installed by turning only. Nails and spikes shall be impact driven. Toe nails shall be installed at an angle of approximately thirty degrees to the member, and started at approximately one-third the length of the nail from the member end.

The diameter of lead holes for nails and spikes shall not exceed:

for $G > 0.60$, $= (0.90)\ D$
for $G \leq 0.60$, $= (0.75)\ D$

where G is specific gravity and D is shank diameter.

Lead holes for wood screws in wood members shall be bored as follows:

(a) The lead hole for the shank shall have a diameter equal to:

for $G > 0.60$, $= (1.0)\ D$
for $G \leq 0.60$, $= (0.875)\ D$

and the same depth as the length of the unthreaded shank.

(b) The lead hole for the threaded portion shall have a diameter equal to:

Lateral resistance:

for $G > 0.60$, $= (1.0)\ D_R$
for $G \leq 0.60$, $= (0.875)\ D_R$

**TABLE 7.1-1.**
**Applicability of adjustment factors for connections for LRFD[1]**

| Adjusted Property = | Reference Property x | Diaphragm | Group Action | Geometry | Penetration Depth | End Grain | Metal Side Plate | Toe-Nail |
|---|---|---|---|---|---|---|---|---|
| | | | | ***Nails, spikes*** | | | | |
| $Z'$ = | $Z$ | $C_{di}$ | | | $C_d$ | $C_{eg}$ | | $C_{tn}$ |
| $Z_W'$ = | $Z_W$ | | | | | $C_{eg}$ | | $C_{tn}$ |
| | | | | ***Wood screws*** | | | | |
| $Z'$ = | $Z$ | | | | $C_d$ | $C_{eg}$ | | |
| $Z_W'$ = | $Z_W$ | | | | | | | |
| | | | | ***Bolts*** | | | | |
| $Z'$ = | $Z$ | | $C_g$ | $C_\Delta$ | | | | |
| | | | | ***Lag screws, drift pins*** | | | | |
| $Z'$ = | $Z$ | | $C_g$ | $C_\Delta$ | $C_d$ | $C_{eg}$ | | |
| $Z_W'$ = | $Z_W$ | | | | | $C_{eg}$ | | |
| | | | | ***Shear plates, split rings*** | | | | |
| $Z_\parallel'$ = | $Z_\parallel$ | | $C_g$ | $C_\Delta$ | $C_d$ | | $C_{st}$ | |
| $Z_\perp'$ = | $Z_\perp$ | | $C_g$ | $C_\Delta$ | $C_d$ | | | |

[1]These adjustment factors shall be applied in addition to the end-use adjustment factors given in Sec. 2.6.

Withdrawal resistance:

for $G > 0.60$, $= (0.9)\ D_R$
for $G \leq 0.60$, $= (0.7)\ D_R$

and the same depth as the length of the threaded portion of the wood screw where G is wood specific gravity and $D_R$ is wood screw root diameter.

No axial (withdrawal) resistance shall be permitted for wood screws, nails, and spikes in lead holes larger than those specified in this section.

**7.4.2 Spacing of fasteners.** Minimum spacing of nails, spikes, or wood screws in a single connection shall be as follows:

**Pitch spacing**: For all angles of lateral loading to grain, the minimum spacing between fasteners in a row shall be at least 10D for wood side plates and 7D for steel side plates.

**Gage spacing**: For all angles of lateral loading to grain, the minimum spacing between rows shall be 5D.

**End distance**: The minimum distance from the end of the member to the center of the nearest fastener shall be:

for lateral tension loading: 15D (wood side plates), 10D (steel side plates);
for lateral compression loading: 10D (wood side plates), 5D (steel side plates).

**Edge distance**: The minimum distance from the edge of the member to the nearest fastener shall be 5D at any unloaded edge, and 10D at any loaded edge.

**7.4.3 Resistance to lateral forces.**

*7.4.3.1 Reference lateral resistance: single shear.* The reference lateral resistance of a connection using steel nails, spikes, or wood screws loaded perpendicular to the fastener axis, installed perpendicular to the member face and subject to single shear shall be the minimum computed using all equations in Table 7.4-1(a) (for nails or spikes) or Table 7.4-1(b) (for wood screws) multiplied by the number of fasteners, $n_f$.

For connections with steel side members, the equation for yield mode $I_s$ in Tables 7.3-1(a) and (b) shall not apply, and the resistance for that mode shall be computed as the resistance for the fasteners bearing on steel side members.

**TABLE 7.4-1(a)**
**Nail or spike reference lateral resistance, Z: one fastener, two-member (single shear) connection.**

| Yield Mode | Applicable Equation | |
|---|---|---|
| $I_s$ | $Z = \dfrac{3.3\ D t_s F_{es}}{K_D}$ | (7.4-1) |
| $III_m$ | $Z = \dfrac{3.3\ k_1 D p F_{em}}{K_D(1 + 2R_e)}$ where: $k_1 = (-1) + \sqrt{2(1 + R_e) + \dfrac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}p^2}}$ | (7.4-2) |
| $III_s$ | $Z = \dfrac{3.3 k_2 D t_s F_{em}}{K_D(2 + R_e)}$ where: $k_1 = (-1) + \sqrt{\dfrac{2(1 + R_e)}{R_e} + \dfrac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}t_s^2}}$ | (7.4-3) |
| IV | $\dfrac{3.3 D^2}{K_D}\sqrt{\dfrac{2F_{em}F_{yb}}{3(1 + R_e)}}$ | (7.4-4) |

Note: $R_e = F_{em}/F_{es}$
$p$ = shank penetration into member holding point
$K_D = 2.2$ for $D \leq 0.17''$ (4.3 mm)
$= 10D + 0.5$ for $0.17'' < D < 0.25''$ ($0.38D + 0.56$ for 4.3 mm $< D <$ 6.4 mm)
$= 3.0$ for $D \geq 0.25''$ (6.4 mm)

**TABLE 7.4-1(b)**
**Wood screw reference lateral resistance, Z: one fastener, two-member (single shear) connection.**

| Yield Mode | Applicable Equation | |
|---|---|---|
| $I_s$ | $Z = \frac{3.3 D t_s F_{es}}{K_D}$ | (7.4-5) |
| $III_s$ | $Z = \frac{3.3 k_3 D t_s F_{em}}{K_D (2 + R_e)}$ <br> where: $k_3 = (-1) + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{F_{yb}(2 + R_e) D^2}{2 F_{em} t_s^2}}$ | (7.4-6) |
| IV | $Z = \frac{3.3 D^2}{K_D} \sqrt{\frac{1.75 F_{em} F_{yb}}{3(1 + R_e)}}$ | (7.4-7) |

Note: $R_e = F_{em}/F_{es}$
$K_D$ = 2.2 for D ≤ 0.17″ (4.3 mm)
= 10D + 0.5 for 0.17″ < D < 0.25″ (0.38D + 0.56 for 4.3 mm < D < 6.4 mm)
= 3.0 for D ≥ 0.25″ (6.4 mm)

*7.4.3.2 Reference lateral resistance: double shear.* For joints containing three wood members with two fastener shear planes, the reference lateral resistance shall be twice the lateral resistance of the weaker single shear joint as illustrated in Fig. 7.4-1(b) and (c). The center member must be thicker than 6D. If fastener penetration in the third member (see Fig. 7.4-1) is less than 12D for nails and spikes, or 7D for wood screws, then the penetration depth factor, $C_d$, applies as specified in Sec. 7.4.3.3.

*7.4.3.3 Adjusted lateral resistance.* The adjusted lateral resistance, Z′, shall be computed by multiplying the reference resistance by applicable adjustment factors in accordance with Sec. 2.6 and 7.1.3. In addition to the adjustment factors in Sec. 2.6 and 7.1.3, the following shall apply.

**Penetration depth**: The reference lateral resistance shall be multiplied by the penetration depth factor, $C_d$, as follows.

For nails and spikes, the actual shank penetration into the member holding the point, p, shall be greater than or equal to 6D.

For 6D ≤ p < 12D, $C_d$ = p/12D. (7.4-8)
For p ≥ 12D, $C_d$ = 1.0.

For wood screws, the actual shank penetration into the member holding the point, p, shall be greater than or equal to 4D.

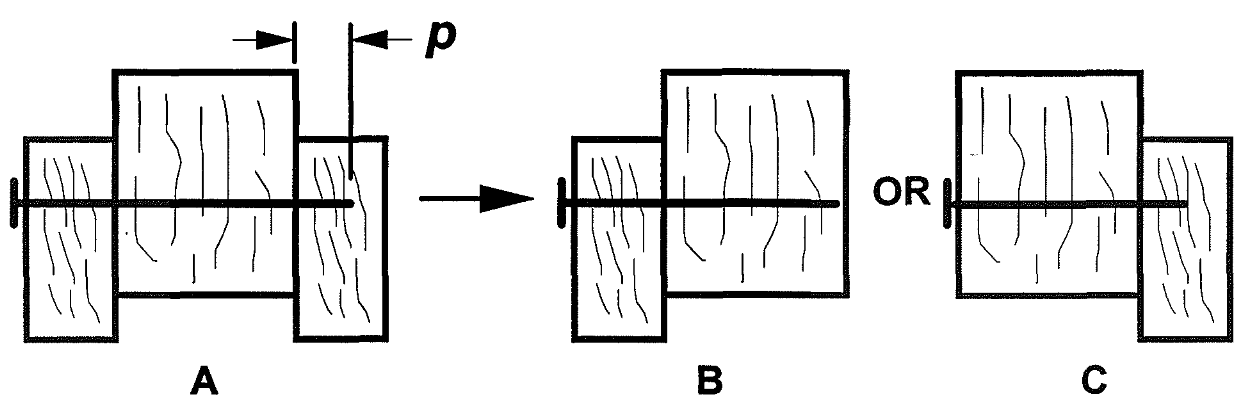



**Figure 7.4-1. Double shear nailed connection with incomplete penetration in the side member.**

For $4D \leq p < 7D$, $C_d = p/7D$. (7.4-9)
For $p \geq 7D$, $C_d = 1.0$.

**End grain**: The reference lateral resistance shall be multiplied by the end grain factor, $C_{eg} = 0.67$, for fasteners inserted into the end grain of wood.

**Toe-nail connections**. The reference lateral resistance shall be multiplied by the toe-nail factor, $C_{tn} = 0.83$, for toe-nail connections.

**7.4.4 Resistance to axial forces.**

*7.4.4.1 General provisions.* The reference resistance of connections using nails, spikes, or wood screws loaded parallel to the fastener axis shall be the minimum of (a) fastener tensile resistance and (b) shank withdrawal resistance.

*7.4.4.2 Fastener tensile resistance.* The tensile resistance of a nail, spike, or wood screw shall be determined in accordance with approved metal design practices, based on the fastener tensile yield strength at the root cross-section. The time effect factor, λ, shall be taken as 1.0 for fastener tensile resistance.

*7.4.4.3 Reference shank withdrawal resistance.* No withdrawal resistance shall be permitted for wood screws, nails, or spikes installed into the end grain of wood.

The reference shank withdrawal resistance of connections with round wire nails and spikes with undeformed shanks from the side grain of wood is:

$$Z_W = 4.59\,D\,G^{2.5}\,p\,n_f \qquad (7.4\text{-}10)$$

where $Z_W$ has units of kips; G is specific gravity of the member holding the point; D is nail or spike diameter, in.; $n_f$ is number of fasteners; and p is length of shank in the member holding the point, in.

The corresponding metric equation to obtain $Z_W$ in Newtons (N), with D and p expressed in mm, is:

$$Z_W = 31.6\,D\,G^{2.5}\,p\,n_f \qquad (7.4\text{-}10\text{-}M)$$

The shank withdrawal resistance of nails and spikes with deformed shanks such as helical threads or annular rings shall be determined by test or shall be calculated using Eq. (7.4-10) with D equal to the least shank diameter.

The reference shank withdrawal resistance of wood screws from the side grain of wood is:

$$Z_W = 9.47\,D\,G^{2}\,p\,n_f \qquad (7.4\text{-}11)$$

where $Z_W$ has units of kips; G is specific gravity of the member holding the point; D is nominal screw diameter, in.; $n_f$ is number of fasteners; and p is length of threaded portion in the member holding the point, in.

The corresponding metric equation to obtain $Z_W$ in Newtons (N), with D and p expressed in mm, is:

$$Z_W = 65.3\,D\,G^{2}\,p\,n_f \qquad (7.4\text{-}11\text{-}M)$$

The minimum length of penetration of the shank of a wood screw into the main member shall be the lesser of 1 in. (25.4 mm) or one-half the nominal screw length.

The length of the threaded portion of a wood screw shall be taken as two-thirds the length of the shank.

*7.4.4.4 Adjusted shank withdrawal resistance.* The adjusted withdrawal resistance, $Z_W'$, shall be computed by multiplying the reference resistance by applicable adjustment factors in accordance with Sec. 2.6 and 7.1.3. In addition to the adjustment factors in Sec. 2.6 and 7.1.3, the following shall apply.

**Toe-nail connections:** The reference shank withdrawal resistance shall be multiplied by the toe-nail factor, $C_{tn} = 0.67$, for toe-nail connections.

**7.5.5 Combined axial and lateral forces.** Connections subjected to forces resulting in an applied force at an angle, α, to the wood surface shall be designed such that:

$$\frac{Z_u \cos\alpha}{\lambda\phi_z Z'} + \frac{Z_u \sin\alpha}{\lambda\phi_z Z_W'} \leq 1.0 \qquad (7.4\text{-}12)$$

where α is angle between the load and the wood surface, degrees (0° < α < 90°); $Z_u$ is the connection force due to factored loads; λ is the applicable time-effect factor given in Table 1.4-2; $\phi_z$ is the resistance factor for connections = 0.65; Z′ is the adjusted lateral resistance; and $Z_W'$ is the adjusted withdrawal resistance.

## 7.5 Bolts, Lag Screws, Drift Pins, and Dowels

**7.5.1 General**

*7.5.1.1 Scope.* The following provisions apply to the design of connections using metal dowel-type fasteners including bolts, lag screws, drift pins, or dowels with 1/4 in. (6.3 mm) ≤ D ≤ 1 in. (25.4 mm).

*7.5.1.2 Fastener properties and dimensions.* Fasteners shall meet the requirements of App. A4 and the applicable documents in Sec. 1.2. The diameter, D, for bolts, lag screws, and drift pins shall be the nominal diameter.

**7.5.2 Installation.**

*7.5.2.1 Lead holes.* The following provisions shall apply to lead holes for bolts, lag screws, drift pins, or dowels installed in wood or wood-based materials. Holes shall be bored perpendicular to the surface of the member, unless other angles are specifically accounted for in the design.

Lead holes shall be accurately aligned. For bolts, lead holes shall not exceed D + 1/32 in. (0.8 mm) for D < 0.5 in. (12.7 mm) and D + 1/16 in. (1.6 mm) for D ≥ 0.5 in. (12.7 mm). Lead holes for drift pins shall be drilled 0- to 1/32-in. (0.8 mm) smaller than the actual pin diameter.

Lead holes for lag screws shall be bored as follows.

(a) The clearance hole for the shank shall have the same diameter as the shank and the same depth as the length of the unthreaded shank.
(b) The lead hole for the threaded portion shall have minimum length equal to the threaded portion of the lag screw and shall have a diameter equal to the following percentages of the shank diameter:

$$G > 0.60 \quad = (0.65)\,D \text{ to } (0.85)\,D$$
$$G > 0.50, \text{ but } \leq 0.60 = (0.60)\,D \text{ to } (0.85)\,D$$
$$G \leq 0.50 \quad = (0.40)\,D \text{ to } (0.70)\,D$$

where G is wood specific gravity and D is lag screw shank diameter.

The larger percentage in each range shall apply to lag screws with larger diameters.

When required to facilitate insertion and prevent damage to the screw, soap or another non-petroleum-based lubricant shall be used on the lag screw or in the lead hole.

*7.5.2.2 Washers.* When a bolt or lag screw head or bolt nut bears on wood or wood-based material, a standard-cut washer, metal strap, or malleable iron washer shall be placed between the wood or wood-based material and the head or nut. The minimum outside washer dimension shall be 2½ times the bolt or lag screw shank diameter. Minimum washer thickness shall be 1/8 in. (3.2 mm).

**7.5.3 Spacing of fasteners.** For bolts, lag screws, dowels, and drift pins, the minimum required edge distance, end distance, and fastener spacing required to develop the reference resistance shall be in accordance with Table 7.5-1. See

**TABLE 7.5-1.**
**Summary of edge distance, end distance, and spacing requirements for connections with bolts, lag screws, drift pins, and dowels.**

| Loading Parallel to Grain | Minimum Dimension Required to Develop Reference Resistance |
|---|---|
| Edge Distance ($b_{opt}$) | |
| $\ell_m/D \leq 6$ (See Note 1) | 1.5D |
| $\ell_m/D > 6$ | Greater of 1.5D or ½ gage spacing perpendicular to grain |
| End Distance ($a_{opt}$) | |
| Tension members | 7D |
| Compression members | 4D |
| Spacing ($s_{opt}$) | |
| Pitch (parallel to grain) | 4D |
| Gage (perpendicular to grain) | 1.5D < 5 inches (See notes 2 & 3) |
| **Loading Perpendicular to Grain** | **Minimum Dimension Required to Develop Reference Resistance** |
| Edge Distance ($b_{opt}$) | |
| Loaded edge | 4D |
| Unloaded edge | 1.5D |
| End Distance ($a_{opt}$) | 4D |
| Spacing ($s_{opt}$) | |
| Pitch (perpendicular to grain) | Limited by requirements of attached member (See note 3) |
| Gage (parallel to grain) | |
| $\ell_m/D \leq 2$ | 2.5D (See note 3) |
| $2 < \ell_m/D < 6$ | $(5\ell_m + 10D)/8$ (See note 3) |
| $\ell_m/D \geq 6$ | 5D (See note 3) |

Notes:
1. $\ell_m$ is defined as the length of dowel fastener in main member or total length of dowel fastener in side members.
2. Larger spacing is required for connections using washers.
3. For dowel-type fasteners, the perpendicular to grain spacing between outermost fasteners in a connection shall not exceed 5 in. (127 mm) unless separate splice plates or other provisions for wood dimensional change are provided.

Sec. 7.5.4.2 for adjustment factors for reduced end distance and pitch spacing.

The perpendicular to grain spacing between outermost fasteners in a connection shall not exceed 5 in. (127 mm) unless provision is made for wood dimensional change.

For staggered fasteners loaded parallel to grain, there are no minimum requirements for pitch spacing between fasteners in adjacent rows. However, for adjacent rows spaced closer than 4D, the net area provisions of Sec. 7.3.3 shall apply.

There is no minimum gage spacing for staggered fasteners loaded parallel to grain if the pitch spacing between fasteners in adjacent rows is greater than or equal to 4D. If the pitch spacing between fasteners in adjacent rows is less than 4D, the minimum gage spacing requirements of Table 7.5-1 shall apply.

### 7.5.4 Resistance to lateral forces.

*7.5.4.1 Reference lateral resistance.* Reference lateral resistances provided in this section apply to connections consisting of a wood, steel, concrete, or masonry main member and either one or two wood or steel side member(s).

Reference lateral resistance of a connection shall be the minimum value found using all equations in Table 7.5-2(a), or Table 7.5-2(b) (for bolts or dowels) or Table 7.5-2(c) (for lag screws), multiplied by the number of fasteners in the connection, $n_f$.

The dowel bearing strength, $F_e$, for concrete or masonry main members shall be taken as the same dowel bearing strength as the wood side member, and the effective thickness of the concrete or masonry main member shall be taken as twice the thickness of the wood side member. Fastener an-

**TABLE 7.5-2(a).**
**Bolt or dowel reference lateral resistance, Z: one fastener, two-member (single shear) connection.**

| Yield Mode | Applicable Equation | |
|---|---|---|
| $I_m$ | $Z = \dfrac{0.83\,D\,t_m\,F_{em}}{K_\theta}$ | (7.5-1) |
| $I_s$ | $Z = \dfrac{0.83\,D\,t_s\,F_{es}}{K_\theta}$ | (7.5-2) |
| II | $Z = \dfrac{0.93\,k_1\,D\,F_{es}}{K_\theta}$ <br> where: $k_1 = \dfrac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)}$ | (7.5-3) |
| IIIm | $Z = \dfrac{1.04\,k_2\,D\,t_m\,F_{em}}{(1 + 2R_e)K_\theta}$ <br> where: $k_2 = (-1) + \sqrt{2(1 + R_e) + \dfrac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}t_m^2}}$ | (7.5-4) |
| $III_s$ | $Z = \dfrac{1.04\,k_3\,D\,t_s\,F_{em}}{(2 + R_e)K_\theta}$ <br> where: $k_3 = (-1) + \sqrt{\dfrac{2(1 + R_e)}{R_e} + \dfrac{2F_{yb}(2 + R_e)D^2}{3F_{em}t_s^2}}$ | (7.5-5) |
| IV | $Z = \left(\dfrac{1.04\,D^2}{K_\theta}\right)\sqrt{\dfrac{2F_{em}F_{yb}}{3(1 + R_e)}}$ | (7.5-6) |

Note: $R_t = t_m/t_s$
$R_e = F_{em}/F_{es}$
$K_\theta = 1 + 0.25\,(\theta/90°)$

**TABLE 7.5-2(b).**
**Bolts or dowel reference lateral resistance, Z: one fastener, three-member (double shear) connection.**

| Yield Mode | Applicable Equation | |
|---|---|---|
| **$I_m$** | $Z = \dfrac{0.83\, D\, t_m\, F_{em}}{K_\theta}$ | (7.5-7) |
| **$I_s$** | $Z = \dfrac{1.66\, D\, t_s\, F_{es}}{K_\theta}$ | (7.5-8) |
| **$III_s$** | $Z = \dfrac{2.08\, k_3\, D\, t_s\, F_{em}}{(2 + R_e) K_\theta}$ <br> where: $k_3 = (-1) + \sqrt{\dfrac{2(1 + R_e)}{R_e} + \dfrac{2 F_{yb}(2 + R_e) D^2}{3 F_{em}\, t_s^2}}$ | (7.5-9) |
| **IV** | $Z = \left(\dfrac{2.08\, D^2}{K_\theta}\right)\sqrt{\dfrac{2 F_{em}\, F_{yb}}{3(1 + R_e)}}$ | (7.5-10) |

Note: $R_e = F_{em}/F_{es}$
$K_\theta = 1 + 0.25\,(\theta/90°)$

chorage to concrete and masonry shall be in accordance with approved design practice.

For connections with steel side members, the equation for yield mode $I_s$ in Tables 7.5-2(a), (b), or (c) shall not apply. For double-shear connections with steel main members, the equation for yield mode $I_m$ in Table 7.5-2(b) shall not apply. Design of steel members and fastener bearing shall be in accordance with approved design practice.

The dowel bearing strength for wood members loaded at an angle to grain, θ, shall be determined using the provisions of Sec. 7.2.3.

*7.5.4.2 Adjusted lateral resistance.* The adjusted lateral resistance, $Z'$, shall be computed by multiplying the reference resistance by applicable adjustment factors in accordance with Sec. 2.6 and 7.1.3. In addition to the adjustment factors in Sec. 2.6 and 7.1.3, the following shall apply.

**Geometry**: The reference lateral resistance shall be multiplied by the geometry factor, $C_\Delta$, where $C_\Delta$

**TABLE 7.5-2(c).**
**Lag screw reference lateral resistance, Z: one fastener, two-member (single shear) connection.**

| Yield Mode | Applicable Equation | |
|---|---|---|
| **$I_s$** | $Z = \dfrac{0.83\, D\, t_s\, F_{es}}{K_\theta}$ | (7.5-11) |
| **$III_s$** | $Z = \dfrac{1.19\, k_4\, D\, t_s\, F_{em}}{(2 + R_e) K_\theta}$ <br> where: $k_4 = (-1) + \sqrt{\dfrac{2(1 + R_e)}{R_e} + \dfrac{F_{yb}(2 + R_e) D^2}{2 F_{em}\, t_s^2}}$ | (7.5-12) |
| **IV** | $Z = \left(\dfrac{1.11\, D^2}{K_\theta}\right)\sqrt{\dfrac{1.75\, F_{em}\, F_{yb}}{3(1 + R_e)}}$ | (7.5-13) |

Note: $R_e = F_{em}/F_{es}$
$K_\theta = 1 + 0.25\,(\theta/90°)$

is the lesser of the geometry factors required for end distance or pitch spacing.

**End distance**: When the end distance measured from the center of the fastener, a, is greater than or equal to $a_{opt}$ specified in Table 7.5-1, then $C_\Delta = 1.0$.

$$\text{When } a_{opt}/2 \leq a < a_{opt}, \quad C_\Delta = a/a_{opt} \qquad (7.5\text{-}14)$$

**Pitch spacing**: When the pitch spacing, s, is greater than or equal to $s_{opt}$ specified in Table 7.5-1, then $C_\Delta = 1.0$.

$$\text{When } 3D \leq s < s_{opt}, \quad C_\Delta = s/s_{opt}. \qquad (7.5\text{-}15)$$

**Penetration**: The actual penetration of the shank and thread of a lag screw into the member holding the point, less the tip length, shall be greater than or equal to 4D. Reference connection lateral resistance shall be multiplied by the penetration depth factor, $C_d$, specified below.

$$\text{For } 4D \leq p < 8D, \quad C_d = p/8D. \qquad (7.5\text{-}16)$$
$$\text{For } p \geq 8D, \quad C_d = 1.0.$$

**End grain**: The reference lateral resistance shall be multiplied by the end grain factor, $C_{eg} = 0.67$, for lag screws inserted into the end grain of wood.

**7.5.5 Resistance to axial forces.**

*7.5.5.1 General.* The reference resistance of connections using lag screws or bolts loaded parallel to the fastener axis shall be the minimum of (a) fastener tensile resistance, (b) lag screw shank withdrawal resistance, or (c) bearing resistance under washers or other end-fixity devices.

*7.5.5.2 Fastener tensile resistance.* The tensile resistance of a dowel-type connector shall be determined in accordance with approved metal design practices, based on the fastener tensile yield strength at the root cross-section. The time-effect factor, λ, shall be taken as 1.0 for fastener tensile resistance.

*7.5.5.3 Reference shank withdrawal resistance.* The effective length of penetration of the threaded portion of a lag screw, p, shall be the actual length of the threaded portion in the member holding the tip, less the length of the tip. The minimum p shall be the lesser of 1 in. or one-half the threaded length.

The reference shank withdrawal resistance of lag screws conforming to ANSI/ASME B18.2.1 (1981) from the side grain of wood is:

$$Z_W = 5.98\, D^{0.75}\, G^{1.5}\, p\, n_f \qquad (7.5\text{-}17)$$

where $Z_W$ is in kips, D is the lag screw shank diameter (in.), G is the wood specific gravity, p is the length of penetration of the fastener (in.), and $n_f$ is the number of fasteners.

The corresponding metric equation to obtain $Z_W$ in Newtons (N), with D and p expressed in mm, is:

$$Z_W = 92.6\, D^{0.75}\, G^{1.5}\, p\, n_f \qquad (7.4\text{-}17\text{-}M)$$

*7.5.5.4 Adjusted shank withdrawal resistance.* The adjusted shank withdrawal resistance, $Z_W'$, shall be computed by multiplying the reference resistance by applicable adjustment factors in accordance with Sec. 2.6 and 7.1.3. In addition to the adjustment factors in Sec. 2.6 and 7.1.3, the following shall apply.

**End grain:** The reference resistance shall be multiplied by the end grain factor, $C_{eg} = 0.75$, for lag screws inserted into the end grain of wood.

*7.5.5.5 Bearing under washers.* The bearing resistance under a washer or plate shall be determined in accordance with Sec. 4.5.

**7.5.6 Resistance to combined axial and lateral forces.** The adjusted resistance of a lag screw connection loaded at an angle, α, to the wood surface, shall be computed as:

$$Z'_\alpha = \frac{Z'\, Z'_W}{Z' \sin^2 \alpha + Z'_W \cos^2 \alpha} \qquad (7.5\text{-}18)$$

where α is the angle between the load and the wood surface, degrees (0° < α < 90°), $Z'$ is the adjusted lateral resistance, and $Z_W'$ is the adjusted withdrawal resistance.

## 7.6 Shear Plates and Split Rings

### 7.6.1 General

*7.6.1.1 Scope.* The following provisions apply to the design of wood-to-wood or wood-to-metal connections using shear plates or split rings with adjacent members secured with either a bolt or lag screw and subject to lateral (shear) forces. Forces other than in-plane shear forces cause severe reduction in lateral resistance and shall be avoided.

For connections made with wood members from different species or species groups, as defined in the National Design Specification (NDS, 1991) for shear plates and split rings, reference resistance shall be based on the weaker strength group.

*7.6.1.2 Connector unit.* For purposes of computing reference lateral resistance, a connector unit shall consist of one of the following:

(a) One split ring with its bolt or lag screw in single shear;
(b) two shear plates used back-to-back in the contact faces of a wood-to-wood connection with their bolt or lag screws in single shear; or
(c) one shear plate with its bolt or lag screw in single shear used in conjunction with a steel side member in a wood-to-metal connection.

A nut and washer shall be placed on each bolt, as specified in Sec. 7.5.2.2. When an outside member is a steel strap or plate of at least 1/8 in. (3.2 mm) in thickness, the washer shall not be required, except for the purpose of extending the bolt or lag screw length to prevent the metal member from bearing on the threaded portion of the bolt or lag screw when used in conjunction with shear plates.

*7.6.1.3 Connector properties and installation.* Connectors shall meet the requirements of App. A4 and the applicable documents in Sec. 1.2. Reference resistances provided in Sec. 7.6 apply only to connections using approved split rings or shear plates in dapped wood or wood-based material members. Daps shall be cleanly cut to the proper depth per manufacturer's instructions and the connector shall be positioned to achieve maximum bearing in adjoining members.

**7.6.2 Spacing of connectors.** The spacings, $A_{opt}$, $B_{opt}$, and $s_{opt}$ (optimum), end distance, $a_{opt}$, and edge distance, $b_{opt}$, required to develop the reference resistance shall be in accordance with Tables 7.6-1, 7.6-2, and 7.6-3. See Sec. 7.6.3.2 for adjustment factors for end and edge distances and spacings that are less than optimum.

If the end of the member is not cut at right angles to its length, the end distance, as defined in Sec. 7.6.3.2, shall not be less than the end distance required for a square-cut member. In no case shall the perpendicular distance from the center of the connector to the sloping end cut of a member be less than the required edge distance.

**7.6.3 Resistance to lateral forces.**

*7.6.3.1 Reference lateral resistance in side grain.* The reference lateral resistance of a split ring or shear plate connection embedded in the side grain of the members and loaded parallel to the grain, $Z\|$, or loaded perpendicular to the grain, $Z\perp$, shall be that of a single connector unit multiplied by the number of connector units.

*7.6.3.2 Adjusted lateral resistance in side grain.* The adjusted lateral resistance parallel to the grain, $Z\|$, or loaded perpendicular to the grain, $Z\perp$, shall be computed by multiplying the reference resistance by applicable adjustment factors in accordance with Sec. 2.6 and 7.1.3. In addition to the adjustment factors in Sec. 2.6 and Sec. 7.1.3, the following shall apply.

**Metal side plates**: When metal side plates are used in connections containing a 4 in. (102 mm) shear plate loaded parallel to grain, the reference lateral resistance shall be permitted to be multiplied by a metal side plate factor, $C_{st}$.

**Penetration depth**: When lag screws are used with split rings or shear plates, penetration of the thread and shank of lag screws into the member holding the point, excluding the length of the tip, shall be $p \geq 4D$. If $p \geq 8D$, then $C_d = 1.0$. If $4D \leq p < 8D$, then the reference connection lateral resistance shall be multiplied by the following penetration depth factor:

$$C_d = \frac{p}{8D} \tag{7.6-1}$$

**Geometry**: Reference lateral resistance shall be multiplied by the geometry factor, $C_\Delta$, where $C_\Delta$ is the lesser of the geometry factors required for edge distance, end distance, or spacing. The smallest $C_\Delta$ factor for any split ring or shear plate connector in a group shall apply to all split ring or shear plate connectors in the group.

**Edge distance**: Table 7.6-1 provides the edge distance, $b_{opt}$, required to develop the reference resistance and the minimum edge distance, $b_{min}$, permitted to develop the reduced resistance for split rings or shear plates installed in the side grain of members that are loaded with a force vector either parallel or perpendicular to grain. For connections loaded between 45° and 90° to grain, the $b_{opt}$ for perpendicular to grain loading shall apply. For angles of load to grain, $0° < \theta < 45°$, the required loaded edge distance for reference resistance is:

$$b_{opt0} = \left(\frac{45° - \theta}{45°}\right)\left(b_{opt\perp} - b_{\min\perp}\right) + b_{\min\perp} \tag{7.6-2}$$

When the loaded edge distance is $b_{min\perp} \leq b < b_{opt\perp}$, the reference connection lateral resistance perpendicular to grain associated with $b_{opt}$ shall be multiplied by:

**TABLE 7.6-1.**
**Edge distances for split ring and shear plate connections.**[1]

| | | Minimum edge distance for load applied:[2] | | | |
|---|---|---|---|---|---|
| | | Parallel to grain | Perpendicular to grain | | |
| Connector | Bolt diameter (in.) | ($b_{min}\| = b_{opt}\|$) | unloaded edge ($b_{min}\perp = b_{opt}\perp$) | loaded edge ($b_{min}\perp$) | loaded edge ($b_{opt}\perp$) |
| 2⅝ in. shear plate | 0.75 | 1.75 | 1.75 | 1.75 | 2.75 |
| 4 in. shear plate | 0.75 or 0.875 | 2.75 | 2.75 | 2.75 | 3.75 |
| 2½ in. split ring | 0.5 | 1.75 | 1.75 | 1.75 | 2.75 |
| 4 in. split ring | 0.75 | 2.75 | 2.75 | 2.75 | 3.75 |

1. For metric conversion, 1 in. = 25.4 mm.
2. $b_{opt}$ = minimum distance required to develop reference resistance. $b_{min}$ = minimum distance permitted to develop reduced resistance (see Sec. 7.6.2).

$$C_\Delta = 0.17\left(\frac{b - b_{\min\perp}}{b_{opt\perp} - b_{\min\perp}}\right) + 0.83 \qquad (7.6\text{-}3)$$

When a member is loaded at an angle to grain other than 0° or 90°, the reference connection resistances for parallel to grain and perpendicular to grain loading, $Z_\|$ and $Z_\perp$, shall be multiplied by $C_\Delta$.

**End distance**: The end distance, $a_{opt}$, required to develop the reference resistance and minimum permitted end distance, $a_{min}$, to develop reduced resistance, are provided in Table 7.6-2 for split ring or shear plate connectors loaded parallel or perpendicular to grain. When members are loaded at an angle to grain, $0° < \theta < 90°$, $a_{opt}$ and $a_{min}$ shall be determined by:

$$a_{\min\theta} = \left(\frac{\theta}{90°}\right)\left(a_{\min\perp} - a_{\min\|}\right) + a_{\min\|} \qquad (7.6\text{-}4)$$

$$a_{opt\,\theta} = \left(\frac{\theta}{90°}\right)\left(a_{opt\perp} - a_{opt\|}\right) + a_{opt\|}. \qquad (7.6\text{-}5)$$

When the end distance is $a_{min} \le a < a_{opt}$, the reference lateral resistance shall be multiplied by:

$$C_\Delta = 0.375\left(\frac{a - a_{\min}}{a_{opt} - a_{\min}}\right) + 0.625. \qquad (7.6\text{-}6)$$

When the end of a member is not square cut, the end distance shall be taken as the minimum distance from any point on the center half of the connector diameter drawn perpendicular to the centerline of the member, to the nearest point on the end of the member measured parallel to the centerline of the member.

**Spacing**: Parallel to grain spacing, $A_{opt}$, and perpendicular to grain spacing, $B_{opt}$, required to develop the reference resistance are provided in Table 7.6-3 for selected angles of load to grain, θ (Fig. 7.6-1). For angles intermediate to those given in Table 7.6-3, $A_{opt}$ and $B_{opt}$ values shall be deter-

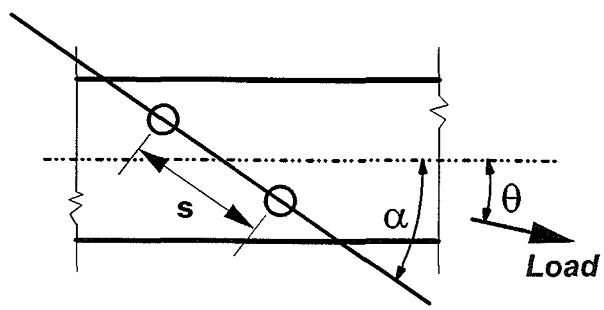



**Figure 7.6-1 Angle of connector axis to grain, α; angle of load to grain, θ.**

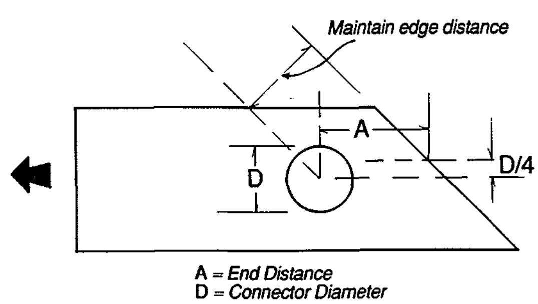



**Figure 7.6-2. End distance for members with sloping end cut.**

mined by straight-line interpolation. Minimum permitted spacings of $A_{min}$ and $B_{min}$ are equal to $A_{opt}/2$ and $B_{opt}/2$, respectively.

When the line joining the centers of adjacent connectors is at an angle, $0° < \alpha < 90°$, to the grain (Fig. 7.6-1), the required spacing for reference resistance shall be:

$$s_{opt} = \frac{A_{opt} B_{opt}}{\sqrt{A_{opt}^2 \sin^2 \alpha + B_{opt}^2 \cos^2 \alpha}} \tag{7.6-7}$$

where $s_{opt}$ is the required spacing along the connector axis for reference connection resistance, in.; $\alpha$ is the angle of connector axis, degrees; and $A_{opt}$, $B_{opt}$ are parameters taken from Table 7.6-3.

The minimum permitted spacing, $s_{min}$, associated with $\alpha$ is $s_{opt}/2$.

When the spacing between split ring or shear plate connectors is $s_{min} \leq s < s_{opt}$, the reference lateral resistance shall be multiplied by:

$$C_\Delta = 0.5\left(\frac{s - s_{min}}{s_{opt} - s_{min}}\right) + 0.5. \tag{7.6-8}$$

The adjusted lateral resistance, $Z_\theta'$, of a connection with shear plates or split rings loaded at an angle to grain, $\theta$, shall be:

**TABLE 7.6-2.**
**End distances for split ring and shear plate connections.[1]**

| Split ring diameter (in.) | Shear plate diameter (in.) | End Distances[2] | Tension (in.) | Compression (in.) |
|---|---|---|---|---|
| | | Parallel to grain loading | | |
| 2½ | 2⅝ | $a_{opt}\parallel$ | 5¼ | 4 |
| 2½ | 2⅝ | $a_{min}\parallel$ | 2¾ | 2½ |
| 4 | 4 | $a_{opt}\parallel$ | 7 | 5½ |
| 4 | 4 | $a_{min}\parallel$ | 3½ | 3¼ |
| | | Perpendicular to grain loading | | |
| 2½ | 2⅝ | $a_{opt}\perp$ | 5½ | 5½ |
| 2½ | 2⅝ | $a_{min}\perp$ | 2¾ | 2¾ |
| 4 | 4 | $a_{opt}\perp$ | 7 | 3½ |
| 4 | 4 | $a_{min}\perp$ | 3½ | 3½ |

1. For metric conversion, 1 in. = 25.4 mm.
2. $a_{opt}$ = minimum distance required to develop reference resistance. $a_{min}$ = minimum distance permitted to develop reduced resistance (see Sec. 7.6.2).

**TABLE 7.6-3.**
**Connector spacing parameters.[1]**

| Type and size of connector | Angle of load to grain (degrees) | $A_{opt}$[2] (in.) | $B_{opt}$[3] (in.) |
|---|---|---|---|
| 2½ in. split ring or 2⅝ in. shear plate | 0 | 6¾ | 3½ |
| | 15 | 6 | 3¾ |
| | 30 | 5⅛ | 3⅞ |
| | 45 | 4¼ | 4⅛ |
| | 60–90 | 3¾ | 4¼ |
| 4 in. split ring or 4 in. shear plate | 0 | 9 | 5 |
| | 15 | 8 | 5¼ |
| | 30 | 7 | 5½ |
| | 45 | 6 | 5¾ |
| | 60–90 | 5 | 6 |

1. For metric conversion, 1 in. = 25.4 mm.
2. $A_{opt}$ = minimum spacing required to develop reference resistance. $A_{min} = A_{opt}/2$ = minimum spacing permitted to develop reduced resistance (see Sec. 7.6.2).
3. $B_{opt}$ = minimum spacing required to develop reference resistance. $B_{min} = B_{opt}/2$ = minimum spacing permitted to develop reduced resistance (see Sec. 7.6.2).

$$Z_\theta = \frac{Z'_\parallel Z'_\perp}{Z'_\parallel \sin^2\theta + Z'_\perp \cos^2\theta} \qquad (7.6\text{-}9)$$

where $Z_\parallel'$, $Z_\perp'$ is adjusted connection resistance for parallel to grain and perpendicular to grain loadings, lb.; and $\theta$ is angle of force vector with respect to a direction parallel to grain, degrees.

*7.6.3.3 Reference strength in end grain.* Shear plates or split rings embedded in the end grain of a member shall be designed in accordance with App. A5.

# CHAPTER 8

# Structural-Use Panels

## 8.1 Scope

This chapter applies to the following structural-use panels: plywood, oriented strand board, and composite panels.

Design of panel-based assemblies shall be in accordance with those provided in App. A6.

## 8.2 Design Requirements

Unless otherwise indicated in this chapter, the design requirements specified in other sections of this standard shall be applicable for structural-use panels.

**8.2.1 Reference conditions**. The reference conditions given in Sec. 2.5 are applicable to structural-use panels with the following exceptions.

(a) Reference resistance values shall apply at continuous exposure to temperatures of 100°F (32°C) and lower. Structural-use panels shall not be exposed to temperatures above 200°F (93°C) for more than very brief periods. For sustained temperature conditions between 100°F and 200°F the temperature adjustments shall be applied.

(b) Strength, stiffness, and reference resistance values are applicable to 24-in. (610 mm) or wider panels. For narrower widths, the width factor specified in Sec. 2.6.5 shall be used.

**8.2.2 Specification of structural-use panels.** Structural-use panels shall be specified by span rating, nominal thickness, exposure rating, and grade.

## 8.3 Reference Resistance

**8.3.1 Panel stiffness and factored reference resistance.** Panel stiffness and factored reference resistance shall be used in structural-use panel design. These load capacity values, which represent the product of material and section properties, shall be determined from tests conducted in compliance with approved standards.

Due to the orthotropic nature of panels, resistance values shall be provided for the primary and secondary strength axes. The appropriate tabulated values shall be applied when designing for each panel orientation. When forces act at an angle to the principal axes of the panel, the resistance of the panel at that angle shall be calculated by adjusting the tabulated values for the principal axes using principles of engineering mechanics.

**8.3.2 Reference strength and elastic properties**. Where required, reference strength and elastic parameters shall be calculated from factored reference resistance and stiffness, respectively, on the basis of tabulated design section properties.

## 8.4 Design Section Properties

**8.4.1 Design thickness**. Nominal thickness shall be used in design calculations. The relationships between span ratings and nominal thicknesses are provided with associated design capacities.

**8.4.2 Design section properties**. Design section properties shall be assigned on the basis of span rating or design thickness and are provided on a per-foot-of-panel-width basis.

