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CSA O86:19
National Standard of Canada

Engineering design in wood

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Δ Preface

This is the eleventh edition of CSA O86, *Engineering design in wood*. It is presented in limit states design (LSD) format and supersedes the previous editions published in 2014, 2009, 2001, 1994, 1989, 1984, 1980, 1976, 1970, and 1959, including their Supplements.

Editions of CSA O86 published in 1959, 1970, 1976, 1980, and 1984 were all developed using working stress design (WSD) theory. The last WSD version, CSA CAN3-O86-M84, *Engineering design in wood (working stress design)*, existed concurrently with the first (1984) and second (1989) LSD versions, *Engineering design in wood (limit states design)*. The WSD version was withdrawn on publication of the 1994 LSD edition.

Three LSD editions were published in 1984, 1989, and 1994 with the CSA designation O86.1. Supplements to each of these editions were published in 1987, 1993, and 1998, respectively. Although the 2001 edition was also based on the LSD method, the O86 designation was reinstated.

Significant changes in this edition include the following:

- editorial changes and some reformatting and restructuring of all clauses for clarity of layout, especially Clauses 11 and 12;
- revisions to design provisions for vibration serviceability of joisted floor systems;
- revisions to add Structural Face-Glued Lumber grade-stamped NLGA SPS 5 and SPS 6 wood products, and Face-Glued/Finger-Joined Lumber "Vertical Stud Use Only" grade-stamped NLGA SPS 5 and SPS 6 wood products;
- modification of Table 6.6 to revise specified strengths for Hem-Fir beams and stringers;
- modification of Table 6.8 to include 38 mm × 64 mm (2 in × 3 in) dimension under the Tension Specified Strengths;
- modification of Table 9.3 to add W24 rated oriented strand board (OSB);
- revisions to clarify shear resistance of members with notches;
- revisions to the K_{Zc} factor for sawn lumber;
- revisions to the application of system factor, K_H;
- corrections to the limit of the length of the notch on the compression side;
- updates to cross-laminated timber (CLT) terminology and symbols;
- deletion of shear form factor for CLT;
- revisions to maximum nail diameter for shearwalls, and new references for nails;
- revisions to diaphragm configuration diagrams;
- revisions to N_u for shearwall and diaphragm;
- revisions to wood member thickness for nail and wood screw joints;
- revisions to Clause 11.9 for the design of CLT shearwalls and diaphragms for platform-type construction;
- revisions to requirements and added references to dowels in various clauses;
- addition of Table A.22 for ASTM nails; and
- editorial revisions to Annex B, on fire resistance of large cross-section wood elements for clarity.

This Standard was prepared by the Technical Committee on Engineering Design in Wood, under the jurisdiction of the Strategic Steering Committee on Construction and Civil Infrastructure, and has been formally approved by the Technical Committee.

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This Standard has been developed in compliance with Standards Council of Canada requirements for National Standards of Canada. It has been published as a National Standard of Canada by CSA Group.

Notes:

1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*
2) *Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users of the Standard to judge its suitability for their particular purpose.*
3) *This Standard was developed by consensus, which is defined by* CSA Policy governing standardization — Code of good practice for standardization *as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this Standard.*
4) *To submit a request for interpretation of this Standard, please send the following information to inquiries@csagroup.org and include "Request for interpretation" in the subject line:*
   a) *define the problem, making reference to the specific clause, and, where appropriate, include an illustrative sketch;*
   b) *provide an explanation of circumstances surrounding the actual field condition; and*
   c) *where possible, phrase the request in such a way that a specific "yes" or "no" answer will address the issue.*

   Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization *and are available on the* Current Standards Activities *page at standardsactivities.csa.ca.*
5) *This Standard is subject to review within five years from the date of publication. Suggestions for its improvement will be referred to the appropriate committee. To submit a proposal for change, please send the following information to inquiries@csagroup.org and include "Proposal for change" in the subject line:*
   a) *Standard designation (number);*
   b) *relevant clause, table, and/or figure number;*
   c) *wording of the proposed change; and*
   d) *rationale for the change.*

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CSA O86:19
Engineering design in wood

1 Scope

1.1 General

This Standard provides criteria in accordance with the *National Building Code of Canada*, for the structural design and evaluation of structures or structural elements made from wood or wood products, including:

a) graded lumber;
b) glued-laminated timber;
c) cross-laminated timber (CLT);
d) unsanded plywood;
e) oriented strand board (OSB);
f) composite building components;
g) shearwalls and diaphragms;
h) timber piling;
i) pole-type construction;
j) prefabricated wood I-joists;
k) structural composite lumber;
l) permanent wood foundations;
m) plank decking;
n) nail-laminated decking; and
o) structural connections.

1.2 Exclusions

This Standard does not include design of timber bridges.

1.3 Design method

This Standard employs the limit states design method.

1.4 Terminology

In this Standard, "shall" is used to express a requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the standard; "should" is used to express a recommendation or that which is advised but not required; "may" is used to express an option or that which is permissible within the limits of the Standard.

Notes accompanying clauses do not include requirements or alternative requirements; the purpose of a note accompanying a Clause is to separate from the text explanatory or informative material.

Notes to tables and figures are considered part of the table or figure and may be written as requirements.

Annexes are designated normative (mandatory) or informative (non-mandatory) to define their application.

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1.5 Units

If a value for measurement as given in this Standard is followed by an equivalent value in other units, the first stated value is to be regarded as the specification.

2 Reference publications

This Standard refers to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA Group

B111-1974 (R2003) (withdrawn)
Wire nails, spikes and staples

G40.20-13/G40.21-13 (R2018)
General requirements for rolled or welded structural quality steel/Structural quality steel

CAN/CSA-O15-15
Wood utility poles and reinforcing stubs

CAN/CSA-O56-10 (R2015)
Round wood piles

CAN/CSA-O80 Series-15
Wood preservation

O112 Series-M1977 (R2006) (withdrawn)
CSA Standards for wood adhesives

O112.6-M1977 (withdrawn)
Phenol and phenol-resorcinol resin adhesives for wood (high-temperature curing)

O112.7-M1977 (withdrawn)
Resorcinol and phenol-resorcinol resin adhesives for wood (room- and intermediate-temperature curing)

O112.9-10 (R2019)
Evaluation of adhesives for structural wood products (exterior exposure)

O112.10-08 (R2017)
Evaluation of adhesives for structural wood products (limited moisture exposure)

O121-17
Douglas fir plywood

CAN/CSA-O122-16
Structural glued-laminated timber

O141-05 (R2014)
Softwood lumber

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O151-17
Canadian softwood plywood

O153-13 (R2017)
Poplar plywood

O177-06 (R2015)
Qualification code for manufacturers of structural glued-laminated timber

O322-15
Procedure for certification of pressure-treated wood materials for use in preserved wood foundations

O325-16
Construction sheathing

S16-19 (in development)
Design of steel structures

S136-16
North American specification for the design of cold-formed steel structural members

S347-14
Method of test for evaluation of truss plates used in lumber joints

S406-16
Specification of permanent wood foundations for housing and small buildings

American Wood Council
Technical Report 10-2018
Calculating the Fire Resistance of Exposed Wood Members

ANSI/APA (American National Standards Institute/APA — The Engineered Wood Association)
ANSI/APA PRG 320-2018
Standard for Performance-Rated Cross-Laminated Timber

ANSI/AWC (American National Standards Institute/American Wood Council)
ANSI/AWC SDPWS-2015
Special Design Provisions for Wind and Seismic 2015 Edition

APEGBC (Association of Professional Engineers and Geoscientists of British Columbia)
Technical and Practice Bulletin 2015
Structural, Fire Protection and Building Envelope Professional Engineering Services for 5- and 6-Storey Wood Frame Residential Building Projects (Mid-Rise Buildings)