## 8.5 Design

**8.5.1 Applicable procedures**. Design procedures provided in this standard are applicable to design of structural-use panels except as noted in this section.

**8.5.2 Flatwise bending**. Structural-use panels shall be designed for flexural capacity by checking bending moment, shear, and deflection limit states. Planar shear shall be used as the shear resistance in checking the shear limit state for panels in flatwise bending. Appropriate beam equations shall be used with the design spans as defined below for each limit state.

(a) Bending moment—distance between centerline of supports.
(b) Shear—clear span.
(c) Deflection—clear span plus one-half the nominal panel thickness.

**8.5.3 Tension in the plane of the panel.** Requirements of Chap. 3 are applicable for axial tension design of structural-use panels with the following additional requirement.

When structural-use panels are loaded in axial tension, the orientation of the primary strength axis of the panel with respect to the direction of loading shall be considered in calculating the tension resistance.

**8.5.4 Compression in the plane of the panel.** Requirements of Chap. 4 are applicable for compression design of structural-use panels with the following additions.

(a) When structural-use panels are loaded in axial compression, to the orientation of the primary strength axis of the panel with respect to the direction of loading shall be considered in calculating the compression resistance.
(b) Panels shall be designed to prevent buckling.

**8.5.5 Panel shear.** Panel shear shall be used as the design resistance when the shear force is applied parallel to the plane of structural-use panels.

# CHAPTER 9

# Shear Walls and Diaphragms

## 9.1 General

**9.1.1 Scope.** Design provisions of this chapter apply to structural-use panel and lumber sheathed shear walls (vertical diaphragms) and horizontal diaphragms acting as elements of the lateral force-resisting system.

## 9.2 Shear Wall and Diaphragm Design

Shear walls and diaphragms shall be designed such that:

$$D_u \leq \lambda \phi_z D' \qquad (9.2\text{-}1)$$

where $D_u$ is diaphragm force due to factored loads (force per unit length); $\lambda$ is 1.0 for lateral force design (wind or seismic) from Table 1.4-2; $\phi_z$ is the resistance factor for shear walls or diaphragms limited by fastener strength = 0.65; and $D'$ is the adjusted shear wall or diaphragm design resistance per unit length.

The adjusted resistance shall be determined using applicable adjustments specified in Sec. 2.6 and in this chapter.

**9.2.1 Design principles.** Shear walls and diaphragms shall be designed according to either the following beam analogy or, alternatively, by more refined structural analysis procedures. Design shall include consideration of sheathing, framing, fasteners, and fastening schedules, all boundary members, boundary member splices, drag struts, and all required connections.

Force transfer to a supporting system not covered by this standard shall be in accordance with applicable building code provisions.

*9.2.1.1* Shear walls and diaphragms and their elements and components shall be analyzed as thin, deep beams with the sheathing resisting in-plane shear (as in a beam web) and the boundary members resisting axial forces (as in beam flanges). Boundary elements shall be provided at shear wall and diaphragm perimeters and at interior openings, discontinuities, and reentrant corners unless not required as shown by analysis. Provision shall be made to dissipate forces from the boundary elements at openings and discontinuities into the body of the shear wall or diaphragm.

*9.2.1.2* Boundary members shall be provided at shear wall and diaphragm perimeters, at all interior openings, and at all discontinuities and re-entrant corners, unless shown by analysis to be redundant.

## 9.3 Required Resistance

The required shear wall or diaphragm resistance is established by the controlling factored lateral load case. Determination of the controlling factored lateral load case shall include wind or seismic forces acting along each of the structure's principal axes and orthogonal effects as specified in the governing building code or ASCE 7-93.

### 9.4 Reference Resistance

**9.4.1 In-plane shear resistance.** The in-plane shear reference resistance, D, shall be obtained from approved tables or determined using principles of engineering mechanics. When the shear resistance is determined using principles of engineering mechanics, the resistance of the structural-use panel sheathing shall be investigated in accordance with the provision of Chap. 8 of this standard.

*9.4.1.1 Adjusted in-plane shear resistance.* The adjusted in-plane shear resistance, D′, shall be computed in accordance with Sec. 2.6, 7.1.3, 7.4.3.3. In addition to the adjustment factors in the aforementioned sections, the following shall apply.

**Diaphragm factor:** The calculated diaphragm resistance shall be multiplied by the diaphragm factor, $C_{di}$, equal to 1.1 for nailed diaphragms that comply with the provisions of this chapter.

**9.4.2 Boundary element resistance.** The reference resistance of boundary elements, including shear wall and diaphragm chords, drag struts and their connections, shall be determined in accordance with the provisions of Chap. 1 through 8 of this standard.

**9.4.3 Shear transfer at shear wall and diaphragm boundaries.** The reference resistance of fastening at all perimeter and interior shear wall and diaphragm boundaries shall be determined in accordance with Sec. 9.4.1 or Chap. 7, as applicable.

### 9.5 Other Design Considerations

Shear walls and diaphragms shall be designed for serviceability limit states in accordance with the provisions of Chap. 10.

# CHAPTER 10

# Serviceability Considerations

### 10.1 General Considerations

The designer shall consider serviceability limit states which include but are not limited to short-term deflection, vibration, creep, dimensional changes, and the effects of deterioration. Serviceability shall be checked using unfactored loads. Deflection under the specified loads shall be limited to avoid damage to structural elements or attached nonstructural elements. Refer to governing codes for requirements concerning the serviceability limit states.

### 10.2 Material and Member Stiffness

The modulus of elasticity used in calculating deflections of members, frames, and components shall be taken as the adjusted mean value, E′.

# APPENDIX A1

# Resistance of Spaced Columns

### A1.1 Geometry and Geometry Limits

The spaced-column direction is the potential buckling direction perpendicular to the connected faces (usually the wider face) of the column component members. This appendix addresses the overall geometry of spaced columns and the resistance as controlled by buckling in the spaced-column direction. In the solid-column direction (usually parallel to the wide component faces), the strength is governed by the provisions of Sec. 4.4.

For spaced columns that are compression members of a truss, a panel point that is braced laterally shall be considered as the end of the spaced column, and the portion of the web members between the individual column component members shall be considered as the end blocks.

Notation and dimensions of a spaced column are shown in Fig. A1.1-1 and include:

$\ell_1$ = overall length in the spaced column direction, in.
$\ell_2$ = overall length in the solid column direction, in.
$\ell_3$ = larger distance from centroid of connectors in an end block to the center of the mid-length area spacer block, in.
$\ell_{ce}$ = distance from centroid of end block connectors to nearer column end, in.
$d_1$ = width of individual component in the spaced column direction, in.
$d_2$ = width of individual component in the solid column direction, in.

End blocks with a thickness at least equal to that of the individual members shall be located at or near the ends of the spaced column and shall have width and length adequate for the connectors required in Sec. A1.4. At least one intermediate spacer block with a width equal to that of the end blocks shall be provided at or near the column mid-length such that $\ell_3 \leq 0.50\ell_1$.

The following maximum length-to-width ratios apply:

In the spaced column direction, $\ell_1/d_1$ shall not exceed 80.
In the spaced column direction, $\ell_3/d_1$ shall not exceed 40.
In the solid column direction, $\ell_2/d_2$ shall not exceed 50.

Spaced columns not meeting the provisions of this appendix shall be designed by considering each component as a separate solid column unless a rational analysis taking into account the column-restraint conditions is used.

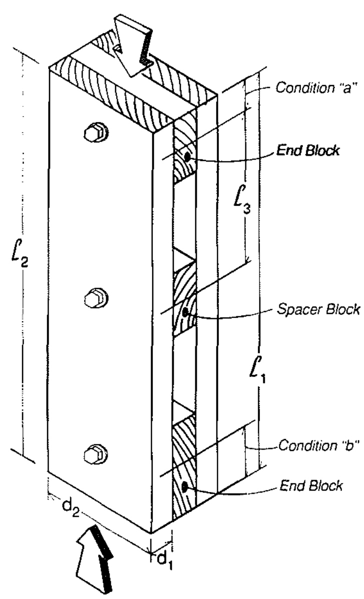
### A1.2 Spaced Column Fixity Conditions

In the spaced column direction, the following two fixity conditions are defined.

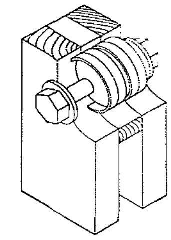
Case a: $\ell_{ce} \leq 0.05\,\ell_1$.
Case b: $0.05\ell_1 < \ell_{ce} \leq 0.10\ell_1$.

If the fixity cases at the two ends differ, case (a) shall be used.

The effective length factor, $K_e$, in the spaced-column direction shall be taken as 0.63 for fixity case(a) and 0.58 for fixity case (b), where there is no sidesway in the spaced column direction. No decrease in these factors shall be taken when the end blocks are thicker than the component members attached to them, nor for overall column-end fixity conditions in the spaced-column direction. For spaced columns with side-




Spaced column



Typical shear plate connection in end block of spaced column

**Figure A1.1-1 Geometry of a spaced column.**

sway in the spaced-column direction, a $K_e$ value greater than 1 and determined according to Sec. 4.2.1 shall be used instead of the 0.58 or 0.63 factor, and external rotation restraint of at least one end of the entire spaced column assembly shall be provided.

In the solid-column direction, the provisions of Sec. 4.2 shall apply.

### A1.3 Resistance of Spaced Columns

The adjusted compressive resistance of a spaced-column shall be taken as the smaller of the adjusted resistances in the spaced-column direction and in the solid column direction. These resistances shall be determined from the equations of Sec. 4.3 and with the resistance factors, time-effect factors, and end-use adjustment factors applicable to solid columns.

In the spaced-column direction, the moment of inertia to be used in Eq. 4.3-4 is that for one column component for the spaced-column direction times the number of component members. The gross area for use in Eqs. 4.3-1 and 4.3-4 shall be that for one component times the number of such components in the column. When the component members are unequal in size, material strength, or material stiffness, the least component I, E, and/or $F_{cn}$ shall be used in the above procedure unless a more detailed analysis is made.

The provisions of the preceding paragraph apply also in the solid-column direction, except that the single-component moment of inertia shall be for the solid-column direction.

### A1.4 Requirements for Connectors in End Blocks

The connectors (split rings or shear plates) in each set of mutually contacting surfaces of the end blocks and column-component members at each end of the spaced column shall provide the following shear resistance as determined by the provisions of Chap. 7:

$$Z' = A_1 K_S \tag{A1.4-1}$$

where $Z'$ is adjusted end block shear resistance, lbs.; $A_1$ is area of one of the column component members in.$^2$; and $K_S$ is end block constant, dependent on $\ell_1/d_1$ and the species group of the connected members. (See Table A1.4-1.)

Spacer blocks in the middle tenth of the column length, $\ell_1$, shall be fastened (nails, bolts, etc.) adequately to hold the column components together and to prevent rotation of the spacer block. Spacer blocks not in the middle third shall be provided with connectors providing the capacity given in Eq. A1.4-1.

The connectors required to meet Eq. A1.4-1 are not additive to those that are required at the end connector for transfer of loads. The larger of the required shear resistance specified by Eq. A1.4-1 and by load transfer within the connection shall apply.

# APPENDIX A2

# Glued Laminated Timber (Glulam)

### A2.1 General

The requirements in Chap. 1 through 7 apply to glued laminated timbers. However, the different shapes and sizes require additional investigations.

**TABLE A1.4-1**
**End block constant.**

| Species Group | Specific Gravity (SG) | $K_S$* |
|---|---|---|
| A | SG ≥ 0.60 | $(\ell_1/d_1 - 11)(20.7)$ but ≤ 1.020 ksi |
| B | 0.50 ≤ SG < 0.60 | $(\ell_1/d_1 - 11)(17.6)$ but ≤ 0.860 ksi |
| C | 0.42 ≤ SG < 0.50 | $(\ell_1/d_1 - 11)(14.6)$ but ≤ 0.715 ksi |
| D | SG < 0.42 | $(\ell_1/d_1 - 11)(10.7)$ but ≤ 0.565 ksi |

*For $\ell_1/d_1 \leq 11$, $K_S = 0$

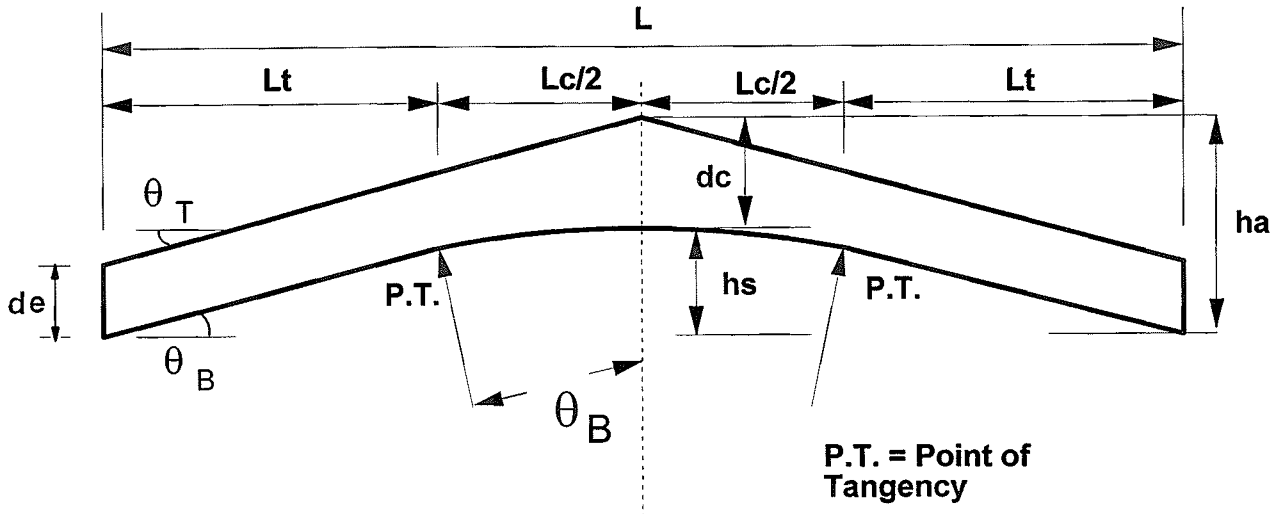



**Figure A2.2-1. Pitched and tapered curved beam.**

When members are pitched, tapered, and curved as shown in Fig. A2.2-1, or arched, as shown in Fig. A2.2-2, the distribution of bending and radial stresses and methods of computing deflections are different from those of prismatic members of constant cross-section.

## A2.2 Pitched and Tapered Curved Beams

**A2.2.1 Moment resistance limited by radial stress.** The moment resistance of symmetrical uniformly loaded rectangular curved beams having the pitched and tapered geometry shown in Fig. A2.2-1 shall be limited by radial stress conditions to:

$$M' = b(d_c)^2F_r'/6K_{sr} \qquad \text{(A2.2-1)}$$

where:

$M'$ = adjusted moment resistance at midspan, kip-in.,
$b$ = width, in.,
$d_c$ = depth of cross-section at the apex, in.,
$F_r'$ = adjusted radial strength, ksi.,
= $F_{rt}'$ when radial stress is in tension, ksi.,
= $F_{rc}'$ when radial stress is in compression, ksi. ($F_{rc}'$ shall be taken equal to $F_{c\perp}'$, the adjusted perpendicular-to-grain compression strength, ksi.),
$K_{sr}$ = radial stress factor,
$K_{sr} = K_{gr}[A+B(d_c/R_m) + C(d_c/R_m)^2]$,
$= K_{gr}K_{ar}$ (A2.2-2)

where:

A, B, and C = constants for use with the angle of the upper tapered surface and shall be obtained from Table A2.2-1,
$K_{gr} = X\text{-}Y(d_c/D_m)$,
= reduction factor depending on member shape determined from Table A2.2-2,
$R_m$ = radius of curvature at member at mid depth, in.

where $L/L_c$ is ratio of full length of member to curved portion of member, and $d_c/R_m$ is ratio of depth at center line to the radius of the mid-depth member.

**TABLE A2.2-1**
**Polynomial approximation to $K_{ar}$ (as a function of the angle of upper tapered surface $\theta_T$).**

| | Factors[1] | | |
|---|---|---|---|
| $\theta_T$ Degrees | A | B | C |
| 2.5 | 0.0079 | 0.1747 | 0.1284 |
| 5.0 | 0.0174 | 0.1251 | 0.1939 |
| 7.5 | 0.0279 | 0.0937 | 0.2162 |
| 10.0 | 0.0391 | 0.0754 | 0.2119 |
| 15.0 | 0.0629 | 0.0619 | 0.1722 |
| 20.0 | 0.0893 | 0.0608 | 0.1393 |
| 25.0 | 0.1214 | 0.0605 | 0.1238 |
| 30.0 | 0.1649 | 0.0603 | 0.1115 |

[1]For intermediate values of $\theta_T$, use straight-line interpolation.

**TABLE A2.2-2**
**Equations for determining $K_{gr}$.**

| | $L/L_c=1.0$ | | $L/L_c=2.0$ | | $L/L_c=3.0$ | | $L/L_c=4.0$ | |
|---|---|---|---|---|---|---|---|---|
| $\theta_T$ | X | Y | X | Y | X | Y | X | Y |
| 2:12 | 0.433 | 0.543 | 0.674 | 0.646 | 0.821 | 0.707 | 0.883 | 0.680 |
| 3:12 | 0.622 | 0.857 | 0.820 | 0.867 | 0.940 | 0.827 | 0.980 | 0.626 |
| 4:12 | 0.705 | 0.850 | 0.880 | 0.863 | 0.972 | 0.823 | 1.000 | 0.233 |
| 5:12 | 0.788 | 0.893 | 0.945 | 0.753 | 0.982 | 0.677 | 1.000 | 0.000 |
| 6:12 | 0.847 | 0.893 | 1.000 | 0.733 | 0.998 | 0.427 | 1.000 | 0.000 |

**A2.2.2 Moment resistance limited by bending stress.** The geometry of the member affects the distribution of bending stresses. The adjusted moment resistance, $M'$, shall be adjusted to reflect the effect of geometry.

The adjusted moment resistance in the curved portion of rectangular beams having the pitched and tapered geometry shown in Fig. A2.2-1 is limited by bending stress conditions to:

$$M' = b\,(d_c)^2 F_b'/6K_{sb} \qquad \text{(A2.2-3)}$$

where:

$M'$ = adjusted moment resistance at mid span,
$d_c$ = depth of cross section at apex, in.,
$b$ = member width, in.,
$F_b'$ = adjusted bending strength, ksi.,
$K_{sb}$ = bending stress factor = $D + E(d_c/R_m) + F(d_c/R_m)^2$,

$$\text{(A2.2-4)}$$

where $R_m$ is radius of curvature at mid-depth of a curved member, in., and D, E, F are dimensionless factors from Table A2.2-3.

The moment resistance at any point from the tangent points to the ends of the beam shall be taken as that of a prismatic member of the same depth at the point at which the moment is being considered.

**A2.2.3 Deflection of pitched and tapered curved beams.** Deflection of pitched and tapered curved beams at the center line shall be determined using the following equation:

$$\Delta_c = 5wL^4/32E'bd_{eb}^3 \qquad \text{(A2.2-5)}$$

where:

$w$ = unfactored uniform load, kips/in.,
$L$ = length of span, in.,
$E'$ = adjusted mean modulus of elasticity, ksi.,
$b$ = width, in.,
$d_{eb}$ = effective depth = $(d_e + d_c)(0.5 + 0.735\tan\theta_T) - 1.41(d_c)\tan\theta_B$,

where:

$d_e$ = depth at end, in.,
$d_c$ = depth at mid span, in.,
$\theta_T$ = slope of top, degrees,
$\theta_B$ = slope of bottom (soffit) at ends, degrees.

As an alternative to the above, other methods of determining deflection shall be used when they are demonstrated to take into account all affected parameters and produce equivalent results.

**A2.2.4 Radial reinforcement.** When the radial tension strength is exceeded, mechanical reinforcing shall be used and shall be sufficient to resist all radial tension forces. However, these radial tension forces shall not exceed that determined by multiplying the area being reinforced by a radial tensile stress equal to one-third the nominal shear strength parallel to grain. When radial reinforcing is used for beams intended for dry end-use conditions, the moisture content of the laminations shall not exceed 12% at the time of manufacture.

**A2.2.5 Adjustment factors.** The adjustment factors for glued laminated timber shall be applied in the same manner as shown in the body of this standard except as noted otherwise. The radial stress factor, $K_{sr}$, the reduction factor for shape, $K_{gr}$, and the bending stress factor, $K_{sb}$, are consid-

**TABLE A2.2-3**
**Coefficients for determining $K_{sb}$.**

| | Factors[1] | | |
|---|---|---|---|
| $\theta_T$ Degrees | D | E | F |
| 2.5 | 1.042 | 4.247 | −6.201 |
| 5.0 | 1.149 | 2.036 | −1.825 |
| 10.0 | 1.330 | 0.0 | 0.927 |
| 15.0 | 1.738 | 0.0 | 0.0 |
| 20.0 | 1.961 | 0.0 | 0.0 |
| 25.0 | 2.625 | −2.829 | 3.538 |
| 30.0 | 3.062 | −2.594 | 2.440 |

1. For intermediate values of $\theta_T$, use straight-line interpolation.

ered to be parts of the calculations for pitched and tapered curved beams rather than end-use adjustment factors.

**A2.2.6 Stress interaction factor**. The stress interaction factor, $K_{si}$, shall not be applied to the design of pitched and tapered curved beams, except that the portions of the beam outside of the curved portion shall be checked by using this factor.

## A2.3 Glued Laminated Timber Arches

**A2.3.1 Types of arches**. The two general types of glued laminated timber arches are three-hinged arches and two-hinged arches, shown in Fig. A2.3-1. The design provisions of Chaps. 1 through 7 shall apply, except as noted in this section.

**A2.3.2 Three-hinged arches**. Design of statically determinate three-hinged arches shall include bending combined with compression parallel to grain in amounts that vary along the member and shear near the member ends. The standard design strength equations for glued laminated members shall apply, except that the volume effect on bending strength is modified, and the interaction provisions for taper cut surfaces (Sec. 5.1.10 and 5.1.11) shall not apply.

**A2.3.3 Two-hinged arches**. Design of statically indeterminate two-hinged arches shall include appropriate analysis methods to determine moments, axial loads, and shears at locations along the arch. Once these forces and moments are known, the design is similar to that used for the three-hinged arch.

**A2.3.4 Axial compressive resistance**. The same procedures used for columns shall be used for calculating the axial compressive resistance.

Although most arches are laterally braced about the Y-Y axis, bracing requirements shall be considered in accordance with Chap. 4 and 6. Arches are generally not braced about the X-X axis. However, design for buckling about this axis shall not be required because of arch action.

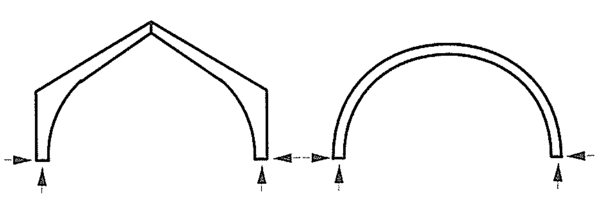


**Figure A2.3-1 - Three-hinged arch (left) and two-hinged arch (right).**

**A2.3.5 Radial stresses in arches**. Radial stresses shall be considered in the design of arches. The strength of arches as affected by radial stresses shall be determined in the same manner as for curved beams.

**A2.3.6 Nominal moment resistance**. The adjusted moment resistance, M′, shall be calculated by use of Eq. 5.2-2 repeated here:

$$M' = M_x' = S_x F_{bx}' \tag{A2.3-1}$$

The volume effect factor, $C_V$, an adjustment included in $F_{bx}'$, is modified for arches by setting the exponents of the width and length ratios equal to zero.

When combined bending and compression exists, the corrected volume effect factor, $C_V'$ is:

for $F_b'(1 - C_V) \leq f_c$

$$C_V' = 1.0 \tag{A2.3-2}$$

for $F_b'(1 - C_V) > f_c$

$$C_V' = C_V + f_c/F_b' \tag{A2.3-3}$$

where $F_b'$ is adjusted bending strength, ksi.; $f_c$ is applied axial compressive stress, ksi; and $C_V$ is volume effect factor.

The unsupported length for compression of tudor-type arches about the X-X axis shall be taken as the length of the rafter portion of the arch along the top and the leg of the arch on the side. For circular, parabolic, and similarly shaped arches, it is usually taken as the distance from the base to crown.

The effective arch length for bending shall be determined and the elastic lateral buckling moment, $M_e$, calculated by Eq. 5.4-4. M′ shall then be calculated by use of Eq. 5.4-1 and compared with the value obtained from Eq. 5.3-1. The lesser of the two values shall apply. The unsupported lengths of the arch segments for bending shall be determined as for any other bending member.

The factor for tapering, $K_{si}$, shall not apply to arches.

**A2.3.7 Interaction of moment and axial forces in arches.** The interaction of axial compression and bending shall be computed in the same manner as that used for beams except that the arch shall be assumed to be braced in the Y-Y direction and the factored moment need not be magnified. On this basis, for arch design, Eq. 6.3-1 reduces to:

$$(P_u/\lambda\phi_c P')^2 + (M_{bx}/\lambda\phi_b M_x') \leq 1 \qquad \text{(A2.3-4)}$$

where $P_u$ is factored axial compression force, kips; $P'$ is adjusted member resistance for axial compression acting alone, kips; $M_{bx}$ is factored moment about the strong axis, kip-in.; and $M_x'$ is adjusted bending resistance about the strong axis, kip-in.

**A2.3.8 Deflection of arches.** The deflection of arches shall be limited in accordance with applicable serviceability requirements. The elastic or short time deflection caused by loads at any point along the arch and in any direction shall be calculated by principles of engineering mechanics. One method commonly used is the method of virtual work. Consideration of long-term deflection caused by creep shall be in accordance with Chap. 10.

The deflection caused by change of moisture content with a resulting change in the angle of curvature of the arch (see Fig. A2.3-2) shall be determined by principles of engineering mechanics. For vertical deflection, the following equation is often applied:

$$\Delta_m = \alpha_s l/2[1 - H_r/(H_r + H_w)]\tan\theta_q \qquad \text{(A2.3-5)}$$

where:

$\Delta_m$ = deflection at the crown, in.,
$l$ = clear span between hinges, in.,
$H_r$ = height of roof portion of the arch, in.,
$H_w$ = height of wall portion of the arch, in.,
$\alpha_s$ = interior angle between axes of straight portions of arch, degrees,

$$\theta_q = -d_q/(1 - d_q), \qquad \text{(A2.3-6)}$$

where $\theta_q$ is the percent change in angle caused by shrinkage and $d_q$ is the percent change in depth of member caused by shrinkage. When $d_q$ is very small compared with unity, $\theta_q$ shall be taken as $-d_q$.

The horizontal deflection of the tudor-type arch is often determined by the following equation:

$$\Delta_h = \alpha_s[H_r H_w/(H_r + H_w)]\tan\theta_q. \qquad \text{(A2.3-7)}$$

A semi-graphical method is also used for tudor-type arches as well as for arches of other shapes.

If an arch swells because of increase in moisture content, the effect will be in the opposite direction to that caused by shrinkage.

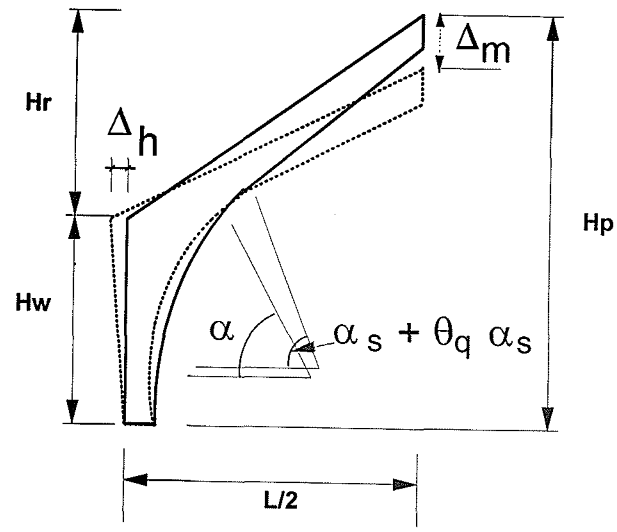



**Figure A2.3-2 - Tudor-type arch deflection caused by shrinkage.**

# APPENDIX A3

# Ponding

## A3.1 Scope

Flat and nearly flat roof systems shall be investigated by structural analysis to assure stability and adequate resistance under ponding conditions unless the roof surface is provided with either sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rain water. Any drains and parapet walls shall be detailed to minimize the occurrence of clogging and unplanned retention of water.

The provisions of Sec. A3.2 apply for one-way roofing systems (parallel beams or trusses with roof sheathing). The provisions of Sec. A3.3 apply for two-way roof framing system (such as sheathed joists or purlins framing into supporting beams).

Additional provisions for ponding design (such as that caused by failure of the roof's primary drainage) may be required by the applicable building code. The provisions of this section shall not be substituted for such building code requirements for roof ponding design.

## A3.2 One-Way Roof Systems

One-way roof systems consist of sheathing attached to flexural members spanning in only one direction (no secondary beams or purlins) and with the flexural members are along the slope. These systems shall satisfy either the following minimum slope or minimum resistance provisions considering the increased moments due to ponding.

**A3.2.1 Minimum slope to drain.** The roof system shall be considered to have sufficient slope to drain, and no further investigation is needed if:

$$\theta_r \geq \lambda_{cr} \frac{(w_p / 144) S_p L_p^3}{24 E'_{05} I_x} + \frac{16 \Delta_i}{5 L_p} \qquad \text{(A3.2-1)}$$

where:

$\theta_r$ = initial slope of the primary roof members, radians,

$\lambda_{cr}$ = creep factor, shall be taken as 1.5 for glued laminated or seasoned sawn lumber and 2.0 for unseasoned sawn lumber,

$L_p$ = length of sloped primary roof members, in.,

$S_p$ = spacing of primary members, in.,

$E_{05}'$ = adjusted modulus of elasticity for the direction of primary member bending at the fifth percentile for the primary roof member, psi.,

$I_x$ = moment of inertia about the bending axis for a single primary roof member, usually the x axis, in.$^4$,

$\Delta_i$ = initial sag of member under no load (negative if member is cambered upward), in.,

$w_p$ = uniform load caused by factors loads 1.2D + 1.2P acting over a width equal to $S_p$, where D = nominal dead load, and P = nominal ponding load caused by rain water, ice, or water/ice trapped by ice dams, exclusive of the magnification due to ponding, which is computed separately, psf.

**A3.2.2 Increased Moment Caused by Ponding**

If the roof system does not have sufficient slope to satisfy Eq. A3.2-1, then the moment due to factored loads based on the load combination of Eq. 1.3-3 shall be multiplied by a moment magnification factor, $K_{rp}$, to account for the increased moment which results from the ponding associated with subcritical roof flexibility:

$$K_{rp} = \frac{1}{\left(1 - \frac{S_p}{\phi_s S_c}\right)} \qquad \text{(A3.2-2)}$$

where $\phi_s$ is resistance factor for stability = 0.85, and $S_c$ is critical spacing of primary roof member, in.

$$= \frac{1}{\lambda_{cr}} \frac{\pi^4 E'_{05} I_x}{(\rho/1728) L_p^4} \qquad \text{(A3.2-3)}$$

where $\rho$ is weight density of liquid (62.4 pcf for water).

If the primary member is a truss with a flat or nearly flat top, the top compression chord shall be checked for adequacy in combined bending, including from ponding loads, and axial load. This shall be done by replacing the strong axis moment magnifier, $B_{xb}$, given by Eq. 6.3-4 by:

$$B_{xb} = \frac{C_{mx}}{\left(1 - \frac{P_u}{\phi_c P_{ex}} - \frac{S_p}{\phi_s S_c}\right)} \geq 1.0 \qquad \text{(A3.2-4)}$$

where $C_{mx}$ is moment shape coefficient as given in Sec. 6.3.

## A3.3 Two-Way Roof Systems

Roof systems consisting of sheathed secondary members supported by primary members that are not sloped adequately to ensure drainage are considered adequate to prevent ponding if both Eqs. A3.3-1 and A3.3-2 are met:

$$(B_p + 0.9 B_s) \geq 0.25 \qquad \text{(A3.3-1)}$$

$$I_r \geq \frac{725 S^4}{E_{05r}} \qquad \text{(A3.3-2)}$$

where:

$$B_p = \frac{L_s L_p^4 \lambda_{cr}}{2.68 I_p E'_{05p}} \qquad \text{(A3.3-3)}$$

$$B_s = \frac{S L_s^4 \lambda_{cr}}{2.68 I_s E'_{05s}} \qquad \text{(A3.3-4)}$$

and:

$L_s$ = length of secondary members (column spacing perpendicular to direction of primary members), in.,
$L_p$ = length of primary members, in.,
$I_p$, $I_s$ = moment of inertia for primary and secondary members, respectively, in.$^4$,
$I_r$ = moment of inertia per foot width of roof sheathing supported on secondary members, in.$^4$/ft.,
$E_{05p}'$, $E_{05s}'$ = adjusted elastic modulus for primary and secondary members, respectively, psi.,
$E_{05r}'$ = adjusted elastic modulus for roof sheathing, psi.,
S = spacing of secondary members, in.,
$\lambda_{cr}$ = creep factor (see Sec. A3.2).

# APPENDIX A4

# Qualification of Fasteners and Connectors

### A4.1 General

Unless otherwise specified, the actual average yield strength ($F_{yb}$) and all dimensions of fasteners and connectors shall be determined by measurement and reported or certified by the manufacturer.

Fasteners and connectors for use with pressure impregnated preservative and fire-retardant treated wood shall be of stainless steel, silicon-bronze, copper, or hot-dip zinc galvanized steel meeting the requirements of the applicable standards included in Sec. 1.2.

### A4.2 Nails and Spikes

These fasteners shall be manufactured from common wire and demonstrate ductile behavior in bending.

### A4.3 Wood Screws

Wood screws shall meet the requirements of ANSI/ASME B18.6.1-1981 and demonstrate ductile behavior in bending.

### A4.4 Bolts, Lag Screws, Drift Pins, and Dowels

The dimensions and quality of bolts and lag screws shall conform to ANSI/ASME B18.2.1-1981.

The average nominal yield strength, $F_{yb}$, of bolts, lag screws, drift pins, and dowels shall be as determined by:

(a) tests of representative samples using the procedures of ASTM F606-86; and
(b) the tabulated yield value for grades 1 through 8, as appropriate, for commercially available low carbon steel bolts complying with the requirements of SAE J429-1985.

### A4.5 Split Rings

Both 2½- and 4-in. diameter split-ring timber connectors shall be manufactured from SAE-1010 hot-rolled carbon steel meeting the requirements of SAE J412-1989. Each ring shall form a closed true circle with the principal axis of the cross section parallel to the geometric axis of the ring. The ring shall fit snugly into the precut grooves in the connected members. One method to accomplish this is with a ring with a section that is beveled from the central portion toward the edges. The thickness of the ring at the central portion shall be larger than at its edges. Any other method that will accomplish the same end result is acceptable. The ring shall be cut through in one place in its circumference to form a tongue and slot.

### A4.6 Shear Plates

Two and five-eighths-inch pressed steel type. Pressed steel shear plates shall be manufactured from SAE-1010 hot-rolled carbon steel meeting the requirements of SAE J412-1989. Each plate shall be a true circle with a flange around the edge extending at right angles from one face of the plate only. The plate portion shall have a central hole for insertion of the bolt or lag screw.

Four-inch malleable iron type. Malleable iron shear plates shall be manufactured according to grade 32510 of ASTM A47-89. Each casting shall consist of a perforated round plate with a flange around the edge extending at right angles to the face of the plate and projecting from one face only.

The plate portion having a central hole for insertion of a bolt or lag screw shall have an integral hub extending from the same face as the flange.

Dimensional tolerances of the connector shall not be greater than those conforming to standard practices for the machining operations involved in manufacturing the connectors.

### A4.7 Metal Connector Plates

Metal connector plates shall be manufactured to meet or exceed the requirements of ANSI/TPI 1-1995.

# APPENDIX A5

# Resistance of Shear Plates Or Split Rings In End Grain

### A5.1 Definitions and Notations

The following definitions apply to the use of shear plates and split rings when these connectors are installed in a surface that is not parallel to the general direction of the grain of the member.

Side-grain surface means a surface parallel to the general direction of the wood fibers ($\alpha = 0°$), such as the top, bottom, and sides of a straight member.

Sloping surface means a surface cut at an angle, $\alpha$, other than 0° or 90° to the general direction of the wood fibers.

Square-cut surface means a surface perpendicular to the general direction of the wood fibers ($\alpha = 90°$).

Axis of cut defines the direction of a sloping surface relative to the general direction of the wood fibers. For a sloping cut symmetrical about one of the major axes of the member, as in Figs. A5.1-1 through A5.1-4, the axis of cut is parallel to a major axis. For an asymmetrical sloping surface (i.e., one that slopes relative to both major axes of the member), the axis of cut is the direction of a line defining the intersection of the sloping surface with any plane that is both normal to the sloping surface and also aligned with the general direction of the wood fibers (see Figs. A5.1-1 and A5.1-5).

$\alpha$ = the least angle formed between a sloping surface and the general direction of the wood fibers (i.e., the acute angle between the axis or cut and the general direction of the fibers, sometimes called the slope of the cut) (see Figs. A5.1-1 through A5.1-6).

$\theta$ = the angle between the direction of applied load and the axis of cut of a sloping surface, measured in the plane of the sloping surface (see Fig. A5.1-4).

$Z_{\parallel}'$ = adjusted resistance of a connector unit in a side-grain surface, when loaded in a parallel to grain direction ($\alpha = 0°$, $\theta = 0°$).

$Z_{\perp}'$ = adjusted resistance of a connector unit in a side-grain surface, when loaded in a perpendicular to grain direction ($\alpha = 0°$, $\theta = 90°$).

$Z_{\perp,90}'$ = adjusted resistance for a connector unit in a square-cut surface, when loaded in any direction in the plane of the surface ($\alpha = 90°$).

$Z_{\parallel,\alpha}'$ = adjusted resistance for a connector unit in a sloping surface, when loaded in a direction parallel to the axis of cut ($0 < \alpha < 90°$, $\theta = 0°$).

$Z_{\perp,\alpha}'$ = adjusted resistance for a connector unit in a sloping surface, when loaded in a direction perpendicular to the axis of cut ($0 < \alpha < 90°$, $\theta = 90°$).

$Z_{\alpha}'$ = adjusted resistance for a connector unit in a sloping surface, when the direction of load is at an angle from the axis of cut.

### A5.2 Design Basis

When shear plates or split rings are installed in a surface that is not parallel to the general direction of the grain of the member, such as the end of a square-cut member, or the sloping surface of a member cut at an angle to its axis, or the surface of the glued laminated timber cut at an angle to the direction of the laminations, adjusted resistance shall be determined in accordance with this appendix and Sec. 7.4 of this standard.

### A5.3 Connectors Installed in Square-Cut or Sloping Surfaces

For connectors installed in square-cut or sloping surfaces, design values shall be determined from the following forms of the Hankinson equation.

(a) Square-cut surface, loaded in any direction ($\alpha = 90°$) (see Fig. A5.1-6):

$$Z_{\perp,90}' = 0.60\, Z_{\perp}' \qquad \text{(A5.3-1)}$$

(b) Sloping surface, loaded parallel to axis of cut ($0 < \alpha < 90°$, $\theta = 0°$) (see Fig. A5.1-2):

$$Z'_{\parallel,\alpha} = \frac{Z'_{\parallel} Z'_{\perp,90}}{Z'_{\parallel} \sin^2 \alpha + Z'_{\perp,90} \cos^2 \alpha} \qquad \text{(A5.3-2)}$$

(c) Sloping surface, loaded perpendicular to axis of cut ($0 < \alpha < 90°$, $\theta = 90°$) (see Fig. A5.1-3):

$$Z'_{\perp,\alpha} = \frac{Z'_{\perp} Z'_{\perp,90}}{Z'_{\perp} \sin^2 \alpha + Z'_{\perp,90} \cos^2 \alpha} \qquad \text{(A5.3-3)}$$

(d) Sloping surface, loaded at angle $\theta$ to axis of cut ($0 < \alpha < 90°$, $0 < \theta < 90°$) (see Fig. A5.1-4):

$$Z'_{\alpha} = \frac{Z'_{\parallel,\alpha} Z'_{\perp,\alpha}}{Z'_{\parallel,\alpha} \sin^2 \theta + Z'_{\perp,\alpha} \cos^2 \theta} \qquad \text{(A5.3-4)}$$

## A5.4 Spacings

The provisions for edge distance, end distance, and spacing given in Sec. 7.4.2 of this standard for connectors in side-grain surfaces shall apply to connectors in square-cut surfaces and sloping surfaces, as follows.

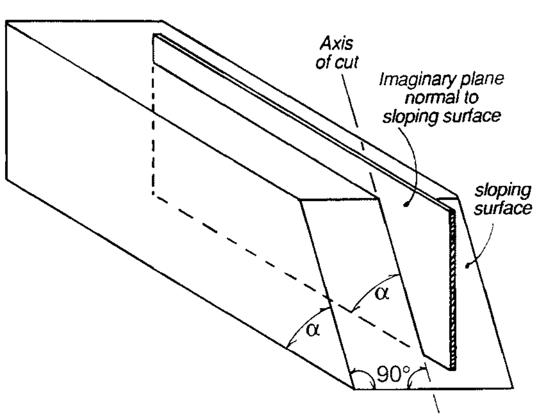



**Figure A5.1-1. Axis of cut for symmetrically sloping end cut at angle α.**

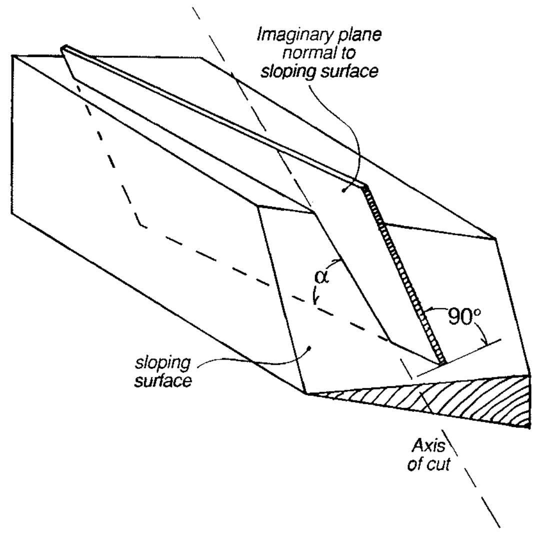



**Figure A5.1-2. Axis of cut for asymmetrical compound sloped end cut.**

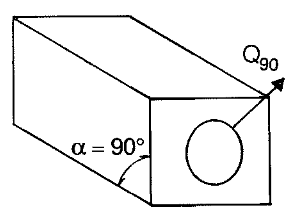



**Figure A5.1-3. Square end cut (α = 90 degrees).**

(a) Square-cut surface, loaded in any direction—apply provisions for perpendicular to grain loading.
(b) Sloping surface with $\alpha$ from 45° to 90° loaded in any direction—apply provisions for perpendicular to grain loading.

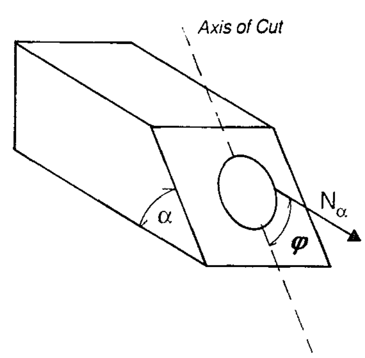



**Figure A5.1-4. Load at angle θ to axis of cut.**

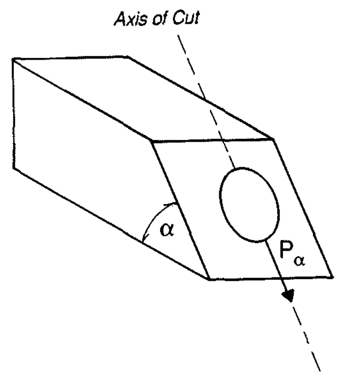



**Figure A5.1-5. Load parallel to axis of cut (θ = 0 degrees).**

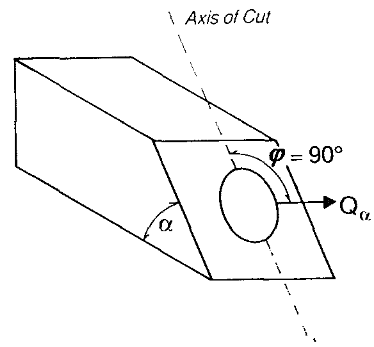



**Figure A5.1-6. Load perpendicular to axis of cut (θ = 90 degrees).**

(c) Sloping surface with α less than 45°, loaded parallel to axis of cut—apply provisions for parallel to grain loading.
(d) Sloping surface with α less than 45°, loaded perpendicular to axis of cut—apply provisions for perpendicular to grain loading.
(e) Sloping surface with α less than 45°, loaded at angle θ to axis of cut—apply provisions for member loaded at angles to grain other than 0° and 90°.

# APPENDIX A6

# Design of Panel-Based Assemblies

### A6.1 Scope

Panel-based assemblies include I-beams, stressed-skin panels, sandwich panels, and curved panels. For design purposes, a panel-based assembly is treated as a single member subsystem composed of panels, lumber or core material components.

The scope of panel-based assemblies is limited to components that are glued together to develop composite action. The provisions in this appendix are limited to panel-based assemblies manufactured in plants. Further, such assemblies shall be subject to an ongoing quality control program. Designs of panel-based assemblies that include the use of prefabricated wood I-joists in are outside the scope of this standard.

### A6.2 Components

Each structural component of these assemblies (structural-use panels, structural framing, lightweight core material, as applicable) shall meet requirements of this standard.

### A6.3 Fabrication

Performance of panel-based assemblies depends on load-carrying capacity and the quality of components, and the integrity of adhesive bonds between components. Adhesives used for bonding components shall be specified by the designer for intended service conditions and shall conform to applicable adhesive specifications.

### A6.4 End Joints

When structural elements are end joined with mechanical fasteners and/or adhesive, the designer shall consider the load-transfer capacity of these joints.

The reference resistance values of end joints used in a design shall be supported with adequate test data or by analysis for the appropriate loading conditions, e.g., tension, compression, bending, and shear.

### A6.5 Design Procedure

The design procedure for panel-based assemblies shall consist of evaluating a series of checking equations representing potential failure modes and serviceability limit states for the particular assembly. All specified checking equations must be satisfied.

### A6.6 Deflection Limitations

Satisfactory performance of panel-based assemblies requires that the ratio of the maximum deflection to the span of individual members or subassemblies be restricted to:

$$\frac{\Delta}{L} \leq \frac{1}{N_d} \qquad \text{(A6.6-1)}$$

where $N_d$, a deflection limitation number, depends on the end-use of the assembly ($N_d$ = 180, 240, 360, etc.) as specified in the governing code.

### A6.7 I-Beams

I-beams are categorized by their components and cross-sectional geometries. As defined in this standard, I-beams include:

(a) panel-lumber (I and box-sections); and
(b) all-panel beams (vertically laminated, multi-web).

The following limit states shall be taken into account.

Strength limit states:
(a) shear resistance of panel webs;
(b) shear transfer of web splices;
(c) flexural resistance of web splices;
(d) shear transfer resistance between web and flange;
(e) flexural resistance of the composite beam;
(f) tension and compression resistance of the appropriate flanges;
(g) compression perpendicular to grain resistance of flanges under the stiffeners;
(h) rolling shear resistance of panels at the stiffener interface; and
(i) lateral stability of the composite beam.

Serviceability limit states:

(a) deflection caused by flexure;
(b) deflection caused by shear; and
(c) combined flexural and shear deflection.

### A6.8 Stressed-Skin Panels

Limit states for design of stressed-skin panels shall include failure modes and deflection limitations. The following limit states shall be taken into account.

Strength limit states:

(a) panel moment resistance caused by either top or bottom skin limitation (tension or compression);
(b) resistance of splice plates (if applicable) for transferring stresses;
(c) rolling shear resistance of the top and bottom skins at the skin-web interfaces;
(d) shear resistance of webs;
(e) transverse load resistance and combined bending and axial resistance if the stressed-skin panel is used as a wall panel; and
(f) skin buckling.

Serviceability limit states:

(a) flexural deflection of panel;
(b) shear deflection of panel;
(c) combined flexural and shear deflection; and
(d) top and bottom skin deflections between webs.

### A6.9 Sandwich Panels

Limit states for sandwich panels shall include failure modes and deflection limitations. The following limit states shall be taken into account.

Strength limit states:

(a) flexural resistance of the panel;
(b) shear resistance of the panel;
(c) column buckling (if used in axial loading);
(d) combined axial and bending resistance (if used under combined loading); and
(e) skin buckling.

Serviceability limit states:

(a) deflection caused by flexure;
(b) deflection caused by shear; and
(c) combined flexural and shear deflection.

### A6.10 Curved Panels

Limit states for curved panels shall depend on the type of panel construction. For stressed-skin type construction, all the limit states of stressed skin panels provided in Sec. A6.8 shall be considered. Similarly, for sandwich-type panel construction, all the limit states of sandwich panels provided in Sec. A6.9 shall be considered. However, in addition to the above, the following limit states shall be taken into account:

(a) effect of curvature on flexural resistance;
(b) effect of curvature on compression or tensile resistance;
(c) flexural resistance caused by radial stress limitations; and
(d) horizontal deflection limitation (for curved flexural panels only).

# Glossary

**Adjusted resistance.** The reference resistance adjusted to include the effects of all applicable adjustment factors resulting from end use and other modifying factors. Time-effect adjustments are not included because they are considered separately.

**American Softwood Lumber Standard.** A voluntary product standard developed by the National Institute of Standards and Technology, U.S. Department of Commerce, in cooperation with wood producers, distributors, and users. The standard establishes the dimensions for various types of lumber products, the technical requirements and the methods of testing, grading and marking, and is designated PS 20-94 (Product Standard 20 issued in 1994).

**American Lumber Standard Committee (ALSC).** A standing committee composed of representatives of producers, distributors, specifiers, and consumers of lumber. The primary function of the committee is to review and consider revisions to the American Softwood Lumber Standard, PS 20-94. ALSC inspectors conduct field checks on certified grading agencies and the committee's independent Board of Review has the power to discipline under the aegis of the Commerce Department, National Institute of Standards and Technology.

**Aspect ratio.** In any rectangular configuration, the ratio of the long side's length to the short side's length.

**Assembly.** A collection of parallel structural members and/or components connected in a manner such that load applied to any one component will affect the stress conditions of adjacent parallel components.

**Assembly effects.** Component interactions that affect the way stress is distributed within an individual component and/or the way loads are distributed to other components in an assembly.

**Boundary elements.** Shear wall and diaphragm members to which sheathing transfers forces. Boundary elements include chords and drag struts at shear wall and diaphragm perimeters, interior openings, discontinuities and re-entrant corners.

**Built-up member.** A member made of structural wood elements that are glued or mechanically connected.

**Clear span.** Inside distance between the faces of supports.

**Composite action.** Interaction between elements connected in such a way that the resulting member strength and stiffness is greater than the sum of the strength and stiffness of the individual elements.

**Composite member.** A member composed of multiple elements connected so as to achieve composite action.

**Composite panel.** A structural-use panel comprised of wood veneer and reconstituted wood-based material and bonded with waterproof adhesive.

**Connection.** An attachment used to transmit forces between two or more members by means of a fastener, an assembly of fasteners, or adhesive, acting alone or in combination with member bearing.

**Connector.** Synonym for fastener.

**Decay.** Decomposition of wood substance caused by action of wood-destroying fungi; the word "rot" means the same as decay.

**Decking.** Solid sawn lumber or glued laminated decking expressed in nominal terms as being "2 in. to 4 in." thick and "4 in. and wider." Decking is usually surfaced to single tongue and groove in 2 in. (51 mm) nominal thickness. In 3 in. (76 mm) and 4 in. (102 mm) nominal thickness, it may be double tongue and groove and worked with rounded or V edges, striated, or grooved.

**Design span.** For simple, continuous, and cantilever beams, the design span is the clear span plus one-half the required bearing length at each support.

**Design resistance.** Resistance (force or moment as appropriate) provided by member or connection; the product of adjusted resistance, the resistance factor, and time-effect factor.

**Design strength.** Material strength (tensile, compressive, etc.) derived in accordance with ASTM 5457-93 procedures and adjusted to reflect end-use conditions.

**Diaphragm.** A sheathed horizontal or nearly horizontal system (e.g., roof, floor) acting to transfer lateral forces to the vertical resisting elements.

**Diaphragm boundary.** A location where shear is transferred into or out of the diaphragm sheathing. Transfer is either to a boundary element or to another force resisting element. Also applied to shear walls.

**Diaphragm chord.** A diaphragm boundary element perpendicular to the applied load which is assumed to take axial stresses analogous to the flanges of a beam. Also applied to shear walls.

**Dowel bearing strength.** The maximum compression strength of wood or wood-based products when subjected to bearing by a steel dowel of specific diameter.

**Dowel-type fasteners.** Includes bolts, lag screws, wood screws, nails, and spikes.

**Drag strut (collector, tie, diaphragm strut).** A shear wall or diaphragm boundary element parallel to the applied load which collects and transfers diaphragm shear forces to the vertical resisting elements or distributes forces within the diaphragm.

**Dry service.** Structures wherein the maximum equilibrium moisture content does not exceed 19%.

**Edge distance.** The distance from the edge of the member to the center of the nearest fastener, measured perpendicular to grain. When a member is loaded perpendicular to grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting.

**Edgewise bending.** Bending about the strong axis.

**Effective width.** In sheathing, the reduced width that, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution.

**End distance.** In the case of square-cut ends, the distance measured parallel to grain from the end of the member to the center of the nearest fastener.

**Equilibrium moisture content.** A moisture content at which wood neither gains nor loses moisture to the surrounding air.

**Exposure durability.** A classification of panels based on raw material composition and adhesive bond durability.

**Exposure 1** - Panels suitable for protected construction and industrial uses. Exposure 1 panels have adequate durability to resist moisture exposure due to long construction delays, or other conditions of similar severity.

**Exposure 2 or IMG (intermediate glue)** - Panels suitable for protected applications that are not continuously exposed to high humidity conditions.

**Exterior** - Panels suitable for permanent exposure to weather or moisture.

**Interior** - Panels suitable for permanently protected interior applications.

**Factored load.** The product of the nominal load and an applicable load factor.

**Fastener.** Generic term for individual mechanical devices such as bolts, nails, metal plates, etc., used in a connection. Synonymous with connector.

**Fiber saturation point.** The moisture content at which the cell walls are saturated with water (bound water) and no water is held in the cell cavities by capillary forces. It is species dependent and usually is taken as 25% to 30% moisture content, based on weight when ovendry.

**Fire-retardant treated wood.** Any lumber or wood product impregnated with chemicals by a pressure process, or by other means, meeting prescribed requirements for resistance to flame spread and resistance to progressive combustion.

**Flatwise bending.** Bending about the weak axis.

**Gage or row spacing.** The center-to-center distance between fastener rows or gage lines.

**Glued laminated timber (glulam).** See structural glued laminated timber.

**Grade.** The classification of structural wood products with regard to strength and utility in accordance with the grading rules of an approved agency.

**Grading rules.** Requirements and specifications for the manufacture, inspection, and grading of designated species of lumber.

**Green lumber.** Lumber of less than nominal 5-in. (127 mm) thickness that has a moisture content in excess of 19%. For lumber of nominal 5-in. (127 mm) or greater thickness (timbers), green shall be defined in accordance with the provision of the applicable lumber grading rules certified by the ALSC Board of Review.

**Horizontal diaphragm.** A sheathed horizontal or nearly horizontal element (roof, floor) acting to transfer lateral forces to the vertical resisting elements.

**I-beams.** Wood I-beams are custom designed and fabricated for specific applications. Lumber flanges and panel webs are bonded with adhesives to form "I", multiweb, or box sections. The design of wood I-beams is in accordance with App. A6 of this standard.

**I-joists (prefabricated).** Structural members manufactured using sawn or structural composite lumber flanges and structural panel webs, bonded together with waterproof adhesives, forming an "I" cross-sectional shape. The design of I-joists is in accordance with ASTM D5055-94.

**Joist (lumber).** Pieces (nominal dimensions 2 to 4 in. (51 to 102 mm) in thickness by 5 in. (127mm) and wider with rectangular cross-section graded primarily with respect to strength in bending when loaded on the narrow face. Typically used as framing members for floor or ceilings.

**Kiln dried.** Lumber that has been seasoned in a chamber to a predetermined moisture content by applying heat.

**Laminated veneer lumber (LVL).** A composite of wood veneer sheet elements with wood fibers primarily extended along the length of the member. Veneer thickness does not exceed 0.25 in. (6.4 mm).

**Limit state.** A condition in which a structure or component is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

**Load duration (time-effect).** The period of continuous application of a given load, or the cumulative period of intermittent applications of the maximum load.

**Load factor.** A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.

**Load sharing.** The load redistribution mechanism among parallel components constrained to deflect together or joined by crossing members such as sheathing or decking.

**Load/slip constant.** The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

**LRFD (Load and Resistance Factor Design).** A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is reached when the structure is subjected to all appropriate load combinations.

**Lumber.** The product of the sawmill and planing mill usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, cross-cutting to length, and matching.

**Lumber Sizes.** Lumber is typically referred to by size classifications. Two of the frequently used size classifications are dimension and timbers. Additionally, lumber is specified by manufacturing classification. Rough lumber and dressed lumber are two of the routinely used manufacturing classifications.

**Boards.** Lumber of less than nominal 2 in. (51 mm) thickness and of nominal 2 in. (51 mm) or greater width.

**Dimension.** Lumber from nominal 2 in. through 4 in. (51 mm through 102 mm) thick and nominal 2 or more in. (51 or more mm) wide.

**Dressed size.** The dimensions of lumber after surfacing with a planing machine. Usually ½ to 3/4 in. (12.7 to 19.0 mm) less than nominal size. The American Softwood Lumber Standard lists standard dressed sizes.

**Rough lumber.** Lumber that has not been dressed (surfaced) but that has been sawed, edged, and trimmed at least to the extent of showing saw or other primary manufacturing marks in the wood on the four longitudinal surfaces of each piece for its overall length. Lumber surfaced on one edge (S1E), two edges (S2E), one side (S1S), or two sides (S2S) is classified as rough lumber in the unsurfaced width or thickness.

**Timbers.** Lumber of nominal 5 in. (127 mm) or greater in least dimension.

**Stress-graded lumber.** Lumber graded for its mechanical properties.

**Machine evaluated lumber (MEL).** Lumber that has been nondestructively evaluated by mechanical grading equipment. Each piece is evaluated and marked to indicate its strength classification. MEL lumber is also required to meet certain visual requirements.

**Machine stress-rated (MSR) lumber.** Lumber that has been evaluated by mechanical stress-rating equipment. Each piece is nondestructively tested and grademarked to indicate the assigned bending strength and modulus of elasticity. MSR lumber is also required to meet certain visual requirements.

**Visually stress-graded lumber.** Structural lumber that has been graded visually to limit strength-reducing and appearance characteristics. Assigned design values are based on the effect of the strength limiting visual characteristics.

**Main member.** In three-member connections, the center member. In two-member connections, the thicker member.

**Mat-formed panel.** A structural-use panel designation representing panels manufactured in a mat-formed process, such as oriented strand board and wafer board.

**Moisture content.** The weight of the water in wood expressed as a percentage of the weight of the wood from which all water has been removed (ovendry).

**Nominal loads.** The loads specified by the applicable code.

**Nominal size.** The approximate commercial size by which lumber products are known and sold in the market. The nominal size is generally greater than the actual dimensions, i.e., a dry 2×4 is surfaced to 1½ in. by 3½ in. (38 mm by 89 mm).

**Oriented strandboard.** A mat-formed structural-use panel comprised of thin rectangular wood strands arranged in cross-aligned layers with surface layers normally arranged in the long panel direction and bonded with waterproof adhesive.

**Ovendry wood.** Wood dried until it is free of any moisture.

**Panel.** A sheet-type wood product.

**Panel rigidity.** Shear rigidity of a panel, the product of panel thickness and modulus of rigidity.

**Panel shear.** Shear developed in a structural-use panel due to in-plane loads, commonly called "shear through the thickness," and is developed in shear walls, diaphragms, and webs of I-joists.

**Panel stiffness.** Flexural or axial stiffness of a panel. The product of panel section property and modulus of elasticity.

**Parallel strand lumber (PSL).** A composite of wood-strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands is not greater than 0.25 in. (6.4 mm) and the average length is not less than 150 times the least dimension.

**Performance rating.** A classification designating end-use applications for which specific performance test procedures and criteria have been established.

**Performance standard.** A standard for trademarked products based on performance. Performance is measured by tests that approximate end-use conditions.

**Pile.** Round timber structural element of any size or length, that is driven or otherwise introduced into the soil for the purpose of providing vertical or lateral support.

**Pitch or spacing.** The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

**Planar shear.** The shear developed in structural-use panels due to flatwise bending, commonly referred to as "rolling shear" in plywood.

**Plank.** A piece of lumber, from 2 to 4 in. (51 to 102 mm) thick, used with the wide face placed horizontally (differs from joist only that latter is used on edge).

**Ply.** A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

**Plywood.** A structural-use panel comprised of plies of wood veneer arranged in cross-aligned layers. The plies are bonded with an adhesive that cures on application of heat and pressure.

**Pole.** A round timber of any size or length, usually used with the larger end in the ground.

**Pole construction.** A form of construction in which the principal vertical members are round poles or sawn timbers (post-frame construction) embedded in the ground and extending vertically above ground to provide both foundation and vertical framing for the structure.

**Prefabricated wood I-joists.** Pre-engineered proprietary structural members that are mass produced to established specifications. An "I" cross-section is formed from sawn or structural composite lumber flanges and structural panel webs, bonded together with exterior exposure adhesives. These are used primarily as joists in floor and roof construction with their engineering properties determined in accordance with ASTM D5055-94.

**Preservative.** A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

**Pressure-preservative treated wood.** Wood products pressure-treated by an approved process and preservative.

**Primary panel (strong) axis.** The axis corresponding with the primary strength direction of structural use panels. Unless otherwise indicated (marked) on the panel, the primary strength axis is in the panel length direction.

**Punched metal plate.** A light steel plate fastening having punched teeth of various shapes and configurations which are pressed into wood members to effect shear transfer. Used with structural lumber assemblies.

**Purlin.** A roof framing member, perpendicular to the trusses or rafter members, which supports the roof sheathing or other common rafter members.

**Rated panel.** A panel rated for conventional floor, roof, and wall applications.

**Reference end use conditions (reference conditions).** Assume standard end-use conditions. Adjustments to resistances are required if design end-use conditions differ from the reference end-use conditions.

**Reference resistance.** The resistance (force or moment as appropriate) of a member or connection computed at the reference end-use conditions prescribed by this standard.

**Reference strength.** Material strength (tensile, compressive, etc.) derived in accordance with ASTM D5457-93 procedures.

**Repetitive member assembly.** A system of closely spaced parallel framing members, which exhibits load-sharing behavior.

**Required member resistance.** Load effect (force, moment, or stress, as appropriate) acting on an element or connection, determined by structural analysis from the factored loads and the critical load combinations.

**Resistance.** The capacity of a structure, component, or connection to resist the effects of loads. It is determined by computations using specified material strengths, dimensions, and formulas derived from accepted principles of structural mechanics, or by field or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

**Resistance factor.** A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

**Row of fasteners.** Two or more fasteners aligned with the direction of load.

**Scarf joint.** A slope overlapping joint bonded with an adhesive.

**Seasoned lumber.** Lumber that has been dried. Seasoning takes place by open-air drying within the limits of moisture contents attainable by this method, or by controlled air drying (i.e., kiln drying).

**Secondary panel (weak) axis.** The axis corresponding with the secondary strength direction of structural-use panels. Unless otherwise indicated (marked) on the panel, the secondary strength axis is in the panel width direction.

**Serviceability limit state.** A limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery under normal usage.

**Shear plate.** A circular metal plate that, by being embedded in adjacent wood faces, or in one wood face, acts in shear to transmit loads from one timber to a bolt and, in turn, to a steel plate or another shear plate.

**Shear wall (vertical diaphragm).** A sheathed wall element that transfers in-plane lateral forces to the base of the wall.

**Sheathing.** Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

**Shrinkage.** The decrease in the dimensions of wood caused by a decrease of moisture content.

**Side member.** The member or connection element adjacent to the main member.

**Slenderness ratio for beams.** The ratio used in lateral stability calculations for bending members.

**Slenderness ratio for compression members.** The ratio of the effective length of a compression member to its radius of gyration.

**Spaced column.** A column with two or more individual members, usually rectangular and with their wide faces parallel, placed with their longitudinal axes parallel, spaced at their ends and in the mid-length region by blocking, and joined at their ends by the end blocks with split rings or shear plates of sufficient shear stiffness to effectively restrain the column ends.

**Span rating.** A panel index number identifying the recommended maximum center-to-center support spacing in inches for roof, floor, and wall applications under normal use conditions.

**Specific gravity.** The ratio of the ovendry weight of a sample to the weight of a volume of water equal to the volume of the sample at some specified moisture content, as green, air-dry, or ovendry.

**Split ring.** A metal ring that, by being embedded into adjacent faces of two wood members, acts in shear to transmit force between the members.

**Stiffener (web).** A piece of wood that is glued or otherwise fastened to the webs between the inner surfaces of the top and bottom flanges of a built-up beam.

**Strength limit state.** A limiting condition affecting the safety of a structure, a structural component, or a mechanical connection.

**Stress grades.** Lumber grades having assigned design stress and modulus of elasticity values in accordance with accepted basic principles of strength grading.

**Stressed skin panel.** A form of construction in which the outer skin, in addition to its normal function of providing a surface covering, acts integrally with the frame members contributing to the strength of the unit as a whole.

**Structural composite lumber (SCL).** In this standard, structural composite lumber is either laminated veneer lumber (LVL) or parallel strand lumber (PSL). These materials are intended for structural use and are bonded with an exterior adhesive.

**Structural glued laminated timber.** An engineered, stress-rated product of a timber-laminating plant comprising assemblies of specially selected and prepared wood laminations securely bonded together with adhesives. The grain of all laminations is approximately parallel longitudinally. They comprise pieces end joined to form any length, pieces placed or glued edge-to-edge to make wider ones, or pieces bent to curved form during gluing.

**Structural-use panel.** A wood-based panel product bonded with a waterproof adhesive. Included under this designation are plywood, oriented strand board, and composite panels. These panel products meet the requirements of PS 1-94 or PS 2-92 and are intended for structural use in residential, commercial, and industrial applications.

**Stud.** Used for vertical framing members in interior or exterior walls of a building, usually 2 × 4 or 2 × 6 sizes and precision end trimmed.

**Time-effect factor.** A factor applied to adjusted resistance to account for effects of duration of load (refer to load duration).

**Tie down.** An anchoring device for a shear wall boundary element that resists overturning of the wall.

**Unbraced length.** The distance between braced points of a member, measured between the centers of gravity of the bracing members.

**Veneer.** Thin wood sheet (ply) from which plywood or other wood products are manufactured, referred to as plies in the glued panel.

**Wet service.** Structures wherein the maximum equilibrium moisture content exceeds 19%.

(This Commentary is not a part of the AF&PA/ASCE Standard for Load and Resistance Factor Design for Engineered Wood Construction and is included for information purposes.)

# COMMENTARY

# Chapter 1 General Provisions

## C1.1 Scope

ASTM D5457-93, "Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design," hereinafter referred to as the "ASTM specification," defines procedures to develop resistance values for wood products for use under this standard. Its primary purpose is to provide unified procedures and methodologies for the derivation of resistance values. Methodologies are provided for calculation directly from data or by format conversion from approved allowable stress values.

The LRFD notation used in this standard differs from that of steel LRFD. In steel LRFD, the term "required strength" is used to denote the force on the member due to factored loads. Similarly, the term "design strength" is used to denote the factored member capacity—which is a product of the resistance factor and another term called the "nominal strength." Thus, steel LRFD uses notation that sometimes includes applicable factors and sometimes does not.

This problem is compounded for wood LRFD for two reasons. First, designers must often "track" both unadjusted and adjusted strength quantities. This alone would confound a simple notation system. A second source of confusion lies in the need to publish not only member strengths (i.e., moment capacity) but also material properties (i.e., the LRFD equivalent of allowable stresses). This need to define both unadjusted (called "reference") and adjusted quantities of both member and material parameters forced the wood LRFD notation down a slightly different track than steel. A description of the notation and its rationale follows.

In this standard, the term "resistance" is used to refer to member capacities (i.e., moment resistance, compression resistance, etc.). This is distinct from the term "strength" which refers to limit state material properties—conceptually a "factored allowable stress." As mentioned previously, the deviation from steel LRFD notation was needed because it is anticipated that designers of wood products will track not only the forces due to factored loads (called "required" resistance in steel LRFD), but also both reference (i.e., unadjusted) and adjusted resistance and sometimes reference and adjusted member strength.

An overview of the LRFD notation in this standard is as follows.

**Loads:** subscripted with a "u," called "force due to factored loads," units = kips of force, kip-inches of moment, etc.

**Member:** reference values are unadjusted, adjusted values have a "prime" (′) attached, called "resistances," units = kips of force, kip-inches of moment, etc.

**Material:** reference values are unadjusted, adjusted values have a "prime" (′) attached, called "strengths," units = ksi, kips, etc.

No separate notation is provided for the product of the resistance and its resistance factor (and the time-effect factor, where applicable). Thus, for moment, the product $\lambda\phi_b M'$ is used regularly but is not given a separate name.

**C1.1.1 Units.** Most of the equations in this standard do not require explicit statement of units. They only require that the user apply the equations consistently. For example, application of Eq. 6.2-1 only requires the user to input the tension quantities in units of force (kN or kips) and the moment quantities in units of force-length (kN-m or kip-in.).

Several equations throughout the standard contain embedded units in one or more constants. For these equations, separate versions of the equation are provided—the first in U.S. customary units and the second, designated with the letter "M," in metric units.

## C1.3 Loads and Load Combinations

The nominal loads, load combinations, and load factors in Sec. 1.3 appear in Sec. 2 through 9 of the ASCE Standard Minimum Design Loads for Buildings and Other Structures (ASCE7-93). The loads specified in Model Building Codes are similar. (See separate discussion of earthquake loads below.) These load requirements are suitable for use with all materials of construction, including engineered wood construction. They do not apply to vehicle loads on bridges, construction loads, and other loads that are outside the scope of ASCE 7-93.

**C1.3.1 Nominal loads.** The commentary for this section summarizes the basis for the nominal loads appearing in ASCE7-93 and relates the nominal values to load survey information where such

**TABLE C1.1-1.**
**Metric conversion units.**

| To convert from | To | Multiply by |
|---|---|---|
| degree Fahrenheit | degree Celsius | t°C = (t°F − 32)/1.8 |
| foot | meter (m) | $3.048\ 000 \times 10^{-1}$ |
| $ft^2$ | $m^2$ | $9.290\ 304 \times 10^{-2}$ |
| $ft^3$ | $m^3$ | $2.831\ 685 \times 10^{-2}$ |
| inch | m | $2.540\ 000 \times 10^{-2}$ |
| pound-force (lbf) | newton (N) | 4.448 222 |
| pound (lb avoirdupois) | kilogram (kg) | $4.535\ 924 \times 10^{-1}$ |
| $lbf/in^2$ (psi) | pascal (Pa) | $6.894\ 757 \times 10^3$ |
| lbf/ft | N/m | $1.459\ 390 \times 10$ |
| $lbf/ft^2$ | Pa | $4.788\ 026 \times 10$ |
| $lb/ft^2$ | $kg/m^2$ | 4.882 428 |
| $lb/ft^3$ | $kg/m^3$ | $1.601\ 846 \times 10$ |

information exists. ASCE 7-93 accounts for the fact that structural loads are random in nature by specifying the nominal load for design at a conservative fractile of the load distribution, wherever possible (White and Salmon, Chap. 2, 1987).

**Dead loads:** Dead loads include the weight of the structure and associated permanent construction and attachments. These are given in Sec. 3 of ASCE 7-93.

**Occupancy live load:** Live loads arise from the weight of building occupants and their possessions and from moveable equipment and fixtures. Nominal live loads for design are given in Sec. 4 of ASCE 7-93. Unlike some codes, ASCE 7-93 differentiates between occupancy live loads, roof live loads, rain and snow loads. Load magnitudes in ASCE 7-93 are based on the combined effects of sustained live loads plus transient live loads. Load surveys have shown that the live load on a floor at a given point in time (the sustained live load) typically is about 20% to 30% of the nominal live load, L, with a coefficient of variation on the order of 0.60, depending on the loaded area. The average maximum sustained live load in 50 years is only about 50% of L. However, load surveys typically miss the transient component of live load which arises from temporary crowding, remodeling, or emergencies. When the transient load is included in the analysis, it is found that the nominal live load, L, in ASCE 7-93 is approximately equal to the mean value of the maximum combined (sustained plus transient) live load to occur in 50 years (Chalk and Corotis, 1980). However, the combined live load may act only for a few weeks in 50 years; for the remaining time, the sustained live load acts alone. Thus, the assumption (AF&PA, 1991) that the full design live load acts cumulatively for 10 years is unduly conservative. Moreover, when combining live load with other variable loads for design purposes, only the sustained load is required since the chance of a joint occurrence of transient live load with a significant peak of the other variable loads is very small.

**Roof live load:** The roof live load, $L_r$, given in Sec. 4.11 of ASCE 7-93, accounts for miscellaneous live loads on the roof due to periodic inspection, maintenance, and repair. The roof live load is important for certain wood structural components, particularly in areas where snow loads are not significant.

**Rain loads:** ASCE 7-93 does not provide specific values for rain load on roofs, leaving these up to the discretion of the engineer of record. The emphasis is on providing adequate and reliable drainage so that significant rain loads cannot occur. Roofs also should be designed so as to preclude instability from ponding loads (see Sec. 5.7 of this Standard).

**Snow loads:** Snow loads generally provide the governing load requirements for the design of roofs in northern and mountainous regions of the United States. The ground snow load is selected as that load value with a probability 0.02 of being exceeded in any year (Ellingwood and Redfield, 1983). The snow map in ASCE 7-93 is obtained by smoothing the site-specific data. The roof load is determined by multiplying the ground snow load by a ground-to-roof conversion factor determined from surveys of snow accumulation on roofs (O'Rourke, et al, 1982).

The mean of the annual extreme roof snow load is typically about 0.2 times the nominal roof snow load in ASCE7-93; the coefficient of variation is site-dependent and seldom less than 0.75. Extreme roof snow loads generally have durations of on the order of days to weeks.

**Wind and earthquake loads:** Wind loads depend on the wind environment and the aerodynamic characteristics of the building. Earthquake loads are a function of seismic zone and the specific configuration of the building.

This standard incorporates the newly developed ASCE 7-93 earthquake load provisions, based largely on the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (BSSC, 1991). Previous earthquake loading provisions (ASCE 7-88, withdrawn with the publishing of ASCE 7-93; the Uniform Building Code through the 1994 Edition) produce loads for use with either allowable stress design methods (on an unfactored basis) or with load and resistance factor design methods (on a factored basis). The ASCE 7-93 provisions address earthquake loads in a fundamentally different manner, wherein the provisions are tailored to the needs of load and resistance factor design (and allowable stress design is permitted merely as a convenience). ASCE 7-93 earthquake design loads are based on the expectation of inelastic behavior and resulting energy dissipation and presumes that certain detailing requirements will be followed. Thus, the use of the 1.0 load factor for earthquake loading is only intended for use with ASCE 7-93 provisions. For proper use with other earthquake loading provisions, the user of this standard needs to be aware of the different formulations of E and must select a load factor appropriate to the formulation being used. LRFD resistances to be used with this standard have been developed in accordance with ASTM D5457 procedures. These procedures have been judged to provide reasonable compatibility with current levels of performance under the load conditions and using the load factors specified in ASCE 7-93. Thus, the use of different load criteria or load factors requires engineering judgment that is beyond the scope of this standard.

In contrast to dead, live, and snow loads, the effects of which are primarily static, extreme winds and earthquakes cause forces that fluctuate rapidly in time. Structurally significant winds occur on a time scale of hours, with the peak loads incorporated into the ASCE 7-93 provisions occurring on a scale of seconds. Structurally significant earthquake loading occurs on a time scale of less than 1minute.

**C1.3.2 Load combinations.** The load combinations are the result of a program to unify the structural design process by developing common bases and load requirements for design with different construction materials (Galambos et al., 1982; Ellingwood et al., 1982). The load combinations were developed using principles of structural reliability theory and probabilistic load modeling techniques.

Probabilistic load modeling research (e.g., Turkstra and Madsen, 1980) has shown that the maximum effect of a combination of loads generally occurs when one of the loads in the combination attains its maximum value during the reference period, taken herein as 50years. The maximum load effect is then

$$U = D + \max_{i} \left[\max_{t} X_i(t) + \Sigma X_j(t)\right] \quad \text{(C1.3-1)}$$

in which D = dead load, and $X_i(t)$ and $X_j(t)$ = loads that vary in time.

Studies have shown that Eq. C1.3-1 is a good approximation for most practical load cases affecting building structures. The term "max $X_i(t)$" is denoted by the "principal (variable) load," while $X_j(t)$ are the "companion actions," and design load combinations with the format of Eq. C1.3-1 are referred to as "companion action" formats. Equation C1.3-1 has been transformed using probabilistic load modeling techniques (Galambos et al., 1982; Ellingwood et al., 1982) into the set of design load combinations that appear in Sec. 1.3.2 with the general format,

$$U = \gamma_d D + \gamma_i X_i + \sum_{j=1}^{n} \gamma_j X_j . \quad \text{(C1.3-2)}$$

The load factors, $\gamma_d$, $\gamma_i$, $\gamma_j$, reflect the uncertainty in the determination of the various loads. Different load factors must be assigned to the permanent and time-varying loads in order to achieve uniform reliability for all combinations of D, $X_i$, and $X_j$. No adjustment is made to the dead load when it is combined with other loads since the dead load is always present.

**C1.3.3 Other loads.** Load factors are provided in the standard for several other types of loads covered in ASCE 7-93 (fluid, soil, ponding, temperature). Note that the load factor for ponding is 1.2, rather than the 1.6 specified for rain load. Ponding is covered in detail in App. A3. Other loads not traditionally covered by ASCE 7-93 may require consideration in design. Statistical data on these loads are limited, and the procedures used to derive the load requirements in Sec. 1.3.1 and 1.3.2 cannot be applied. Designers are advised to give such loads careful consideration.

**C1.3.4 Counteracting loads.** Counteracting loads are especially important in light structures, where the stabilizing effects of gravity loads may not be adequate to counter lateral forces.

## C1.4 Design Basis

**C1.4.1 Limit states design.** This standard is based on the concepts of limit states design. A structure reaches a limit state when it ceases to fulfill its intended purpose in some way. Two general types of limit states apply for building structures: ultimate limit states and serviceability limit states. Ultimate limit states relate to requirements for safety under extreme load conditions, and include rupture, instability, and loss of equilibrium. Codes and specifications emphasize the ultimate limit states because of the paramount importance of public safety in design. Serviceability limit states relate to functional requirements under ordinary or service conditions, and include unacceptable deformations and vibrations. Limit states vary from member to member and several may have to be considered in design. Generally, one dominant limit state serves as the design basis; the others then are checked.

Wood structures traditionally have been designed using allowable stress concepts. In allowable stress design, the computed elastic response of a structure to a set of unfactored nominal loads is compared to an allowable stress set at some fraction of the limiting stress of the material (e.g., stress at which rupture or instability occurs). Serviceability concerns are reflected in the choice of limiting deflections. By keeping stresses low and elastic throughout the structure, the allowable stress criteria not only ensured safety but indirectly took care of many serviceability problems as well. However, modern design and construction practice and the use of high-strength materials have caused stresses at service loads to increase and exposed a number of shortcomings of allowable stress design (Allen, 1976; Galambos et al., 1982). Limit states design, with its explicit consideration of each source of uncertainty, has the potential to permit code writers and designers to pay closer attention to the relations between structural loads, behavior, and performance requirements (AISC, 1994).

**C1.4.2 Structural analysis.** The force or moment due to factored loads acting upon structural members and connections is determined by structural analysis for appropriate factored load combinations in Sec. 1.3.2. Elastic analysis is permitted unconditionally by this standard. Nonlinear behavior of members and connections is permitted, provided that substantiating data on behavior is available and approved by the authority having jurisdiction.

If the relationship between loads and structural response is nonlinear, load factors should be applied to nominal loads prior to performing the structural analysis.

Load patterns or combinations that produce critical forces may not be the same in all members. The designer is advised to take these differences into account in determining the forces due to factored loads.

*C1.4.2.1 Modulus of elasticity.* For strength or stability limit states, the design equations specify the use of $E_{05}'$ rather than $E'$ for the value of the modulus of elasticity. This is consistent both with allowable stress design and with other strength properties. Default values are typically based on the assumption of a normal distribution with coefficients of variation of 0.11 (MSR lumber, glulam, structural composite lumber), 0.15 (machine evaluated lumber), and 0.25 (other products), respectively, multiplied by a 1.03 flexure-to-axial stiffness adjustment factor. This assumption yields $E_{05}'$ values equal to $0.84E'$, $0.78E'$, and $0.61E'$, respectively, for the three product groups.

**C1.4.3 Strength limit states.** The resistance criteria appearing in this standard for engineered wood construction, including resistance factors and time effect factors, have been derived to be compatible with the loads and load combinations for limit states design that appear in Sec. 2.4 of ASCE 7-93. These resistance criteria are not applicable to design of wood structures using load requirements that differ from those in ASCE 7-93.

The basic requirement for safety is expressed as:

$$\text{Factored member resistance} \geq \text{forces due to factored loads} \qquad (C1.4\text{-}1)$$

in which the forces due to factored loads is defined by the response to the appropriate factored load combinations in 1.3.2. In LRFD, the factored member resistance is given by:

$$\text{Factored member resistance} = \lambda \, \phi \, R' \qquad (C1.4\text{-}2)$$

in which $R'$ = adjusted resistance, $\phi$ = resistance factor, and $\lambda$ = time effect factor. The adjusted re-

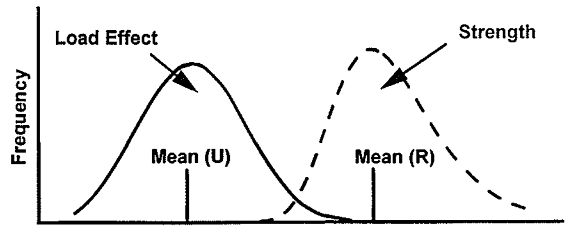



**Figure C1.4-1(a) Frequency distribution of strength, R and load, U.**

sistance, R′, depends on wood species, grade, moisture, size, and other factors reflecting end-use conditions. The resistance factor $\phi$ reflects variabilities associated with the resistance and the mode and consequence of failure. The time effect factor, $\lambda$, accounts for the time-dependent strength of wood under load. The time effect factor depends on the temporal characteristics of the principal load in the combination and thus on the load combination considered.

LRFD is one particular form of probability-based limit states design. In probability-based limit states design, a code performance objective, expressed in terms of a desired probabilistic measure of reliability, is transformed to a set of conventional safety checking equations such as those represented by Eq. C1.4-2 (Ellingwood et al., 1982). This transformation is carried out by the code committee; the designer need not deal with the complexities of reliability analysis, and the end product has a conventional appearance. The process is illustrated briefly by the following. The resistance, R, and the dimensionally consistent structural effect of the applied loads, U (Eq.C1.3-1), for a given limit state are assumed to be statistically independent random variables. Figure C1.4-1(a) illustrates the frequency distributions for R and U. The limit state occurs when R < U. The frequency distribution of R − U is described in Figure C1.4-1(b). The limit state probability, the probability

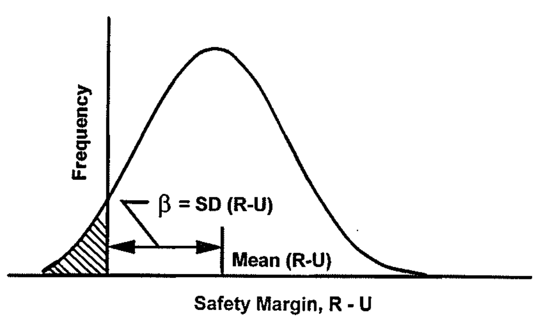



**Figure C1.4-1(b) Frequency distribution of safety margin, R-U.**

that R − U < O, is represented by the shaded area under the frequency function of R − U. This probability can be decreased by increasing the mean of R − U. For a prescribed limit state probability, the mean of R − U must be $\beta$ standard deviations of R − U above zero. The factor, $\beta$, is denoted as the reliability index. It typically varies between 2 and 5, and is a convenient relative measure of reliability for code work.

Design equations such as Eq. C1.4-2 traditionally contain factors of safety and set the relative position of the frequency function for R with respect to that of U. Once a target reliability measure is identified by a code committee or regulatory authority, a set of design equations can be derived that are consistent with this reliability. For wood structures, this process is complicated by the fact that the strength of wood is sensitive to the rate of application of loads and to their duration (Itani and Faherty, 1984). Several improved models for analyzing damage accumulation in wood are now available (Gerhards and Link, 1986; Foschi and Barrett, 1982). These damage accumulation models can be used, along with stochastic process models of common structural loads, to obtain estimates of limit state probabilities or reliability indices for wood structures, taking rate and duration of load effects into account (Hendrickson, Ellingwood, and Murphy, 1987). Conversely, if a target reliability measure is set, corresponding values of R′, $\phi$, and $\lambda$ in C1.4-2 can be determined for use in design with the load combinations in Sec.1.3.

**Illustrations of reliability assessment procedure.** Consider a glulam roof beam supporting a dead load of 10 psf and a snow load of 40 psf, designed using existing working stress design procedures. The safety check is

$$1.15\, F_b S_x = D + S \qquad \text{(C1.4-3)}$$

in which 1.15 is the load duration adjustment for snow, $F_b$ is the allowable stress ("normal" load duration), $S_x$ is the section modulus in bending, and D and S are nominal dead and snow loads (ASCE 7-93). The evaluation of stochastic damage accumulation requires the stress ratio, SR(t), as a function of time, t. In terms of the applied loads, the stress ratio is

$$SR(t) = \frac{D + S(t)}{S_x\, MOR} \qquad \text{(C1.4-4)}$$

in which D is random dead load, S(t) is the snow load, modeled by a stochastic pulse process, and

MOR is the modulus of rupture determined from a conventional strength test. The snow load pulse process is developed so as to be consistent with the distribution of the annual extreme, which is modeled by a lognormal distribution (Hendrickson et al., 1987; Murphy et al., 1988).

For this example, the strength values of glulam beams in flexure were taken from an analysis of full-scale beam tests conducted as part of an industry research project. Statistical analysis of these data showed that the MOR (corrected for size and load-span ratio) is described by a two-parameter Weibull distribution with a mean of 6865 psi (47 MPa) and coefficient of variation of 0.15. The 5th-percentile MOR, $F_{05}$, is 5061 psi (35 MPa) and $F_{05}/2.1$ is 2410 psi (17 MPa), almost precisely the allowable stress in bending.