ASME (The American Society of Mechanical Engineers)
B18.2.1-2012
Square and Hex Bolts and Screws (Inch Series)

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B18.6.1-1981 (R2016)
Wood Screws (Inch Series)

ASTM International (American Society for Testing and Materials)

A36/A36M-14
Standard Specification for Carbon Structural Steel

A47/A47M-99 (2014)
Standard Specification for Ferritic Malleable Iron Castings

A307-14e1
Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength

A653/A653M-07
Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
Note: *ASTM A653/A653M replaces ASTM A446/A446M. Their chemical and mechanical requirements are the same.*

C1002-18
Standard Specification for Steel Self-Piercing Tapping Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Wood Studs or Steel Studs

C1396/C1396M-17
Standard Specification for Gypsum Board

D5055-16
Standard Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists

D5456-18
Standard Specification for Evaluation of Structural Composite Lumber Products

D5457-17
Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design

D7147-11 (2018)
Standard Specification for Testing and Establishing Allowable Loads of Joist Hangers

F1667-18
Standard Specification for Driven Fasteners, Nails, Spikes, and Staples

Canadian Geotechnical Society

Canadian Foundation Engineering Manual, 4th ed. (2006)

CCMC (Canadian Construction Materials Centre)

Registry of Product Evaluations
Note: *See CCMC website.*

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CWC (Canadian Wood Council)

Concluding Report on Development of Design Procedures for Vibration Controlled Spans Using Engineered Wood Members (1997)

Standard Practice Relating Specified Strengths of Structural Members to Characteristic Structural Properties (2001)

Wood Design Manual including CSA O86 supplemented with commentary material

European Committee for Standardisation

EN 1995-1-2:2004
Eurocode 5: Design of timber structures — Part 1-2: General — Structural fire design

ISO/IEC (International Organization for Standardization/International Electrotechnical Commission)

ISO/IEC 17065:2012
Conformity assessment — Requirements for bodies certifying products, processes and services

NLGA (National Lumber Grades Authority)

Δ *Standard Grading Rules for Canadian Lumber* (2017), including *National Grading Rules for Dimension Lumber*

SPS 1-2017
Special Products Standard for Fingerjoined Structural Lumber

SPS 2-2019
Special Products Standard for Machine Graded Lumber

SPS 3-2017
Special Products Standard for Fingerjoined "Vertical Stud Use Only" Lumber

SPS 4-2014
Special Products Standard for Fingerjoined Machine Graded Lumber (FJ-MGL)

SPS 5-2015
Special Products Standard for Face-glued Fingerjoined Lumber — Vertical Stud Use Only

SPS 6-2015
Special Products Standard for Structural Face-Glued Lumber

NRC (Codes Canada)

National Building Code of Canada, 2020

Structural Commentaries (User's Guide — National Building Code 2015: Part 4 of Division B)

SAE International (Society of Automotive Engineers)

SAE Handbook (2008)

J429-2014
Mechanical and Material Requirements for Externally Threaded Fasteners

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TPIC (Truss Plate Institute of Canada)
TPIC-2019
Truss Design Procedures and Specifications for Light Metal Plate Connected Wood Trusses

Underwriters Laboratories of Canada
CAN/ULC-S101-14
Standard Methods of Fire Endurance Tests of Building Construction and Materials

Other publications
Bulmanis, N. S., Latos, H. A., and Keenan, F. J. 1983. "Improving the bearing strength of supports of light wood trusses", *Canadian Journal of Civil Engineering* 10: 306–312.

Lum, C., Jones, E., and Hintz, B. 1996. *Design of wood trusses for small buildings*. In Proceedings of the International Wood Engineering Conference, Vol. 1. New Orleans.

3 Definitions, symbols, spacing, and dimensions

3.1 Definitions

The following definitions shall apply in this Standard:

Adhesive (or **glue)** — a substance capable of holding materials together by surface attachment for structural purposes.

Analogue member — the line representation of a truss member for the purposes of structural analysis.

Aspect ratio of a shearwall segment — the ratio of the height to the length of the segment.

Axis (direction) —

Major axis (or **major strength direction)** — the direction with the greater strength in bending:
a) in plywood, the direction parallel to the face grain;
b) in OSB, the direction of alignment of the strands in the face layers of the panel; and
c) in CLT, general direction of the grain of laminations in the outer layers.

Minor axis (or **minor strength direction)** — the direction with the lesser strength in bending:
a) in plywood, the direction perpendicular to the face grain;
b) in OSB, the direction perpendicular to the alignment of the strands in the face layers of the panel; and
c) in CLT, the direction perpendicular to the grain of laminations in the outer layers.

Beam — a structural element that primarily resists loads applied laterally to its longitudinal axis.
Note: *For sawn lumber, a category of sawn timber whose larger dimension exceeds its smaller dimension by at least 51 mm; the beam is usually graded for use in bending with the load applied to the narrow face. Grading rules sometimes designate beams as "beams and stringers".*

Board — a piece of lumber that is less than 38 mm in its smaller dimension.

Butt joint — a square joint between the ends of two pieces of wood or panel.

Capacity — in relation to prefabricated wood I-joists or structural composite lumber, the numeric value determined from strength and stiffness test data by calculations specified in ASTM D5055 or

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ASTM D5456, respectively, carried out to characterize strength properties of I-joists or structural composite lumber or their component materials.

Notes:

1) *The term is used in combination with a specific property, such as tensile capacity or shear capacity.*
2) *See **Specified capacity** for the term's application to panel products.*

Certification organization (CO) — an impartial body possessing the necessary competence and reliability to operate a certification system in which the interests of all parties concerned with the functioning of the system are represented, and accredited as such by agencies having a national mandate to accredit certification organizations operating within their countries' borders.

Note: *The Standards Council of Canada (SCC) has a national mandate to accredit certification organizations for operation in Canada.*

CLT grade — a class of CLT panels determined by the combination of grades of laminations in the longitudinal and transverse layers.

Companion load — a specified variable load that accompanies the principal load in a given load combination.

Cross-laminated timber (CLT) — an engineered wood product made of at least three orthogonal layers of graded sawn lumber that are laminated by gluing with structural adhesives and that meets the requirements of ANSI/APA PRG 320.

CLT panel — a fabricated CLT component of any size and any aspect ratio.

Layer — an arrangement of laminations of the same thickness, grade, and species combination laid out essentially parallel to each other in one plane.

Longitudinal — a layer with laminations oriented parallel to the major direction.

Transverse — a layer with laminations oriented perpendicular to the major direction.

Layup — an arrangement of layers determined by the number, orientation, thickness, and grade of laminations.

Density — mass per unit volume. In the case of wood, density is usually expressed as kilograms per cubic metre at a specified moisture content.

Diaphragm — a horizontal or nearly horizontal system that acts to transmit lateral forces to the vertical resisting elements. It also includes horizontal bracing systems.

Dimension lumber — lumber 38 to 102 mm, inclusive, in its smaller dimension.

Documented — having written technical substantiation that the use of a material, design, practice, or construction method satisfies the intent of this Standard.

Dowel — a round mild-steel bar without a head or threads.

Dressed size — the cross-sectional dimensions in millimetres of lumber after planing.

Drift pin — a dowel used to provide a laterally loaded connection between overlapping timbers in structures such as cribwork.

Edge distance — the distance from the edge of the member to the centre of the nearest fastener.

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End distance — the distance measured parallel to the axis of a piece from the centre of a fastener to the square-cut end of the member.

Note: *In the case of a connector, if the end of the member is not square-cut, the end distance is taken from any point on the centre half of the connector diameter drawn perpendicular to the centreline of the piece to the nearest point on the end of the member measured parallel to the axis of the piece (see Figures 12.2 and 12.3).*

Equilibrium moisture content — the moisture content at which wood or wood products neither gain nor lose moisture when surrounded by air at a given relative humidity and temperature.

Factored load — the product of a specified load and its applicable load factor.

Factored resistance — the product of resistance and its applicable resistance factor.

Fibre saturation point — the moisture content at which the cell walls are saturated and the cell cavities are free of water; approximately 25% to 30% moisture content.

Flat truss — a truss in which the slope of the top chord does not exceed 2 in 12.

Girder truss — a truss that is used as a main supporting member for secondary framing systems, such as other trusses, joists, or rafters.

Glue — see **Adhesive**.

Glued-laminated timber (or **glulam)** — the wood product that is made by bonding under pressure graded laminating stock whose grain is essentially parallel and that meets the requirements of CAN/CSA-O122.

Note: *Also referred to as "structural glued-laminated timber".*

Grade — the designation of the quality of a piece of wood.

Hold-down — a connection or a system that is designed to provide the overturning restraint for the lateral-load-resisting system.

Importance factor — a factor applied to snow, wind, and earthquake loads to obtain the specified load and take into account the consequences of failure as related to the limit state and the use and occupancy of the building.

Joist — a piece of dimension lumber 114 mm or more in its larger dimension, intended to be loaded on its narrow face.

Lamination — a thin element of wood of appreciable width and length consisting of one or more pieces that can be joined end-to-end.

Limit state — a condition of a structure in which the structure ceases to fulfill the design function.

Load duration — the period of continuous application of a given load or the aggregate of periods of intermittent applications of the same load.

Lumber —

> **Machine evaluated lumber** — structural lumber that has been graded by means of a non-destructive test and visual grading, conforming to the requirements for machine stress-rated lumber, with the exception that the process lower fifth percentile modulus of elasticity (MOE) equals or exceeds 0.75 times the characteristic mean MOE for the grade.

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Machine stress-rated lumber — structural lumber graded by means of a non-destructive test and visual grading, in accordance with the requirements of CSA O141 and NLGA SPS 2.

Rough lumber — lumber as it comes from the saw.

Sawn lumber — the product of a sawmill not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing mill, and cross-cutting to length.

Structural lumber — lumber in which strength is related to the anticipated end use as a controlling factor in grading or selecting.

Visually stress-graded lumber — structural lumber that has been graded in accordance with the provisions of the NLGA *Standard Grading Rules for Canadian Lumber.*

Lumber sizes — under the metric system
a) rough lumber is designated by its actual dry or green size;
b) dressed dry (S-Dry) lumber is designated by its actual finished size;
c) dressed green (S-Grn) dimension lumber is designated by its anticipated dry size at 19% moisture content;
d) dressed green timber is designated by its actual green size; and
e) sizes are rounded to the nearest millimetre.

Mid-panel moment — the maximum moment between panel points.

Moisture content — the mass of water in wood expressed as a percentage of the mass of the oven-dry wood.

Nominal resistance — resistance calculated using resistance factors of 1.0.

OSB — an acronym for oriented strandboard, i.e., a strandboard panel containing layers of aligned strands, generally with the strands in the face layers aligned in the direction of the panel length. Panels are marked to show the direction of alignment of face layers.

Construction sheathing OSB — OSB that complies with CSA O325 for protected construction uses such as roof sheathing, wall sheathing, and floor sheathing in light-frame construction applications and other permitted engineering applications in this Standard.

Panel length — with respect to the design of lumber members in metal-plated trusses (see Clause 6.5.12), the distance between two adjacent panel points.

Panel point — with respect to the design of lumber members in metal-plated trusses (see Clause 6.5.12), a point representing the intersection of two or more analogue member lines and/or a normal line from a bearing surface.

Panel point moment — with respect to the design of lumber members in metal-plated trusses (see Clause 6.5.12), the moment computed at an analogue panel point.

Perimeter member — an element at edges of openings or at perimeters of shearwalls or diaphragms.

Pitch — the rise in run, usually expressed as a fraction of 12, of the upper surface of a roof member.

Pitch break — the point at which a truss chord member changes slope.

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Planar shear (rolling shear) — in structural panels and CLT panels, the shear that occurs in the plane of the panel and leads to strains in a plane perpendicular to the grain direction.
Note: *It is also referred to as "shear in-plane".*

Plank — a piece of dimension lumber 114 mm or more in its larger dimension, intended to be loaded on its wide face.

Ply — a thin layer or sheet of wood (veneer) or several pieces, laid with adjoining edges to form one layer in a plywood panel.
Note: *The layers can be edge-glued.*

Plywood —

Regular unsanded grade — any one of the following three grades: Select Tight Face, Select, or Sheathing as defined in CSA O121 and CSA O151.

Standard construction — panels that meet the requirements for standard constructions as defined in CSA O121 and CSA O151.

Pole-type construction — a form of construction in which the principal vertical members are round poles or sawn timbers embedded in the ground and extending vertically above ground to provide both foundation and vertical framing members for the structure.

Post — a timber with its larger dimension not more than 51 mm greater than the smaller dimension, usually graded for use as a column.
Note: *Grading rules sometimes designate posts as "posts and timbers".*

Prefabricated wood I-joist — a structural member manufactured using structural lumber, structural glued-laminated timber, or structural composite lumber flanges, and structural panel webs, bonded together with structural adhesives, forming an "I" cross-sectional shape.
Note: *To avoid confusion with plywood web beams as defined in Clause 10.5, "prefabricated wood I-joist" refers to high-volume, mass-produced proprietary products primarily used as joists in the construction of floor and roof systems.*

Principal load — the specified variable load or rare load that dominates in a given load combination.

Product evaluation report — a report published by a third-party evaluation agency that confirms the evaluation of the proprietary product and that the resulting resistance and stiffness values for design are derived in compliance with Clause 16 and the *National Building Code of Canada*.

Resistance factor, ϕ — a factor applied to the resistance of a member or connection for the limit state under consideration, which takes into account the variability of dimensions and material properties, quality of work, type of failure, and uncertainty in the prediction of resistance.

Serviceability limit states — those states that restrict the intended use and occupancy of the structure. They include deflection, joint slip, vibration, and permanent deformation.

Service condition —

Dry-service condition — a climatic condition in which the average equilibrium moisture content of wood is 15% or less on average over a year without exceeding 19%.

Wet-service condition — all service conditions other than dry.

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Shearwall — a wall system designed to resist lateral forces parallel to the plane of the wall. It can consist of one or more shearwall segments in the plane of the wall.

Shearwall segment — a section of a shearwall with uniform construction that forms a structural unit designed to resist lateral forces parallel to the plane of the wall.

Shrinkage — the decrease in the dimensions of wood or wood products due to a decrease of moisture content.

Slab floor — a basement floor system in which a concrete slab or equivalent provides lateral support at the bottom of the foundation studs.

Slenderness ratio for bending members — the ratio used in lateral stability calculations of a bending member.

Slenderness ratio for compression members — the ratio of the effective length of a compression member to its actual dimensions.

Slip resistance of a connection — a serviceability state; a resistance that corresponds to a specific level of slip.

Space frame — a three-dimensional structural system without structural stud walls that is composed of members interconnected to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

Spacing of fasteners — the distance between fasteners measured between centres.

Specified capacity — the assigned strength capacity, or stiffness or rigidity capacity, for use in the prediction of strength resistance or deflection.

Specified loads — those loads defined in the appropriate building code, or those loads determined by use and occupancy for structures other than buildings, or such larger loads as selected for the design.

Specified strength — the assigned strength for use in the prediction of strength resistance.