**Target reliability index.** Assuming that the mean load pulse duration ranges from 1 to 2 weeks and that the probability of measurable snow on the roof during the snow season is in the range 0.2–0.4 at any given time, the reliability indices, $\beta$, for roof beams designed by existing specifications vary from 2.1 to 2.2. Similar analyses performed for glulam beams designed by ASD and subjected to occupancy live load lead to $\beta$'s of about 2.6 to 2.7, depending on the assumed duration of the transient live load, L. The differences from the snow load combination arise from the inconsistent treatment of the effect of temporal variation in live and snow load in the current factors used to adjust for duration of load (AF&PA, 1991). There is no apparent justification for this difference in terms of desired structural performance.

The target reliability for a particular limit state can be set from an evaluation of reliability measures associated with existing acceptable design practice (AF&PA, 1991). In the absence of data to suggest that roof members are underdesigned, a target reliability index of 2.4 for flexure might be selected for both load combinations. For comparative purposes, the reliability indices of compact steel beams designed by the AISC LRFD specification are about 2.4 and 2.2 for load combinations involving live and snow load, respectively, at comparable ratios of L/D and S/D. This small difference arises from the decision to specify the same load factor within ASCE 7-93 for live and snow load in the interest of simplicity (Ellingwood et al., 1982).

**Standardization of reliability concepts.** As mentioned in the scope, ASTM D5457-93 is being used as the consensus basis for developing resistance values for wood products for use under this standard. This ASTM specification provides two options by which LRFD resistance values can be developed. The first option, format conversion from allowable stress design, permits those products with code approved allowable stress design values to convert those values to the LRFD basis. The conversion procedure requires multiplication by a simple numerical factor that was derived on the basis of minimizing the change from allowable stress design across common design load cases. Format conversion preserves the same level of reliability inherent in allowable stress design procedures. Since this reliability level is not quantified, users of this option cannot claim compliance with a specific reliability index. The second option requires analysis of test data for the derivation of a resistance value. To provide stable and usable results across a broad range of product lines, the ASTM specification gives precise data analysis and design value calculation procedures. The rationale behind the procedures in the ASTM specification are discussed further in Gromala et al., 1994.

**Time-effect factor.** The time-effect factor, $\lambda$ (the LRFD equivalent of the load duration factor), was determined by the following procedure (Ellingwood and Rosowsky 1991). Consider a beam designed to support a roof at a site in the northern U.S. where snow load governs design. The safety check in LRFD would be:

$$\lambda \, \phi_b \, F_b' \, S_x = 1.2D + 1.6S \qquad \text{(C1.4-5)}$$

in which D and S are dead and snow load given in ASCE 7-93, $F_b'$ = Adjusted bending strength, $S_x$ = section modulus, and $\lambda$ and $\phi_b$ are constants. If the product $\lambda\phi_b F_b'$ is known, $S_x$ can be determined and the beam can be designed. A reliability index, $\beta$, can then be computed for this particular design for two conditions: (1) load duration effects present, using the procedure outlined above, and (2) load duration effects absent. In the second case, failure occurs due to either overload or an understrength beam, $\lambda$ equals 1.0 by definition, and the product $\lambda\phi_b F_b'$ simply equals $\phi_b F_b'$. Repeating the analysis yields a relation between $\beta$ and the product $\lambda\phi_b$ or $\phi_b$; this is illustrated in Fig. C1.4-2 which was prepared using an exponential damage rate model developed at the U.S. Forest

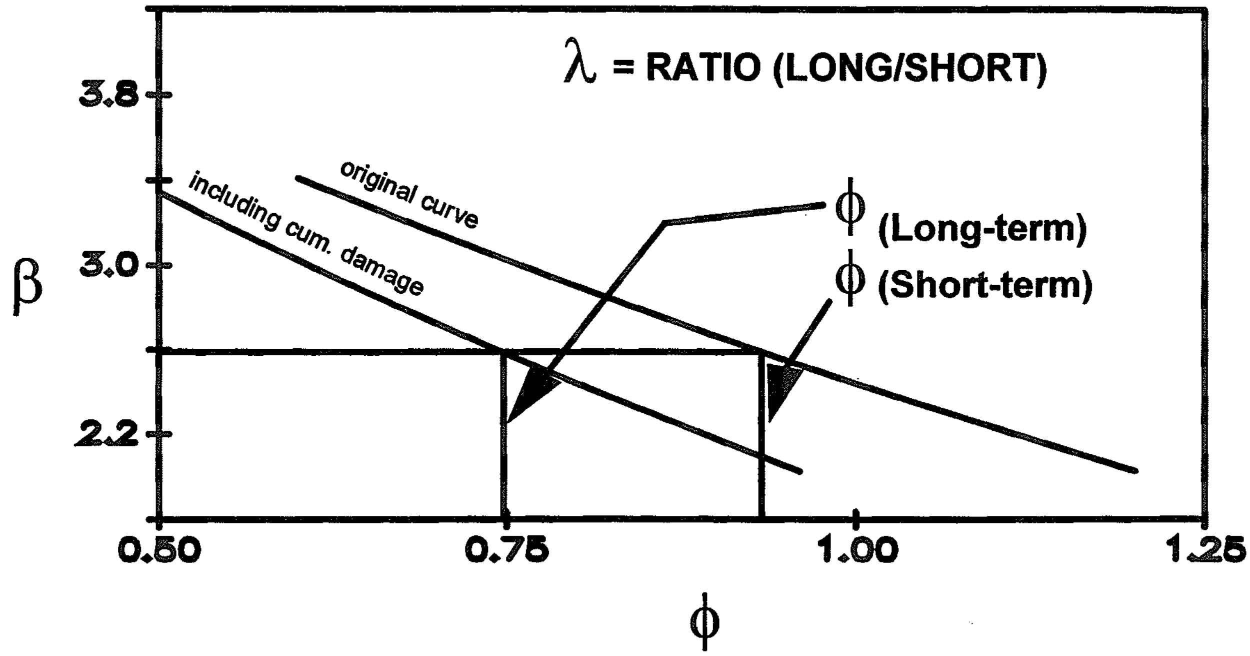



**Figure C1.4-2**

Products Laboratory (Gerhards and Link, 1986). Entering Fig. C1.4-2 at the target reliability, one obtains the necessary $\phi_b F_b'$ and $\lambda$.

If the factors $F_b'$, $\phi$, and $\lambda$ are set so that the target reliability index, $\beta$, for flexure is approximately 2.4 for all load combinations, the $\lambda$'s obtained from plots such as Fig. C1.4-2 are about 0.77 for load combinations involving occupancy live load or snow load. If the damage model developed at Forintek (Foschi and Barrett, 1982) were used instead, the $\lambda$'s would be 0.83 for the same target reliability. This procedure led to the choice of time effect factors of 0.80 for both snow and floor (occupancy live) load.

Survey data on miscellaneous roof live loads are unavailable. As an alternative, a live load pulse process was constructed with the assumptions that (1) events giving rise to significant roof live load occur approximately four times during a 50 year period, and (2) the mean and coefficient of variation of the 50 year maximum roof live load range from 0.8–1.0 times the nominal load in ASCE 7-93 and from 0.25–0.30, respectively. These statistics are consistent with those for other live load cases for which data are available. With these assumptions, $\lambda$ = 0.78 when $\beta$ =2.4.

Limited live load survey data exist for warehouses, storage and industrial buildings, libraries, and similar heavily loaded occupancies (Chalk and Corotis, 1980). In such occupancies, the transient component of the live load is negligible and the temporal variations in load arise mainly from fluctuations in the sustained component of the live load. Making plausible assumptions concerning the temporal variation of sustained loads, it was found that $\lambda \cong 0.70$ for a target $\beta$ of 2.4.

The time-effect factor for wind and earthquake load cases was chosen based on judgment. The potential for damage to accumulate during wind or earthquake events giving rise to fluctuating stresses with frequencies of 2 Hz or greater has been evaluated. Results show that it is necessary to scale wind or earthquake excitations upward from the ASCE 7-93 load levels by about a factor of 4 before damage accumulates using the available damage accumulation models (Gerhards and Link, 1986). On the basis of these studies, the time-effect factors for wind and earthquake are set equal to 1.0 in this Standard.

It is significant from a designer's perspective that the time-effect factor is identical for the gravity load cases of floor live, snow and roof live, in contrast to the different factors used in allowable stress design. As discussed above, this is because, when measured on a reliability basis, the factor used for floors in ASD is conservative. Conversely, the factor used for roof live load was judged to be close enough to the other two cases that the same value could be used. The end result is that time-effect factors more closely align with standards of other countries, in which relatively short-term loads receive a 1.0 factor (relative to short term test values), intermediate duration loads receive a 0.80 factor and long-term loads receive the lowest factor.

Engineered wood construction comprises a wide range of producers and product types. Thus, one might expect that the specified values of $\phi$ and R should vary across product lines in order to maintain the code objective of a consistent reliability index. To facilitate codification and to make the standard usable across a wide a range of wood products, one basic set of $\phi$-factors was selected to reflect in a general way the relative variability in strength and differences in failure modes and consequences. Further adjustments to account for differences in product variability are built into the reference resistance, R, in accordance with ASTM D5457-93.

**C1.4.4 Serviceability limit states.** Serviceability limit states relate to functional requirements of the building under ordinary service conditions. Excessive deformations that are unsightly or that lead to nonstructural damage and excessive structural motions that cause discomfort to building occupants are examples of unserviceability. Serviceability depends on the usage of the building and the perceptions of the building occupants. Serviceability considerations are particularly important in lightweight structures.

Serviceability normally should be checked using service loads. Service loads are generally taken as the unfactored nominal loads; lesser values may be taken if substantiating information is provided. Additional guidelines are provided in Chap. 10.

**C1.4.5 Existing structures.** Guidance for selecting appropriate values of adjusted resistance, R′, for members and connections in existing structures can be found in several references (Meyer and Kellogg, 1982 and ASCE, 1982, for example). Effects of structural degradation should be evaluated and needed corrective measures taken. Structural degradation of wood structures can take many forms including, but not limited to, decay, insect damage, splitting at critical connections, and delamination.

## References-Commentary Section 1


Allen, D.E. 1976. *Limit States Design - A Probabilistic Study.* Canadian J. Civil Engr. 2(1):36–49.

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC, 125 p.

American Institute of Steel Construction. 1994. *Load and Resistance Factor Design Specification for Structural Steel Buildings,* Second Edition. Chicago, IL.

American Society of Civil Engineers. 1982. *Evaluation, Maintenance and Upgrading of Wood Structures.* New York, NY.

American Society of Civil Engineers. 1993. *Minimum Design Loads for Buildings and Other Structures.* ASCE 7-93. New York, NY.

American Society for Testing and Materials. 1993. *Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design.* ASTM D5457-93. Philadelphia, PA.

Bodig, J., Cheung, K., and Cunningham, T.P., Jr. 1995. *Engineered Wood Construction: Structural Properties for LRFD.* J. Str. Div., ASCE 121(9):1346–1351. New York, NY.

Building Seismic Safety Council. 1991. *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings.* Report FEMA 222 and 223 (two volumes), Federal Emergency Management Agency, Washington, DC.

Chalk, P. and Corotis, R.B. 1980. *A Probabilistic Model for Design Loads.* J. Str. Div., ASCE 106(10):2017-2030. New York, NY.

Ellingwood, B., MacGregor, J.G., Galambos, T.V., and Cornell, C.A. 1982. *Probability Based Load Criteria: Load Factors and Load Combinations.* J. Str. Div., ASCE 108(5):978–997. NewYork, NY.

Ellingwood, B. and Redfield, R. 1983. *Ground Snow Loads for Structural Design.* J. Str. Engr., ASCE 109(4):950–964. New York, NY.

Ellingwood, B. and Rosowsky, D. 1991. *Duration of Load Effects in LRFD for Wood Construction.* J. Str. Engr., ASCE 117(2):584–599. New York, NY.

Ellingwood, B., Hendrickson, E., and Murphy, J. 1988. *Load Duration and Probability Based De-*

*sign of Wood Structural Members.* Wood and Fiber Science 20(2):250–265.

Foschi, R. and Barrett, J.D. 1982. *Load Duration Effects in Western Hemlock Lumber.* J. Str. Div., ASCE 108(7):1494–1510. New York, NY.

Galambos, T.V., Ellingwood, B., MacGregor, J.G., and Cornell, C.A. 1982. *Probability Based Load Criteria: Assessment of Current Design Practice.* J. Str. Div., ASCE 108(5):959–976. New York, NY.

Gerhards, C. and Link, C. 1986. *Effect of Loading Rate on Bending Strength of Douglas-Fir 2by4's.* Forest Prod. J.36(2):63–66. Madison, WI.

Gromala, D.S., Pollock, D.G., and Williamson, T.W. 1994. *Development and Implementation of a Load and Resistance Factor Design System in the United States.* In Proceedings of the 1994 Pacific Timber Engineering Conference. Queensland University, Brisbane, Australia.

Hendrickson, E., Ellingwood, B., and Murphy, J. 1987. *Limit State Probabilities of Wood Structural Members.* J. Str. Engr., ASCE 113(1):88–106. New York, NY.

Itani, R. and Faherty, K. eds. 1984. *Reliability Based Design.* In Structural Wood Research: State of the Art and Research Needs, American Soc. of Civil Engr., New York.

International Conference of Building Officials. 1994. *Uniform Building Code.* Whittier, CA.

Madsen, B. 1975. *Strength Values for Wood and Limit States Design.* Canadian J. Civil Engr. 2(3):270–279.

Meyer, R. W. and Kellogg, R. M. 1982. *Structural Uses of Wood in Adverse Environments.* Society of Wood Science and Technology, Van Nostrand Reinhold.

Murphy, J. F., ed. 1988. *Load and Resistance Factor Design for Engineered Wood Construction—A Prestandard Report.* Am. Soc. of Civil Engrs., New York, NY.

O'Rourke, M., Redfield, R., and von Bradsky, P. 1982. *Uniform Snow Loads on Structures.* J. Str. Div., ASCE 108(12):2781–2798. New York, NY.

Turkstra, C. and Madsen, H. 1980. *Load Combinations in Codified Structural Design.* J. Str. Div., ASCE 106(12):2527–2543. New York, NY.

White, R.N. and Salmon, C.G., eds. 1987. *Loads.* In Chapter 2 of Building Structural Design Handbook, John Wiley & Sons, New York, NY.

# COMMENTARY

# Chapter 2 Design Requirements

### C2.2 Gross And Net Areas

The effects of materials removed from the gross area, A, of a wood member are recognized by considering the net area, $A_n$, in calculating the strength of a member. For further information for consideration of net area associated with connections, refer to the Commentary for Chap.7. Note that nominal sizes are often used to describe some wood products (i.e., 2×4). Nominal sizes, when they differ from actual sizes, are not appropriate for use in design calculations. See product manufacturer's literature for sizes to be used in design calculations.

### C2.3 Stability

Commentary on stability issues associated with individual member design can be found in the commentary to Chap. 4.

Special consideration should be given to secondary effects which can result from the deflected shapes of wood structures. So-called P-Δ effects can occur in wood structures, especially in cases involving long members with multiple loads. An example is the case of a single wood pole overhead line utility structures. Methodology for handling P-Δ effects resulting in this case from combined horizontal and vertical loading can be found in references by Goodman et al., 1981.

### C2.4 Lateral Support

Detailed calculations for the effects of unsupported length on design values for beam and column members are found in Chap. 4, 5, and 6. For bending members when the depth of the member does not exceed its thickness, no lateral support is required. When the depth of a bending member exceeds its thickness, lateral support shall at least be provided at points of bearing which shall prevent rotation and/or lateral displacement at those points, as cited in Sec. 2.4 of the standard. If adequate intermediate supports are provided, the corresponding reduction in unsupported length may be considered.

For forces acting in a direction parallel to a truss or beam, column bracing may be provided by knee braces or, in the case of trusses, by extending the column to the top chord of the truss where the bottom and top chords are separated sufficiently to provide adequate bracing action. In a direction perpendicular to the truss or beam, bracing may be provided by wall construction, knee braces, or bracing between columns. Such bracing between columns should be installed preferably in the same bays as the bracing between trusses.

Truss bracing is often provided by: (a) diagonal lateral bracing between top chords of trusses (omitted when the lateral support of the compression chords is adequate); (b) vertical sway bracing installed in each third or fourth bay at intervals of approximately 35 feet; (c) bottom chord lateral bracing, installed in the same bays as the vertical sway bracing and extending from side wall to side wall; and (d) struts, installed between bottom chords at the same truss panels as vertical sway bracing and extending continuously from end wall to end wall.

When roof joists or purlins are used between arches or compression chords, or when roof joists or purlins are placed on top of an arch or compression chord, and are securely fastened to the arch or compression chord, the unsupported length should be calculated using the depth of the arch or compression chord or calculated using the least dimension of the arch or compression chord between points of lateral support.

When planks are placed on top of an arch or compression chord, and securely fastened to the arch or compression chord, or when sheathing is nailed properly to the top chord of trussed rafters, the depth rather than the width of the arch, compression chord, or trussed rafter may be used as the least dimension.

When stud walls in light frame construction are adequately sheathed on at least one side, the depth, rather than width of the stud, may be taken as the least dimension.

## C2.5 Reference Conditions

The reference environmental conditions are self-explanatory, representing a range of design conditions under which the reference resistance values require no adjustment.

The requirement that products be new, rather than reused, is often a point of question. Design resistance values provided in this standard are for new products placed in service for the first time. This standard does not cover the procedures needed to establish design resistance values for products used previously. The reasons for this restriction are related to the difficulty of knowing the grade of used material and the grading rules in effect at the time of manufacture, the potential that the material had significant, and potentially damaging, overloads during its history, and the potential for decay and insect damage.

This standard provides consideration for most typical exposure conditions. It does not address the design considerations needed for unique exposures such as contact with specific chemicals, radioactive radiation, steaming, etc. For these unique applications, the designer should consult the available literature or conduct experiments to aid in developing modifiers for design resistance values.

## C2.6 Adjusted Resistance and Adjusted Strength

**C2.6.2 Adjustment factors for end-use.** A set of reference end-use conditions is assumed for the conditions as given in this section. Numerous references exist describing test results for wood products under various conditions of moisture, temperature, and duration of load and are listed in this commentary.

Designers of wood structures must adjust reference resistances as specified in Sec. 2.5 for their specific end-use conditions. It is anticipated that the reference conditions will be adequate to cover a large proportion of typical designs of covered protected structures thus avoiding the need for additional adjustment in the majority of cases.

**Moisture:** An adjustment factor, $C_M$, shall be applied to calculate adjusted member resistances under moisture conditions of service outside of the reference end-use conditions cited in Sec. 2.5.

Reference moisture conditions cover the range commonly encountered with protected structures (dry use conditions). In this condition, the yearly average equilibrium moisture content does not exceed 15% and the maximum equilibrium moisture content does not exceed 19%. For conditions where the equilibrium moisture content exceeds these limits, the designer shall apply the appropriate end-use moisture use adjustment factor, $C_M$.

**Temperature:** An adjustment factor, $C_t$, shall be applied to calculate adjusted member resistance to account for effects of end-use and in-service temperatures which do not conform to standard end-use conditions cited in Sec. 2.6. The following guidelines are provided to assist the designer in applying the temperature end-use adjustment factor, $C_t$.

As wood is cooled below normal temperatures, its strength increases. When heated, its strength decreases. This temperature effect is immediate and its magnitude varies, depending on the moisture content of the wood. Up to 150°F, the immediate effect is reversible. The member will recover essentially all its strength when the temperature is reduced to normal. Prolonged heating to temperatures above 150°F can cause a permanent loss of strength.

In some geographic regions, structural members are periodically exposed to elevated temperatures. However, the concurrent relative humidity is generally low and, as a result, wood moisture contents are also low. The immediate effect of the periodic exposure to the elevated temperature is less pronounced because of this dryness. Also, independent of temperature changes, wood strength properties generally increase with a decrease in moisture content. In recognition of these offsetting factors, it has been traditional practice to use the design values without adjustment for ordinary temperature fluctuations and occasional short-term heating to temperatures up to 150°F for wood products and up to 200°F for structural panels.

**Fire-retardant treatment:** This standard does not provide specific recommendations for adjustment factors for fire-retardant treated wood products. Appropriate factors to apply to nominal resistance should be obtained from suppliers of fire-retardant treated products.

**Preservative treatment:** An adjustment factor, $C_{pt}$, may be required for some materials if an approved pressure-impregnation process has been used to preservatively treat the wood prior to its use. The factor is 1.0 for many common types of treatments. However, the designer is responsible for verifying via applicable codes and standards, or from suppliers of preservatively treated wood products what factor should be used in design.

**C2.6.3 Adjustment factors for member configuration.** Adjustment factors for effects of member geometry and configuration within the structure are applied to adjust the reference resistance when the application differs from the reference conditions. This category was developed to provide a framework by which designers can review the long, and often cumbersome, list of adjustment factors. This category includes the generic size factor, stability factors for both bending and compression, assembly factors, the bearing area factor, and the form factor.

**C2.6.4 Additional adjustments for structural lumber and glued laminated timber.** This section includes factors that apply only to lumber and glulam. The factors for shear stress, buckling stiffness, and flat-use are taken directly from current ASD requirements (AF&PA, 1991). Size effects for glulam beams are currently quantified on the basis of length, width, and depth effects which are combined into the volume factor ($C_V$). The curvature factor for glulam is also taken directly from ASD requirements.

**C2.6.5 Additional adjustments for structural panels.** For panel products, a panel width effect factor ($C_W$) is applied if a panel width is less than 24 in. In addition, the latest tabulation of panel properties includes a grade/construction factor, $C_G$, to modify basic stress values to better reflect specific layup properties.

**C2.6.6 Additional adjustments for timber poles and piles.** The untreated factor, $C_u$, requires further explanation. Because timber piles are typically produced and used in a treated condition, this is chosen as the reference condition. On this basis, for piles that are not treated an increase factor is justified.

**C2.6.7 Additional adjustments for structural connections.** The factors for structural connections are consistent with those used in ASD (AF&PA, 1991).

## References Commentary Chapter 2


American Institute of Timber Construction. 1994. *Timber Construction Manual.* Wiley-Interscience, Fourth Edition.

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction. AF&PA.* Washington, DC, 125 p.

Goodman, J. R., M. D. Vanderbilt, M. E. Criswell, and J. Bodig. 1981. *Probability-Based Design of Wood Transmission Structures.* Electric Power Research Institute, Palo Alto, CA. EL-2040.

Meyer, R. W. and R. M. Kellogg, Editors. 1982. *Structural Uses of Wood in Adverse Environments.*

Society of Wood Science and Technology, Van Nostrand Reinhold Company.

USDA Forest Products Laboratory. 1987. *Wood Handbook-Wood as an Engineering Material.* Agricultural Handbook 72, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

# COMMENTARY

# Chapter 3 Tension Members

## C3.1 General

**C3.1.1 Scope.** Although the tensile strength of clear wood in tension is high, knots and other natural defects can significantly reduce this strength. In addition, because of the reduction from gross area to net area, the connection regions of wood tension members often control the member resistance. End and edge distances and details that minimize horizontal splitting (in the longitudinal direction) can be critical in assuring that the tensile resistance of members as given by the provisions of this chapter will be provided. Special connection design considerations are included in Chap.7. Information on connection details that minimize localized tension problems and are effective for both solid-sawn and glued-laminated timbers are contained in TCM-94 and in textbooks on structural timber design.

Members with combined tension and moments from either eccentricity of the tension loading or from lateral loads must be designed using the interaction formulas of Chap. 6.

**C3.1.2 Member design.** The adjusted resistance equation in this chapter follows the general form of Sec. 1.4.3.

Adjustment factors given in Sec. 2.6 are intended to be applied to the reference property (resistance, strength, or stiffness) values rather than being applied in the final design equations. This procedure is especially important for design equations that use two or more material properties or are nonlinear in one or more parameters.

## C3.2 Tension Resistance Parallel to Grain

**C3.2.1 Tension resistance.** The adjusted tension resistance of a member connected at its ends by bolts or other connectors extending through the member is usually controlled by the critical net section.

The tensile strength of wood members, especially away from areas of connector holes, is known to depend upon the size of the member. The effects of size are accounted for in the adjusted strength values. Note that size adjustment factors for sawn lumber have been judged to be adequate for all lengths available from a sawmill. If a design includes long members under high tensile stresses ($> 0.80\ \lambda\phi_t T'$) the additional length effects should be investigated (Green et al., 1989; Showalter et al., 1987; Lam & Varoglu, 1990). If an adjustment for length is indicated, the smaller of the net section member strength, computed without the length adjustment, and the gross section member strength, computed with the length adjustment, should be used.

Tension members should not be notched because wood material failure along the grain can easily originate at the base of a notch. Because of the orthotropic nature of wood, with its shear strength along-the-grain and tension perpendicular to grain being much lower than for tension parallel to grain, a gradual slope is needed on any cut surface to allow a smooth stress flow in areas of reduced section. A much more gradual transition is needed than for isotropic materials such as steel. If the area of a tension member must be reduced, a gradual symmetrical taper on each side of the member with a slope of no more than 1 on 12, relative to the member axis, is advised.

**C3.2.2 Special considerations for unsymmetrical net areas.** Connection regions should be designed whenever possible so that the centroid of the net section is the same as that of the member gross section. The purpose of Sec. 3.2.2 is to require that members loaded with a significant eccentricity due to connector placement be checked as members with combined tension plus end moment flexural loading.

This section allows some small eccentricity when three or more connectors are used in the connection region under consideration before a combined stress analysis (Sec. 6.2) is required. A load eccentricity of 5% of the member width results in a local maximum flexural stress equal to 130% of the average tensile stress in the absence of eccentricities and/or end moments. Connections with several connectors have some rotational stiffness and can provide some end moment which, along with the tendency of a member loaded in tension to stay straight, helps assure a fairly uniform tensile strain over the tension member section. The actual stress increase is thus less than the amount indicated considering only the eccentricity at the connection.

### C3.3 Tension Resistance Perpendicular to Grain

Wood is usually weakest in tension perpendicular to grain, and failure in this mode is brittle. For both design efficiency and structural safety, tension perpendicular to grain should be avoided whenever possible.

### C3.4 Resistance of Built-up and Composite Members

A built-up member is defined as a member with two or more parallel components of the same material, while a composite member may include a mixture of solid wood, wood products, metal, and other components of different member resistances and stiffnesses.

The primary design concerns for both types of members in tension are that the components be well connected so that the axial strains in all components are equal or nearly equal and that the effects of any splices be properly accounted for in the member resistance assessment.

Roof diaphragm chords and drag struts are often built-up tension members formed from several top plates or other multiple wood members. The cross section area of at least one component must be considered ineffective as the net section can be placed at any of the butt splices.

For composite members made of different material types, the designer is cautioned that even if well connected, various components can reach their tensile resistance at different axial strains. Unless these components have ductility in tension, as is the case for mild steel, the first component to reach its failure strain can limit the resistance of the composite member. Thus, unless the elastic moduli and ductilities of all pieces are similar, the composite resistance may be significantly less than the sum of the individual component resistances.

The components in a composite tension member should generally be placed so the resulting member is symmetrical. The design of a composite member may need to recognize the effects of the possibly quite different time-dependent behaviors and coefficients of thermal expansion among the various materials used in the composite member.

### References Commentary Chapter 3

American Institute of Timber Construction. 1994. *AITC 104-94, Typical Connection Details.* Contained in "Timber Construction Manual." Fourth Edition. Wiley-Interscience.

American Society for Testing and Materials. 1994. *Standard Method of Testing Small Clear Specimens of Timber, ASTM D143-94.* ASTM. Philadelphia, PA.

Green, D.W., R.W. Wolfe, L.A. Soltis, and J.W. Evans. 1989. *On Design Use Adjustments to Lumber Properties Data.* USDA Forest Service, Forest Products Laboratory, Madison, WI.

Lam, F. and E. Varoglu. 1990. *Effect of Length on Tensile Strength of Lumber.* Forest Products Journal, Vol. 40, No. 5, Forest Products Research Society, Madison, WI, pp. 37–42.

Showalter, K.L., F.E. Woeste, and B.A. Bendtsen. 1987. *Effect of Length on Tensile Strength in Structural Lumber.* USDA Forest Service Research Paper FPL-RP-482, Forest Products Laboratory, Madison, WI.

# COMMENTARY

# Chapter 4 Compression Members and Bearing

### C4.1 General

**C4.1.1 Scope.** Compression can act on a member along the member axis, as in a column, or over a part of the end, edge, or any surface of a member, in which case that local area is said to be loaded in bearing. Both concentrically-loaded columns and bearing upon a wood surface are included in this chapter.

Axial compression is often combined with bending from either an eccentricity of the applied axial load or from transverse loads or end moments applied onto the column. In these cases, the provisions of Sec. 6.3 for combined loading apply. Depending on the member slenderness, significant second-order (sometimes called P-delta) effects can occur and must be considered in these members.

**C4.1.2 Member design.** The adjusted member resistance equations of this chapter follow the general form of Sec. 1.4.3. Adjustment factors given in Sec. 2.6 are intended to be applied to the reference property (resistance, strength, or stiffness) values rather than being applied in the final design equations. This procedure is especially important for design equations that use two or more material properties or are nonlinear in one or more parameters.

The resistance factor, $\phi_c$, is used in the equations for both member compressive strength and for bearing.

While many of the design considerations in Chap. 4 focus on overall member performance, the designer must remain aware that local member resistance and stability in regions of concentrated applied loads, either axial or transverse, may control design. These considerations become more critical for built-up and composite I-shaped members, especially those with thin webs. Axial loads must be applied over enough of the cross section or through a long enough connection length that local areas of extremely high bearing and local stresses are prevented. Good design practices and analysis assumptions paralleling those for web crippling and web buckling of metal wide flange sections are recommended for designs of concentrated load regions of thin web I members (AISC, 1994). Detailed design equations for these local effects have not been given in the standard, in part because the behavior and resulting analysis is quite dependent upon the geometry and properties of the thin-web sections for which these local effects are most critical. These web properties differ considerably among the available proprietary products.

## C4.2 Slenderness and Effective Length Considerations

**C4.2.1 Effective column length.** The effective length factor, $K_e$, accounts for the column end restraint and sidesway conditions which change the length of the half sine wave buckled shape relative to an end-braced (no sidesway) column pinned at both ends. Note that the unbraced column length and the effective length factor may be different in each direction.

The effective length, $K_e\ell_u$, can be visualized as the length of the half sine wave curve fit to the shape of the column with the specified end and sway conditions. For example, a column perfectly fixed at each end and with no sidesway at its ends will buckle into a shape having an inflection point at one-fourth the column length from each end, producing a half sine shape in the middle half of the column height. Thus, $K_e = 0.50$ for such an ideal fixed-fixed column.

The effective length factor, $K_e$, is between 0.50 (both ends fixed) and 1.00 (both ends pinned) for column lengths braced against sidesway, and is between 1.00 (both ends rotationally fixed) and infinity (both ends pinned) when the member is unbraced against sidesway. Thus, the use of $K_e = 1.00$ is conservative for members without sidesway, but nonconservative when sidesway is present. For this reason, Sec. 4.2 allows $K_e = 1.00$ to be used as a default value only when no sidesway is present.

The design $K_e$ values for several common design cases are given in Fig. C4.2-1. This figure is the same as that in App. G of the 1991 NDS (AF&PA, 1991). Note that the recommended design values exceed the theoretical values in cases when end fixity is present. This reflects the fact that perfect end fixity is not achieved in actual structures.

**C4.2.2 Column slenderness ratio.** The slenderness ratio appearing in the column buckling equations is the effective column length divided by the radius of gyration. This is a generalized form of the traditional $K\ell_u/d$ factor in the NDS for rectangular columns. The radius of gyration for a rectangular member is 0.289 times the appropriate side dimension. The radius of gyration of a circular section is one-fourth the diameter.

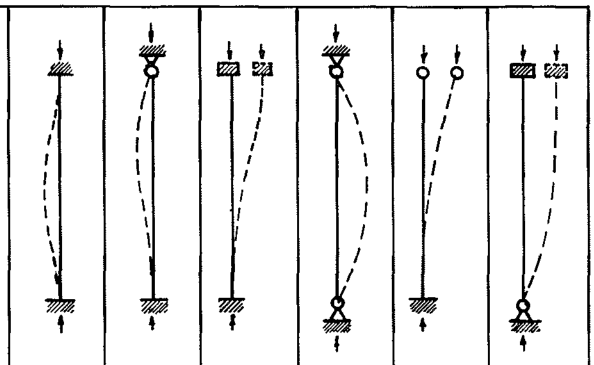
For composite columns consisting of two or more components with different material stiffnesses, the radius of gyration is to be determined using an appropriate transformed gross area. Transformed section radius of gyration values should be calculated only for well-connected composite columns.

The $K\ell_u/r$ limit of 175 when applied to a solid rectangular column corresponds to a $\ell/d$ value of approximately 50 and is consistent with NDS provisions for ASD.

## C4.3 Resistance of Solid Columns Concentrically Loaded in Compression

**C4.3.1 Design material values and design factors.** When member stiffness becomes critical to strength, rather than serviceability, limit states, the standard requires the use of the lower 5th percentile value, $E_{05}$, rather than E in the design equations. Explicit use of $E_{05}$ in the behavioral equations (as opposed to reducing E by some factor) permits

Buckling modes



| | | | | | | |
|---|---|---|---|---|---|---|
| Theoretical $K_e$ value | 0.5 | 0.7 | 1.0 | 1.0 | 2.0 | 2.0 |
| Recommended design $K_e$ when ideal conditions approximated | 0.65 | 0.80 | 1.2 | 1.0 | 2.10 | 2.4 |

End condition code

- Rotation fixed, translation fixed
- Rotation free, translation fixed
- Rotation fixed, translation free
- Rotation free, translation free

***Figure C4.2-1. Effective length factors for wood column design.***

presentation of stability formulas in a familiar strength of materials format. Note that this procedure is also used in allowable stress design, but the reduction from tabulated E to $E_{05}$ is embodied in numerical constants in ASD. The $E_{05}$ value to be used is that corresponding to the buckling direction being considered. As discussed in commentary Sec. C1.4.2.1, default values of $E_{05}$ related to E are commonly used for several wood product lines.

**C4.3.2 Resistance of prismatic columns.** As in allowable stress design, some of the limit states related to column behavior are a combination of strength and stiffness considerations. This presents a difficulty in LRFD because in LRFD the time-effect factor is applied **after** the member resistance is computed (as opposed to ASD, where load duration effects are factored into the behavioral equations). Because member stiffness is assumed to be independent of time under load (as in ASD), this presents a difficulty in presentation format. This dilemma was solved by inserting $E/\lambda$ into the member strength equations (instead of just E). The $\lambda$ in the divisor then cancels out the $\lambda$ in the $\lambda\phi_c P'$ design equation, retaining a time independent buckling limit state.

The column resistance must be checked in each possible buckling direction, with the lower value controlling the design. The continuous column equation given in this section has the same basis as that found in NDS-91. The equation for $C_P$ is written as a stability-related correction factor to be applied to the adjusted short column resistance, $P_0'$. The $C_P$ factor is a function of the ratio of the Euler buckling load to the short column resistance. As noted in other portions of this commentary, this basic ratio is modified by including the time effect factor and the ratio of the compression to stability resistance factors. This algebraic modification is required to provide proper limiting behavior (for the extremes of very short and very long columns). A similar equation form is also used to transition between braced and unbraced beams.

To provide the correct inclusion of the time-effect factor and the stability and compression resistance factors, $\alpha$ has been defined as $\phi_s/\lambda\phi_c$ times the original $\alpha$ value. This original $\alpha$ is equal to the Euler buckling load divided by the very short column strength. At large slenderness values, the continuous column equation gives a stability-related geometry correction factor, $C_P$, approaching $\alpha$ from below. When the resulting adjusted column resistance, P′, which thus approaches the modified $\alpha$ times $AF_c'$, is multiplied by $\lambda\phi_c$ to get the design member strength, the $\lambda$ and $\phi_c$ tend to increasingly cancel at higher slenderness ratios and the adjusted member resistance approaches $\phi_s$ times the Euler critical buckling load from below. This is the desired result of the time-effect factor not being applicable to the elastic buckling case and the long column being subject to a stability resistance factor. For very short columns, $C_P$ approaches unity for all c values and the definition of $\alpha$ has no effect. Thus, the very short column is subject to the time-effect factor and the resistance factor for axial compression. Most columns are between these extremes and are thus partially subject to all of three factors, $\phi_s$, $\phi_c$, and $\lambda$.

The stability resistance factor, $\phi_s$, accounts for uncertainties such as the variation in the design material stiffness, $E_{05}$, along with the effects of the member initial crookedness permitted for compression members, very small accidental eccentricities, and effects of creep. Creep is much less influential than initial crookedness for typical columns (Itani et al., 1986), partly because the maximum design loads are typically of short duration. When members are subject to large ratios of dead to live load and/or when the members are subject to large and frequent temperature and/or moisture cycles, creep can more significantly affect the strength of long column members. For beam-columns, the flexural creep due to bending stresses effectively causes an increasing initial curvature of the column and thus the flexural creep acts to reduce the column strength. The effect of creep can be modeled by decreasing the $E_{05}$ value. Section 7 of ASTM D245 (ASTM, 1993) notes that "the buckling of a long column is sensitive to the duration of load" and proceeds to suggest the usual time-effect factors be applied to the buckling load.

The material stiffness value, $E_{05}$, used in Eq. 4.3-4 is the longitudinal E value. This value exceeds the modulus of elasticity based on flexure and, in NDS-91, is computed as 3% larger. Conversion factors for $E_{05}$ consistent with NDS procedures were discussed in C1.4.2.1.

**C4.3.3 Resistance of notched or bored prismatic columns.** For cases in which the net area is locally much smaller than the gross area, it is possible for the buckling load to exceed the short column resistance based on the net section. For this reason, the latter is set as an upper limit on the member resistance in compression.

The second part of this section provides judgment-based criteria when the reduced section is either long enough or in a critical enough location that the net section is to be used in computing the column resistance. The stated criteria are more definite than the previously used requirement of net section use "when the reduced section occurs in the critical part of the column length that is most subject to potential buckling" (AF&PA, 1991). The designer may base the buckling load on a more detailed analysis recognizing the actual nonprismatic shape of the column.

**C4.3.4 Resistance of tapered columns.** The provisions for determining an equivalent prismatic column for a buckling load determination of columns falling into the four basic cases, case 1 though 4, are based on numerical studies of tapered members with various end conditions (Criswell, 1991). These provisions replace the generally, but not always, conservative past practice of defining this equivalent diameter at one-third of the length from the small end. To illustrate the need for different equivalent diameters for the same tapered member, visualize a tapered member in a flag pole configuration with the large end fixed, then the same member in a cantilevered pile arrangement with the small end fixed. As expected, the flag pole member has a significantly larger buckling load. This results because the member portion subject to the largest moments in the buckled configuration is at the larger end for the flag pole, while this most critical region is adjacent to the smaller end for the cantilever pile. Thus, the flag pole (case 1 in Sec. 4.3.4) has a larger equivalent prismatic diameter than does the cantilever pile (case 2). The provisions for columns with more restraint are approximate. The requirement that $D_1/D_2$ exceed 1/3 is a judgmental limit of applicability for the simplified linearized equations.

As for all columns, an effective length factor should be applied to the length of a tapered column if the column has other than the basic pinned end, no sidesway conditions. When sidesway is prevented, use of $K_e = 1.00$ is acceptable and is conservative. The compression resistance of tapered columns is limited (see also Sec. 4.3.3) to more than the $\lambda\phi_c F_c$ times the area of the small end.

As with prismatic columns, the buckling resistance of tapered columns should be checked about both principal axes if any doubt exists as to which buckling direction controls. Note that for singly tapered columns, a somewhat different equivalent prismatic member section location is given for the two principal axes. Thus, two equivalent sections must be located and used if the controlling direction cannot be determined by inspection.

## C4.4 Resistance of Spaced, Built-up, and Composite Columns

**C4.4.1 Spaced columns.** The design provisions for spaced columns have been placed in an appendix, App. A1, because these provisions are quite lengthy relative to the limited use of such members in new construction.

**C4.4.2 Built-up columns.** Unless the components of a built-up column are rigidly attached, the interlayer slip between the components will reduce the assembly's stiffness because of the resulting incomplete composite action. Thus, the resistance of a built-up column in compression can vary between that of a rigidly connected column if the connector stiffness is very high and/or the column is quite short to a smaller value equal to the sum of the individual component strengths, each component acting independently. This lower limit is approached if the column is of intermediate to long length and the fastener stiffness is very low. The slenderness values for which the connector stiffness has the largest effect on column resistance is in the intermediate $K\ell/r$ range of about 70 to 100, as the connector forces needed to prevent individual component buckling are highest in that range. The alternate method consisting of summing the individual component resistances is increasingly conservative as the connection between adjacent layers becomes stiffer.

Some design guidance for built-up columns is contained in the 1955 "Wood Handbook" (Forest Products Laboratory, 1955). This information is not contained in either the 1974 or 1987 versions of this book. More recently, Malhotra and Van Dyer (1977) have presented information on built-up columns. Jumaat (1991) presents an analysis method for both spaced and built-up members. The Canadian provisions for Limit States Design in Wood (CSA, 1989) allow 60%, 75%, and 80% of the fully composite compressive resistance to be used for nailed, bolted, and split-ring connected built-up members, respectively, if specified minimum connectors are provided. Otherwise, the resistances of the individual components acting independently are to be summed.

**C4.4.3 Composite columns.** Transformed section concepts must be used if the well-connected composite column includes components of different material stiffnesses, as both the critical buckling expression and the column stress equations assume a homogeneous member with one effective material stiffness. Analyses of partially connected composite columns must consider both differences in material component stiffnesses and the connector stiffness characteristics.

## C4.5 Resistance in Bearing

The end bearing strength is different than the compressive strength because bearing strength is not reduced by knots in the grade. Thus, end bearing strength is a function of specific gravity while the compression strength is influenced by knots as permitted for the grade. The side bearing strength is equal to the perpendicular to grain compression strength.

For purposes of this standard the time-effect factor is, contrary to recent allowable stress design practice, applied to the compression perpendicular to grain strength. Thus, the design practice returns to that in the National Design Specifications prior to the 1982 Edition. The 1982 Edition was the first in which the load duration factor was first excluded from "compression perpendicular to grain design values based on a deformation limit." Because this change is significant, additional background is provided below.

The 1982 change from a compression perpendicular to grain design value defined at the proportional limit to that at a deformation of 0.04 in. (1.02 mm) (using the standard testing specimen described in ASTM D143 (ASTM, 1994)) resulted in a significant increase in allowable stresses. According to ASTM D2555 section X1.9 (ASTM, 1988), the 0.04 in. deformation values used in the 1982 and 1986 NDS (AF&PA, 1982 and 1986) are about 1.6 times the proportional limit values. Although ASTM D2555 states that duration of load modifications apply to compression perpendicular to grain values only if not deformation based, the short-term laboratory test results for both proportional limit and deformation-based values are modified by ASTM D2555 to 10 year loading design values using identical adjustment factors, which by ASTM D2555 Sec. 6.2 include adjustment for normal duration of load and a factor of safety. Thus, time-effects have in fact been considered in determining deformation-based compression perpendicular to grain normal duration (ten year) design values.

The reasoning behind not applying the traditional duration of load factor to the deformation-based compression perpendicular values in the 1982 and 1986 NDS supplements often has been that since the load duration factor does not apply to the modulus of elasticity, E, it should not apply to other stiffness or deformation defined quantities. A more pragmatic reasoning may be that not applying the load duration factor (which exceeds unity for load durations less than ten years) effectively offsets some of the large jump in normal duration design values which occurred in 1982. Time-effects certainly affect the long-term strengths and deformations defined at a deformation of 0.04 in. (1.02 mm) at least as much as for the lower stresses defined based on a proportional limit.