Stiffener — a piece of wood of rectangular cross-section that extends between the inner surfaces of the top and bottom flanges of a plywood web beam and is glued or otherwise connected to the webs.

Strandboard — a mat-formed structural panel made of specialized wood wafers having a length at least twice their width.

Strength resistance of a connection — a resistance based on the geometry and on the ultimate load-carrying capacity of the structural materials of a connection.

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 entire unit.

Structural composite lumber (SCL) — the wood product that is either laminated veneer lumber (LVL), parallel strand lumber (PSL), laminated strand lumber (LSL), or oriented strand lumber (OSL), as defined in ASTM D5456 and manufactured for use in structural applications.

Structural glued-laminated timber — see **Glued-laminated timber**.

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Structural panel — plywood or OSB panels of specific grades and quality to which specified capacities have been assigned, or are assignable (e.g., standard constructions of regular grades of unsanded Douglas fir plywood and Canadian softwood plywood manufactured and marked in accordance with CSA O121 and CSA O151, and construction sheathing OSB manufactured and marked in accordance with CSA O325).

Stud wall system — a structural system without a complete vertical load-carrying space frame. Studs are spaced not more than 610 mm on centre.

Suspended floor — a basement floor system in which a floor assembly is attached to continuous foundation studs at a point above the bottom of the studs.

Timber — a piece of lumber 114 mm or more in its smaller dimension.

Truss plate — a light-gauge metal plate connector, intended for use in structural lumber assemblies that can have integral teeth of various shapes and configurations.

Ultimate limit states — those states concerning safety, including the maximum load-carrying capacity, overturning, sliding, fracture, and deterioration.

Waferboard — a mat-formed panel made predominantly of wood wafers of a minimum and controlled length, a controlled thickness, and a variable or predetermined width, bonded together with a waterproof and boilproof binder.

Wood preservation —

Preservative treatment — impregnation under pressure with a wood preservative.

Wood preservative — any suitable substance that is toxic to fungi, insects, borers, and other living wood-destroying organisms.

Δ 3.2 Symbols

The following symbols shall apply in this Standard. Deviations from these usages and additional nomenclature are noted where they occur.

A = cross-sectional area, mm^2

A_b = bearing area, mm^2

A_{eff} = effective cross-sectional area of CLT panels, mm^2

A_g = gross cross-sectional area, mm^2

A_n = net cross-sectional area, mm^2

B_a = specified axial stiffness of structural panels, N/mm

B_b = specified bending stiffness of structural panels, N•mm^2/mm

B_r = factored buckling resistance for plywood assemblies, kN/m^2

B_v = specified shear-through-thickness rigidity of structural panels, N/mm

b = width of member, mm

b_p = width of structural panel, mm

C_B = slenderness ratio for bending members

C_C = slenderness ratio for compression members

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d = depth of member, mm

d_F = diameter of fastener, mm

d_p = depth of structural panel in plane of bending, mm

E = specified modulus of elasticity, MPa

E_S = modulus of elasticity for stiffness calculations, MPa

E_{05} = modulus of elasticity for design of compression members, MPa

$E_\perp$ = transverse modulus of elasticity, MPa

$(EI)_e$ = effective stiffness of structural panel assemblies, N•mm^2

Δ $(EI)_{eff,f}$ = effective flatwise bending stiffness of CLT panels, N•mm^2

f_b = specified strength in bending, MPa

f_c = specified strength in compression parallel to grain, MPa

f_{cp} = specified strength in compression perpendicular to grain, MPa

f_f = specified fracture shear strength at notch, MPa

f_s = specified strength in rolling shear, MPa

f_t = specified strength in tension parallel to grain, MPa

f_{tg} = specified strength in tension parallel to grain at gross section of glued-laminated timber, MPa

f_{tn} = specified strength in tension parallel to grain at net section of glued-laminated timber, MPa

f_{tp} = specified strength in tension perpendicular to grain, MPa

f_v = specified strength in shear, MPa

G = shear modulus, MPa

$G_\perp$ = rolling shear modulus, MPa

$(GA)_{eff,f}$ = effective shear stiffness in flatwise bending of CLT panels, N

I = moment of inertia, mm^4

I_{eff} = effective out-of-plane moment of inertia of CLT panels, mm^4

J = factors affecting the resistance of a connection, used with appropriate subscripts

J_{hd} = hold-down effect factor for shearwall segments

K_B = length-of-bearing factor

K_C = slenderness factor for compression members

K_D = load-duration factor

K_E = end-fixity factor for spaced compression members

K_e = effective-length factor

K_F = end-use factor for foundations

K_{fi} = adjustment factor for fire resistance

K_H = system factor

K_L = lateral-stability factor for bending members

K_M = bending capacity modification factor

K_N = notch factor

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K_R = radial stress factor
K_{rb} = adjustment factor for bending moment resistance of CLT panels
K_S = service-condition factors, used with appropriate subscripts
K_{Sb} = service-condition factor for bending
K_{Sc} = service-condition factor for compression parallel to grain
K_{Scp} = service-condition factor for compression perpendicular to grain
K_{SE} = service-condition factor for modulus of elasticity
K_{SF} = service-condition factor for connections
K_{Sf} = service-condition factor for fracture shear
K_{St} = service-condition factor for tension parallel to grain
K_{Stp} = service-condition factor for tension perpendicular to grain
K_{Sv} = service-condition factor for shear
K_T = treatment factor
K_X = curvature factor for glued-laminated timber
K_Z = size factor
K_{Zb} = size factor for bending for sawn lumber
K_{Zbg} = size factor for bending for glued-laminated timber
K_{Zc} = size factor for compression for sawn lumber and for CLT
K_{Zcg} = size factor for compression for glued-laminated timber
K_{Zcp} = size factor for bearing
K_{Zt} = size factor for tension for sawn lumber
K_{Ztp} = size factor for tension perpendicular to grain for glued-laminated timber
K_{Zv} = size factor for shear for sawn lumber
L = length, mm
L_e = effective length, mm
L_p = length of penetration of fastener into main member, mm
ℓ = span, mm
M_f = factored bending moment, N•mm
M_r = factored bending moment resistance, N•mm
m_p = specified strength capacity of structural panels in bending, N•mm/mm
N_r = factored compressive resistance at an angle to grain, N
= factored lateral strength resistance of a connection, N or kN
N_{rs} = factored lateral slip resistance of a connection, N
N_s = lateral slip resistance of a connection, N
N_u = lateral strength resistance of a connection, N or kN
n_F = number of fasteners in a connection
P_f = factored axial load in compression, N
P_r = factored compressive resistance parallel to grain, N

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= factored lateral strength resistance of a connection parallel to grain, N or kN

P_{rw} = factored withdrawal resistance of a connection from side grain, N

P_u = lateral strength resistance of a connection parallel to grain, N or kN

p_p = specified strength capacity of structural panels in axial compression, N/mm

Q_r = factored compressive resistance perpendicular to grain or to plane of plies, N

= factored lateral strength resistance of a connection perpendicular to grain, N or kN

Q_u = lateral strength resistance of a connection perpendicular to grain, N or kN

q_p = specified strength capacity of structural panels in bearing, MPa

R = radius of curvature at centreline of member, mm

r_{eff} = effective radius of gyration, mm

S = section modulus, mm^3

$S_{eff,f}$ = effective section modulus in flatwise bending of CLT panels, mm^3, used with appropriate subscripts

T_f = factored axial load in tension, N

T_r = factored tensile resistance parallel to grain, N

t_p = specified strength capacity of structural panels in axial tension, N/mm

V_f = factored shear force, N

V_{hd} = factored basic shear resistance calculated with J_{hd} = 1.0, kN

V_r = factored shear resistance, N

= factored shear-through-thickness resistance of structural panels, N

V_{rp} = factored planar shear resistance of structural panels, N

V_{rs} = factored shear resistance of a shearwall segment, kN

v_p = specified strength capacity of structural panels in shear-through-thickness, N/mm

v_{pf} = specified strength capacity of structural panels in planar shear (due to in-plane forces), MPa

v_{psb} = specified strength capacity of structural panels in planar shear (due to bending), N/mm

W_f = factored total load, N

w = specified total uniformly distributed load, kN/m^2

X = factors affecting capacities of plywood and plywood assemblies, used with appropriate subscripts

Z = volume, m^3

ϕ = resistance factor

3.3 Spacing and dimensions

In this Standard, the following shall apply:

a) The following centre-to-centre member spacing dimensions may be used interchangeably:
   i) 300 mm and 305 mm;
   ii) 400 mm and 406 mm; and
   iii) 600 mm and 610 mm.

b) The following panel dimensions may be used interchangeably:
   i) 1200 mm and 1220 mm; and

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ii) 2400 mm and 2440 mm.

4 Objective and design requirements

4.1 Objective

The objective of the provisions in this Standard is the achievement of assurances that the structure, when correctly designed and built, will be fit for the intended use.

Δ 4.2 Limit states

The structure or portion thereof is considered fit for use when the structure, its components, and its connections are designed such that the requirements of Clauses 4.3, 5.1.2, and 5.1.3 are satisfied.

Note: *The objective of limit states design calculations is to limit the probability of reaching a state beyond which it ceases to fulfill the function for which it was designed. The states that concern safety are called ultimate limit states (ULS, see Clause 5.1.2) and include exceeding the load-carrying capacity, loss of equilibrium, instability, and fracture. The states that restrict the normal use and occupancy are called serviceability limit states (SLS, see Clause 5.1.3) and include deflection and vibration.*

4.3 Design requirements

4.3.1 Structural adequacy

All members shall be so framed, anchored, tied, and braced together as to provide the strength and rigidity necessary for the purpose for which they are designed. All structural members shall be of a size and quality to carry all loads and other forces that can be reasonably expected to act upon them during construction and use without exceeding the strength or serviceability limit states.

4.3.2 New or special systems of design and construction

New or special systems of design or construction of wood structures or structural elements not already covered by this Standard may be used where such systems are based on analytical and engineering principles and reliable test data, or both, that demonstrate the safety and serviceability of the resulting structure for the purpose intended.

4.3.3 Structural integrity

The general arrangement of the structural system and the interconnection of its members shall provide positive resistance to widespread collapse of the system due to local failure.

4.3.4 Basis of design

Design in accordance with this Standard is based on the assumption that

a) the specified loads are realistic in size, kind, and duration;
b) the wood product is normal for its species, kind, and grade;
c) consideration is given to service conditions, including possible deterioration of members and corrosion of fasteners;
d) the temperature of the wood does not exceed 50 °C, except for occasional exposures to 65 °C;
e) the design is competent, fabrication and erection are good, grading and inspection are reliable, and maintenance is normal; and
f) wood products are used as graded or manufactured for a designated end use.

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4.3.5 Quality of work

The quality of work in fabrication, preparation, and installation of materials shall conform throughout to accepted good practice.

4.3.6 Design drawings

4.3.6.1 Scale and dimensions

Where design drawings are required, they shall be drawn to a scale that legibly conveys the required information. The drawings shall show a complete layout of the structure or portion thereof that is the subject of the design, with members suitably designated and located, including dimensions and detailed descriptions necessary for the preparation of shop and erection drawings. Governing heights, column centres, and offsets shall be dimensioned.

4.3.6.2 Standard references

Design drawings shall designate the design standards used, as well as material or product standards applicable to the members and details depicted.

4.3.6.3 Information on loading

When needed for the preparation of shop drawings, the governing loads, reactions, shears, moments, and axial forces to be resisted by all members and their connections shall be shown on drawings and supplemental material, or both.

4.3.6.4 Camber

If camber is required for beams, girders, or trusses, the magnitude of such camber shall be specified on the design drawings.

5 General design

5.1 Limit states design

5.1.1 Methods of analysis

The load effect on all members and connections shall be determined in accordance with recognized methods of analysis generally based on assumptions of elastic behaviour.

Note: *Effects include forces, moments, deformations, and vibrations.*

5.1.2 Ultimate limit states

Design for ultimate limit states shall include

a) establishing the value of the effect of the factored loads individually and with the load combinations specified in Clause 5.2; and

b) confirming by rational means that for each load effect in Item a), the factored load effect does not exceed the corresponding factored resistance, as determined in accordance with the appropriate clauses of this Standard.

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5.1.3 Serviceability limit states

Design for serviceability limit states shall include

a) establishing the value of the effect of the specified loads individually and with the load combinations specified in Clause 5.2; and

b) confirming by rational means that for each load effect in Item a), the structural effect falls within the limits specified in the appropriate clauses of this Standard.

5.1.4 Resistance factors

The resistance factors, ϕ, are given in the appropriate sections of this Standard for all applicable limit states for wood members and connections.

5.2 Loading

5.2.1 Loads applied to buildings

Except as provided for in Clause 5.2.2, the specified loads, load effects, and combinations to be considered in the design of a building and its elements shall be those given in Clauses 5.2.3 and 5.2.4.

Note: *Specified loads, load effects, and combinations specified in this Standard are in accordance with the provisions of the* National Building Code of Canada*, the Codes Canada* Structural Commentaries (User's Guide — National Building Code 2015: Part 4 of Division B)*, and the Canadian Geotechnical Society's* Canadian Foundation Engineering Manual*. In the case of seismic design, Clause 11 includes provisions additional to those in Division B of the* National Building Code of Canada*.*

5.2.2 Loads applied to other structures

Where load requirements other than those in Clause 5.2.1 are specified, the appropriateness of the applicable factored resistance in this Standard shall be considered.

5.2.3 Specified loads

5.2.3.1 Loads to be considered

Specified loads shall be determined in accordance with the requirements of the applicable building code.

5.2.3.2 Importance factors

For determining the specified Snow (*S*), Wind (*W*), or Earthquake (*E*) loads of Clause 5.2.3.1, importance factors shall be applied in accordance with the applicable building code.

5.2.4 Load combinations

5.2.4.1 Load combinations for ultimate limit states

The effect of factored principal plus companion loads shall be determined in accordance with the load combinations in the applicable building code.

5.2.4.2 Load combinations for serviceability limit states

The effect of principal plus companion loads shall be determined in accordance with the load combinations in the applicable building code.

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5.3 Conditions and factors affecting resistance

5.3.1 General

Specified strengths and capacities in this Standard are based on the standard-term duration of loads, dry-service conditions, and untreated materials. The specified strengths and capacities for materials and connections shall be multiplied by the modification factors specified in this Clause and in the appropriate clauses of this Standard.

Note: *The basis for derivation of specified strengths for sawn lumber members is described in the CWC* Standard Practice Relating Specified Strengths of Structural Members to Characteristic Structural Properties*. The principles described therein have also been used to guide derivations for other products in this Standard.*

5.3.2 Load-duration factor, K_D

5.3.2.1 General

Except as specified in Clause 5.3.2.2, the specified strengths and capacities shall be multiplied by a load-duration factor, K_D, in accordance with Table 5.1, but not exceeding 1.15.

Table 5.1
Load-duration factor, K_D
(See Clauses 5.3.2.1 and 9.4.1.1.)