In this LRFD standard, material strengths are based on short-term values. Application of the time-effect factor, $\lambda$, which is less than unity except for impact, provides an appropriate part of the load duration reduction fully utilized in ASTM D2555 to obtain the 10 year normal duration value. From a practical sense, the use of the same time-effect factors for both end bearing and side bearing also simplifies the design procedure for bearing, including for bearing at an angle to the grain. Thus, for the designer, the application of the time-effect factor to this property actually simplifies the design process. It should also be noted that, for most design cases, the calibration process produces virtually identical bearing area requirements in LRFD as previously used in ASD.

**C4.5.1 Resistance in end bearing.** Because strength in end grain bearing is larger than in parallel to grain compression, end bearing can be critical only when the load on a column is introduced on a bearing plate or surface which is smaller than the column net section area. The bearing strength is based on the net area of bearing on the end of the member.

When two wood members meet in an end bearing compression splice, it is important that the member ends be cut square so that the bearing stresses are uniformly distributed over the member ends. Such a square cut helps maintain the structural geometry and minimizes the unintentional load eccentricities and/or alignment problems that are introduced as a result of incomplete bearing over the member surface. Established practice, continued by the provisions of this section, has been to require a metal bearing plate between highly loaded members in end bearing compression splices.

The member flexural strength is interrupted by an end bearing compression splice and the member ends at the splice become effectively hinged for purposes of determining column behavior. To avoid translation and subsequent buckling behavior of such an end bearing connection, it is important that the members be laterally supported in both directions at or very close to the splice.

**C4.5.2 Resistance in side bearing.** Side bearing typically occurs at the supports and at concentrated loading points of flexural members. While the bearing stress is actually largest at the edge of a simple support (because of the beam's deflected shape being sloped at the support) conventional practice is to assume this bearing stress to be uniformly distributed.

Note that the time effect factor applies to the compression perpendicular to grain value, as was discussed previously.

The increase in side bearing for a bearing lengths, $L_p$, of less than 6 in. (152 mm) is the same as in the 1991 NDS (AF&PA, 1991), but is expressed in equation rather than tabular form. The minimum distance of the entire bearing area away from the member end has been clarified. When side bearing is through a washer, the washer diameter may be used as the $L_p$ value.

**C4.5.3 Bearing at an angle to grain.** Hankinson's formula is used as a practical and adequate way to provide a transition between side bearing and end bearing conditions. When bearing at an angle to grain results in a compressive force with a component directed toward the end of the member, the adequacy of the shear strength of the surface from the root of the notch to the member end should be investigated to determine if the strength is adequate to prevent a sliding block shear failure along this parallel to grain path (see Fig. C4.5-1).

A provision allowing bearing within 10 degrees of the perpendicular to the member direction has been added to allow the provisions of Sec. 4.5.2, including the increase for short bearing lengths, to be used for sloped-to-drain roofs and other slightly inclined members. Equation 4.5-4 gives a value little different from $A_gF_{c\perp}$ when $\theta_b$ = 80 degrees (i.e., when the load is 10 degrees from a direction perpendicular to the member). The limit of 10 degrees is based largely on judgment.

## C4.6 Radial Compression in Curved Members

Radial compression arises in curved members when the applied moment acts to close the member (i.e., decrease the radius of curvature and cause

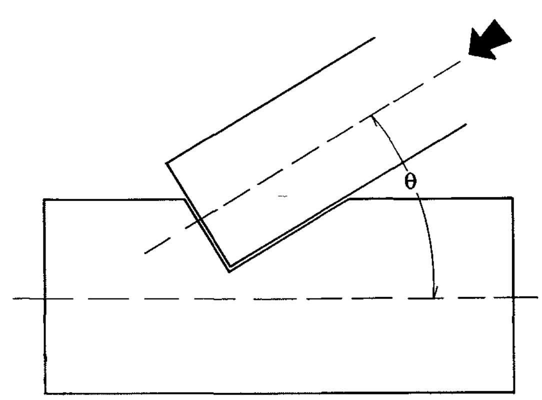



**Figure C4.5-1 Block shear conditions at a bearing notch near member end.**

flexural compression on the inside of the member). Although not often critical, this stress should be checked to determine that it does not exceed the adjusted perpendicular to grain compressive strength, $F_{c\perp}'$, times the applicable time-effect factor, $\lambda$, and the resistance factor for compression and bearing, $\phi_c$.

### References Commentary Chapter 4


American Forest & Paper Association. 1991. *National Design Specification for Wood Construction. AF&PA.* Washington, DC.

American Institute of Steel Construction. 1994. *Load and Resistance Factor Design Specification for Structural Steel Buildings, Second Edition.* Chicago, IL.

American Society for Testing and Materials. 1994. *Standard Method of Testing Small Clear Specimens of Timber, ASTM D143-94.* ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1993. *Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber. ASTM D245-93.* Philadelphia, PA.

American Society for Testing and Materials. 1988. *Establishing Clear Wood Strength Values. ASTM D2555-88.* Philadelphia, PA.

Canadian Standards Association. 1989. *Engineering Design in Wood (Limit States Design).* CAN3-086.1-M89. Canadian Standards Association, Rexdale (Toronto), Ontario.

Criswell, M.E. 1991. *New Design Equations for Tapered Columns.* Wood Design Focus, Vol. 2, No. 3, Fall 1991.

Forest Products Laboratory. 1955. *Wood Handbook–Basic Information on Wood as a Material of Construction with Data on its Use in Design.* USDA, Forest Products Laboratory, Madison, WI.

Galambos, T.V., editor. 1988. *Guide to Stability Design Criteria for Metal Structures, Fourth Edition.* Structural Stability Research Council, John Wiley & Sons.

Jumaat, M.K. 1991. *Analysis of Built-up Timber Columns Using Matrix Progression Methods.* Journal of Structural Engineering, ASCE, Vol. 117, No. 7, July 1991, pp. 1911–1928.

Malhotra, S.K., and D.B. Van Dyer. 1977. *Rational Approach to the Design of Built-Up Timber Columns.* Wood Science, Vol. 9, No. 4.

Newlin, J.A., and J.M. Gahagan. 1930. *Tests of Large Timber Columns and Presentation of the Forest Products Laboratory Column Formula.* Technical Bulletin No. 167, USDA, Forest Products Laboratory.

Ylinen, A. 1956. *A Method of Determining the Buckling Stress and the Required Cross-Sectional Area for Centrally Loaded Straight Columns in Elastic and Inelastic Range.* Publications of the International Association for Bridge and Structural Engineering, IABSE, Volume 16, Zurich, Switzerland, pp. 529–550.

Zahn, J.J. 1986. *Design of Wood Members Under Combined Load.* Journal of Structural Engineering, ASCE, Vol. 112, No. 9, September 1986, pp. 2109–2126.

Zahn, J.J. 1990. *Interaction of Crushing and Buckling in Wood Columns and Beams.* U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI.

Zahn, J.J. 1992. *Re-examination of Ylinen and Other Column Equations.* J. Str. Engr., ASCE 118(10):2716–2728. New York, NY.


# COMMENTARY

# Chapter 5 Flexural Members, Bending, and Shear

### C5.1 General

**C5.1.1 Scope.** The provisions of Chap. 5 address the strength limit states of members loaded in flexure. Both flexural bending and flexural shear are included. Torsion is also addressed in this chapter. Serviceability limit states are not considered, although these conditions, most notably deflections and the associated dynamic characteristics, often control the practical design of flexural members. The designer is referred to Chap. 10 and its commentary for guidance on serviceability considerations for flexural members.

Ponding is also included briefly in this chapter, as ponding is a condition that can affect the safety of roofs and roof flexural members. Specific provisions to prevent unsatisfactory performance due to ponding on flat or nearly flat roofs are contained in App. A3.

When flexure is combined with axial loads, either tension or compression, provisions of this chapter are used to provide the bending-related quantities needed for the interaction equations of Chap. 6.

**C5.1.2 Member design.** As with Chap. 3 and 4, the adjusted resistance for flexural members is found by multiplying the reference resistance by the time-effect factor, $\lambda$, and the appropriate resistance factor. This format requires the time-effect factor and resistance factors to additionally appear in some stability-related terms so that these factors are correctly considered in the member design strength quantities.

Proper design must consider localized resistance and stability as well as overall resistance and stability. This is particularly critical under concentrated loads and for members fabricated into efficient structural shapes (I-shapes, etc).

**C5.1.3 Design span.** The definition of member span as the clear span plus one-half the required bearing length for each support reflects usual practice and a recognition that the span length for determining moments should be slightly larger than the clear span. This definition of span somewhat complicates the flexural member design and analysis, as the required bearing length, and thus the effective member span needed to design the member, are both functions of the loading magnitude on the flexural member. Thus, it is common practice to perform preliminary designs on the basis of centerlines of bearing (a slightly conservative calculation), only rechecking the minimum design span when required by the design.

**C5.1.4 Notching of flexural members.** As discussed in Chap. 3 for tension members, use of notches in flexural members, especially on the tension side, is discouraged because of the susceptibility of the member to along-the-grain cracking starting at the inside corners of the notch. Once initiated, such a crack may extend into the member in such a way that the remaining beam section above the notch is reduced.

The effect of a notch can be reduced by tapering the notch or by rounding the inside corners of the notch. Because of the orthotropic nature of wood, with its much larger stiffness and strength in the longitudinal direction which generally aligns with the member axis, a much flatter taper is necessary to provide a smooth flow of stresses without local overstress in shear or perpendicular to grain stresses than for isotropic materials.

The restrictions on notches contained in the second paragraph of Sec. 5.1.4 are those in the 1991 NDS (AF&PA, 1991) and the Timber Construction Manual (AITC, 1994) with an added sentence prohibiting notches in the large negative moment regions of interior and cantilever supports.

The more restrictive provisions for notches on the tension side in areas of significant moment than for compression side notches reflects the serious effects of tension side notches. For a simply-supported, uniformly loaded member, the bending moment reaches one-half the maximum value at 15% of the span length from the supports. Tension notches should not be used at any location where the bending stress exceeds 50% of the maximum member moment. Thus, notches on the tension face should generally not be used at intermediate supports of continuous beams and at supports of cantilevers.

**C5.1.5 Member orientation and support conditions.** Solid sawn members are often graded and glued-laminated members are often designed and manufactured to meet certain span, support, and loading conditions. Glued-laminated beams and other manufactured members may be manufactured with a definite bottom or tension face. For this reason, custom-designed members must be installed in the proper locations in the building.

**C5.1.6 Partial composite action of parallel member assemblies.** The strengthening and especially the stiffening of a parallel member floor, roof, or wall system with sheathing connected with nails, screws, staples, or elastomeric glues can be quite significant, although these increases are less than would result with "rigid" glues and complete composite action. An incomplete composite or T-beam action results as the connectors are stiff enough to be able to transfer some axial forces into the sheathing, but flexible enough that appreciable interlayer slip may occur between the sheathing and primary flexural member. The sheathing also acts as a wide beam crossing the primary flexural members and is able to laterally distribute concentrated loads and to redistribute uniform loads among the joists when the joists are not of the same stiffness. These basic system behaviors of partial composite or T-beam action and crossing beam or two-way action are described in several references including Criswell (1981), Foschi (1984), Polensek (1976), and Vanderbilt et al. (1974).

The magnitude of the system benefits because of partial composite action and load sharing varies considerably with the relative flexural member to sheathing stiffnesses, fastener types, and spacings, conditions at the sheathing joints, and other parameters. The extent of load redistribution in uniformly loaded parallel member systems depends on the variability of the stiffness of these parallel flexural members. Methods to properly address the systems benefits of sheathed parallel member systems include mathematical models implemented by computer programs (Thompson et al., 1975; Foschi, 1984). Other simplified models more limited in scope also exist. Section 5.3 contains provisions suitable for routine design which approximately and conservatively recognize systems benefits; these provisions replace the 15% increase for repetitive member use contained in other specifications (AF&PA, 1991).

**C5.1.7 Moment resistance of square and circular prismatic members.** Square members bent about the diagonal and circular members have long been credited with a flexural resistance above that given by the equations of Sec. 5.2, conventionally through the use of a form factor (AF&PA, 1991). There is limited evidence that this increase does indeed occur. Several explanations for this increase have been offered, all centered around the observation that the width of the most highly stressed portions of these cross sections are narrow relative to the average member width. The usual factors of 1.414 for square sections bend about the diagonal and 1.180 for circular members originated from tests of small clear Sitka spruce specimens performed in the early 1920's as a part of a program to define the behavior of wood for use in aircraft (Newlin & Trayer, 1924) and were later reported by Markwardt (1938).

The increases contained in Sec. 5.1.7 have been rounded slightly below the traditional values to reflect the lack of recent test results on typical wood members.

Any form factor effect is included in the basic material strength values for poles and pile, thus the provisions for circular prismatic members do not apply to poles and piles.

**C5.1.8 Moment resistance of box beams and I-beams.** Two design questions that arise for box beams, I-beams, and similar configurations are (a) when should the flanges be evaluated as tension and compression chord members (with strength based on $F_t'$ and $F_c'$ material strengths) rather than as part of a flexural member (with $F_b'$ the relevant material strength), and (b) when designed as a flexural member, is there any shape or form effect?

Work by Newlin and Trayer (1924), including tests of small clear spruce I-beams and T-beams, formed from single pieces of wood and small box beams with full depth webs, is a useful reference on this topic. In addition, the following form factor was given in the 1985 Uniform Building Code (ICBO, 1985) for lumber I-beams and box beams:

$$C_f = 0.81\left[1 + \left(\frac{d^2 + 143}{d^2 + 88} - 1\right)C_g\right] \qquad \text{(C5.1-1)}$$

where:

$C_g$ = support factor = $p^2(6 - 8p + 3p^2(1 - q)) + q$,
$p$ = ratio of depth of compression flange to full depth of beam,
$q$ = ratio of thickness of web or webs to the full width of beam.

When using this equation, the form factor was assumed to include any size effect factor.

**C5.1.9 Moment resistance of nonprismatic members.** The maximum bending stress in a nonprismatic member generally does not occur at the location of the maximum bending moment. This results because the section modulus of a nonprismatic member changes along the length and thus the maximum cross section flexural stress is not proportional to the bending moment.

Closed form solutions for the maximum bending stress and its location along the member are possible for some simple geometries and loadings. For example, the maximum flexural stress for a uniformly tapered cantilever pole subjected to a single transverse load is at a section where the diameter is 1.50 times the diameter at the loaded point (or at the base of the cantilever if the diameter there is less than 1.50 times the pole diameter at the load). The maximum flexural stress location for a uniformly-tapered constant-width solid rectangular cantilever beam loaded with a concentrated load at its tip is where the member depth is twice that at the load point (or at the cantilever's fixed end if the taper is not enough to double the member depth). For general loadings on tapered beams, several sections may need to be investigated to closely locate the critical section.

**C5.1.10 Tapering of members.** For reasons similar to those discussed in Sec. C5.1.4 for notches, members generally should not be tapered by cutting the tension side of a member. This practice of tension side tapering by cutting has traditionally not been allowed by the glued-laminated lumber industry, and is to be discouraged for other lumber products, as failure can easily initiate on the cut face.

Glued-laminated members are typically made by using the laminations of the higher stiffness and high expected strength near the top and bottom of the member cross section. Thus, cutting a taper on the compression side (which is permitted) will usually remove outer lamination material of higher quality than that which will be exposed at the taper cut, unless the member has been specifically designed and manufactured for such a geometry. When tapering results in lower grades of laminations being at the member surface, this must be considered by a reduced material bending strength.

**C5.1.11 Stress interaction at a cut face of a member.** At a beam compression face formed by a taper cut, the flexural stress is required by statics to be parallel to the cut surface rather than parallel to the grain. When this flexural stress is expressed in components parallel and perpendicular to the lamination direction (and thus the grain direction), compressive stresses accompanied by shear occurs in both the parallel to grain and perpendicular to grain directions.

**C5.1.12 Moment resistance of composite members.** The concerns of layer connectivity, strain compatibility, and ductility discussed in Sec. C3.4 and C4.4 also apply for flexural members.

**C 5.1.13 Moment resistance of built-up members.** Flexural members such as headers over large doors (including garage doors) and windows, as well as supporting girders, often consist of several adjacent joists or other beams. If the loading on this built-up member is applied equally to all plies, the load sharing increase is applicable.

When loads are applied to one side of the beam and connection details do not limit the twisting of the supporting beam, the torsional moments can be sizeable. The torsional constant, J, for a beam consisting of n adjacent members each b wide by d in depth is more nearly n times the J value for each component member than the much larger J value (because of the narrow dimension being cubed) for a solid member of width, b, and depth, d. The value of the effective torsional constant is between these extremes and depends on the stiffness of the connectors between the individual components.

## C5.2 Conditions of Lateral Support

*C5.2.1.1 Consideration of lateral support conditions.* The resistance of flexural members bent about the strong axis and not fully braced by a floor or other sheathing or decking system is reduced by the lateral torsional buckling behavior as a function of the distance between lateral bracing of the compression side of the member increases and the member depth-to-width ratio. The basic phenomena of lateral torsional buckling is contained in texts on structural stability and buckling, with a good coverage of the topic available in Chap. 5 of SSRC Guide (Galambos, 1988).

The consideration of lateral stability of members during construction is especially critical for trusses, slender I-beams, and similar members with a high ratio of strong axis to weak axis strength and stiffness. These members are typically braced and sheathed before service loads are applied. When such members are subject to construction loads, adequate bracing must be in place to prevent failures during construction. Members must be braced to supports capable of resisting the possible member motions; the bracing of slender members only to each other will not necessarily prevent a general failure in which all members move together laterally. Bracing of wood trusses, including bracing during construction, is discussed in TPI (1991).

*C5.2.1.2 General requirements for lateral bracing.* The requirement that members not fully braced in the lateral direction be held in position both laterally and in rotation at the supports represents good design practice and is consistent with previous specifications (AF&PA, 1991), although the need for rotational support is herein more explicitly stated. Recent tests of glued-laminated members supported on their bottom surface by hangers (Peterson, 1991) demonstrated that this support configuration is effective in preventing end rotation unless the member has a quite high depth-to-width ratio. Design guidance is expected from this ongoing research.

Flooring, purlins, and similar members can usually provide lateral support to larger bending members in positive moment regions. Intermediate (i.e., other than at supports, where rotational restraint is also desired) lateral bracing needs to support at least the compressive face of the member. In areas of negative moment, sheathing or flooring attached to the top face is not fully effective as bracing, as they are attached to the tension face, and diagonal "kickers" or similar bracing supporting the lower face or lower proportion of the beam are necessary. Bracing of parallel member systems must either consist of sheathing elements, as exists in most floor and roof systems, or be attached to supports precluding all the braced members from moving laterally together in the same direction.

For solid sawn members, the rules of thumb in Sec. 5.2 have often been used (AF&PA, 1991) for bracing requirements based on the ratio of nominal depth to width.

*C5.2.1.3 Effective laterally unsupported length.* Wood beams without full lateral support may be analyzed and designed by either of two approaches. The general mechanics equations for laterally unsupported beams requiring section flexural and torsional properties (Chap. 5, Galambos, 1988) may be used. An alternate procedure long used for solid rectangular wood beams is based on the definition of an effective unbraced length, $\ell_e$, which can be used in a buckling load expression containing only flexural quantities. In the derivation of the expressions for this unbraced length (Hooley and Madsen, 1964), torsional stiffness has been considered and a ratio of the material stiffness in shear to the longitudinal modulus of elasticity, G/E, of 0.064 was assumed. Since this second method is limited to rectangular shapes, it cannot be used for trusses, I-shaped sections, and other nonrectangular shapes.

The maximum beam slenderness factor value of 50 is the same as in the 1991 NDS (AF&PA, 1991) and thus reflects the generally accepted design limit. This slenderness can be shown to function very similarly to the column slenderness measures $\ell$/d or $\ell$/r.

**C5.2.2 Moment resistance of laterally supported beams.** The basic bending strength equation uses the usual linearly elastic flexural expression. The bending strength equations and the $F_{bx}'$ and $F_{by}'$ material strengths they contain apply for bending producing flexural stresses in the direction of the member length. Cross grain bending, as occurs when a roof diaphragm pulls on the top of a ledger beam bolted to a wall, is to be avoided, as the cross grain bending strength of wood is low and quite unpredictable. Such cross grain bending results in tension perpendicular to grain and can result in a member splitting over at least part of its length.

**C5.2.3 Moment resistance for beams without full lateral support.**

*C5.2.3.1 Strength and stiffness.* The initial provisions of this section parallel those for columns (Sec. C4.3.1). The exclusion of the volume factor, $C_V$, for glued laminated members when computing bending resistance controlled by lateral torsional buckling follows the philosophy that this factor, which is less than unity for large beams, accounts for the effects of member size in reducing the tensile face bending capacity. As the resistances computed for lateral torsional buckling are controlled mainly by the behavior of the compression face, not the tension face, the volume effect factor and similar factors are typically not applied.

Note that, due to the nonlinearities in this equation, the applicable time-effect factor, $\lambda$, from Table 1.4-2 should be used as a multiplier of the M′ values rather than as a multiplier of the adjusted bending strength, $F_{bx}'$.

*C5.2.3.2 Prismatic beams.* As for columns, the bending resistance of a beam without full lateral support is written as a beam stability-related modification factor, $C_L$, times the bending resistance of the flexural member if fully braced. The equation for $C_L$ is similar to that found in the 1991 NDS (AF&PA, 1991).

Section C4.3.2 explains the presence of the resistance factors and the time-effect factor in the $\alpha$ term which also apply to the member bending strength equations and its equivalent parameter $\alpha_b$.

Two equations are given for the elastic buckling moment for the laterally unbraced beam, with proper use governed by the designer's previous choice of using the general provisions for lateral torsional buckling or the alternate effective length. For solid rectangular members, for which both equations apply, which equation gives the larger $M_e$ value depends upon the ratio of member width to depth, b/d, the value of $c_b$, and the ratio of effective to actual unbraced length, $\ell_e/\ell_u$, used in the specialized Eq. 5.2.7. Since Eq. 5.2.7 more completely models the effects of moment diagram shape for the specific common cases included in Sec. 5.2.3 and it gives the higher $M_e$ values in a majority of problems involving rectangular sections, consideration of only Eq. 5.2.7 for rectangular beams is appropriate. Seldom will significantly larger bending resistance result from the use of the more general equation, Eq. 5.2-8, for rectangular beams.

Equation 5.2-7 is similar to that for the allowable flexural stress of very slender bending members contained in the NDS (AF&PA, 1991) and, like the NDS expression, follows from the development reported by Hooley and Madsen (1964). In both Eq. 5.2-7 and the NDS expression, the effective unbraced length has been increased 15% to recognize that lateral bracing to prevent rotation is in practice not perfect, just as end fixity in columns seldom reaches that for an idealized column.

The moment shape effect, along with the torsional properties of the rectangular member, are included in the equivalent unbraced length expressions. Thus, the alternate method elastic buckling moment expression in Eq. 5.2-7 does not include the $C_b$ factor. Equation 5.2-7 can be written in terms of a critical stress times the strong axis section modulus, $S_x$:

$$M_e = \frac{1.20E'_{y05}}{\ell_e d / b^2}(S_x) . \tag{C5.2-1}$$

This form more clearly shows that the elastic buckling moment depends upon the square of the beam slenderness factor $K_s = (\ell_e d/b^2)^{1/2}$ and is the same, except for the coefficient and the definition of E (mean or 5% lower exclusion limit), as the NDS expression.

The theoretical background for Eq. 5.2-8 equation for the elastic buckling moment is given in Chap. 5 of Galambos (1988). The moment shape factor has long been used in structural steel design (AISC, 1994) to recognize the shape of the moment diagram, and thus the compression force in the beam which controls the lateral torsional motion, affects the elastic buckling moment. A beam with a uniform moment (ratio of end moments, $M_1/M_2 = 1.00$) is the most critical; this moment diagram shape gives $C_b = 1.00$. When the unbraced beam segment is loaded with equal but opposite end moments that bend the member into an S-shape reverse curvature, the compression region of either side of the beam is only one-half of the unbraced length and, with the moment diagram decreasing linearly from each end to a zero value at midspan, much of the beam side within this length is not highly stressed. These conditions reduce the tendency of the compression zone of the beam to buckle and move laterally, giving a resistance increase reflected in this case by the maximum $C_b$ value of 2.3.

The equation given for J, the torsion constant for St. Venant's torsion, is for the elastic behavior of a rectangular member and is the same equation as was used in the derivation of the equivalent unbraced length approach (Hooley & Madsen, 1964). For other shapes, J can be obtained from texts on advanced strength of materials.

Equation 5.2-8 can be simplified if G′ is taken as $E_{y05'}/16$, $(E_{y05}' G')^{1/2}$ is then $E_{y05}'/4$. Expressing J as $J = 4 I_y(1-0.63(b/d))$ results in $(JI_y)^{1/2} = 2I_y(1-0.63(b/d))^{1/2}$. The following variations of Eq. 5.2-8 then result:

$$M_e = \frac{\pi C_b E'_{y05} I_y}{2.30 \ell_u}\sqrt{1 - 0.63\frac{b}{d}} \tag{C5.2-2}$$

$$= \frac{1.366 C_b E_{yo5n} I_y}{\ell_u}\sqrt{1 - 0.63\frac{b}{d}} . \tag{C5.2-3}$$

The similarity and differences of the two buckling moment equations are more evident when Eq. C5.2-7 is compared with Eq. 5.2-8. By setting the $M_e$ values from these two equations equal to each other, it can be shown these two equations give the same results for rectangular beams if the ratio of effective to actual unbraced lengths is as follows:

$$\frac{\ell_e}{\ell_u} = \frac{1.757}{M_b\sqrt{1 - 0.63\frac{b}{d}}} \tag{C5-2.4}$$

For lower ratios of $\ell_e/\ell_u$, Eq. 5.2-7 gives a higher $M_e$ value than Eq. 5.2-8.

The lateral torsional buckling behavior is controlled by the compression portion of the beam, while the volume factors deal with the member strength as controlled by the tension side of the member. Thus, the equation for the nominal bending strength for members without full lateral bracing, Eq. 5.2-4, does not contain a volume factor. For large members with a volume factor, $C_V$, significantly less than one, and with the beam-stability related factor, $C_L$, not much less than one, Eq. 5.2-2 for the member in a laterally-supported configuration can give a member bending strength less than that given by Eq. 5.2-4. This possibility leads to the provision that the smaller of these two member bending strengths is to be used.

## C5.3 Moment Resistance of Assemblies

As was noted previously, sheathed and other assemblies with parallel flexural members can benefit from the systems behaviors of partially composite (T-beam) action along the members and load sharing action arising from the sheathing crossing the parallel framing members. The magnitude of these two effects depends on: (a) the stiffness of the connectors attaching sheathing to framing, (b) the relative stiffness of framing to sheathing, (c) framing spacing, (d) sheathing joints, (e) material strength and stiffness variabilities, and (f) loading pattern. Connector stiffness depends upon type, size, and spacing of connectors. Composite action resulting from sheathing on one side only (as is typical for floors and roofs) increases stiffness more than it reduces the maximum flexural stresses (Criswell, 1981).

The provisions of Sec. 5.3 contain simple and generally conservative expressions for systems benefits for uniformly load assemblies. Guidance on the lateral distribution of concentrated loads is available from App. D of the 1991 NDS (AF&PA, 1991), and from studies based on analytical models (Sazinski and Vanderbilt, 1979).

**C5.3.2 Adjustment factors for uniformly-load assemblies.** Separate adjustment factors are given for assembly stiffness and assembly strength. The composite action factor, $C_E$, used as a multiplier of the member flexural stiffness, may be used in investigating serviceability requirements.

**C5.3.3 Composite action factor.** The composite action factor, $C_E$, recognizes primarily the benefits of the partial composite action by which the flexural member is stiffened by the attached sheathing. Thus, the $C_E$ factor depends upon the stiffness of the connectors attaching the sheathing to the flexural members.

The conditions required to use the $C_E$ factor follow past practice and its values have been chosen to assure the increases given by the $C_E$ factor are reliably achieved. Sheathing gaps/joints interrupt the partial composite action. The values provided are applicable with the 4-ft spacing of sheathing joints along the flexural members, the spacing usually provided with the use of 4-ft by 8-ft panel products. Boards, decking, and other non-panel products introduce enough discontinuities to prevent significant partial composite action from developing. In contrast, the $C_E$ values are quite conservative for sheathing systems with few gaps or with glued sheathing joints.

As an alternative to the approach discussed above, it is possible to compute the composite action factor using principles of engineering mechanics. In its simplest form, a composite action calculation would first assume that the connection between the various bending members in an assembly is adequate for the members to act as one composite unit and that the connection between the various members of the assembly is capable of distributing the horizontal shear flow due to the vertical load to each of the assembly members. This shear flow acts in the plane between the assembly members (i.e., the connection interface in a sheathed assembly). Using this approach, the magnitude of the shear flow would be computed as $f_v = VQ/It$, where $f_v$ is the horizontal shear flow per unit length of the interface, V is the vertical applied shear, Q is the statical moment of the area of the assembly, I is the moment of inertia of the assembly, and t is the member thickness.

Using this approach, if mechanical fasteners of sufficient size and spacing are provided in the assembly to transfer the computed horizontal shear flow, the section properties of the composite section can also be computed. One challenge using this procedure is quantifying the stiffness of the interface connection. For example, mechanical fasteners (i.e., nails) are fairly rigid during initial loading and become less stiff at higher loads. Adhesives, on the other hand, range from fully rigid to highly elastomeric.

**C5.3.4 Load sharing factor.** The load sharing factor, $C_r$, may be used as a multiplier of the single member adjusted flexural strength, $F_b'$. The $C_r$ factor primarily recognizes the benefits of the sheathing in laterally redistributing a part of the tributary area uniform load away from the members with lower stiffness values. Thus, the value of the $C_r$ factor depends upon the coefficient of variation of the member stiffness. It also depends upon the positive correlation between flexural strength and flexural stiffness.

## C5.4 Resistance of Members in Shear

Controlling shear stresses in solid sawn, glued-laminated, laminated veneer lumber, and other wood products with the wood material oriented in one direction, often called horizontal shear, is generally oriented along the wood's longitudinal-tangential and longitudinal-radial planes. Shear stresses in the tangential and radial planes, the so-called rolling shear, does not often control design in solid-sawn members and is addressed in Chap. 8 when panel products are involved. Shear in a plane perpendicular to the grain is not a practical concern for wood. Splitting or checking at the ends of flexural members from seasoning and other causes reduces shear strength in the end regions of many solid sawn members and complicates the behavior in shear at such locations. The concepts of fracture mechanics have been used with generally good results to study the strength in shear of beams with end splits (Murphy, 1979). A recent state-of-the-art paper on the shear design of wood beams has been prepared by Soltis & Gerhardt (1988).

**C5.4.1 Calculation of design shear force.** For typical wood beams, loads placed very close to the support are carried at least partially along a diagonal compression path to the support and the region of the beam above the support has significant compression perpendicular to grain stress. These are among the reasons for the usual practice, continued in this standard, for loads placed within a distance equal to the member depth from the face of the support to be excluded from the calculated design shear force the member must resist.

For other cases, such as when the member is supported by bolts or other connections supporting the beam at locations along the side of the beam, the favorable support conditions noted above are not present and the exclusion of loads very near the support is not considered prudent.

Design for shear at and near the supports of built-up and composite components, including manufactured sections with thin webs and/or with load-bearing connections, such as between web and chord, should follow the recommendations of the manufacturer.

**C5.4.2 Flexural shear resistance.** The linearly elastic flexural shear equation from basic mechanics is used as the basis for the member shear resistance expression. This equation is given first in its general form (Eq. 5.4-1). Note that the largest shearing stress occurs where the ratio b/Q is the smallest. This is at the neutral axis for rectangular, round, and I-shaped sections, but for unusual section shapes, can be elsewhere. Equation 5.4-2 restates the flexural shear strength equation for a rectangular shape.

Equation 5.4-1 is appropriate for the investigation of shear adequacy at web-flange junctions and other planes of potential weakness if Q, the first moment of the beam cross section area outside the section location of interest taken about the neutral axis, is properly determined. Equation 5.4-1 is derived from the flexural stress distribution given by the elastic Mc/I beam equation and thus is subject to the same limitations as is the elastic bending stress equation. One of these limitations is that Eq. 5.4-1 applies only when all material in the cross section has the same material stiffness. Thus, for composite members with components of differing stiffness, a transformed section analysis must be performed.

The tabulated horizontal shear strength, $F_v$, for solid-sawn members considers the possible effect of end splits; this reduction is not used for glued-laminated members. Thus, it is appropriate to increase the material shear strength at locations well away from the ends of solid sawn members. Equation 5.4-3 provides a linear increase starting at locations 3d from the member end which double the shear strength at locations 6d and more from the end. This increase, which applies for most support locations of continuous and cantilever bending members, is consistent with the provisions of section 4.4.2 of the 1991 NDS (AF&PA, 1991).

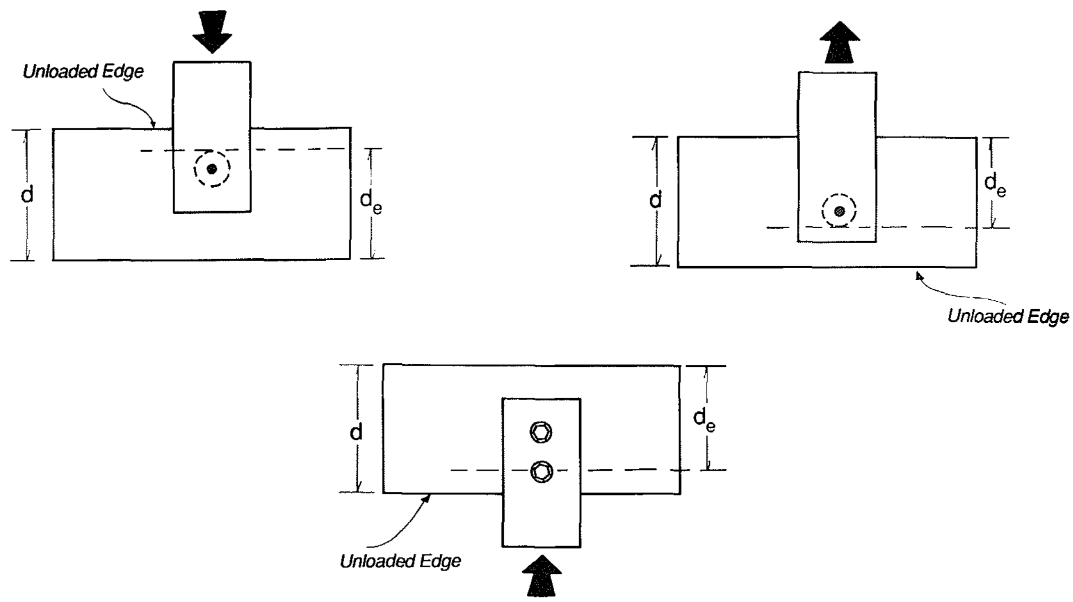
**C5.4.3 Shear resistance in the vicinity of notches.** Just as for tension and flexural bending, a notch introduces a stress concentration that reduces the member strength in flexural shear. In beams, such notches are most often placed on the lower member face at and close to supports. The $d_n/d$ term acts as a reduction factor that increases in its effect as the notch becomes deeper relative to the overall member depth. This term is also contained in the 1991 NDS (AF&PA, 1991). Provisions and limitations of Sec. 5.1.4 must also be met at notches.

A provision allowing this $d_n/d$ reduction term to be replaced by $1 - (d - d_n)\sin\theta/d$ has been added to both recognize the benefits of a gradual taper leading into the fully notched section and to encourage the use of such a taper. This modified expression results in less reduction when notch ends are tapered. For example, if $d_n = 0.8d$ for a notch with a 1 on 1 (45 degree) taper, this allows 0.86 to be used as the reduction term instead of 0.8. For a flatter 1 (vertical) on 4 (horizontal) notch, the term increases to 0.95.

Note that the increase in shear strength away from the member ends given by Eq. 5.4-3 is not contained in Eq. 5.4-4; thus, this increase does not apply in the vicinity of notches.

**C5.4.4 Shear resistance in the vicinity of connections.** Connections supporting flexural members or transferring appreciable transverse loads into flexural members can lead to local shear distress. At such locations, the portion of the member that is on the unloaded side (relative to the forces transferred by the connection) of the connection is ineffective and that portion of the member depth is excluded from the effective member depth, $d_e$, at the connection region. Figure C5.4-1 shows the effective member depth, $d_e$, for several connections.

Many connections that do not transfer large loads can be placed along the length of a beam. It is not reasonable to expect connections transferring small loads to affect overall member strength in shear. In order to clarify when the shear in the connection region needs to be checked, a phrase has been introduced which requires that Eq. 5.4-5, or Eq. 5.4-6 when applicable, needs to be satisfied only if the transverse load or support force intro-




Bolt or Lag Screw Connections

**Figure C5.4-1. Definition of effective member depth at a connection.**

duced by the connection is large relative to the member strength. The definition of the connection contribution to section shear force trigger the use of Eq. 5.4-5 is judgmental, as adequate experimental data are not available.

Note that the shear conditions at a connection are to be satisfied at the edge of the connection, not at a distance, d, away from this edge.

The increase in member horizontal shear strength permitted by Eq. 5.4-6 at locations at least 3d away from the ends (i.e., out of the area of possible end splits) is consistent with the provisions of Sec. 3.4.5 of the 1991 NDS (AF&PA, 1991), except that a smooth transition of this increased strength is provided over the 3d to 6d distances away from member ends, similar to what is provided for members away from notches and connections by Eq. 5.4-3, rather than as a jump at 5d from the member end. The $d_e/d$ factor is dropped at the 3d location rather than the 5d location of the 1991 NDS provisions. A smaller maximum increase (50% versus 100%) is allowed for connection areas than for flexural members in general. This difference can partly be explained by the effects of the very local introduction of forces on the member at the connector locations and the possible localized tension parallel to grain that can occur in these areas and could hasten the formation of splits near the individual connectors. The provision that the shear resistance in the connection vicinity not exceed that for the gross section can control when the gross section shear resistance is not permitted to increase at locations away from the member ends. This is the case with glued laminated members, but not for solid sawn members for which the use of Eq. 5.4-3 is permitted. When no increase is permitted for the gross section shear resistance away from the ends, the gross section shear will control when the $1 + (x - 3d)/6d$ factor of Eq. 5.4-6 exceeds the value $d/d_e$.

## C5.5 Resistance of Members in Torsion

Timber members are sometimes, but not often, designed to resist torsional loadings. In general, it is advised that the structure be designed to avoid wood member torsional stiffness and strength being the principal or only path for the carrying of load.

A wood member loaded to failure in torsion displays a longitudinal, parallel to grain splitting. A rectangular wood member has a fairly large torsion constant, J, but low relevant material strength and stiffness. Thus, wood is not very efficient as a torsion member and the design principles for its use as such have not been well defined. The contents of Sec. 5.5 are those in the Timber Construction Manual (AITC, 1994), including the torsional shear strength definition for glued laminated members. The torsional shear strength limit for solid sawn members is from the Wood Handbook (Forest Products Laboratory, 1987).

## C5.6 Curved or Pitched/Tapered Curved Glued Laminated Beams

**C5.6.1 Adjustment of flexural resistance for curvature.** An originally straight member bent into a curved shape retains most of the flexural stress caused from this bending process, although the process of material creep will reduce this stress somewhat. This stress, which can be thought of as a residual stress, is generally recognized as having some effect on the member flexural strength. The provisions of Sec. 5.8.1 are the same as those in the 1991 NDS (AF&PA, 1991). The minimum radii of curvatures of 100 times the lamination thickness, t, for hardwoods and southern pine and 125t for other softwoods have been found by experience to be reasonable. At these minimum radii, the curvature factor, $C_c$, values are 0.800 and 0.872 for hardwoods/southern pine and for other softwoods, respectively.

**C5.6.2 Radial tension and compression in curved members.** This section calls for the radial stresses that arise as a basic requirement of curved beam behavior (refer to the analysis of curved beams in mechanics of materials texts) to be considered in the design process. These stresses, particularly radial tension, can govern the design of a curved member.

The wood material strength in radial tension (which is effectively a tension perpendicular to grain that arises from the member shape) is very low, especially for some of the softwood species, including Douglas-fir. Accordingly, radial reinforcement is sometimes used for such members especially when the governing design loads are other than wind or earthquake. Design procedures for this reinforcement are contained in AITC (1994).

*C5.6.2.1 Curved members of constant cross section.* Equations for the determination of the design radial stress depend upon the beam geometry. The member strength equation in this section is based on an expression that closely approximates the maximum radial stress, which occurs near the mid-depth of a uniformly curved beam bent about one of the primary axes of its rectangular cross-section and responding in the linearly elastic range. The provisions of adjusted strength in radial stress, including the increased values for some species for wind and earthquake loadings, follow those of the 1991 NDS. Note that the text of Sec. 5.6.2.1 states that radial stress shall be adjusted for moisture and temperature only. This differs slightly from the 1991 NDS provision because the time effect in LRFD is applied afterwards, rather than being included within the calculation of the adjusted moment resistance.

*C5.6.2.2 Pitched and tapered glued laminated beams.* The analysis of pitched and tapered beams, geometries of which are possible with lamination techniques, is complicated by the wide variety of geometries which can be involved and the absence of any accurate, simple closed form equation that can be used for this analysis. Design procedures that have evolved for the consideration of radial stresses in such members are contained in App. A2.

## References Commentary Chapter 5

American Association of State Highway and Transportation Officials. 1983. *Standard Specifications for Highway Bridges.* 13th Edition, AASHTO, Washington, DC, p. 394.

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington DC.

American Institute of Steel Construction. 1994. *Load and Resistance Factor Design Specification for Structural Steel Buildings.* Second Edition. Chicago, IL.

American Institute of Timber Construction. 1994. *Timber Construction Manual.* Fourth Edition, Wiley-Interscience.

American Society for Testing and Materials. 1993. *Standard Specification for Establishing and Monitoring Structural Capacity of Prefabricated Wood I-Joists.* ASTM. Philadelphia, PA.

Criswell, M.E. 1981. *New Floor Design Procedures.* Proceedings of a Conference on Design and Performance of Light-Frame Structures, Denver, CO, September 1981, Proceeding 7317, Forest Products Research Society, Madison, WI, pp. 63–86.

Foschi, R.O. 1984. *Structural Analysis of Wood Floor Systems.* Journal of Structural Engineering, ASCE, Vol. 108, No. ST7, July 1984.

Galambos, T.V., editor. 1988. *Guide to Stability Design Criteria for Metal Structures, Fourth Edition.* Structural Stability Research Council, John Wiley & Sons.

Hooley, R.F., and B. Madsen. 1964. *Lateral Stability of Glued Laminated Beams.* Journal of the

Structural Division, ASCE, No. 90, ST3, June 1964, pp. 201–218.

Hooley, R.G. and R.H. Devall. 1972. *Lateral Buckling of Simply Supported Glued Laminated Beams.* Report to Laminated Timber Industry of Canada, Department of Civil Engineering, University of British Columbia, November 22, 1972.

International Conference of Building Officials. 1985. *Uniform Building Code, 1985 edition.* ICBO, Whittier, CA.

Markwardt, L.J. 1938. *Form Factors and Methods of Calculating the Strength of Wooden Beams.* Paper presented at Conferences of Timber and Concrete, Cambridge, MA, USDA Forest Products Laboratory, Madison, WI.

Murphy, J.F. 1979. *Strength of Wood Beams with End Splits.* Research Paper FPL 347, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

Murphy, J.F. 1986. *Strength and Stiffness Reduction of Large Notched Beams.* Journal of Structural Engineering, ASCE, Vol. 112, No. 9, September 1986, pp. 1989–2000.