Load duration	K_D	Explanatory notes
Short term	1.15	Short term means the condition of loading where the duration of specified loads is not expected to last more than 7 days continuously or cumulatively throughout the life of the structure. Examples include wind loads, earthquake loads, impact loads, falsework, and formwork, as well as standard- and/or long-term loads in combination with the above.
Standard term	1.00	Standard term means the condition of loading where the duration of specified loads exceeds that of short-term loading, but is less than long-term loading. Examples include snow loads, live loads due to occupancy, wheel loads on bridges, and long-term loads in combination with the above.
Long term	0.65	Long term means the condition of loading under which a member is subjected to continuous specified load. Examples include dead loads or dead loads plus live loads of such character that they are imposed on the member for as long as the dead loads themselves. Such loads include those usually occurring in tanks or bins containing fluids or granular material, loads on retaining walls subjected to lateral pressure such as earth, and floor loads where the specified load can be expected to be continuously applied, such as those in buildings for storage of bulk materials. Loads due to fixed machinery are considered long term.

Note: *Load duration requires professional judgment by the designer. Explanatory notes in this Table provide guidance to designers about the types of loads and load combinations for which each modification factor should be applied.*

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5.3.2.2 Long-term load-duration factor

If the specified long-term load, P_L, is greater than the specified standard-term load, P_s, a load-duration factor of 0.65 may be used or K_D may be calculated as follows:

$K_D = 1.0 - 0.50 \log (P_L/P_s) \geq 0.65$

where

P_L = specified long-term load

P_s = specified standard-term load based on Snow (S) and Live (L) loads acting alone or in combination

= S, L, $S + 0.5L$, or $0.5S + L$, determined using importance factors equal to 1.0

5.3.2.3 Combined loads

When the total specified load is made up of loads acting for different durations, the design shall be based on the most severe combination. The load duration factor, K_D, for the load of the shortest duration may apply for that load combination, except as specified elsewhere in this Standard.

Note: *The most severe combination requires consideration of both the effect of the factored load combinations and the corresponding load-duration factor (which affects the resistance). The load combination that produces the most critical effect might also correspond to a high load-duration factor (i.e., with a relatively higher resistance) and therefore might not produce the most severe combination. Instead, a load combination producing a lower effect but with a corresponding lower load duration (i.e., with a lower resistance) might produce the more severe combination.*

5.3.3 Service-condition factor, K_S

Where materials or connections are not used in dry service conditions, specified strengths, specified capacities, and moduli of elasticity shall be multiplied by the service-condition factor, K_S, specified in the appropriate clauses of this Standard.

5.3.4 Preservative and fire-retardant treatment factor, K_T

5.3.4.1 General

Except as permitted in Clause 5.3.4.4, specified strengths and capacities shall be multiplied by the treatment factor, K_T, specified in the appropriate clauses of this Standard.

5.3.4.2 Preservative treatment

When conditions conducive to decay or other deterioration are likely to occur in the case of permanent structures, wood should be pressure treated with preservative in accordance with the requirements of the CAN/CSA-O80 Series of Standards. If possible, all boring, grooving, cutting, and other fabrication should be completed before treatment. Fabrication that is carried out after pressure treatment shall be treated locally in accordance with the CAN/CSA-O80 Series of Standards.

5.3.4.3 Untreated wood

Untreated wood in permanent structures shall not be in direct contact with masonry, concrete, or soil when moisture transfer can occur. Any method that eliminates transfer of moisture (e.g., a minimum of 10 mm air space around a member in a wall) shall be considered adequate protection.

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5.3.4.4 Fire-retardant treatment

Where wood is impregnated with fire-retardant or other strength-reducing chemicals, K_T shall be determined in accordance with the results of appropriate tests or shall not exceed the value of K_T specified in the appropriate clause of this Standard.

5.3.5 System factor, K_H

Specified strengths may be multiplied by a system factor, K_H, specified in the appropriate clauses of this Standard.

Note: *See Clause A.5.3.5 for additional information on system factors.*

5.3.6 Size factor, K_Z

Specified strengths shall be multiplied by the size factor, K_Z, specified in the appropriate clauses of this Standard.

5.3.7 Lateral-stability factor, K_L

The specified bending strength shall be multiplied by the lateral-stability factor, K_L, specified in the appropriate clauses in this Standard.

5.3.8 Reduction in cross-section

5.3.8.1 Net section

The net section, obtained by deducting from the gross section the area of all material removed by boring, grooving, dapping, notching, or other means, shall be checked in calculating the resistance of a member.

5.3.8.2 Limitation

The net section shall not be less than 75% of the gross section.

5.4 Serviceability requirements

5.4.1 Modulus of elasticity

The modulus of elasticity for stiffness calculations, E_S, shall be calculated as follows:

$E_S = E\ (K_{SE}\ K_T)$

where

E = specified modulus of elasticity, MPa

K_{SE} = service-condition factor

K_T = treatment factor

5.4.2 Elastic deflection

The elastic deflection of structural members under the load combinations for serviceability limit states shall not exceed 1/180 of the span. For members having cambers equaling at least dead load deflection, the additional deflection due to live, snow, and wind loads shall not exceed 1/180 of the span. Deflection under the load combinations for serviceability limit states shall be limited to avoid damage to structural elements or attached non-structural elements.

Note: *See Clause A.5.4.2 for additional information on deflection of a wood frame system under static loads.*

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5.4.3 Permanent deformation

Structural members that support long-term loads shall be designed to limit permanent deformation. In the absence of a more accurate evaluation of acceptable deflection limits, an upper limit of 1/360 of the span shall be imposed on the elastic deflection due to long-term service load combinations.

5.4.4 Ponding

Roof framing systems shall be investigated by rational analysis to ensure performance under ponding conditions unless

a) the roof surface is provided with a slope toward points of free drainage which prevents accumulation of rain water; or
b) for a simply supported system subjected to a uniformly distributed load, the following condition is satisfied:

$$\frac{\Sigma\Delta}{w} < 65$$

where

$\Sigma\Delta$ = sum of deflections due to this load, mm, of all the components of the system (decking, secondary beams, primary beams, etc.)

w = specified total uniformly distributed load, kN/m^2

5.4.5 Vibration

Δ

5.4.5.1 General

Special consideration shall be given to structures subjected to vibration to ensure that such vibration is acceptable for the use of the structure.

Note: *Additional information can be found in the commentary on serviceability criteria for deflections and vibrations in the* Structural Commentaries (User's Guide — National Building Code 2015: Part 4 of Division B)*, and for joisted floor and CLT floor vibration in CWC* Commentary on CSA O86*.*

Δ

5.4.5.2 Floor vibration

For joisted floor systems that consist of a wood-based subfloor structurally attached to a series of equally-spaced floor joists, design shall consider vibration serviceability arising from normal human activities, such as walking action. Recommended design methods are provided in Clause A.5.4.5. The floor joist products include, but are not limited to, sawn lumber, glulam joists, prefabricated wood I-joists, parallel chord trusses, and structural composite lumber (SCL).

For CLT floors, a suitable recommended design method is provided in Clause A.8.5.3.

Note: *Additional information can be found in the commentary on serviceability criteria for deflections and vibrations in the* Structural Commentaries (User's Guide — National Building Code 2015: Part 4 of Division B)*, and for joisted floor and CLT floor vibration in CWC* Commentary on CSA O86*.*

5.4.6 Building movements due to change in moisture content

Anticipated building movements due to shrinkage and swelling shall be indicated in the drawings by the designer.

Note: *See Clause A.5.4.6 for information on shrinkage and swelling of wood members due to moisture changes.*

5.5 Lateral brace force for wood truss compression webs

The lateral brace used to resist out-of-plane deflection in axial compression web members in light-gauge metal plate connected wood trusses shall be designed for a force equal to 1.25% of the axial

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compressive force in the web member. This value shall apply equally to both single-braced and double-braced webs. In repetitive-member systems, the lateral force is cumulative. Connection of the lateral bracing to truss webs shall be calculated in accordance with Clause 12.

5.6 Fire resistance

Where applicable, design for fire resistance shall be in accordance with the applicable building code.

Note: *See Annex B for a design methodology for determining the fire resistance of large cross-section wood members. In the absence of recognition of the design methodology in the applicable building code, the information can be used in the development of a proposal for an alternative solution meeting the objectives of the* National Building Code of Canada.

6 Sawn lumber

6.1 Scope

The design data and methods specified in Clause 6 apply only to structural lumber complying with the requirements of CSA O141.

6.2 Materials

6.2.1 Identification of lumber

6.2.1.1 General

Design in accordance with this Standard is predicated on the use of lumber that is graded in accordance with the NLGA *Standard Grading Rules for Canadian Lumber* and identified by the grade stamp of an association or independent grading agency in accordance with the provisions of CSA O141.

Note: *A list of approved agencies can be obtained from the Canadian Lumber Standards Accreditation Board.*

6.2.1.2 Canadian lumber

In this Standard, Canadian species are designated according to species combinations given in Table 6.1, which reflects marketing practice. These combinations should be used for general design purposes.

Note: *The designer is strongly advised to check availability of species, grade, and sizes before specifying. See Clause A.6.2.1.2.*

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Table 6.1
Species combinations
(See Clause 6.2.1.2.)

Species combinations	Stamp identification	Species included in the combination
Douglas Fir-Larch	D Fir-L (N)	Douglas fir, western larch
Hem-Fir	Hem-Fir (N)	Pacific coast hemlock, amabilis fir
Spruce-Pine-Fir	S-P-F	Spruce (all species except coast Sitka spruce and Norway spruce), Jack pine, lodgepole pine, balsam fir, alpine fir
Northern Species	North Species	Any Canadian species graded in accordance with the NLGA rules

Notes:
1) *Names of species in this Table are standard commercial names. Additional information on botanical names and other common names is given in CSA O141.*
2) *The NLGA* Standard Grading Rules for Canadian Lumber *contains other species combinations not shown in this Table. If the species can be identified, however, it can be possible to group it in one of the species combinations for the purpose of assigning specified strengths.*

6.2.1.3 US lumber

For US commercial species combinations graded in accordance with the *National Grading Rule for Dimension Lumber*, the design data may be determined using the species combination equivalents in Table 6.2.

Table 6.2
Species combination equivalents
(See Clause 6.2.1.3.)

US combination	Equivalent Canadian combination
Douglas Fir-Larch	Douglas Fir-Larch
Hem-Fir	Hem-Fir

Note: *The NLGA* Standard Grading Rules for Canadian Lumber *incorporates the* National Grading Rule for Dimension Lumber, *a uniform set of grade descriptions and other requirements for softwood dimension lumber that form a required part of all softwood lumber grading rules in the United States. Thus, all dimension lumber throughout Canada and the United States is graded to uniform requirements.*

6.2.2 Lumber grades and categories

6.2.2.1 Visually stress-graded lumber

Grade categories, limiting dimensions, and structural grades for which design data are assigned in this Standard are listed in Table 6.3. These grades are specified in the NLGA *Standard Grading Rules for Canadian Lumber.*

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Table 6.3
Visual grades and their dimensions
(See Clause 6.2.2.1.)

Grade category	Smaller dimension, mm	Larger dimension, mm	Grades
Light framing	38–89	38–89	Construction, Standard
Studs	38–89	38 or more	Stud
Structural light framing	38–89	38–89	Select structural No. 1, No. 2, No. 3
Structural joists and planks	38–89	114 or more	Select structural No. 1, No. 2, No. 3
Beams and stringers	114 or more	Exceeds smaller dimension by more than 51	Select structural No. 1, No. 2
Posts and timbers	114 or more	Exceeds smaller dimension by 51 or less	Select structural No. 1, No. 2
Plank decking*	38–89	140 or more	Select, Commercial

* *"Plank decking" is defined as "decking" under NLGA rules.*

6.2.2.2 Machine stress-rated (MSR) and machine evaluated lumber (MEL)

The design data specified in this Standard apply to lumber that is graded in accordance with NLGA SPS 2 and is identified by the grade stamp of a grading agency accredited for grading by mechanical means.

Note: *A list of accredited agencies can be obtained from the Canadian Lumber Standards Accreditation Board.*

6.2.3 Finger-joined and face-glued lumber

6.2.3.1 General

Except as limited in Clause 6.2.3.2, 6.2.3.3, or 6.2.3.4, the design data specified in this Standard apply to finger-joined and face-glued lumber that has been produced in accordance with NLGA SPS 1, SPS 3, SPS 4, SPS 5, or SPS 6.

Note: *Finger-joined and face-glued lumber is produced to specifications that permit the same specified strength and stiffness to be assigned as non-finger-joined and non-face-glued lumber of the same grade, species, and size.*

6.2.3.2 NLGA SPS 3 and SPS 5 "vertical stud use only" lumber

6.2.3.2.1 Applications

Finger-joined and face-glued lumber that has been produced in accordance with NLGA SPS 3 and SPS 5, respectively, shall be used only under the following conditions:

a) applications where the primary loading is in compression, with only short-duration stresses in bending or tension, such as due to wind or earthquake loads; and
b) applications where it is protected from wet-service conditions and not in an environment where the temperature can be expected to exceed 50 °C for an extended period.

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6.2.3.2.2 Fire-rated wall assemblies

For SPS 3 and SPS 5 "vertical stud use only" lumber to be used in a fire-rated wall assembly, the grade stamp shall include the Heat Resistant Adhesive (HRA) designation.

6.2.3.3 NLGA SPS 4 "dry use only" lumber

Finger-joined lumber that has been produced in accordance with NLGA SPS 4 and designated "dry use only" shall be used only where protected from wet-service conditions.

6.2.3.4 NLGA SPS 6 structural face-glued lumber

For SPS 6 structural face-glued lumber to be used in a fire-rated assembly, the grade stamp shall include the HRA designation.

6.2.4 Remanufactured lumber

Dimension lumber and timbers that are resawn or otherwise remanufactured shall be regraded in accordance with Clause 6.2.1.

6.2.5 Mixed grades

When mixed grades are used, the specified strength shall be that of the grade having the lowest value.

6.3 Specified strengths and moduli of elasticity

6.3.1 Visually stress-graded lumber

6.3.1.1 General

The specified strengths and moduli of elasticity for visually stress-graded lumber are tabulated as follows:

a) structural joists and planks, structural light framing, and studs grade categories of lumber in Table 6.4;
b) light framing grades in Table 6.5;
c) beams and stringers grade categories of lumber in Table 6.6; and
d) posts and timbers grade categories of lumber in Table 6.7.

6.3.1.2 Plank decking

The specified strengths and moduli of elasticity for plank decking shall be derived from Table 6.4 using the following grade equivalents:

Decking grade	Equivalent lumber grade
Select	Select structural
Commercial	No. 2

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6.3.2 Machine stress-rated and machine evaluated lumber

The specified strengths and moduli of elasticity for machine stress-rated lumber are given in Table 6.8. The specified strengths and moduli of elasticity for machine evaluated lumber are given in Table 6.9. Specified strengths in shear are not grade dependent and shall be taken from Table 6.4 for the appropriate species.

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Table 6.4
Specified strengths and moduli of elasticity for structural joists and planks, structural light framing, and studs, MPa

(See Clauses 6.3.1.1, 6.3.1.2, 6.3.2, 6.5.3.1, 6.5.5.2.4, 6.5.5.2.5, 6.5.6.2, 6.5.8, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, 12.4.4.4, 12.4.4.5, A.6.5.5.3.6, and A.5.4.2.2 and Tables 6.8, 6.9, and 8.2.)