Newlin, J.A., and G.W. Trayer. 1924. *The Influence of the Form of a Wooden Beam on Its Stiffness and Strength, Part II: Form Factors of Beams Subjected to Transverse Loading Only.* Technical Report 181, Report to National Advisory Committee on Aeronautics, Annual Report No. 9, pp. 375–393.

Peterson. 1991. Report on glued-laminated beam tests at Oregon State (in preparation).

Polensek, A. 1976. *Finite Element Analysis of Wood-Stud Walls.* Journal of the Structural Division, ASCE, Vol. 102, No. ST7, July 1976.

Sazinski, R.J. and M.D. Vanderbilt. 1979. *Behavior and Design of Wood Joist Floors.* Wood Science, Vol. 11, No. 4, pp. 209–220.

Soltis, L.A., and T.D. Gerhardt. 1988. *Shear Design of Wood Beams–State of the Art.* General Technical Report FPL-GTR-56, USDA Forest Service, Forest Products Laboratory, Madison, WI, 9 pp.

Thompson, E.G., J.R. Goodman, and M.D. Vanderbilt. 1975. *Finite Element Analysis of Layered Wood Systems.* Journal of the Structural Division, ASCE, Vol. 101, No. ST12, December 1975, pp. 2659–2672.

Truss Plate Institute. 1991. *Handling, Installing and Bracing Metal Plate Connected Wood Trusses (HIB-91).* Truss Plate Institute, Madison, WI.

USDA Forest Products Laboratory. 1965. *Deflection and Stresses of Tapered Wood Beams.* Research Paper FPL34, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

USDA Forest Products Laboratory. 1987. *Wood Handbook–Wood as an Engineering Material.* Agricultural Handbook 72, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

Vanderbilt, M.D., J.R. Goodman, and M.E. Criswell. 1974. *Service and Overload Behavior of Wood Joist Floor Systems.* Journal of the Structural Division, ASCE, Vol. 100, No. ST1, January 1974, pp. 11–29.

Zahn, J.J. 1988. *Combined Load Stability Criterion for Wood Beam-Columns.* Journal of Structural Engineering, ASCE, Vol. 114, No. 11, November 1988, pp. 2612–2628.

# COMMENTARY

# Chapter 6
# Members With Combined Bending and Axial Loads

## C6.1 General

**C6.1.1 Scope.** The provisions of this chapter apply when loadings result in two or more of the following: axial tension or compression, strong axis bending, and weak axis bending. Such loadings include axial tension or compression combined with bending about either or both of the member's principal axes. Skew bending (bending about both principal axes) without axial loading also is included. Other interaction of stresses can occur; for example, flexural shear and torsion interact, as do flexural shear and tension perpendicular to grain. However, these do not have nearly the design importance as does the case of members under combined bending and axial loads.

**C6.1.2 Member design.** The interaction equations of this chapter provide limits on how two or more of the conditions considered individually in

Chap. 3 through 5 may be combined. Thus, this chapter draws extensively on the member resistance equations contained in the previous three chapters. The resistance factors of these previous chapters also apply in Chap. 6.

Note that the adjusted member resistance values to be used from Chap. 3 through 5 do not contain the $\lambda$ and $\phi$ values unless these terms are involved in the consideration of stability effects for either columns (Sec. 4.3.2) or laterally unbraced beams (Sec. 5.2.3). The time-effect factor, $\lambda$, used in all terms of the interaction equations is to have the same value, namely that specified in Sec. 1.4.3 for the loading being considered. This is consistent with the LRFD philosophy of considering several load combinations, each with a different dominant load. This does differ from a design philosophy that allows or requires different time related factors on each term depending on the duration of load most dominant in producing the type of load (axial, bending) considered by the individual terms.

## C6.2 Resistance in Combined Bending and Axial Tension

The interaction equations of this chapter are expanded versions of the basic checking equation requiring the design member resistance, including the time-effect factor and the appropriate strength-related resistance factors as multipliers, to exceed the design values from the factored loads in the combination being considered:

$$\text{(Supply)} \qquad \lambda\phi R' > R_u \text{ (Demand)} \qquad \text{(C6.2-1)}$$

where:

$R'$ = adjusted member resistance,
$R_u$ = required member resistance.

By dividing both sides of this equation by $\lambda\phi R'$, Eq. C6.2-1 can be rewritten as:

$$R_u/\lambda\phi R' < 1 \qquad \text{(C6.2-2)}$$

Equation C6.2-2 is still valid if both sides are squared (or both raised to any power).

How two or more of the type of terms on the left side of Eq. C6.2-2 should be combined is a basic feature of the interaction diagram. An often used and generally conservative approach is to limit the sum of all such terms to less than or equal to 1. This linear interaction form is most appropriate when the stresses from two elastic analyses are additive, i.e., superposition is valid and both cases involve the same type and direction of stress. The first equation in Sec. 6.2-1 is an example of such an approach. A more permissive interaction equation results when any or all of the terms, each being less than one, are raised to a power greater than one before the ratios are summed. These forms are more appropriate when inelastic action occurs or the interaction is among different types of stresses. The stress interaction correction factor in Sec. 5.1.11 is derived from such a three-term interaction equation with each term containing a different stress and each being squared before the summation. Analyses, tests results, or a combination of methods, depending upon the interaction involved, can be used to define the appropriate interaction equation form (Zahn 1988).

With combined bending and axial tension, the axial tensile stress is additive to the flexural tension on one face and reduces the magnitude of the flexural compression on the other face. Because the member resistance can be controlled by the conditions at the compressive face when the beam is not fully laterally braced, a separate interaction equation for each of the two beam faces results. In general, for the member in combined tension and bending to be accepted as adequate, both the first equation and also the second equation (compression face) with the appropriate inclusion or exclusion of $T_u$ are to be satisfied at all locations along the beam. Usually, this can be done by checking the section where the moment is a maximum, or in cases of biaxial bending, the locations of the maximum moment for each direction.

The first tension plus bending interaction equation, Eq. 6.2-1, is a linear expression that effectively limits the maximum tension arising from the superposition of axial and flexural tension. Because the tensile flexural face is of concern, the strong axis member resistance is defined as the strong axis bending strength with the beam stability factor, $C_L$, equal to one. Thus, the $M_s'$ value in the denominator of the second term is to be computed from Eq. 5.2-2, regardless of the actual lateral support conditions. Equation 6.2-1 recognizes no benefit of the axial tension load in reducing member flexural deflections, and thus the flexural moments, by acting to straighten the member. This second order effect is generally small and the usual practice in structural design is to neglect it.

The second tension plus bending interaction equation for tension plus flexure about one or both axes, Eq. 6.2-2, recognizes that the tensile loading may not reduce the compressive flexural stress enough to preclude a lateral torsional failure mode governed by compression if the unbraced beam or beam segment is quite slender and the axial tension is quite small. Note that $M_x'$, which includes any lateral torsional buckling behavior, is to be used in Eq. 6.2-2, not the $M_s'$ value used in Eq. 6.2-1. This second equation will not control if the member is fully braced laterally and the member's section modulus for the top and bottom face are equal.

In Eq. 6.2-2, with the compression side of the beam controlling, the tension loading is beneficial as it allows a larger moment to be applied. If during the actual loading the structure is expected to experience, the tension load will not necessarily occur simultaneously with the maximum moment condition, the second equation must be examined with the beneficial $T_u$ value set equal to zero. The resulting equation is the same as Eq. 6.3-1 with an axial compression load of zero and all moment magnifiers equal to zero.

The first term of Eq. 6.2-2 can be obtained from the 1991 NDS (AF&PA, 1991) equation for the compressive face for flexure and axial tension, $(f_b - f_t)/F_b \leq 1$, by multiplying all terms by the strong axis section modulus, $S_x$, noting that $f_t = P/A$, changing to the LRFD notation for forces and moments, and introducing the $\phi_b$ and $\lambda$ factors so the first term correctly goes to $M_{ux}/\lambda\phi_b M_x'$ when the applied tension force, $T_u$, is zero. This procedure results in a $(S_x/A)$ multiplier of the $T_u$ term. For rectangular members, $S_x/A = d/6$.

For large $T_u$ values, the first term of Eq. 6.2-2 can become negative. Before $T_u$ is large enough for this to occur, Eq. 6.2-1 will have become the controlling equation. Eq. 6.2-2 controls for a larger range of $T_u$ values as the beam stability factor, $C_L$, decreases in value.

## C6.3 Member Resistance in Biaxial Bending and in Combined Bending and Axial Compression

Bending moment in members subjected to both bending and compression forces (often called beam-columns) is magnified due to the axial force acting through a lever arm equal to the member deflection. This additional moment is often described as a P-delta moment. In members with combined bending about one or both axes and in axial compression, the second order effects add to the demand and must be included in the interaction equation.

The general form of Eq. 6.3-1, including the squaring of the axial compression term, is derived from work by Zahn (Zahn, 1986 & 1988). As noted earlier, the squaring of this axial term results in a more permissive interaction criterion than for a linear combination of terms.

Portions of the moment terms which reflect the second order effects, and thus can be identified as moment magnifiers, have been separated out (Eqs. 6.3-4 through 6.3-7). This approach also allows the moment magnification to be made considering the moment diagram shape and the sidesway condition of the member being examined. The design specifications for structural steel (AISC, 1986) and for reinforced concrete (ACI, 1992) contain a similar treatment of the moment magnification. The notation used in Sec. 6.3 is closer to that of the ACI Code than to the AISC, although not identical to either.

Note that when Eq. 5.2-5 is used for the assessment of the strong axis moment resistance, $M_x'$, for use in the Eq. 6.3-1 interaction equation, the moment shape term, $C_b$, appearing in Eq. 5.2-5 for $M_x'$ is to be taken as unity. This is because a similar moment shape term is included in the moment magnifier equations.

Unlike the 1991 NDS (AF&PA, 1991), moments from all sources, including axial load eccentricity, are included together, rather than the moment from axial load eccentricity and that from transverse loads being kept separate.

In Eq. 6.3-1, the general interaction equation, the factored moments about the two principal axes are defined as also including second-order effects when axial compression is present. To indicate that these moments are to include magnification, to account for second-order effects, they are designated as $M_{mx}$ and $M_{my}$ rather than $M_{ux}$ and $M_{uy}$, with the u subscript used for only the first-order moments. The usual method for assessing their values is through Eqs. 6.3-2 and 6.3-3. In these equations, the first-order moments (moments calculated with the structure in its original position, rather than in the deflected position for which equilibrium must actually be met), $M_{ux}$ and $M_{uy}$, are divided into two categories. The motive for this separation is that different magnification factors need to be applied to each of these categories of moments. Thus:

$$M_{ux} = M_{bx} + M_{sx}, \qquad \text{(C6.3-1)}$$

$$M_{uy} = M_{by} + M_{sy} \qquad \text{(C6.3-2)}$$

The first category of first order moments, $M_{bx}$ and $M_{by}$, are the moments from loads which cause no appreciable structural sidesway of the member; this includes the gravity loads on an unbraced frame and all loads in a braced frame. Using b from the word braced as a subscript, these first-order moments are denoted as $M_{bx}$ and $M_{by}$. Since a majority of wood structures are braced rather than rigid frame, this first category is the only one which needs to be considered in most cases. It should also be noted that sway, which more broadly defined is the relative translation of the member ends, is almost always prevented for horizontal members (i.e., beams) subject to combined loading by their vertical support by columns.

Moments from loads resulting in appreciable sidesway, such as wind or other lateral loads acting on a rigid frame, constitute the second family of first order moments. The sidesway moments are denoted (with s from the word "sway") as $M_{sx}$ and $M_{sy}$. Rigid frame wood buildings are sometimes used; examples include many pole building and construction in which column members are given moment constraint at their upper end by their being attached to both the top and bottom chords of parallel chord trusses or when diagonals between the column and the floor or roof member are used.

The moment magnifiers all include a denominator term that becomes smaller, thus increasing the moment magnifier, as the ratio of the axial force to the Euler column buckling load for the direction being considered gets larger. When the applied axial load $P_u$ equals the compression resistance factor, $\phi_c$, times the buckling load $P_e$, which is the limiting condition approachable for a concentrically loaded column only when the column has a high slenderness ratio, the moment magnifier becomes infinitely large. This means that the beam-column which is about to buckle from the axial load acting alone can sustain no applied moment.

When the member ends are braced against sidesway, the magnification of the maximum moment also depends upon the moment distribution along the member. This is reflected in the moment shape coefficients $C_{mx}$ and $C_{my}$. For a member bent into a single curvature by equal end moments, the maximum P-delta moment occurs at midheight and always results in a larger total (first order plus P-delta) moment. For a member bent into an S-shaped reverse curve by end moments equal in magnitude, the member's flexural deflection is largest at about one-quarter of the member length from each end. The resulting maximum P-delta moment acts where the first order moment is about one-half of that at the member ends. Maximum first-order moments cannot be reduced by the P-delta moments. Thus, the moment magnifiers all must be limited to one or larger.

Equation 6.3-8, which gives $C_m$ when only end moments act, is the same as that used (when allowance is made for different sign conventions for $M_1/M_2$) in the design codes for reinforced concrete (ACI, 1989) and for structural steel (AISC, 1986).

The moment magnification factors of Eqs. 6.3-4 through 6.3-7 contain no explicit recognition of the creep deflections accompanying long term flexural loading and the resulting increase of P-$\Delta$ moments. The influence of creep and other time-effects on the performance of beam-columns is not well defined. When long duration bending moments act, the effects of creep deflections may be recognized by reducing the modulus of elasticity value used to determine the buckling loads $P_u$ and $M_{ux}$.

### C6.4 Columns Loaded on Side Brackets

These procedures are a continuation of those that have long been in the NDS (AF&PA, 1991). The replacement lateral load, $P_s$, is specified to produce approximately the same effect as the actual loading. Through a simple consideration of the moments in the actual and the replacement loadings, it is easy to show (Criswell, 1986) that the replacement loading gives a maximum moment equal to 0.75 of that in the real member, but produces a more critical single curvature moment diagram rather than the actual reverse curvature shape. Additional conservatism is provided by placing the side bracket load so that it acts along the entire length of the column rather than only from the bracket down.

### C6.6 Trusses

**C6.6.1 Sheathed truss compression chords.** Sheathing attached to a truss compression chord, as often is the case for the top chord, increases the flexural stiffness of the chord through the partial composite behavior described in Sec. 5.5.3 for parallel member beam systems. This increased effective EI flexural stiffness is recognized with the $C_T$ buckling stiffness factor which acts as a multiplier on the strong axis moment of inertia. The $C_T$ values are the same as in the 1991 NDS (AF&PA,1991).

## References Commentary Chapter 6

American Concrete Institute. 1992. *Building Code Requirements for Reinforced Concrete (ACI 318-92) and Commentary - ACE 318R-92.* ACI, Detroit, MI, 353 p.

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC.

American Institute of Steel Construction. 1994. *Load and Resistance Factor Design Specification for Structural Steel Buildings, Second Edition.* Chicago, IL.

Criswell, M.E. 1986. *Design of Columns.* In "Wood: Engineering Design Concepts," Volume IV, Clark C. Heritage Memorial Series on Wood, Forest Products Laboratory in cooperation with The University of Wisconsin, published by The Pennsylvania State University, pp. 291–364.

Criswell, M.E., J.R. Goodman, and J. Bodig. 1988. *The Stiffness Contribution of Sheathing to Truss Compression Chord Members.* Proceedings of the 1988 International Conference on Timber Engineering, September 19–22, 1988, Seattle, WA, Forest Products Research Society, Madison, WI, pp. 439–447.

Zahn, J.J. 1986. *Design of Wood Members Under Combined Load.* Journal of Structural Engineering, ASCE, Vol. 112, No. 9, September 1986, pp. 2109–2126.

Zahn, J.J. 1988. *Combined-Load Stability Criterion for Wood Beam-Columns.* Journal of Structural Engineering, ASCE, Vol. 114, No. 11, November 1988, pp. 2612–2628.

Zahn, J.J. 1988. *Empirical Failure Criteria with Correlated Resistance Variables.* J. Str. Engr., ASCE 116(11):3122–3137. New York, NY.

## COMMENTARY

# Chapter 7 Mechanical Connections

### C7.1 General

**C7.1.2 Connection design.** Connection resistance is based on the use of (a) statistical models fit to test data, (b) verified analytical models using yield theory (McLain and Thangjitham, 1983; Soltis, et al., 1986; Soltis and Wilkinson, 1987), or (c) test data directly. These estimates have been combined with traditional safety levels and cast into an LRFD format. The development of connection design provisions is described in McLain et al. (1993). Note that fasteners are assumed to be installed in clear, relatively flat grain material. Design values for connections do not account for localized growth characteristics such as knots, excessive slope of grain, pitch pockets, and the like which may influence connection capacity. Also, they do not consider processing-related characteristics such as splits, wane, or voids.

The time-effect factor, $\lambda$, developed for solid wood products, as described in Sec. 1.4, is applied to connections with two exceptions. First, the factor $\lambda$ is not applied if the resistance of a nonwood connecting element or of the fastener governs the resistance of the connection. Second, for impact load $\lambda = 1.0$. There are no data to support an increase in connection resistance associated with impact loading. Hence, the more conservative $\lambda = 1.0$ is specified.

**C7.2.3 Dowel bearing strength.** The dowel bearing strength of wood or wood-based material indicates the ability of a material to resist lateral embedment of a rod. It is defined as the load at which the initial slope of the load-embedment curve, when offset by 5% of the dowel diameter, intersects the curve. It is a material property and is available from the manufacturer or found in the supplements to the specification. For solid wood, $F_e$ is a function of density, rod diameter, and direction of embedment with respect to grain. A Hankinson-type formula is used to interpolate between parallel to and perpendicular to grain loadings. For further information see Wilkinson (1991), or Smith, et al. (1988).

Note that the dowel bearing strength used in LRFD is the same value used in allowable stress design (ASD). This differs from other ASD properties, which are generally multiplied by a conversion factor prior to their use in LRFD. The scaling of connection resistances is accomplished within the resistance equations themselves, not by scaling of the dowel bearing strength values.

**C7.3.1 Simple connections.** Pinned connections with no fixity should be used in design and construction unless provision is made for moment at the connection. In general, moment connections should be avoided with dowel-type fasteners, split rings, and shear plates because of the high tension perpendicular to grain forces that may result. These forces may cause brittle failure of the connection.

**C7.3.3 Member stress at connection.** Historically, net section requirements for connections with staggered rows of bolts and lag screws loaded parallel to grain have been based on a fastener spacing of 8D. It has been assumed that for a row with less than 8 diameter spacing, the fasteners in adjacent rows are considered aligned; for spacing greater than 8D, adjacent rows are considered staggered (AF&PA 1991). This philosophy is extended to the current pitch (s) and gage (g) requirements, the group action factor ($C_g$), as well as net section requirements for bolts, lag screws, drift pins, or 1/4 in. (6.3 mm) or larger diameter dowels. It is based on the observation that a 4D pitch requirement between fasteners in adjacent rows equals an 8D pitch distance in one row. See Fig. C7.3-1.

If any fastener in a row is spaced less than a 4D pitch distance from a fastener in any adjacent row for parallel to grain loading:

(a) The critical net section requirement assumes fasteners in adjacent rows are aligned and placed at the critical section. For example, if there are two rows of such fasteners, the critical net section would be the gross section area minus the area of two holes.
(b) The gage requirements are the same as for aligned multiple rows.
(c) For calculation of $C_g$, $n_i$ is equal to the actual number of fasteners in each individual row since the actual staggered rows are assumed aligned.

If any fastener in a row is spaced 4 or more diameters pitch distance from any fastener in an adjacent row for parallel to grain loading:

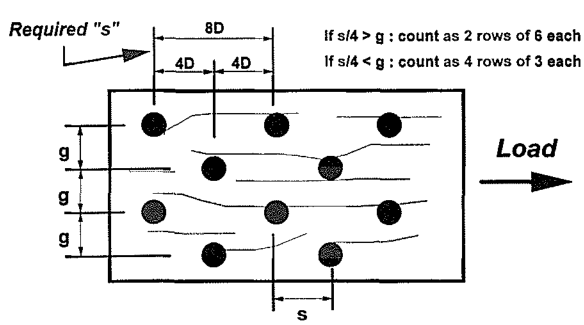



**Figure C7.3-1. Fastener spacing, showing pitch (s) and gage (g).**

(a) The critical net section includes only one fastener from adjacent rows. For example, if there are two rows of fasteners and s > 4D then the critical net section would be the gross section area less the area of one hole.
(b) There is no minimum gage spacing requirement. As the gage approaches zero, two staggered rows with 8D pitch become one row with 4D pitch.
(c) Calculation of $C_g$ is based on the gage distance between rows. If the gage is less than or equal to 1/4 the pitch, the two staggered rows are considered as one row with $n_i$ equal to the total number of fasteners in the two rows. If the gage distance is more than 1/4 the pitch (for one row), the two staggered rows are independent and $n_i$ is equal to the actual number of fasteners in each row.

The net section at a split ring/shear plate unit is determined by subtracting from the full cross-sectional area of the member the projected area of that portion of the connector groove within the member and that portion of the bolt hole not within the connector groove located at the critical plane.

*C7.3.6.1 Group Action Factor:* When a connection contains one or more serial rows of bolts, lag screws, or dowels with D > 1/4 in. (6.3 mm) or split rings, shear plates, or similar devices, there is a reduction in resistance because of uneven distribution of force to each fastener. This reduction does not apply to small diameter fasteners with D < 1/4 in. (6.3 mm). Formula 7.3-1 for $a_i$ was obtained by algebraic simplification of an elastic analysis of load sharing in a row of fasteners (Lantos, 1969; Zahn, 1990).

The row factor term $a_i$ is always less than $n_i$ and approaches $a_\infty$ as $n_i$ increases. $a_\infty = (1 + j)/(1 - m)$. This term is useful for design purposes to find the maximum possible row strength for a given configuration.

## C7.4 Nails, Spikes, and Wood Screws

**C7.4.1 General.** The pennyweight system of nail classification has been used in trade for many decades. However, there are no widely accepted standards that relate pennyweight with nominal diameter or length. The only partial specification is Federal Specification FFN-105B (GSA 1974) which identifies nails by pennyweight, diameter, and length. However, this specification does not serve as an overall standard for dimensions or yield properties for use in the marketplace. Given the lack of accepted standards, the designer should specify required diameter, length and $F_y$ for nails used in engineered connections. Specifying pennyweight is not adequate for engineering purposes.

The use of hardened steel nails for engineered nail connections can be advantageous, particularly for connections involving specialized hardware. In the absence of accepted quality standards for these fasteners, particularly with regard to thread, the designer/manufacturer must qualify nail properties by test using industry guidelines.

**C7.4.2 Spacing of fasteners.** Minimum spacings for nails, spikes, and wood screws will ensure that each fastener is utilized to its fullest capacity. These spacings will minimize splitting of the wood near the fastener in most species. Predrilling holes may be required in some hardwoods. Because of the ductility of these fasteners there is no adjustment for multiple fasteners.

The capacity of nail and spike connections using lead holes is generally greater than for connections that are not predrilled. This rises from associated minimization of wood splitting. However, lead holes can be too large and strength will be reduced. For this reason, lead holes must not exceed those specified.

**C7.4.3 Resistance to lateral forces.** The equations for the nominal lateral strength of a small-dowel connection are based on a yield theory for dowel connections. To insure the expected action of the fastener in the connection a minimum penetration is specified. When the length of penetration of the shank into the member holding the point ($l_p$) is less than 6D for nails and spikes or 4D for screws, lateral capacity may be erratic and variable. This is because the withdrawal resistance of the point may not be great enough to allow formation of the expected yield mode.

Lateral resistance is also a function of the properties of the nail, spike, or wood screw. There are no widely accepted values of these properties and this information should be sought from manufacturers for key connections. Loferski and McLain (1991) sampled nails from varied regions of the U. S. and determined bending strength, $F_y$. Their results are comparable to those reported by Smith et al. (1988). For calibration purposes, $F_y = 130 - 214D$ (ksi) was used to set the relative safety level to that of current practice. For a 16d nail with $D = 0.162$ in. (3.2 mm), $F_y = 95$ ksi.

A distinction between "large" and "small" dowels lies in their embedding response parallel to and perpendicular to grain in solid wood. For "small" driven fasteners (i.e., nails, spikes, etc.) there is no discernable effect of grain orientation—that is, parallel and perpendicular to grain strengths are nearly equal. With "large" dowels (i.e., bolts, lag screws, etc.) there is a clear difference between strengths at different grain orientations and Hankinson's formula is generally used for intermediate loading angles. Soltis, et al. (1987) found that the orientation effects boundary between large and small dowels was about 1/4 in. (6.3 mm), but did vary somewhat by density. This effect is offset somewhat by differences in $F_e$ for "driven" vs. "non-driven" fasteners. Consequently, there is no need to consider grain orientation for standard nails or screws. However, for connections with dowel having $D > 1/4$ in. (6.3 mm) and driven into prebored holes, the bolt or lag screw design provisions may be appropriate.

Connections with fasteners in double shear may be designed as the sum of two single shear connections. The thickness limit on the center member precludes a failure mode unique to three member connections (Aune and Patton-Mallory, 1986). Minimum penetration of the point in a side member should be provided to make sure that the expected yield mode is physically possible. This specification applies only to symmetric connections and fasteners should be inserted from both sides to minimize eccentricity.

Nails and wood screws inserted in end grain are inefficient in resistance to lateral loading and may result in brittle failure of the connection under some circumstances. Alternative fastening schemes that take advantage of fastening strength in side grain should be considered.

**Head Pull-Through.** The fastener and side member should be chosen to avoid head pull-through failures. Resistance to head pull-through can be established by test or analysis. Failures due to head pull through can occur in areas subject to high negative pressures caused by high wind speeds. Although determination of head pull design values is difficult, the designer should be aware of the need to consider it when determining the type and spacing of fasteners. Recent revisions to standards in high wind areas (SBCCI 1991) and to recommended nailing schedules (APA 1992) specifically address this issue.

**C7.4.4 Resistance to axial forces.** The withdrawal resistance of proprietary fasteners such as helically or annularly threaded nails and spikes depends on the geometry of the thread or ring. Research has shown that improvement in withdrawal strength over that of a smooth shank by "threading" may be negligible or substantial depending on quality of thread (Stern, 1986). Since there are no universally accepted thread quality specifications there is no way to count on a specific increase in capacity over that of a plain-shank nail. Design resistance can default to that of a plain-shank nail with a shank diameter equal to that of the deformed shank nail. There is no reduction in withdrawal strength of a threaded nail due to changing moisture conditions (USDA, 1987).

Equations 7.4-10 and 7.4-11 are the basis of current NDS withdrawal tables. New information, based on analyses of a large collection of available data from a wide variety of sources, will likely be the basis of future versions of this section of the specification. Only shank withdrawal is considered by use of these equations. In general, engineered connections that depend solely on nail, spike, and wood screw axial withdrawal should be avoided because withdrawal strength is subject to wide variation. This is particularly true for some fastener types if the connection experiences any change in moisture content. Screw connections in withdrawal are generally more reliable than nail or spike connections and may be disassembled and reassembled without significant loss of strength, if properly fabricated.

**C7.4.5 Combined axial and lateral loading.** Equation 7.4-12 was developed by DeBonis and Bodig (1975) for smooth shank nails and is a conservative interpretation of their experimental results. For fasteners with a high withdrawal resistance such as wood screws, or for nails and spikes with deformed shanks, this interaction form becomes more conservative.

## C7.5 Bolts, Lag Screws, Drift Pins, and Dowels

**C7.5.3 Spacing of fasteners.** Design criteria for spacing, end, and edge distance of bolted connections are based on experimental observation (Trayer, 1932). Trayer had broad experience with bolted connections for aircraft components during the 1920's and based his recommendations on this experience. Trayer recognized that the stress distribution beneath the bolt for various $\ell/D$ ratios affects the spacing and end distance requirements needed to develop maximum capacity of the connection. He concluded, however, that spacing and end distance requirements based on small $\ell/D$ ratios are conservative for larger $\ell/D$ ratios. Later researchers have confirmed Trayer's observations. These observations are also incorporated for other large diameter dowel-type connections.

How holes are drilled and aligned significantly affects the load-deformation behavior of connections, especially bolted connections. Unpublished research at the U. S. Forest Products Laboratory indicates that slightly oversized holes do not materially affect yield loads, but deformation at yield is approximately 10% to 20% greater. If the hole is drilled at a two degree angle from a perpendicular to the surface, the load is approximately half of the yield load for perpendicular-drilled holes for the same deformation.

The assumed load distribution to fasteners in a row is based on research of bolted connections in carefully machined and fit holes. The distribution of loads becomes unknown if one or more holes is reamed for field fitting or is misfabricated. For example, if three out of four holes in a row are reamed it is possible for total load to be placed on the fourth fastener (Wilkinson, 1986). Inspection should be provided to prevent deleterious field fitting. For connections with multiple fasteners, it is highly advisable to bore holes in a quality controlled environment, preferably prior to any preservative treatment.

Holes should be drilled smooth. Rough holes produce flatter load deformation curves which result in higher than normal levels of deformation at service load levels. Ultimate strength is relatively unaffected (USDA, 1987).

The requirement for a maximum dimension of 5 in. (127 mm) between the outermost fasteners perpendicular to grain is based on experience with shrinkage cracks developed in connections with steel side plates. It is possible for wood or wood-based members to be placed in-service at 19% moisture content and subsequently dry out to a lower moisture content. The restraint caused by shrinkage in the wood and no shrinkage in the steel side plates may result in cracks due to high tension perpendicular to grain stresses. These cracks may result in a weakened connection when subject to design load. The 5-in. (127 mm) requirement is a spacing judged to minimize this shrinkage problem.

**C7.5.4 Resistance to lateral forces.** Characteristic loads for bolts, lag screws, or dowels (as well as nails and wood screws) are predicted from a strength of materials-based yield theory. These provisions are consistent with the NDS (AF&PA, 1991). Analytical models were developed by Johansen (1949) to predict connection yield load. The models have been shown by McLain and Thangjitham (1983), Soltis et al. (1987, 1986), and others to accurately predict connection behavior.

The yield models use embedding strength, $F_e$, fastener yield strength, $F_y$, and connection geometry to predict a connection yield load for two- and three-member connections. Based on mechanics, six possible yield modes were identified for single-shear connections and four modes for double-shear connections. These are shown graphically in Fig. C7.5-1. Each mode is identified by number and action. Modes I and II actions are bearing dominated. Mode III results from the formation of a single plastic hinge in the dowel near each shear plane. Mode IV exhibits two yield points in the fastener near each shear plane. The connection yield load is defined as the load at which the initial slope of the connection load-slip curve, when offset by 5% of the fastener diameter, intersects the curve (see Fig. C7.5-2).

This definition of yield load has been used by Harding and Fowkes (1984) and is a more repeatable experimental load than the traditionally used proportional limit load.

The yield equations for bolts have been calibrated to historical practice over a portion of the design space using $F_e$ values taken from industry guidelines and $F_y$ = 45 ksi (310.3 MPa).

For lag screws, the complete derived yield equations have been simplified based on a calibration that is strictly tied to the ASME/ANSI B18.2.1 (1981) geometry. The designer should be aware that lag screws with both cut and rolled threads are found in the marketplace. Screws with rolled threads generally have a shank diameter that is less than nominal size. Either type of screw may be used for lateral resistance so long as actual diameter is used for computation of resistance. The use of "full body diameter" screws is generally more efficient for lateral loading.

*C7.5.4.2 Adjusted lateral resistance.* If less than maximum resistance of a connection is needed, the spacing and end distance requirements may be reduced according to a straight line interpolation. However, in no case should the capacity be reduced to more than one half the maximum capacity. These provisions are based on successful historical practice.

**C7.5.5 Resistance to axial forces.** The withdrawal resistance of lag screws is based on an analysis of available data from Newlin and Gahagan (1938) and Carroll (1988). No screws with rolled threads (i.e., reduced body diameters) were tested but it seems reasonable to expect that the withdrawal performance of all fasteners meeting the standard will be adequate. This is not true of lateral resistance.

**C7.5.6 Resistance to combined axial and lateral forces.** Recent research by McLain and Carroll (1990) indicates that either the vectorial approach of the current allowable stress design (AF&PA, 1991) or the Hankinson's interpolation approach of the Timber Construction Manual (AITC, 1985) may be used depending on the design basis. The latter is preferred for use with yield load as it remains conservative for larger diameter screws.

## C7.6 Split Rings and Shear Plates

**C7.6.1 General.** Shear plates and split rings are connectors devised to increase the bearing and shear area of bolts or lag screws. Shear forces are transferred between adjoining members through the ring or plate; the bolt or lag screw serves principally to keep the members in contact but do provide some resistance. Their design basis is testing discussed by Scholten (1944). *The Wood Handbook* (USDA, 1987) provides a contemporary summary of those results.

**C7.6.3 Resistance to lateral forces.** It is assumed that the faces of all members are brought into contact when the connectors are installed. In addition, seasonal variations in moisture content (after the wood has reached moisture equilibrium) must be considered. When joints are made in unseasoned wood, joint tightening should take place periodically until moisture equilibrium is reached.

The connector axis is defined by a line joining the centers of any two adjacent connectors located in the same face of a member in a joint. The angle of the connector axis is that with respect to the longitudinal axis of the member. This angle is a factor in the determination of the required spacing of connectors for a given load as illustrated in Fig. 7.4-1.

The tabulated edge distances are for unloaded and loaded edges. The unloaded edge distance is a minimum value and is used both for parallel to grain loading and for the unloaded edge distance when the load acts at an angle to grain. The distances given for the loaded edge are the minimum and that required to realize full design load.

The edge distances for connectors were determined by tests on lumber. Design values for 2-1/2

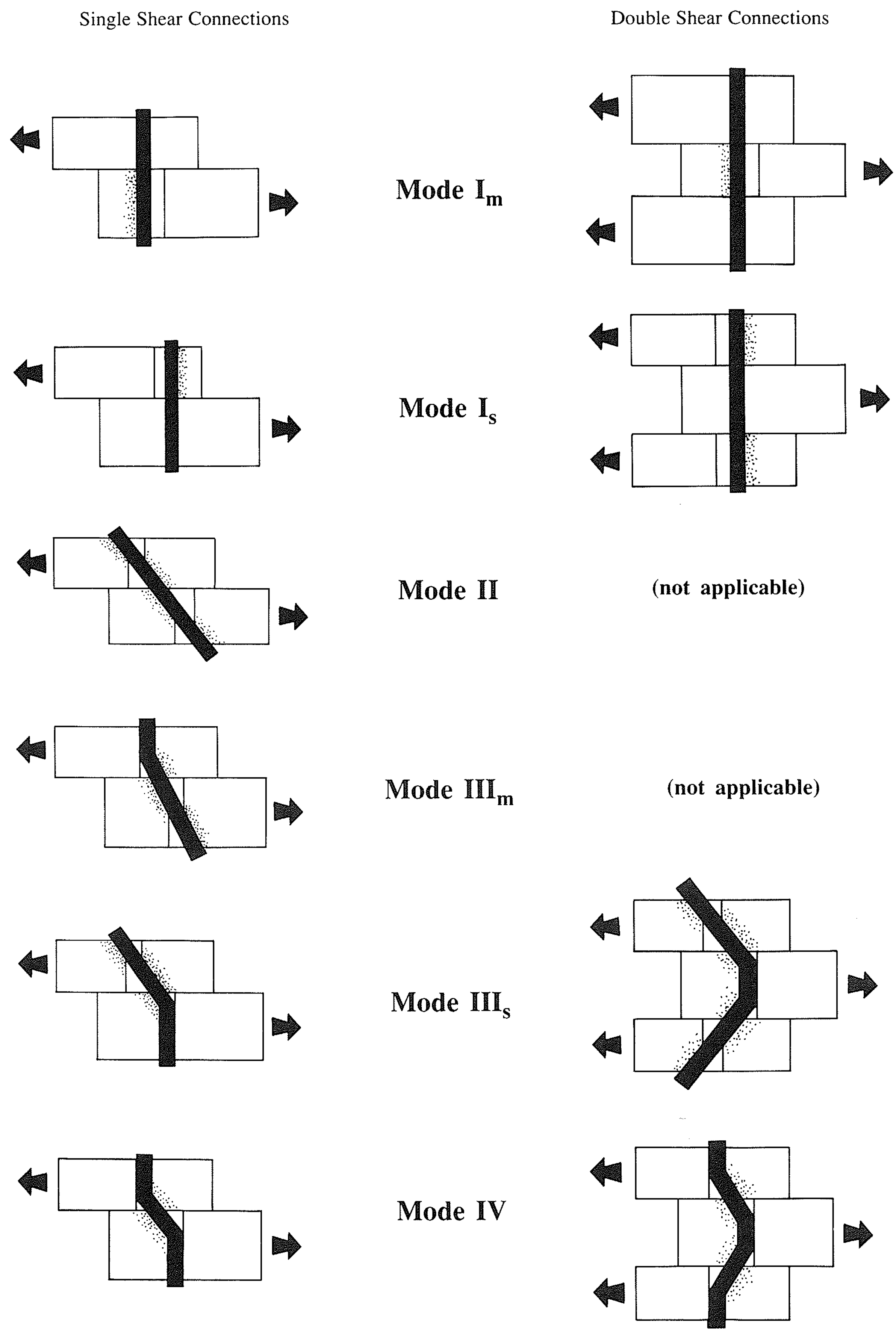



**Figure C7.5-1. Yield modes for laterally-loaded connection with dowel-type fastener.**

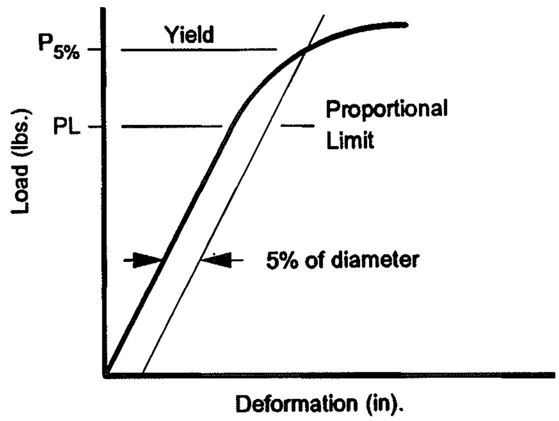



**Figure C7.5-2. Definition of yield for laterally-loaded connection with dowel-type fastener.**

in. (65 mm) and 2-5/8 in. (67 mm) connectors were based on edge distances that allow the use of a connector in nominal 4 in. (102 mm) - wide pieces (net 3.5 in.; 89 mm) and the use of a 4 in. (102 mm) connectors in nominal 6 in. (152) - wide pieces (net 5.5 in.; 140 mm). Glued laminated timbers are slightly smaller in width than sawn lumber of the same nominal width. The modification factors were developed from the work of Scholten (1944) who explored the use of less than reference edge distances. A 2-1/2 in. (65 mm) or 2-5/8 in. (67 mm) connector should not be used in a member less than 3 in. (76 mm) wide, and a 4 in. (102 mm) connector should not be used in a member less than 5 in. (127 mm) wide.

## References Commentary Chapter 7


American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC.

American Institute of Timber Construction 1994. *Timber Construction Manual.* Wiley-Interscience, Fourth Edition.

American Plywood Association. 1992. *Roof Sheathing Fastening Schedules for Wind Uplift.* T92-28. Tacoma, WA.

ANSI/ASME B18.2.1-1981. *American National Standard for Square and Hex Bolts and Screws—Inch Series.* American Society of Mechanical Engineers, New York, NY.

ANSI/ASME B18.6.1-1981. *American National Standard for Wood Screws.* American Society of Mechanical Engineers, New York, NY.

ASTM A307-84. 1984. *Specification for Carbon Steel Externally Threaded Standard Fasteners.* American Society for Testing and Materials, Philadelphia, PA.

Aune, P. and M. Patton-Mallory. 1986. *Lateral load-bearing capacity of nailed joints based on the yield theory.* USDA Forest Service Research Reports. FPL-469 and 470. U. S. Forest Products Laboratory, Madison, WI.

Carroll, J.D. 1988. *Withdrawal and combined load capacity of threaded-fastener wood joints.* M.S. thesis. Dept. of Wood Science and Forest Products. Virginia Tech. Blacksburg, VA.

DeBonis, A.L. and J. Bodig. 1975. *Nailed wood joints under combined loading.* Wood Science and Technology 9(2):129–144.

General Services Administration (GSA). 1974. *Federal Specification, Nails, Brads, Staples, and Spikes: Wire, Cut and Wrought.* (FFN-105B) (with 4 supplements). Federal Supply Service, U. S. Govt. Printing Office, Washington, DC.

Gerhards, C.C. 1982. *Effect of moisture content and temperature on the mechanical properties of Wood: An Analysis of Immediate Effects.* Wood and Fiber, 14(1), 1982.

Harding, N. and Fowkes, A.H.R. 1984. *Bolted timber joints.* Proceedings of Pacific Timber Engineering Conference, Volume III Wood Science, May. pp. 872–883.

Johansen, K.W. 1949. *Theory of timber connections.* Publications, International Association for Bridge and Structural Engineering. 9:249–262.

Lantos, G. 1969. *Load distribution in a row of fasteners subjected to a lateral load.* Wood Science 1(3):129–136.

Loferski, J.R. and T.E. McLain. 1991. *Static and impact properties of common wire nails.* ASTM J. of Testing and Evaluation, 19(4):297–304.

McLain, T.E., L. Soltis, D. Pollock, T. Wilkinson. 1993. *LRFD for Engineered Wood Structures: Connection Behavioral Equations.* J. Structural Engineering, 119(10):3024–3038.

McLain, T.E. and J.D. Carroll. 1990. *Combined load capacity of threaded fastener wood connections.* J. Structural Engineering, 116(9):2419–2432.

McLain, T.E. and S. Thangjitham. 1983. *Bolted wood joint yield model.* Journal of the Structural

Division, American Society of Civil Engineers. 109(8):1820–1835.

Nelson, J.A. and R.W. Petterson. 1983. *The Effects of Hot Water Exposure on the Strength and Stiffness of Douglas-Fir and Redwood.* Cooling Tower Institute Paper No. TP 264A.

Newlin, J.A. and Gahagan, J.M. 1938. *Lag-screw Joints: Their Behavior and Design.* Technical Bulletin No. 597. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI.

Scholten, J.A. 1944. *Timber-connector joints, their strength and design.* Tech Bulletin 865. U.S. Dept. of Ag. Washington, DC.

Smith, I., L.R.J. Whale, C. Anderson, B.O. Hilson, and P.D. Rodd. 1988. *Design properties of laterally loaded nailed or bolted wood joints.* Can. J. Civil Eng. 15(4):633–643.

Soltis, L.A. and Wilkinson, T.L. 1987. *Bolted Connection Design.* General Research Report FPL-GRT-54. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI.

Soltis, L.A., Hubbard, F.K., and T.L. Wilkinson. 1986. *Bearing strength of bolted timber joints.* J. Struct. Eng. ASCE. Vol. 112, No. 9 pp. 2141–2154.

Soltis, L.A., S. Karnasudirdja, and J.K. Little. 1987. *Angle to grain strength of dowel-type fasteners.* Wood and Fiber Science 19(1):68–80.

Southern Building Code Congress International. 1993. *SBCCI SSTD 10-93 Standard for Hurricane Resistant Residential Construction.* Birmingham, Alabama.

Stern, E.G. 1986. *Specifications for fasteners used for assembly of wood pallets, wood-frames, trussed rafters, trusses, buildings and pole-type structures.* Forest Products Journal 36(1):25–26.

Trayer, G.W. 1932. *Bearing strength of wood under bolts.* Tech. Bull. No. 332, Washington, DC, U. S. Dept. of Agriculture, 40 p.

USDA Forest Products Laboratory. 1987. *Wood Handbook—Wood as an Engineering Material.* Agricultural Handbook 72, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

Wilkinson, T.L. 1986. *Load distribution among bolts parallel to load.* J. Struct. Engineering, Vol. 112, No. 4, pp. 835–851.

Wilkinson, T.L. 1991. *Dowel Bearing Strength.* Research Paper FPL-RP-505. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI.

Zahn, J.J. 1990. *Design Equations for Multiple-Fastener Wood Connections.* J. Str. Engr., ASCE 117(11):3477–3486. New York, NY.

# COMMENTARY

# Chapter 8 Structural-Use Panels

## C8.1 Scope

Structural-use panels are manufactured to the requirements of applicable standards. Structural-use panels enclose structures and support occupancy loads, transferring loads from covered areas to the primary structural framework. The loads distributed over the sheathing typically induce bending and shear stresses in the panels. The sheathing must resist the applied loads without excessive deformation.

Another mode of transferring sheathing loads to framework elements is through panel shear (i.e., diaphragm behavior). This is an important structural application involving composite action between panels and structural framing.

Equitable assessment of various types of wood-based sheathing materials led to development of a performance-based evaluation system (APA PRP-108), in which end-use requirements are the criteria for acceptable performance, independent of the material composition of the panel element. This development activity has culminated in promulgation of PS2-92, "Performance Standard for Wood-Based Structural-Use Panels," by the U.S. Department of Commerce, National Institute of Standards and Technology.

A description of the performance-based evaluation system and its relationship to the development of LRFD for structural-use panels is presented by O'Halloran et al. (1988).

## C8.2 Design Requirements

**C8.2.1 Reference conditions.** Reference conditions represent applications typically encountered by the designer. The reference resistance, R, may be used directly in design equations when the end-use conditions coincide with reference end-use conditions. Reference end-use conditions are presented in section 2.5.

**C8.2.2 Specification of structural-use panels.** Structural-use panels are classified by span ratings. Along with the selected span rating, the designer must specify nominal panel thickness, panel exposure rating, and, for plywood panels, panel grade.

Structural-use panels are qualified for use upon satisfying requirements of either prescriptive manufacturing standards (e.g., PS1-83), or performance standards (e.g., PS1-83, PS2-92). Both approaches accommodate a range of panel thicknesses for each span rating, which requires that the designer specify both span rating and nominal panel thickness.

Structural-use panels exposure durability classifications include Exterior and Exposure 1.

**Exterior** panels have a fully waterproof bond and are designed for applications subject to permanent exposure to the weather or to moisture.

**Exposure 1** panels have a fully waterproof bond and are designed for applications where long construction delays or other conditions of similar severity may be expected prior to providing protection.

## C8.3 Reference Resistance

Panel-based structures represent a special class of wood products, mostly due to their unique aspect ratios and applications. Unless otherwise indicated, the primary axis, $X_p$, of a structural-use panel is in the long axis direction and the secondary axis, $X_s$, is normal to the axis, with both axes in the plane of the panel. While other dimensions may be available, panel length is usually 8 ft and panel width is usually 4 ft.

**C8.3.1 Panel stiffness and factored reference resistance.** The design values include factored reference resistance, $\lambda\phi R$, panel bending stiffness (flexural rigidity), EI, panel axial stiffness, EA, and panel rigidity. The $\lambda\phi R$ values are given as factored reference bending moment capacity, $\lambda\phi M$, factored reference shear capacity, $\lambda\phi V$, factored reference tensile capacity, $\lambda\phi T$, and factored reference compression capacity, $\lambda\phi P$. These values are tabulated on a one foot of panel width basis, simplifying the distributed load (psf) conversion to linearly distributed load (plf).

**C8.3.2 Reference strength and elastic properties.** Structural-use panel design is accomplished with panel load capacities—panel stiffness and factored reference resistance. For unusual applications requiring elastic and strength properties of structural-use panels, the designer will compute these properties using tabulated design section properties.

The tabulated design section properties correspond to reference (design) thicknesses, which are tabulated as the nominal thickness for each span rating. Due to the orthotropic nature of structural-use panels, unique property values exist in the primary axis and the secondary axis directions.

Note that the term shear "through the thickness" is the result of in-plane shearing of the sheet. This is contrasted with the term "rolling shear," which refers to shear that tends to slide, or roll, one "ply" of the sheet over another (Fig. C8.3-2).

## C8.4 Design Section Properties

The design section properties are provided for the primary and secondary axes on a per foot of width basis. For cases where the normal stress is parallel to the primary axis, the section properties designated for the primary axis, *p*, should be used. Such is the case for bending applications where the panel is installed with the primary axis across supports. When the normal stress is in the cross-panel direction, the secondary axis, *s*, section properties should be used. This condition occurs when the panel is installed with the secondary axis across supports.

**C8.4.1 Design thickness.** The design thickness values for structural-use panels are associated with design resistances so that the engineer may design with the material as if it were a homogeneous plane anisotropic plate—a plate with different properties in its two principal directions. In most applications, the designer need not be concerned with the actual multilayer makeup of the panels.

## C8.5 Design

Generally, structural-use panels are designed to carry uniform loads over multiple spans.

**C8.5.2 Flatwise bending.** Flatwise panel bending design utilizes standard beam equations (Eq. 5.1-1 and 5.1-2). For single or multispan application, the moment due to factored loads (kip-in.) is computed as:

$$M_u = w\ell^2/k \qquad \text{(C8.5-1)}$$

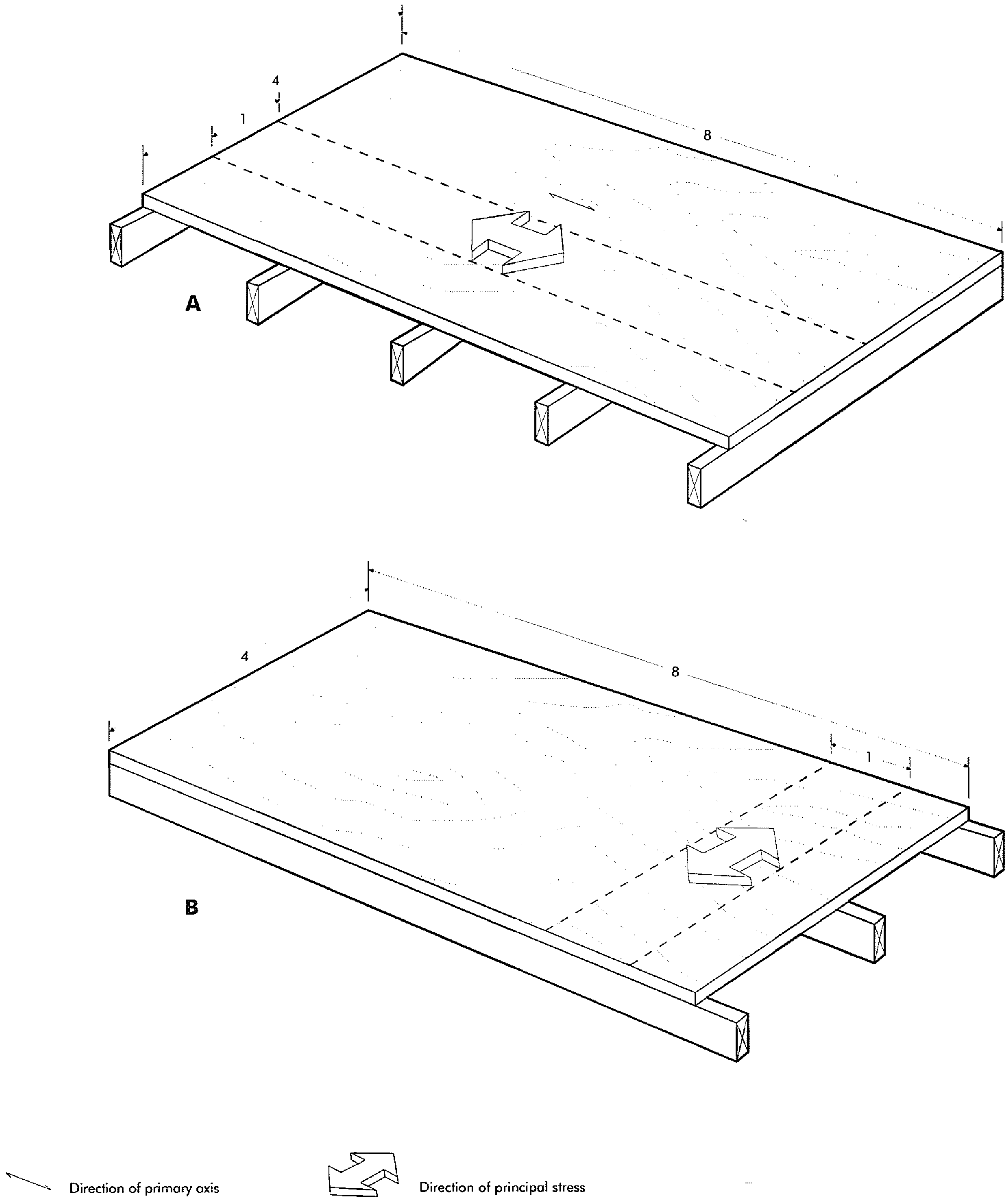
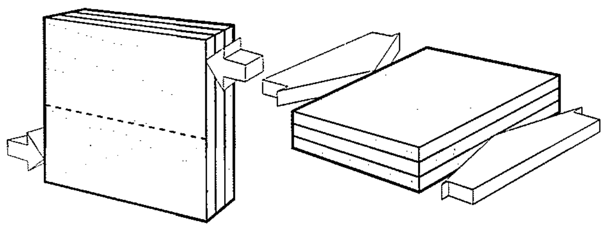



**Figure C8.3-1. Panels with force applied along the primary (A) and secondary (B) axes.**



A B

**Figure C8.3-2. Illustration of shear through the thickness (A) and rolling shear (B).**

where w is the factored uniform applied load (ksf), $\ell$ is the design span (center-to-center of supports, in.) and k is a constant (which includes units conversion) equal to 96 for single-span or two-span (continuous) applications and equal to 120 for three or more span (continuous) applications.

Similarly, the shear force (kips) due to factored loads can be computed as:

$$V_u = w\ell/k \quad \text{(C8.5-2)}$$

where w is the factored uniform applied load (ksf), $\ell$ is the clear span (center-to-center of support minus support width, in.) and k is a constant (which includes units conversion) equal to 24.0 for single-span applications, 19.2 for two-span (continuous) applications and 20.0 for three or more span (continuous) applications.

Deflections are computed as:

$$\Delta = w\ell^4/k \quad \text{(C8.5-3)}$$

where $\Delta$ is the maximum deflection (in.), w is the unfactored uniform applied load for deflection calculations (ksf), $\ell$ is the design span (clear span plus one-half panel thickness, in.) and k is a constant (which includes units conversion) equal to 0.92 for single span applications, 2.22 for two-span (continuous) applications and 1.74 for three or more span (continuous) applications.

**C8.5.4 Compression in the plane of the panel.** Compressive load capacities of structural-use panels are usually controlled by buckling due to high slenderness ratios. Slenderness ratios are typically high because panel thickness is small relative to other panel dimensions and boundary conditions.

**C8.5.5 Panel shear.** Design applications that require checking shear-through-the-thickness include the panel element in shear walls and diaphragms, panel webs of I-joists, and panel gusset plates. This would also be the appropriate resistance for checking punching shear.

## References Commentary Chapter 8


American Plywood Association. 1988. *Plywood in Hostile Environments*. RR-132. Revised December 1988. Tacoma, WA.

American Plywood Association. 1989. *Plywood Design Specification*. Revised June 1989. Tacoma, WA.

American Plywood Association. 1989. *Performance Standards and Policies for Structural-Use Panels*. APA PRP-108. Revised June 1989. Tacoma, WA.

American Plywood Association. 1989. *Design Capacities of APA Performance-Rated Structural-Use Panels*. TN-N375. Revised September 1989. Tacoma, WA.

American Plywood Association. 1990. *Design/Construction Guide—Residential & Commercial*. Revised June 1990. Tacoma, WA.

American Society of Civil Engineers. 1987. *Evaluation, Maintenance and Upgrading of Wood Structures, A Guide and Commentary*. ASCE Committee on Wood, New York, NY.

American Society for Testing and Materials. 1987. *Standard Methods of Testing Structural Panels in Flexure, ASTM D3043-87*. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1976. *Standard Test Method for Shear Modulus of Plywood, ASTM D3044-94*. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1976. *Standard Method of Testing Plywood in Compression, ASTM D3501-94*. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1989. *Standard Methods of Testing Plywood in Shear Through-the-Thickness, ASTM D2719-89*. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1990. *Standard Method of Testing Plywood in Planar Shear, ASTM D2718-95*. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1990. *Standard Method of Testing Plywood in Tension, ASTM D3500-90*. ASTM. Philadelphia, PA.

Bodig, J., Cheung, K., and Cunningham, T.P., Jr. 1995. *Engineered Wood Construction: Structural*

*Properties for LRFD.* J. Str. Div., ASCE 121(9):1346–1351. New York, NY.

Canadian Standards Association. 1978. *Canadian Softwood Plywood.* CAN-0151.1-M78. Canadian Standards Association, Rexdale (Toronto), Ontario.

Canadian Standards Association. 1978. *Douglas-fir Plywood.* CAN3-0121.1-M78. Canadian Standards Association, Rexdale (Toronto), Ontario.

Canadian Standards Association. 1992. *Oriented Strand Board and Waferboard.* CAN-0437.0-M92. Canadian Standards Association, Rexdale (Toronto), Ontario.

Canadian Standards Association. 1991. *Construction Sheathing.* CAN-0325.0-M91. Canadian Standards Association, Rexdale (Toronto), Ontario.

Canadian Wood Council. 1990. *Wood Design Manual.* Ottawa, Ontario.

Meyer, R.W. and R.M. Kellogg, Editors. 1982. *Structural Use of Wood in Adverse Environments.* Van Nostrand Reinhold, Co., New York, NY.

O'Halloran, M.R., J.A. Johnson, E.G. Elias, and T.P. Cunningham. 1988. *Considerations of Reliability-Based Design for Structural Composite Products.* Forest Products Journal, 38(4): 35–43.

O'Halloran, M.R. 1975. *Plywood in Hostile Environments.* American Plywood Association, Research Report 132. Tacoma, WA.

TECO Performance Standards and Policies for Structural Use Panel. March, 1989.

USDA Forest Products Laboratory. 1987. *Wood Handbook: Wood as an Engineering Material.* Agricultural Handbook 72, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

U.S. Department of Commerce. 1994. *Construction and Industrial Plywood. PS 1-94.* Washington, DC.

U.S. Department of Commerce. 1992. *Performance Standard for Wood-Based Structural-Use Panels. PS 2-92.* Washington, DC.

# COMMENTARY

# Chapter 9 Shear Walls and Diaphragms

### C9.1 Scope

Elements of the lateral force resisting system have been defined by the Seismology Committee of the Structural Engineers Association of California (SEAOC) as follows:

**Diaphragm:** "... a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term 'diaphragm' includes horizontal bracing systems."

**Shear Wall:** "... a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm or a structural wall)."

### C9.2 Shear Wall and Diaphragm Design

*C9.2.1.1.* Design of wood shear walls and diaphragms is a lateral force design process. In resisting and transferring lateral forces, shear walls and diaphragms act as thin, deep beams comprised of structural-use panel sheathing connected to structural framing. The sheathing acts as "web" material and boundary members function as "flanges" (chords). It is assumed that chords resist axial forces and webs resist shear. Induced moment is resisted by the couple of chord forces, with any moment resistance provided by the webs being ignored. Shear stresses are assumed to be distributed uniformly through the depth of nailed shear walls and diaphragms. This beam analogy procedure has been substantiated by extensive laboratory testing.

### C9.4 Reference Resistance

With the exception of tabulated capacities for high-load diaphragms, design capacities of wood shear walls and diaphragms are limited to the capacity of the sheathing-to-framing connection. Tabulated capacities for high-load diaphragms are limited by either connection capacity or sheathing capacity in shear-through-the-thickness (web shear).

Wood shear wall and diaphragm design capacities may be taken from approved tables, or calculated according to principles of engineering mechanics. Traditionally, the application of engineering mechanics has been guided observing the results of strictly controlled tests conducted in accordance with applicable standards. The principles of engineering mechanics method has proven to be generally supportive of the tabulated capacities. Likewise, test information has served to verify both the tabulated values as well and the assumptions of the engineering mechanics approach. To assure compatibility between tabulated resistances and the underlying design assumptions, limits have been placed on the applicability of the engineering mechanics procedure.

Considerations in the development of shear wall and diaphragm (in-plane shear) design capacities include:

- sheathing-to-framing connection capacity,
- sheathing thickness,
- sheathing configuration,
- sheathing capacity in shear-through-the-thickness,
- sheathing (web) buckling,
- blocking,
- fastener spacing,
- lines (rows) of fasteners at sheathing edges,
- framing capacities,
- framing width (face receiving fastener).

There are three major phases in design of wood shear walls and diaphragms:

(1) determination of the required resistance through determination of the controlling factored lateral loads;
(2) detailing the diaphragm to provide the required design resistance; and
(3) establishing a complete load path to transfer design forces to the supporting system. Shear wall and diaphragm action requires adequate load (force) transfer connections between associated structural elements.

**C9.4.1 In-plane shear resistance.** In-plane shear resistance is the resistance offered by basic or standard configurations of shear walls and diaphragms. Such resistance is generally limited to the capacity of the sheathing-to-framing connection and corresponds to specific detailing of element components. Openings and other irregularities are usually not reflected in element resistance except where established by approved tests.

**C9.4.2 Boundary element resistance.** Boundary members must be designed to carry the axial forces associated with the in-plane shear resistance established in Sec. 9.4.1. This includes determination of the required number of chords and design of chord splices.

With in-plane shear resistance established and peripheral boundary elements checked to assure that they can carry associated forces, consideration must be given to the effects of openings and other discontinuities. Interior boundary members must be provided around discontinuities if the forces are high enough. Sheathing resistance and associated connection resistances must be checked as necessary to carry localized forces due to discontinuities.

## C9.5 Other Design Considerations

Serviceability considerations are addressed in the model codes in several ways, as discussed below:

**Dimensional Limits.** Dimensions of wood shear walls and diaphragms are typically limited as follows:

maximum span-to-width ratio for diaphragms is 4,
maximum height-to-width ratio for blocked shear walls is 3-1/2,
maximum height-to-width ratio for unblocked shear walls is 2.

**Diaphragm Deflection.** Deflection of blocked wood diaphragms (Eq. C9.5-1) and shear walls (Eq. C9.5-2) may be estimated from the following equations. However, dimensional limits must be satisfied independent of calculated deflections. Specific deflection criteria have not been established. Acceptability of calculated deflections is a consideration reserved for engineering judgment.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma\Delta_c X}{2b} \quad \text{(C9.5-1)}$$

where Δ is the calculated deflection (in); v is the maximum shear due to unfactored design loads in the direction under consideration (plf); L is the diaphragm length (ft); b is the diaphragm width (ft); E is the elastic modulus of the chords (psi); A is the area of chord cross section (in$^2$); G is the modulus of rigidity of the plywood (psi); t is the effective thickness of plywood for shear (in); $\Sigma(\Delta_c X)$ is the sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its

**TABLE C9.5-1.**
**Fastener slip equations, $e_n$.**

| Fastener | Minimum Penetration (in.) | For Maximum Loads up to (lbf) | Approximate Slip, $e_n$ (in.)[(a)(b)] Green/Dry | Dry/Dry |
|---|---|---|---|---|
| 6d common nail | 1-1/4 | 180 | $(V_n/434)^{2.314}$ | $(V_n/456)^{3.144}$ |
| 8d common nail | 1-7/16 | 220 | $(V_n/857)^{1.869}$ | $(V_n/616)^{3.018}$ |
| 10d common nail | 1-5/8 | 260 | $(V_n/977)^{1.894}$ | $(V_n/769)^{3.276}$ |
| 14-ga staple | 1 to 2 | 140 | $(V_n/902)^{1.464}$ | $(V_n/596)^{1.999}$ |
| 14-ga staple | 2 | 170 | $(V_n/674)^{1.873}$ | $(V_n/461)^{2.776}$ |

(a) Fabricated green/tested dry (seasoned); fabricated dry/tested dry. $V_n$–fastener load.
(b) Values based on Structural 1 sheathing fastened to Group II lumber. Increase slip by 20% when sheathing is not Structural 1.

distance (ft) to the nearest support, and $e_n$ is the nail deformation (in.).

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \qquad \text{(C9.5-2)}$$

where $\Delta$ is the calculated deflection (in); v is the maximum shear due to unfactored design loads at the top of the wall (plf); A is the area of boundary element cross section (in$^2$); h is the wall height (ft); b is the wall width (ft); $d_a$ is the deflection due to anchorage details (rotation and slip at tie-down bolts); E is the elastic modulus of boundary element (vertical member at shear wall boundary) (psi); G is the modulus of rigidity of sheathing (psi); t is the effective thickness of sheathing for shear (in.), and $e_n$ is the nail deformation (in.).

**Diaphragm Rotation.** Design for diaphragm rotation typically include the following limits.

Diaphragms should not be considered for transfer of lateral forces by rotation to masonry or concrete building elements.

Diaphragm depth normal to the open side should be the lesser of 25 ft (7.6 m) or two-thirds the diaphragm width.

*Exceptions:* (a) single story structures with a depth normal to the open side of 25 feet (7.6m) or less may have a depth equal to the width; (b) diaphragm depth normal to the open end may be increased to a depth-to-width ratio of 2 if calculated deflections are supportive.

Diaphragm rotation is a consideration that arises in buildings having a side that is open. It is recommended that shear resistance be provided along the open side as the preferred option to consideration of diaphragm rotation.

## References Commentary Chapter 9


American Plywood Association. 1991. *Design/Construction Guide: Diaphragms.* Revised October 1991. Tacoma, WA.

American Plywood Association. 1990. *Plywood Diaphragms.* APA Research Report 138. Revised April 1990. Tacoma, WA.

American Plywood Association. 1990. *Structural Panel Shear Walls.* APA Research Report 154. July 1990. Tacoma, WA.


**TABLE C9.5-1M.**
**Fastener slip equations, $e_n$.**

| Fastener | Minimum Penetration (cm.) | For Maximum Loads up to (N) | Approximate Slip, $e_n$ (cm.)[(a)(b)] Green/Dry | Dry/Dry |
|---|---|---|---|---|
| 6d common nail | 3.18 | 800 | $2.54(V_n/434)^{2.314}$ | $2.54(V_n/456)^{3.144}$ |
| 8d common nail | 3.65 | 979 | $2.54(V_n/857)^{1.869}$ | $2.54(V_n/616)^{3.018}$ |
| 10d common nail | 4.13 | 1156 | $2.54(V_n/977)^{1.894}$ | $2.54(V_n/769)^{3.276}$ |
| 14-ga staple | 2.54 or 5.08 | 623 | $2.54(V_n/902)^{1.464}$ | $2.54(V_n/596)^{1.999}$ |
| 14-ga staple | 5.08 | 756 | $2.54(V_n/674)^{1.873}$ | $2.54(V_n/461)^{2.776}$ |

(a) Fabricated green/tested dry (seasoned); fabricated dry/tested dry. $V_n$–fastener load.
(b) Values based on Structural 1 sheathing fastened to Group II lumber. Increase slip by 20% when sheathing is not Structural 1.

Applied Technology Council. 1976. *A Methodology for Seismic Design and Construction of Single Family Dwellings.* ATC-4. Palo Alto, CA.

Applied Technology Council. 1981. *Guidelines for the Design of Horizontal Wood Diaphragms.* ATC-7. Palo Alto, CA.

Breyer, Donald E. 1993. *Design of Wood Structures.* McGraw-Hill.

Building Officials and Code Administrators International, Inc. 1993. *National Building Code, 1993.* Country Club Hills, IL.

Diekmann, Edward F. 1986. *Design of Wood Diaphragms.* Clark C. Heritage Memorial Series on Wood, Wood: Engineering Design Concepts, Vol. IV. The Pennsylvania State University.

Canadian Wood Council. 1990. *Wood Design Manual*, Ottawa, Ontario.

Council of American Building Officials. 1995. *One and Two Family Dwelling Code.* Falls Church, VA.

International Conference of Building Officials. 1994. *Uniform Building Code.* Whittier, CA.

Southern Building Code Congress International, Inc. 1991. *Standard Building Code.* Birmingham, AL.

Structural Engineers Association of California, Seismology Committee. 1988. *Recommended Lateral Force Requirements and Tentative Commentary.* San Francisco, CA.

# COMMENTARY

# Chapter 10 Serviceability Considerations

## 10.1 General Considerations

Serviceability limit states are conditions in which the functions of a building or structure are disrupted because of excessive elastic deflections, local damage, or deterioration of building components or because of occupant discomfort. While safety generally is not an issue with serviceability limit states, such limit states may have severe economic consequences. The move toward limit states design coupled with the increasing use of the computer as a design tool, the use of lighter architectural materials, and the uncoupling of the nonstructural components from the structural frame, results in building systems that are relatively flexible and lightly damped. Serviceability criteria are essential to ensure functional performance and economy of design for such building structural systems (Committee on Serviceability, 1986).

There are three general types of unserviceability that may be experienced.

Excessive deflections or rotation that may affect the appearance, functional use, or drainage of the structure, or may cause damaging transfer of load to elements and attachments not designed to carry load.

Excessive vibrations produced by the activities of building occupants, mechanical equipment, or the wind, which may cause occupant discomfort.

Deterioration, including weathering, rotting, and discoloration.

The response of the structure to service loads normally can be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under service loads may require examination with respect to this long-term behavior. Service loads used in analyzing creep or other long-term effects may be significantly less than those used to analyze elastic deflections or other short-term or reversible structural behavior.

Serviceability limits for a specific building can be arrived at only after a careful analysis of all functional and economic requirements and constraints by the building owner, engineer, and architect. Building occupants are able to perceive structural deflections, motion, cracking, or other signs of possible distress at levels that are much lower than those that would indicate that structural failure was impending. Such signs of distress may be taken as an indication that the building is unsafe and diminish the commercial value of the property.

**Short-term Deflection (Static).**

*Vertical.* Historically, common deflection limits for horizontal members have been 1/360 of the span for floor subjected to full nominal live load and 1/240 of span for roof members. Placing a limit on deflection in terms of fraction of span is tantamount to limiting the member curvature or elastic flexural strain, which may explain the close relation between such limits and the onset of nonstructural damage to attachments. Deflections of about 1/300 of the span (for cantilevers, 1/150 of length) are visible and may lead to general architectural damage or cladding leakage. Deformations greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

In certain instances, it may be necessary to place a limit (independent of span), typically 10–12 mm (approx. 0.5 in.), on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements (ISO 4356). For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about 10 mm (3/8 in.) unless special provision is made for differential movement (Cooney and King, 1988).

Load combinations for checking static deflections of floor beams can be developed using reliability analysis (Galambos and Ellingwood, 1986). Duration-of-load (time) effects can be disregarded in the serviceability analysis. Assuming that the current deflection limits represent satisfactory performance of floors with regard to visually objectionable sagging and local nonstructural damage, the limit state for excessive deflection of a uniformly loaded beam is,

$$\Delta_{max} - \frac{kw_L \ell^4}{EI} = 0 \qquad \text{(C10.1-1)}$$

in which $\Delta_{max} = \ell/360$ or $\ell/240$, as appropriate, k = factor reflecting end restraint conditions, $\ell$ = span, EI = flexural rigidity, and $w_L$ (plf) = s·L, in which L is live load (psf) and s = beam spacing. The current serviceability check is

$$\frac{kw_{Ln} \ell^4}{(E'I)} < \Delta_{max} \qquad \text{(C10.1-2)}$$

in which E′I = mean flexural rigidity adjusted to end-use conditions and $w_{Ln} = sL_n$, in which $L_n$ in this example is nominal live load (in ASCE 7-93). C10.1-2 can be solved for the required moment of inertia, I; substituting this I in C10.1-1 and rearranging leads to the limit state,

$$\frac{E}{E'} - \frac{w_L}{w_{Ln}} = 0\,. \qquad \text{(C10.1-3)}$$

Note that this limit state is independent of span, $\ell$, $\Delta_{max}$, and k, provided that the end restraint conditions assumed in the nominal deflection check closely model the actual conditions. This analysis does not consider specifically the load-sharing effects within a floor system, although such effects could be included as part of the factor, k.

The modulus of elasticity, E, and live load, $w_L$, are random. Current practice is to allow the adjusted mean of E′, to be used in checking deflections; thus, E = 1.0E′. The coefficient of variation ($V_E$) in E is about 0.20 for many grades and species of dimension lumber. For glued-laminated beams, $V_E$ is about 0.10. The load statistics are different from those used for the ultimate limit states because the event horizon in a serviceability analysis is substantially shorter than the 50 to 100 years that is considered in safety analysis. On an annual basis, $L_m$ (the mean value of the maximum live load) = $0.47L_n$ and $V_L$ = 0.58 for the total live load, sustained plus transient components on influence areas less than 37 $m^2$ (400 $ft^2$), this live load would have a duration of a few days or less. If one were concerned with deflection due to creep or other long-term effects, only the sustained live load would be of interest because of the short duration of the transient component. For the sustained live load, $L_m = 0.24L_n$ and $V_L$ = 0.60.

Using the above load statistics, the serviceability reliability index, β, associated with current practice (on an annual basis) is 1.5 considering the total live load and is about 2.9 considering only the sustained live load. A similar analysis performed for roof systems subjected to snow load showed that for annual maximum snow load on a roof, β = 2.4. These reliability indices are insensitive to the variability in stiffness, which typically is much less (0.1–0.2) than the variability of load (0.6 or more). They imply that serviceability limit state probabilities range from about 0.002 to 0.07 on an annual basis. Thus, there is a non-negligible probability of serviceability failure using current practice.

Load combinations for checking static deflection serviceability limit states can be devised using the above reliability analyses as a basis. There is little evidence to suggest that current deflection guidelines are inadequate in most buildings; thus, the above analysis indicates that loads corresponding to an annual probability of 0.05 of being exceeded would be appropriate. For serviceability limit states involving gravity loads having a short-term effect on the structure,

$$D + L \qquad (C10.1\text{-}4)$$

$$D + 0.5\,S \qquad (C10.1\text{-}5)$$

in which D, L, and S are the nominal dead, live, and snow loads (in ASCE 7-93) (the subscript n has been dropped for convenience).

For serviceability limit states involving long-term or permanent effects,

$$D + 0.5\,L \qquad (C10.1\text{-}6)$$

***Lateral (drift).*** Lateral deflections of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control, 1988). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions.

No standard practice exists for selecting the wind loads for checking lateral deflections; these may range from a simple uniform load of 20 psf (0.96 kPa) on the windward face of the building to the nominal (unfactored) wind load from ASCE 7-93. The use of the design (factored) wind load in checking serviceability is excessively conservative. An appropriate load combination for checking short-term effects might be,

$$D + 0.5\,L + 0.7\,W \qquad (C10.1\text{-}7)$$

obtained using a procedure similar to that used to derive C10.1-4, 5, and 6. Due to its transient nature, wind load need not be considered in analyzing the effects of creep or other long-term actions.

Deformation limits should apply to the structural assembly as a whole. The stiffening effect of nonstructural walls and partitions may be taken into account in the analysis of drift if substantiating information regarding their effect is available. Where load cycling occurs, consideration should be given to the possibility that increases in residual deformations may lead to incremental structural collapse.

**Short-term deflection (dynamic)**

Structural motions of floors or of the building as a whole can cause discomfort for the building occupants. Traditional static deflection checks are not sufficient to ensure that annoying vibrations of building floor systems or buildings as a whole will not occur (Committee on Serviceability Research, 1986). While control of stiffness is one aspect of serviceability, mass distribution and damping are also important in controlling vibrations. The use of new materials and building systems may require that the dynamic response of the system be considered explicitly. Simple dynamic models usually are sufficient to determine whether there is a potential problem and to suggest possible remedial measures.

In general, excessive structural motion is prevented by measures that limit building or floor accelerations to levels that are not disturbing to the occupants or do not damage service equipment. Perception and tolerance of individuals to vibration is dependent on their expectation of building performance (related to building occupancy) and to their level of activity at the time the vibration occurs (ANSI 3.29, 1983). Continuous vibrations (over a period of minutes) leading to accelerations on the order of 0.005g to 0.01g are annoying to most people engaged in quiet activities, whereas those engaged in spectator events may tolerate steady-state accelerations on the order of 0.02g to 0.05g. Thresholds of annoyance for transient vibrations (lasting only a few seconds) are considerably higher and depend on the amount of structural damping present (Murray, 1981). For a finished floor with (typically) 5 to 10% damping, peak transient accelerations of 0.05g to 0.1g may be tolerated.

Wind-induced vibrations are not a serviceability problem for the majority of wood frame structures. The specialized literature should be consulted for those infrequent cases in which wind-induced vibrations may be significant (Committee on Serviceability, 1986).

Floor vibrations due to activities of the building occupants may be a problem in wood construction. Many common human activities impart dynamic forces to a structure at frequencies (or harmonics) in the range of 2 to 6 Hz (Allen and Rainer, 1976; Allen, et al., 1985). If the fundamental frequency of vibration of the floor system is in this range and if the activity is rhythmic in nature (e.g., dancing, aerobic exercise, running or jogging, cheering at spectator events), resonant amplification may occur. Most floor systems in light frame wood construction have fundamental frequencies of vibration in the range 12 to 30 Hz (Polensek, 1975; Atherton, et al., 1976). This fundamental frequency range is well above the excitation frequency of common human activities (e.g., common pedestrian motion imparts forces with harmonics at about 2 and 4 Hz), and thus resonance is unlikely to occur. However, the floor may move in a quasi-static fashion in response to moving personnel loads. The most troublesome characteristic of wood floors in residences seems to be motion and noise produced as people walk about the room, causing the furniture to shake and the contents of cabinets to rattle (Onysko, 1986). Limiting the deflection under uniform live load to 1/360 of span, or other fractions, is of limited effectiveness in dealing with this sort of floor vibration.

In contrast to long-span reinforced concrete or steel floor systems, the response of wood floors is dependent on the people and furniture present, which may represent a significant fraction of the total mass of the vibrating system. Although the damping is difficult to estimate, it typically is higher in wood than in concrete or steel construction as a result of partial composite action of the joist-subfloor-finish floor system and, to a lesser extent, connection behavior. The mean damping of full-scale nailed wood-joist floor systems supporting people may be 12% or more, while without people, it may be 5% or less (Polensek, 1975). Nonstructural partitions and floor coverings also contribute unpredictably to stiffness and damping.

Floor vibration criteria for wood construction can be developed using relatively simple dynamic models (Allen and Rainer, 1976; Ellingwood and Tallin, 1984; Ohlsson, 1988; Smith and Chui, 1988). In a recent study of floor vibration in residences (Onysko, 1986), it was found that the deflection under a concentrated load of 100 kg (981 N or 224 lb) provided the best measure for identifying floors with excessive springiness under occupant movement. These results suggest that a simple and relatively effective way to minimize objectionable vibrations to walking and other common human activities would be to control the floor stiffness, as measured by displacement under concentrated load. The following limit on static deflection under a concentrated load of 1 kN (225 lb) has been proposed (Onysko, 1988):

$$\delta = 7.5/\ell^{1.2} \text{ mm } (1.2/\ell^{1.2} \text{ inches}); \quad \ell > 3 \text{ m } (\ell > 10 \text{ ft}) \qquad \text{(C10.1-8)}$$

$$\delta = 2 \text{ mm } (0.08 \text{ inch}); \quad \ell < 3 \text{ m } (\ell < 10 \text{ ft}) \qquad \text{(C10.1-9)}$$

in which $\ell$ = span. This relation is plotted in Fig. C10-1, along with similar recommendations from other studies. It is emphasized that this deflection check is solely a means for determining that the floor system has sufficient stiffness, and should not be construed to mean that the design load is 1 kN.

Additional justification for limiting the deflection to some absolute value rather than to some fraction of span can be obtained by considering the dynamic characteristics of a floor system modeled as a uniformly loaded simple span. The fundamental frequency of vibration, $f_o$, of this system is given by,

$$f_o = \frac{\pi}{2\ell^2}\sqrt{\frac{EI}{\rho}} \qquad \text{(C10.1-10)}$$

in which EI = flexural rigidity of the floor, $\ell$ = span, and $\rho$ = mass per unit length; $\rho$ = w/g, in which g = acceleration due to gravity (9.8 m/s$^2$),

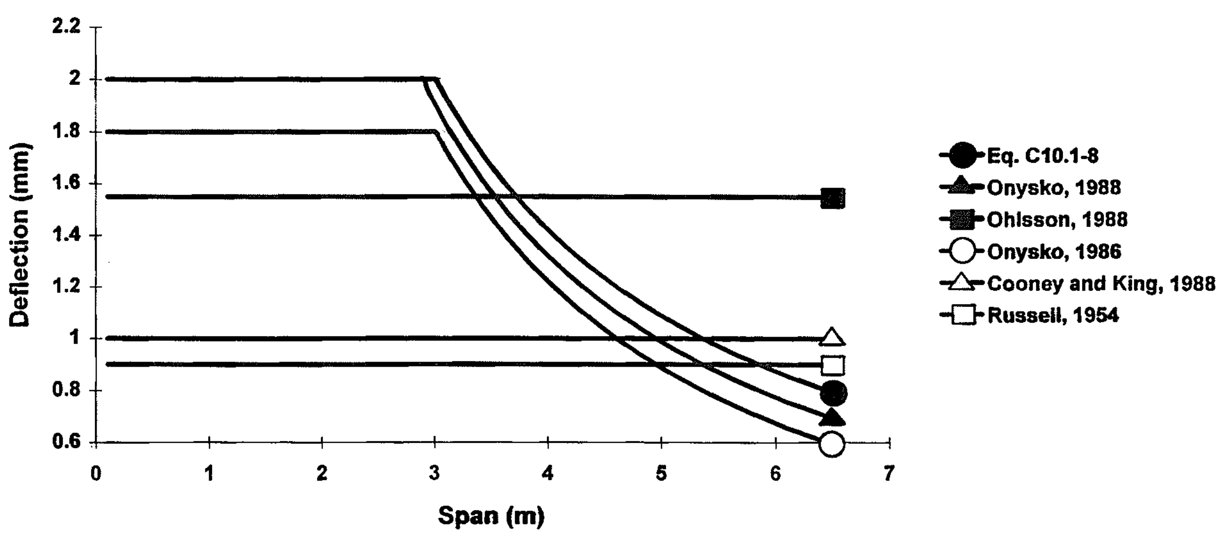



**Figure C10.1-1. Deflection limits for mitigating floor vibration in wood construction.**

and w = dead load plus participating live load. The maximum deflection due to a load, w, is:

$$\delta_{st} = \frac{5\,w\,\ell^4}{384\,EI} \qquad \text{(C10.1-11)}$$

Substituting EI from this equation into C10.1-10, we obtain,

$$f_o = \frac{0.56}{\sqrt{\delta_{st}}} \qquad \text{(C10.1-12)}$$

in which $\delta_{st}$ has the units of meters. If the fundamental frequency of vibration of the floor system is to be kept above about 12 Hz because human sensitivity to vibration decreases above this frequency (ANSI S3.29-1983), Eq. C10.1-11 indicates that the static deflection must be limited to about 2 mm. This limit is consistent with the general requirement in Eq. C10.1-9 and Fig. C10-1; note that it was derived for a uniform rather than concentrated load.

Lateral bracing or load-sharing bridging between the floor joists is an effective way of providing additional effective stiffness to the floor system as a whole under common occupancy (concentrated) loads (Onysko, 1988). The bracing system should allow truss action to develop in the joist-subfloor system. The bracing should be spaced approximately every 2 m on center to be fully effective.

**Long-term Deflection (creep)**

Average stiffness values are intended for the calculation of immediate deformation under load as discussed above. Under sustained loading, wood members exhibit additional time-dependent deformations due to creep, which usually occur at a slow but persistent rate over long periods of time. Creep rates are greater for members that are drying under load or are exposed to varying temperature and relative humidity than for members in a stable environment.

In certain applications, it may be necessary to limit deflection under long-term loading to specified levels. This can be done by applying a creep factor, $\lambda$, to the immediate deflection. It has been a customary practice to use a creep multiplier of 1.5 for glued-laminated timber or seasoned sawn lumber, or 2.0 for unseasoned sawn lumber, when calculating total long-term deflection which includes creep. This limit state should be checked using load combination in Eq. C10.1-6.

## C10.2 Material and Member Stiffness

The use of the mean value of modulus of elasticity has been customary practice for serviceability design of structures made of virtually all construction materials for many years.

## References Commentary Chapter 10

Ad Hoc Committee on Serviceability Research 1986. *Structural Serviceability: A Critical Appraisal and Research Needs.* J. Str. Div., ASCE 112(12):2646–2664. New York, NY.

Allen, D.E. and Rainer, J.H. 1976. *Vibration Criteria for Long-Span Floors.* Canadian. J. Civil Engr. 3(2):165–173.

Allen, D.E., Rainer, J.H., and Pernica, G. 1985. *Vibration Criteria for Assembly Occupancies.* Canadian J. Civil Engr. 12(3):617–623.

American National Standards Institute. 1983. *American National Standard Guide to the Evaluation of Human Exposure to Vibration in Buildings.* ANSI S3.29-1983. New York, NY.

American Society of Civil Engineers. 1993. *Minimum Design Loads for Buildings and Other Structures.* ASCE 7-93. New York, NY.

ASCE Task Committee on Drift Control of Steel Building Structures. 1988. *Wind Drift Design of Steel-Framed Buildings: State of the Art.* J. Str. Div., ASCE 114(9):2085–2108. New York, NY.

Atherton, G.H., Polensek, A., and Corder, S.E. 1976. *Human Response to Walking and Impact Vibration of Wood Floors.* Forest Products J. 26(10):40–47. Madison, WI.

Cooney, R.C. and King, A.B. 1988. *Serviceability Criteria for Buildings.* BRANZ Report SR14, Building Research Association of New Zealand, Porirua, New Zealand.

Ellingwood, B. and Tallin, A. 1984. *Structural Serviceability: Floor Vibrations.* J. Str. Div., ASCE 110(2):401–418. New York, NY.

Ellingwood, B. 1989. *Serviceability Guidelines for Steel Structures.* AISC Engr. J. 26(1):1–8.

Galambos, T.V. and Ellingwood, B. 1986. *Serviceability Limit States: Deflections.* J. Str. Div., ASCE 112(1):67–84. New York, NY.

International Standards Organization. 1977. *Bases for the Design of Structures—Deformations of*

*Buildings at the Serviceability Limit States.* ISO 4356.

Murray, T. 1981. *Acceptability Criteria for Occupant-Induced Floor Vibrations.* AISC Engineering J. 18(1):62–70.

Ohlsson, S. 1988. *Ten Years of Floor Vibration Research—A Review of Aspects and Some Results.* Proceedings, Symposium on Serviceability of Buildings, pp. 435–450. National Research Council of Canada. Ottawa.

Onysko, D.M. 1986. *Serviceability Criteria for Residential Floors Based on a Field Study of Consumer Response.* Forintek Canada Corp. Report. Ottawa.

Onysko, D.M. 1988. *Performance and Acceptability of Wood Floors—Forintek Studies.* Proceedings, Symposium on Serviceability of Buildings, pp. 477–494. National Research Council of Canada. Ottawa.

Polensek, A. 1975. *Damping Capacity of Nailed Wood-Joist Floors.* Wood Science 8(2):141–151.

Russell, W.A. 1954. *Deflection Characteristics of Residential Wood-Joist Floor Systems.* Research Paper No. 30, U.S. Housing and Home Finance Agency, Washington, DC.

Smith, I. and Chui, Y.-H. 1988. *Design of Lightweight Floors to Avoid Human Discomfort.* Canadian J. Civil Engr. 15:254–262.

## COMMENTARY

# Appendix A1 Resistance Of Spaced Columns

### CA1.1 Geometry and Geometry Limits

The restraint by the connected end blocks of a spaced column results in the buckled shape of the column components approaching the full sine wave shape of the fixed end column rather than the half sine wave shape of a simple column without sidesway. Thus, in one direction, the spaced column direction, the column resistance is significantly affected by the spaced column action. In the other direction, the spaced column behaves as adjacent (usually two) solid columns. Because the component widths in the spaced-column direction are usually smaller, often by two to five times, than the width in the solid-column direction, the spaced-column direction often controls the overall member strength despite the restraint provided in the spaced column direction.

The provisions of App. A1 follow closely those of the current NDS provisions (AF&PA, 1991).

Following current design practice, end blocks at least equal in thickness to the column-component members are required. Although some bonuses for thicker end blocks are contained in some codes (e.g., British), no such increase has been historically provided in U.S. codes.

The requirement that spacers be located such that $\ell_3 \leq 0.50\ell$ assures that the buckling of the individual column components between the spacer and end block will be no more critical than the overall column behavior in the spaced-column direction. The maximum length-to-breadth ratios are those historically used in the design of spaced columns.

### CA1.2 Spaced-Column Fixity Conditions

The restraint provided in the spaced-column direction is dependent on the stiffness of the end block connectors and is improved by the placement of these connectors farther from the column end, provided that buckling of the ends of the column over the $\ell_c$ end dimension is precluded. As a simplification, only two categories of end fixity, rather than a continuous variation with the $\ell_{ce}/\ell_1$ ratio, are provided.

The spaced-column effective length factors of 0.63 and 0.58 are equivalent, to two significant digits, to the 2.5 and 3.0 values for $C_x$ in the NDS (AF&PA, 1991). These factors have been expressed as effective length factors to more accurately reflect that the spaced-column action affects the component effective length, not the section stiffness as the use of $C_xE$ in the NDS implies. Note that these $K_e$ values are less (i.e., more end restraint) than the design value given in Sec. 4.2 for a column nominally fixed at both ends. This use of $K_e$ also makes more obvious that the $K_e$ values of Sec. 4.2 should not be used as additional multipliers to further reduce $K_e\ell$.

Provisions for the effective length of spaced columns with sidesway in the spaced-column direction recognize that the minimum $K_e$ for any member with end constraint from any source must have $K_e\ell$ at least equal to $\ell$, and that restraint of at least one end of the entire column assembly is needed to prevent the column from merely rotating about its base as a "pinned-base flagpole."

The restraint of the end blocks does not affect the effective length of the spaced column assembly in the solid-column direction.

### CA1.3 Resistance of Spaced Columns

The continuous (Ylinen) column equation is used for spaced columns designed according to this section. The basing of the spaced-column properties on the least E, I, and/or $F_c'$ of the individual components is a simple and generally conservative approach. A more detailed analysis may be carried out when component properties differ significantly. Such an analysis should consider the effects of any lack of member cross-section symmetry and compatibility requirements arising from different member E and I values.

### CA1.4 Requirements for Connectors in End Blocks

Connectors in the end blocks provide the necessary stiffness of the end block to column connection region. The connector stiffness is determined by the number and type of connectors. Shear plates or split rings are required, because nails, bolts, and other fasteners would not provide sufficient stiffness even if they provided equal strength. Spaced columns with end blocks joined with nails, glue, bolts, and other means are addressed in the British Standard Code of Practice (BSI, 1971), with gluing credited with providing the stiffest connection.

The provisions for connectors in spacers outside of the middle tenth follow from the increasing shear forces the block will experience as it is moved away from the symmetry position at midheight. This provision has the effect of highly encouraging the spacer block to be put in the middle tenth of the spaced column's overall length.

Note that the end block connectors required by section A1.4 provide the stiffness needed to provide component end restraint and thus are not additive to those required for any load-transfer function. The number of connectors needed at a spaced column end is thus the larger of the number required for any force transfer between the end block and column components (as when the end block is part of a beam supported by the spaced column) and the number required by this section.

The species groups are the same as those in the 1991 NDS (AF&PA, 1991). The equations for the end block constant, $C_{eb}$, are equivalent to the tabular values in the 1991 NDS times 2.165. This multiplier is the connector resistance factor, $\phi_z = 0.65$, times 3.33, a multiplier to convert from allowable stress design of the 1991 NDS to connector strength values for LRFD use.

## References

American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC.

British Standards Institution. 1971. Council for Codes of Practice. *British Standard Code of Practice: The Structural Use of Timber.*

# COMMENTARY

# Appendix A2 Glued Laminated Timber (Glulam)

### CA2.1 General

The word *glulam* is a contraction of the term *glued laminated timber* and both are used interchangeably throughout the specification. The design of glued laminated timber follows the same general design procedure used for lumber and sawn timbers. However, because of the unique features of glued laminated timbers, additional design procedures are required. When glulam is used for flexural strength, the outer zones have higher grades of lumber than the inner zones, and a number of combinations of grades have been devised with varying strengths and stiffness.

Many glued laminated beams are tapered, and the effects of tapering must be taken into account. As a result of tapering, it is not possible to set up tables of nominal moments for every depth, and the designer must calculate the adjusted moment resistance, M′, by multiplying the adjusted bending strength, $F_b'$, times the section modulus at the location where the flexural resistance is being calculated. The combination of both tapering and curvature requires special treatment.

The effect of size, depth, width, and length (volume effect) is treated differently than for sawn lumber.

The ability to produce curved members requires additional procedures to account for induced radial stresses and the effect of curvature on flexural stresses.

Fastenings in glued laminated timber are designed in the same manner as for sawn lumber. The combinations of lumber used in a member can vary in species and density. The higher-strength grades are used on the outer top and bottom zones of glued laminated timber, and a lower-strength lumber may be used in the middle. This should be taken into account when designing the fastenings. When timber connectors such as shear plates or split rings are placed in the outer zones, the design values for the higher-strength lumber can be used. When connectors are used in the side faces of the member, the fastening design value associated with the type of lumber used in the core of the member should be used.

## CA2.2 Pitched and Tapered Curved Beams

**CA2.2.1 Moment resistance limited by radial stress.** The geometry of pitched and tapered curved beams causes different distributions of both flexural stresses and radial stresses from that calculated for prismatic members or members of constant cross-section. The method of design presented here is based on theoretical studies, including finite element analysis as well as structural tests of beams. It has been simplified for design by use of factors included in the tables and charts shown herein.

The adjusted moment resistance limited by radial stress is the adjusted moment resistance of a prismatic member divided by a factor, $K_{sr}$, as shown in Eq. A2.2-1. The factor $K_{sr}$ is obtained by determining the constants A, B, and C from Table A2.2-1 for various roof slopes. Interpolation is used for slopes not listed. Also, the factor $K_{gr}$ is obtained by first determining the ratio of total horizontal length of the member to the horizontal length of the curved portion, $\ell/\ell_c$, and using equations in Table A2.2-2.

**CA2.2.2 Moment resistance limited by bending stress.** To account for the effect of geometry on bending stress, the moment resistance limited by bending is determined by modifying the adjusted moment resistance for a prismatic member of the same size by dividing by $K_{sb}$.

The moment resistance limited by bending is then compared with the moment resistance limited by radial tension and the lesser of the two controls.

**CA2.2.3 Deflection of pitched and tapered curved beams.** The deflection can be closely approximated by Eq. A2.2-5. Note that the adjusted mean modulus of elasticity, $E'$, is shown in the equation as is customary in deflection calculation. The load, w, used in the calculation is the actual load, which has not been factored.

**CA2.2.4 Radial reinforcement.** Radial reinforcement commonly consists of long lag screws threaded full length or steel reinforcing rods embedded in an epoxy adhesive. Metals other than steel also may be used if properly designed. The design of radial reinforcement is not included in this specification but shall be performed using a limit states design approach. However, when used, the designer must calculate the radial tension forces involved, and the reinforcement must resist the whole force. Because of shrinkage and other factors, the wood and the steel may not resist the force in unison, and the reinforcement must be capable of resisting all forces. Furthermore, the reinforcement cannot be used to resist more force than obtained by multiplying the area that is being reinforced by the adjusted shear strength of the wood divided by three. To offset effects of shrinkage, the moisture content of the wood at time of fabrication should not exceed 12%.

**CA2.2.5 Adjustment factors.** End-use adjustment factors for glued laminated timbers are $C_M$ factor for moisture, $C_t$ for temperature, and $C_{pt}$ for preservative treatment.

The factors used for calculating the resistance of pitched and tapered curved beams, $K_{sr}$, $K_{gr}$, and $K_{sb}$, are considered part of the calculations rather than adjustment factors.

## CA2.3 Glued Laminated Timber Arches

**CA2.3.1 Types of arches.** For convenience, arches are classified structurally as either two- or three-hinged arches. They are made in many shapes and include names based on shape, such as: tudor, radial, gothic, parabolic, etc. Regardless of shape, they are analyzed structurally as being either two-hinged or three-hinged.

**CA2.3.2 Three-hinged arches.** Three-hinged arches are very popular primarily because the arch can be made in two sections, which facilitates shipping and handling. The connections on smaller arches may include steel plates, connectors, or bolts that are treated as though they were pins. The larger arches are usually designed with true hinge pins.

Three-hinged arches have almost any shape, such as arcs of circles, parabolas, or "tudor"-type arches. Some are also of constant cross-section or tapered. Design concerns for these statically determinate structures include bending combined with compression parallel to grain in amounts that vary along the member and shear near the member ends. The standard design strength equations for glued laminated members apply, except that the volume effect on bending strength is modified, and the interaction provisions for taper cut surfaces (Sec. 5.1.10 and 5.1.11) are not applied.

**CA2.3.3 Two-hinged arches.** Two-hinged arches are slightly more efficient than three-hinged arches. All but the very smallest need to be manufactured in two pieces for shipment and are joined together with a moment splice at the job site. The moment splice is usually located at the midpoint. For very large arches, which need to be shipped in three pieces, the moment splices can be located near the third points. The moment splices should be designed in such a manner that minimizes tension perpendicular to grain. Also, the distance perpendicular to grain between fasteners connecting steel plates to the wood should be limited to prevent the restraint of the rigid side plates from checks or cracks in the wood as it shrinks.

Two-hinged arches are statically indeterminate, and appropriate analysis methods must be used to determine moments, axial loads, and shears at locations along the arch. Once these forces and moments are known, the design is similar to that used for the three-hinged arch.

**CA2.3.4 Axial compressive resistance.** Arches are considered members loaded both in bending and axial compression. Arches are not checked for buckling about the X-X axis. However, they must be checked for buckling about the Y-Y axis if not laterally braced in that direction.

**CA2.3.5 Radial stresses in arches.** The radial force induced in arches is usually radial compression, which seldom controls the design but should be checked. Unbalanced loads and horizontal loads occasionally will induce radial tension. Usually the wood alone is capable of resisting this force. If necessary, radial reinforcement should be used.

**CA2.3.6 Nominal moment resistance.** The nominal moment resistance of arches is determined by the use of Eq. A2.3-1, restated here:

$$M' = M_x' = S_x F_{bx}' C_V. \qquad \text{(A2.3-1)}$$

The volume effect factor $C_V$ is modified so that the effect of length and width is made equal to unity resulting in the equation $C_V = (d/12)^{0.1}$. Experience indicates that using a volume effect factor for depth only is adequate.

Arches generally have compressive forces that tend to diminish the tensile forces on the tension side of an arch subjected to bending forces. If the applied axial compressive stress, $f_c$, is as large or larger than $C_V$ times the adjusted bending strength, $F_b'$, the volume effect is canceled out because it only affects the tension side of a bending member. Thus, the volume effect factor can be taken as unity when Eq. A2.3-2 is satisfied.

When the axial compression force is smaller than $F_b'(1 - C_V)$, the adjusted volume effect factor is computed by Eq. A2.3-3.

**CA2.3.7 Interaction of moment and axial forces in arches.** Arches are assumed to be braced in the Y-Y direction. In addition, the effect of moment magnification in arches in the X-X direction is very small and is not usually taken into account. Therefore, the third term in Eq. 6.3-1 equals zero, and $M_{mx} = M_{bx}$, yielding the simplified form of Eq. A2.3-4, restated here:

$$(P_u/\lambda\phi_c P_o)^2 + (M_{bx}/\lambda\phi_b M_x') \leq 1. \qquad \text{(A2.3-4)}$$

**CA2.3.8 Deflection of arches.** The deflection of arches is caused by elastic deformation under short time loads, creep associated with long time loads, and change in shape caused by transverse shrinkage in the curved portion of the arch.

The elastic or short-time deformation can be calculated using appropriate engineering analysis, such as the method of virtual work or other similar methods. The mean modulus of elasticity is used in the calculations. The long-time deflection caused by creep under dry use conditions is about half of the dead-load deflection. This deflection is added to that calculated for the short time load. The unfactored loads are used in calculating the deflection.

The deflection attributable to change in the transverse dimension caused by moisture content change is downward when shrinkage occurs, as is the usual case for dry conditions of use.

For a Tudor-type three-hinged arch, the deflection caused by shrinkage can be approximated closely by Eq. A2.3. Because $d_q$ is very small, $\theta_q$ can be taken as $-d_q$ with small error. The minus sign indicates a downward deflection movement.

As stated in the main text, the average of radial shrinkage and tangential shrinkage is recommended for calculating the deflection.

# COMMENTARY

# Appendix A3 Ponding

## CA3.1 Scope

Ponding results when rain gathers on a flat or nearly flat roof and causes sufficient deflections in the roof that additional water is held in the deflected shape. Additional water adds more load and more deflection.

Obvious ways to prevent ponding failures include providing adequate slope of the roof to assure drainage and to camber flat roofs so that water will not be retained. It must be noted that these methods are effective only if positive drainage is assured. Parapet walls can retain water and lead to roof failure if roof drains become clogged.

Ponding is both a stiffness and a strength problem; if the roof is too flat to avoid the accumulation of water, the roof system must be adequately stiff to avoid retaining liquid. The stiffness requirements must recognize whether the roof system consists of parallel members in a one-way system, or of a two-way system in which joist or purlin secondary members are supported by larger primary flexural members. In both cases, the deflection of the roof sheathing is ignored, which is equivalent to assuming that the additional ponding volume resulting from the sheathing deflections is negligibly small relative to that from the primary and secondary flexural members.

## CA3.2 One-Way Roof Systems

The provisions of this section are limited to roof systems consisting of parallel primary flexural members spaced closely enough that the roof sheathing deflections are small relative to the deflection at midspan of the primary roof members that directly support this roof sheathing. These provisions follow those proposed by Zahn (1988) and Zahn and Murphy (1988).

**CA3.2.1 Minimum slope to drain.** The expression for the minimum roof slope to assure drainage, Eq. A3.2-1, consists of two parts—the end rotation of an initially flat simply supported member supporting its factored roof load and a second term including the effects of initial sag or camber. The expression gives the requirement for zero slope at the low-end member (assuming that the roof members run in the direction of the roof slope). The ponding load is assumed to be uniformly distributed, whereas in fact the ponding liquid would likely follow the deflected member shape more in a flat-roof system. This unconservative load assumption is effectively offset by applying the creep factor, which recognizes the long-term flexural deflections caused by dead and other sustained loads, to the entire load, including rain loads. The creep factors are those from the 1991 NDS (AF&PA, 1991).

The use of 1.2D + 1.2P for ponding, rather than the load 1.6R term in Eq. 1.3-3, follows from the ponding load provisions in section 2.4.3 of ASCE-7 (ASCE, 1993).

**CA3.2.2 Increased moment caused by ponding.** If the minimum slope given by Eq. A3.2-1 is not provided, then a moment magnification factor, $K_{rp}$, should be computed by Eq. A3.2-2. Because this equation is based on an initially flat (with dead-load deflection ignored) roof member, no credit is given for initial slope and/or camber if not sufficient to meet Eq.A3.2-1.

Equation A3.2-3 gives the maximum beam spacing (contributing ponding load to a single beam) providing a beam stiffness that is adequate to spill water rather than to accumulate water. The equation is formulated in this manner to provide a familiar "$1 - P/P_{critical}$" type of magnification factor. In design, the engineer would likely recast Eq. A3.2-3 into the following form:

$$K_{rp} = \frac{1}{1 - \dfrac{\lambda_{cr}\, S_p\, L_p^4\, (\rho/1728)}{\phi_s\, \pi^4\, E'_{05}\, I_x}} \qquad \text{(A3.2-1)}$$

which algebraically reduces to:

$$K_{rp} = \frac{1}{1 - \dfrac{\lambda_{cr}\, S_p\, L_p^4}{2300\, E'_{05}\, I_x}} \qquad \text{(A4.2-2)}$$

Note that this equation is similar to Eq. 5-102 in TCM-94, with differences being the explicit use of

$\lambda_{cr}$ and $E_{05}'$ (discussed in the TCM text), and in a difference in default units (all inches and pounds here).

As in other chapters of the standard, in the absence of other guidance, $E_{05}'$ may be taken as $1.03 \cdot (1 - 1.645 \cdot COV) \cdot E'$ for all products. Default values for COV are typically 0.11 (MSR lumber, glulam, structural composite lumber), 0.15 (machine evaluated lumber) and 0.25 (other products). This results in $E_{05}'$ values equal to 0.84E′, 0.78E′, and 0.61E′, respectively, for the three product groups.

Equation A3.2-4 adds the ponding-related term to the similar axial load-related term in the magnification factor for flat and near-flat upper-chord truss members, members that act in combined compression and bending.

### CA3.3 Two-Way Roof Systems

The minimum member stiffness criteria for two-way roof systems follow from the provisions in the AISC LRFD Specification (AISC, 1994), which in turn follow from Marino (1966). Eq. A3.3-1 is a simplified expression giving a generally conservative stiffness criteria. More complete and refined stiffness criteria are given in the K2 Appendix and Commentary to AISC (1994).

The $B_p$ and $B_s$ (p = primary roof member, s = secondary roof member) are the $C_p$ and $C_s$ factors of section K2, AISC 1994, modified to include the $E_{05}$ value for wood (rather than the assumed constant 29,000,000 psi for steel), the specification of spans and spacing in inches, and the creep factor $\lambda_{cr}$. The constants in these expressions have been scaled so that Eq. A3.3-1 has the same right side value as the corresponding expression in AISC 1986; this facilitates the application of the appendix and commentary content of AISC 1994 to two-way roof systems using engineered wood construction.

## References


American Forest & Paper Association. 1991. *National Design Specification for Wood Construction.* AF&PA. Washington, DC, 125 p.

American Institute of Steel Construction. 1994. *Load and Resistance Factor Design Specification for Structural Steel Buildings,* Second Edition. Chicago, IL.

American Society of Civil Engineers. 1993. *Minimum Design Loads for Buildings and Other Structures.* ASCE 7-93. New York, NY.

Marino, F.J. 1966. *Ponding of Two-Way Roof Systems.* AISC Engineering Journal. American Institute of Steel Construction, Chicago, Illinois, July 1966.

Zahn, J.J. 1988. *Combined Load Stability Criterion for Wood Beam-Columns.* Journal of Structural Engineering. ASCE, Vol. 114, No. 11, November 1988, pp. 2612–2628.

Zahn, J.J. and J.F. Murphy. *Reliability Analysis of Ponding Collapse of Flat Roofs.* Proceedings, 1988 International Conference on Timber Engineering. September 19–22, 1988, Seattle, Washington, Forest Products Research Society, Madison, WI, Vol. I, pp. 151–155.


# COMMENTARY

# Appendix A4 Qualification Of Fasteners and Connectors

### CA4.1 General

Standardization of fasteners and connectors used in wood construction is an evolving field. While some types of connectors are produced and distributed in strict accordance with applicable standards, other types have little such documentation by the time they reach the job site. This appendix provides guidance on standardization issues for the various fastener types.

### CA4.2 Nails and Spikes

This category of fastener has the lowest level of standardization of the common fastener types. The appendix provides the requirement for demonstrated ductile behavior based on the observation that ductility appears to be the most important engineering characteristic of this type of fastener.

### CA4.3 Wood Screws

The discussion of this type of fastener is similar to the previous one, with the addition of a reference standard.

### CA4.4 Bolts, Lag Screws, Drift Pins, and Dowels

These fasteners have the broadest range of relevant standards, and are often specified and marketed on the basis of their reference standard or grade designation.

### CA4.5 Split Rings

Tests of these fasteners shows that both steel quality and fabrication quality impact the strength of these connections. The states requirements are intended to guide the user toward proper specification of these fasteners.

### CA4.6 Shear Plates

Same issues as for split rings, above.

## COMMENTARY

# Appendix A5 Resistance Of Shear Plates Or Split Rings In End Grain

### CA5.1 Definitions and Notations

The definitions and notations in this LRFD Standard were developed to match, as closely as possible, the provisions of the National Design Specification (NDS).

### CA5.2 Design Basis

This section points out the limitation in the general equations and instructs the user where to obtain information for nonstandard design cases.

### CA5.3 Connectors Installed in Square-Cut or Sloping Surfaces

Shear plates and split rings provide the designer with many alternatives with respect to fastener placement relative to the member and to the applied loads. The terminology in this section is rather complex, but is needed to fully define all relevant design cases. The terminology is consistent with the NDS.

### CA5.4 Spacings

This list of design conditions is provided to further clarify to the user the appropriate equations for a given joint configuration.

## COMMENTARY

# Appendix A6 Design Of Panel-based Assemblies

### CA6.1 Scope

The performance of panel-based assemblies depends on two major factors:

(a) quality of the components; and
(b) integrity of the joints.

Because it is difficult to produce dependable joints under various field conditions, the provisions in this standard are limited to panel-based assemblies manufactured in controlled factory environments within the parameters of an ongoing quality-assurance program.

### CA6.2 Components

The panel-based assemblies included in the standard are those that are manufactured from structural-use panels and structural framing meeting requirements of this standard.

### CA6.3 Fabrication

Fabrication of panel-based assemblies requires use of adhesives that comply with adhesive specifications. In general, interior-type adhesives shall conform with ASTM Specification D3024 or D4689. Exterior-type adhesives shall conform with ASTM Specification D2559. Adhesives conforming to APA Specification AFG-01 may be required for specific assemblies or by certain regulatory agencies.

### CA6.4 End Joints

For special applications, especially when using mechanical connectors, the designer may need to consider the deformation-limit state of these joints.

## CA6.5 Design Procedure

The design equations for assemblies generally account for load sharing among the individual components. Preliminary computations such as location of neutral axis, modified section moduli, weighted statical moments, etc., are required for terms in the design equations.

Panel-based assemblies should be designed according to the latest available knowledge on the methodology of design. Up-to-date design references are available in various technical literature. Detailed design information is also available in various APA publications (listed in the references section of this commentary).

## CA6.6 Deflection Limitations

The designer must establish the specific deflection requirements from the vertical deflection limits specified in the governing code.

For panel-based assemblies, deformation caused by shear can be significant, and the designer is advised to calculate both the shear and flexural deflection components to determine actual design deflection of a panel-based assembly.

More restrictive limitations may be required for special conditions, such as for support of vibrating machinery or for beams over larger windows.

Camber may be provided for purposes of appearance or utility. In general, required cambers have no effect on strength or actual stiffness of panel-based assemblies.

Where roof and floor panel-based assemblies are cambered, a recommended amount is 1.5 times the deflection caused by dead load only. This will provide a nearly level beam under conditions of no live load after set has occurred.

Additional camber may be introduced as desired to provide for drainage or appearance. Roof members must be designed to prevent ponding of water. This may be done either by cambering or by providing a slope or increased stiffness such that ponding will not occur. For more details on ponding, see App. A3 of this standard.

## CA6.7 I-beams

I-beams represent one class of panel-based assemblies. In general, the webs are manufactured from plywood, oriented strandboard or composite panels, whereas the flanges may be made of structural framing or structural-use panel material.

Figure CA6.7-1 illustrates cross-sections of typical I-beams. They may be of uniform cross-section or of different cross-sections at locations where the stresses are higher. The cross-section may also be different at locations where stiffeners are used.

Because several possible failure modes exist for an I-beam, a designer must consider all failure possibilities. Section A6.7 of the standard lists these failure modes.

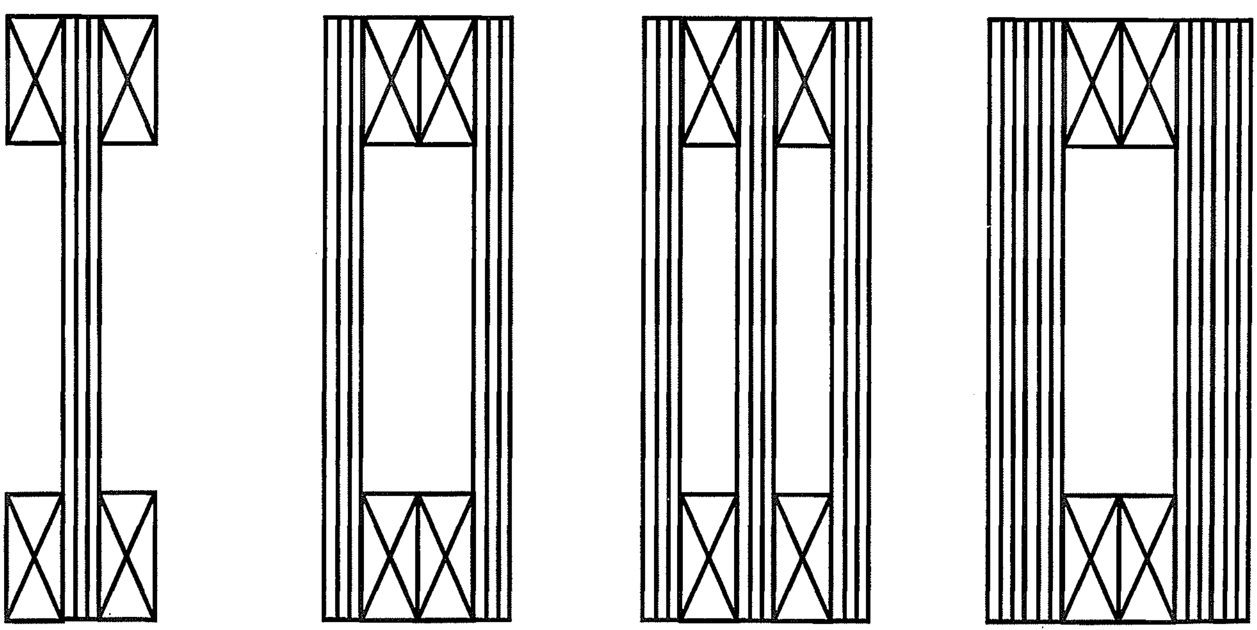


**Figure CA6.7-1. Typical cross sections of I-beams.**

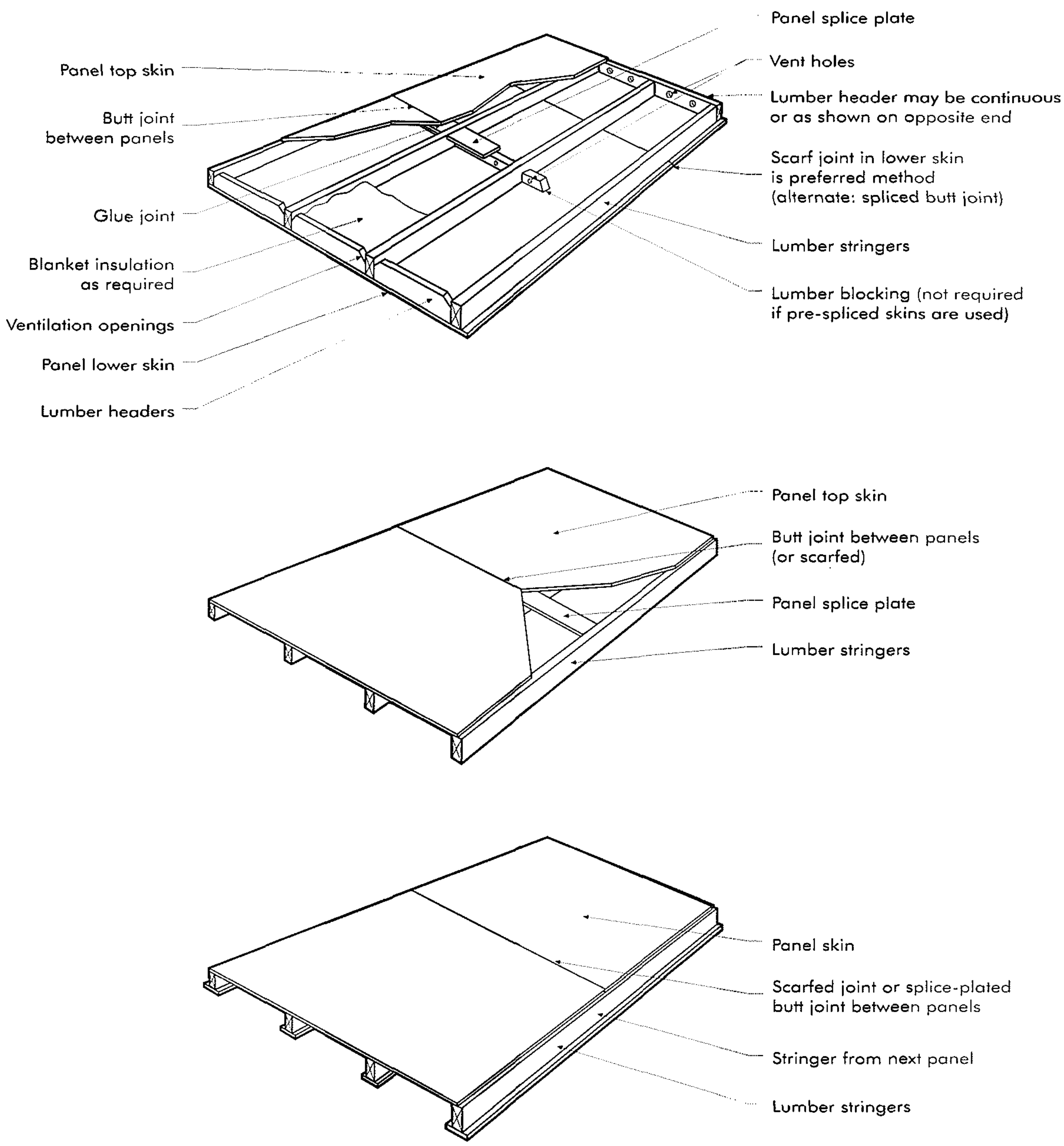



**Figure CA6.8-1. Types of stressed-skin panels.**

## CA6.8 Stressed-Skin Panels

There are two main categories of stressed-skin panels: balanced construction and unbalanced construction. One type of balanced construction and three forms of unbalanced construction are most common. These are:

(a) two-sided stressed-skin panel with identical top and bottom skins;
(b) two-sided stressed-skin panel with the top and bottom skins that are dissimilar because of differing thicknesses, number of layers, grades, types of structural-use panels, or other factors related to mechanical property differences;
(c) one-sided stressed-skin panel; and
(d) T-flanged stressed-skin panel in which the top skin is continuous but the bottom of the panel consists of discontinuous lumber flanges glued to the bottoms of the stringers.

Stressed-skin panels act as a series of fused I-sections to form a panel capable of offering structural resistance as well as performing a sheathing function. Composite action formulas for computing deflection levels and resistances are used to ensure acceptable behavior.

The construction details of the three most common stressed-skin panels are illustrated in Fig. CA6.8-1. Although all major possible components are shown, often one or more of these may not be present. End joints and blanket insulations may not be included in some stressed-skin panels. Furthermore, top and bottom skins may be identical or different.

As with other panel-based assemblies, the possibility of multiple failure modes exists. In considering these failure modes, it helps to visualize a stressed-skin panel as a series of I-sections or T-sections connected by a top skin and sometimes also by a bottom skin. Because of the continuity of the skin from beam to beam, the design limit states also include strength and deflection limitations across the width of the panel, which most often represents the secondary axis direction.

Because loading of a stressed-skin panel is usually through its top skin, transverse strength and deflection limitations seldom need to be considered for the bottom skin. Removal of these limit states allows the optimization of the entire stressed-skin panel by specifying a thinner and/or lower-grade panel for the bottom skin relative to the top skin.

## CA6.9 Sandwich Panels

A structural sandwich panel is an assembly consisting of a lightweight core securely laminated between two relatively thin, strong structural-use panel facings. The facings are plywood, oriented strandboard, or composite panels, and the core material is generally a foam such as polystyrene, polyurethane, or paper honeycombs.

The structural design of a sandwich panel may be compared with that of an I-section and follows

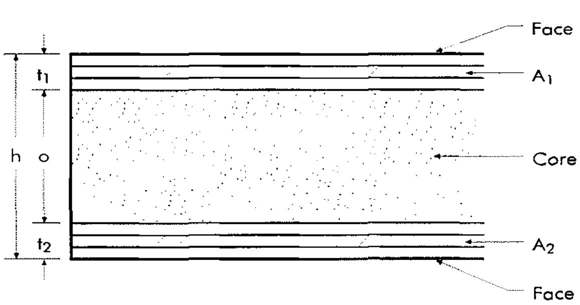



**Figure A6.9-1. Cross section of a plywood-faced sandwich panel.**

the design of a stressed-skin panel. However, because the flexibility of the core material additional failure modes such as shearing, buckling, or wrinkling of the skins might occur. Consequently, modifications to the composite-action formulas used for stressed-skin panels must be made.

Load resistance of sandwich panels is determined in similar fashion to that of stressed-skin panels. The faces of the sandwich panel represent the stressed-skins, whereas the core serves as a spacer and resists shear.

Figure CA6.9-1 shows the cross-section of a plywood-faced sandwich panel. The top face may be of a different thickness, $t_1$, than the bottom face, $t_2$. The overall sandwich panel thickness, t, is the sum of the face thicknesses and the core thicknesses, $t_c$. The faces are bonded to the core by an adhesive.

Structural-use panels are ideal for the facings of sandwich panels. They are strong, light in weight, easily finished, dimensionally stable, and easily repaired if damaged. A variety of core materials may be used. Among these are polystyrene foams, polyurethane foams, and paper honeycombs. In addition to resistance to shearing forces, for some applications such as exterior wall panels and roof panels, the core should possess high resistance to heat and vapor transmission. The designer may need to consider the suitability of the core material to a particular application. Factors to consider include resistance to degradation by heat, age, and moisture, compatibility with adhesives, etc.

The design limit states considered for sandwich panels depend on the type of loading. The three primary load types commonly encountered include:

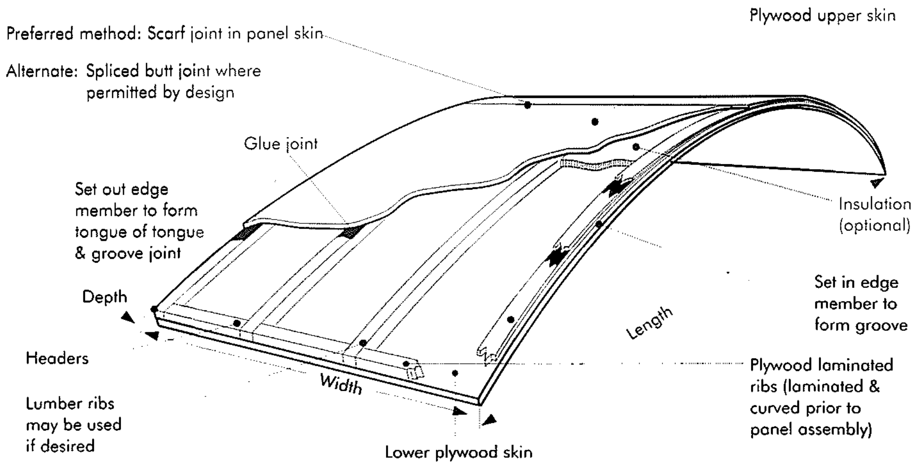
(a) axial compression;
(b) flatwise bending; and
(c) combined axial and flexural loading.

In axial compression, buckling limitations of the panel are of primary significance. In flatwise bending the faces are designed primarily to resist the tension and compression stresses, whereas the core is designed for shear resistance.

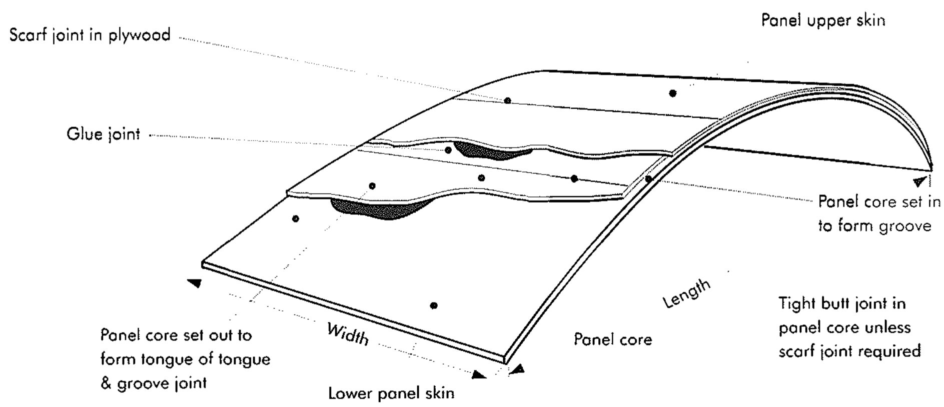
The interfaces between the faces and the core must resist shear. This also requires the design of the faces for planar shear resistance. Insufficient bonding or a low stiffness core could cause the buckling of the faces (skins), which also needs to be designed as one of the several limit states.

Combined axial and bending loading could cause the sandwich panel to fail either by the failure modes of axial loading or the failure modes of

## Curved Structural-Use Panel




## Solid Structural-Use Panel Core




## Light Sandwich Core

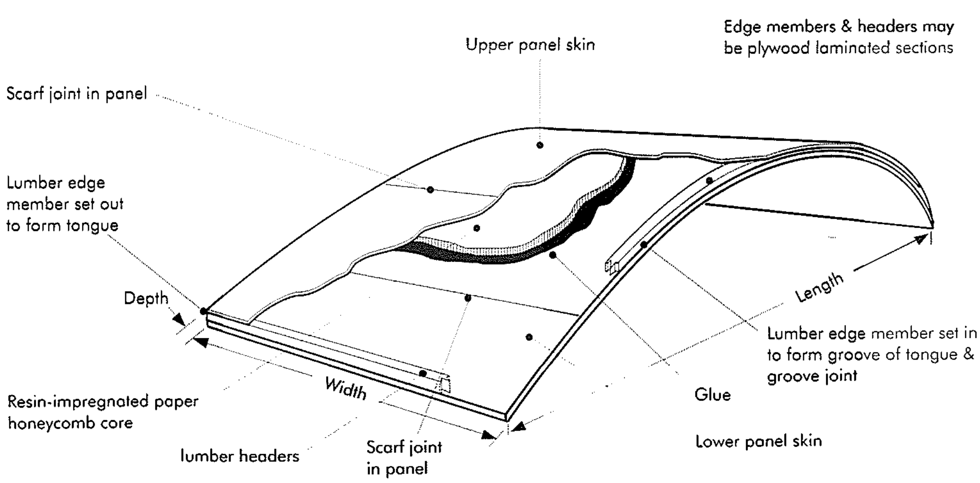



**Figure CA6.10-1. Typical types of curved panel assemblies.**

bending. Thus, for this case, both sets of limit states need to be evaluated.

### CA6.10 Curved Panels

There are two basic types of curved panels: curved flexural panels and arch panels. Curved flexural panels are defined as panels that act as simple beams similar to conventional flat panels. The supporting structure provides for free horizontal deflection; consequently, horizontal thrust is not a design consideration. Generally, the panels are relatively thick, and the radius of curvature affects stresses only slightly.

Arched panels develop thrust. Arches that are continuous from support to support are called two-hinged arches and are statically indeterminate. Two-arch segments may be joined together to form a three-hinged arch, which is statically determinate if the joint is not a moment resisting connection.

The curved panels can be sandwich or stressed-skin types. Generally, medium curvature (span-to-rise ratios of 3 to 8) is most practical.

The design of curved panels is essentially the same as that for flat panels. The effect of panel curvature, however, elevates flexural stresses and introduces radial stresses. Thus, the basic panel resistance must be modified for panel curvature, and the radial stress must be checked. In addition, deflection calculations are performed by different methods than those for flat panels. Connections to the supports and between panels require special attention.

Curved panels can be constructed either as stressed-skin panels or as sandwich panels. The major distinction is the degree of curvature built into the curved panels.

Figure CA6.10-1 illustrates the three most common curved panel constructions: showing a ribbed panel, a multilayer plywood panel, and a sandwich panel with lumber header and edge members added for protection.

The more complex a panel-based assembly, the higher the possibility that multiple failure modes exist. As a consequence, the number of design-limit states is also increased.

For a curved panel, depending on its construction, either all the stressed-skin panel limit states or all the sandwich panel limit states have to be considered by the designer. However, additional stresses are built into a curved panel during its manufacture because straight components are bent to attain the required curvature of the panel.

In addition to the built-up stresses, a curved panel, when loaded as a two-hinged arch or as a three-hinged arch, develops additional radial stresses and horizontal deformations or reaction forces. These additional stresses and deformation further complicate the design of curved panels, requiring a check of additional limit states beyond those required for stressed-skin panels or sandwich panels.

## References

American Plywood Association. 1991. *Performance Standards and Policies for APA Structural-Use Panels*. APA. Tacoma, WA.

American Plywood Association. 1993. *Plywood Design Specification*. APA. Tacoma, WA.

American Plywood Association. 1990. *Plywood Design Specification Supplement One—Design and Fabrication of Plywood Curved Panels*. APA. Tacoma, WA.

American Plywood Association. 1990. *Plywood Design Specification Supplement Two—Design and Fabrication of Plywood - Lumber Beams*. APA. Tacoma, WA.

American Plywood Association. 1990. *Plywood Design Specification Supplement Three—Design and Fabrication of Plywood Stressed - Skin Panels*. APA. Tacoma, WA.

American Plywood Association. 1990. *Plywood Design Specification Supplement Four—Design and Fabrication of Plywood Sandwich Panels*. APA. Tacoma, WA.

American Plywood Association. 1989. *Plywood Design Specification Supplement Five—Design and Fabrication of All-Plywood Beams*. APA. Tacoma, WA.

American Plywood Association Specification AFG-01. *Adhesives for Field-Gluing Plywood to Wood Framing*. APA. Tacoma, WA.

American Society of Civil Engineers. 1987. *Evaluation, Maintenance and Upgrading of Wood Structures, A Guide and Commentary*. ASCE Committee on Wood, New York, NY.

American Society for Testing and Materials. 1984. *Adhesives for Structural Laminated Wood Products for Use Under Exterior (Wet Use) Exposure*

*Conditions*. ASTM D2559-84. ASTM. Philadelphia, PA.

American Society for Testing and Materials. 1984. *Protein-Based Adhesives for Structural Laminated Wood Products for Use Under Interior (Dry Use) Exposure Conditions*. ASTM D3024-84. ASTM. Philadelphia, PA.

Bodig, J. and B.A. Jayne. 1982. *Mechanics of Wood and Wood Composites*. Van Nostrand Reinhold, Co., New York, NY.

U.S. Department of Commerce. 1994. *Construction and Industrial Plywood. PS 1-94*. Washington, DC.

U.S. Department of Commerce. 1992. *Performance Standard for Wood-Based Structural-Use Panels. PS 2-92*. Washington, DC.

USDA Forest Products Laboratory. 1987. *Wood Handbook-Wood as an Engineering Material.* Agricultural Handbook 72, USDA, Forest Service, Forest Products Laboratory, Madison, WI.

# INDEX