Species combination	Grade	Bending, f_b	Longitudinal shear, f_v	Compression: Parallel to grain, f_c	Compression: Perpendicular to grain, f_{cp}	Tension parallel to grain, f_t	Modulus of elasticity: E	Modulus of elasticity: E_{05}
Douglas Fir-Larch	SS	16.5		19.0		10.6	12 500	8500
	No. 1/No. 2	10.0	1.9	14.0	7.0	5.8	11 000	7000
	No. 3/Stud	4.6		7.3		2.1	10 000	5500
Hem-Fir	SS	16.0		17.6		9.7	12 000	8500
	No. 1/No. 2	11.0	1.6	14.8	4.6	6.2	11 000	7500
	No. 3/Stud	7.0		9.2		3.2	10 000	6000
Spruce-Pine-Fir	SS	16.5		14.5		8.6	10 500	7500
	No. 1/No. 2	11.8	1.5	11.5	5.3	5.5	9500	6500
	No. 3/Stud	7.0		9.0		3.2	9000	5500
Northern Species	SS	10.6		13.0		6.2	7500	5500
	No. 1/No. 2	7.6	1.3	10.4	3.5	4.0	7000	5000
	No. 3/Stud	4.5		5.2		2.0	6500	4000

Note: *Tabulated values are based on the following standard conditions:*

a) *286 mm larger dimension;*
b) *dry-service conditions; and*
c) *standard-term duration of load.*

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Table 6.5
Specified strengths and moduli of elasticity for light framing, MPa, applicable to sizes 38 × 38 mm to 89 × 89 mm

(See Clauses 6.3.1.1, 6.5.5.2.5, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, 12.4.4.4, and 12.4.4.5.)

				Compression			Modulus of elasticity	
Species combination	**Grade**	**Bending, f_b**	**Longitudinal shear, f_v**	**Parallel to grain, f_c**	**Perpendicular to grain, f_{cp}**	**Tension parallel to grain, f_t**	**E**	**E_{05}**
Douglas Fir-Larch	Const.	13.0	3.2	16.0	7.0	6.6	10 000	5500
	Stand.	7.3		13.1		3.7	9000	5000
Hem-Fir	Const.	14.3	2.7	16.9	4.6	7.0	10 000	6000
	Stand.	8.0		13.9		3.9	9000	5500
Spruce-Pine-Fir	Const.	15.3	2.6	13.1	5.3	6.2	9000	5500
	Stand.	8.6		10.8		3.5	8000	5000
Northern Species	Const.	9.9	2.2	11.9	3.5	4.5	6500	4000
	Stand.	5.5		9.8		2.5	6000	3500

Notes:

1) *The size factor K_Z for light framing grades shall be 1.00, except that K_{Zc} shall be calculated in accordance with Clause 6.5.5.2.4 and K_{Zcp} shall be determined in accordance with Clause 6.5.6.4.*
2) *Tabulated values are based on the following standard conditions:*
   a) *89 mm width (except for compression properties);*
   b) *dry-service conditions; and*
   c) *standard-term duration of load.*

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Table 6.6
Specified strengths and moduli of elasticity for beams and stringers, MPa

(See Clauses 6.3.1.1, 6.5.2.2, 6.5.5.2.5, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, 12.4.4.4, 12.4.4.5, and 14.3.)

Species combination	Grade	Bending, f_b*	Longitudinal shear, f_v	Compression: Parallel to grain, f_c	Compression: Perpendicular to grain, f_{cp}	Tension parallel to grain, f_t	Modulus of elasticity: E*	Modulus of elasticity: E_{05}*
Douglas Fir-Larch	SS	19.5	1.5	13.2	7.0	10.0	12 000	8000
	No. 1	15.8		11.0		7.0	12 000	8000
	No. 2	9.0		7.2		3.3	9500	6000
Hem-Fir	SS	16.8	1.2	13.0	4.6	7.4	11 500	8000
	No. 1	14.4		12.4		6.3	11 000	7500
	No. 2	14.4		12.4		6.3	11 000	7500
Spruce-Pine-Fir	SS	13.6	1.2	9.5	5.3	7.0	8500	6000
	No. 1	11.0		7.9		4.9	8500	6000
	No. 2	6.3		5.2		2.3	6500	4500
Northern Species	SS	12.8	1.0	7.2	3.5	6.5	8000	5500
	No. 1	10.8		6.0		4.6	8000	5500
	No. 2	5.9		3.9		2.2	6000	4000

** Specified strengths for beams and stringers are based on loads applied to the narrow face. When beams and stringers are subject to loads applied to the wide face, the specified strength for bending at the extreme fibre and the specified modulus of elasticity shall be multiplied by the following factors:*

	f_b	E or E_{05}
Select structural	*0.88*	*1.00*
No. 1 or No. 2	*0.77*	*0.90*

Notes:

1) *Beams and stringers have a smaller dimension of at least 114 mm, with a larger dimension more than 51 mm greater than the smaller dimension.*

(Continued)

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Table 6.6 (Concluded)

2) *An approximate value for modulus of rigidity may be estimated at 0.065 times the modulus of elasticity.*
3) *With sawn members thicker than 89 mm that season slowly, care should be exercised to avoid overloading in compression before appreciable seasoning of the outer fibre has taken place; otherwise, compression strengths for wet-service conditions shall be used.*
4) *Tabulated values are based on the following standard conditions:*
   a) *343 mm larger dimension for bending and shear and 292 mm larger dimension for tension and compression parallel to grain;*
   b) *dry-service conditions; and*
   c) *standard-term duration of load.*
5) *The designer is strongly advised to check availability of species, grade, and sizes before specifying. See Clause A.6.2.1.2.*

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Table 6.7
Specified strengths and modulus of elasticity for posts and timbers grades, MPa

(See Clauses 6.3.1.1, 6.5.2.2, 6.5.5.2.5, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, 12.4.4.4, and 12.4.4.5.)

Species combination	Grade	Bending, f_b	Longitudinal shear, f_v	Compression: Parallel to grain, f_c	Compression: Perpendicular to grain, f_{cp}	Tension parallel to grain, f_t	Modulus of elasticity: E	Modulus of elasticity: E_{05}
Douglas Fir-Larch	SS	18.3	1.5	13.8	7.0	10.7	12 000	8000
	No. 1	13.8		12.2		8.1	10 500	6500
	No. 2	6.0		7.5		3.8	9500	6000
Hem-Fir	SS	13.6	1.2	11.3	4.6	7.9	10 000	7000
	No. 1	10.2		10.0		6.0	9000	6000
	No. 2	4.5		6.1		2.8	8000	5500
Spruce-Pine-Fir	SS	12.7	1.2	9.9	5.3	7.4	8500	6000
	No. 1	9.6		8.7		5.6	7500	5000
	No. 2	4.2		5.4		2.6	6500	4500
Northern Species	SS	12.0	1.0	7.5	3.5	7.0	8000	5500
	No. 1	9.0		6.7		5.3	7000	5000
	No. 2	3.9		4.1		2.5	6000	4000

Notes:

1) *Posts and timbers have a smaller dimension of at least 114 mm, with a larger dimension not more than 51 mm greater than the smaller dimension.*
2) *Posts and timbers graded to beam and stringer rules may be assigned beam and stringer strength.*
3) *An approximate value for modulus of rigidity may be estimated at 0.065 times the modulus of elasticity.*
4) *With sawn members thicker than 89 mm that season slowly, care should be exercised to avoid overloading in compression before appreciable seasoning of the outer fibre has taken place; otherwise, compression strengths for wet-service conditions shall be used.*
5) *Tabulated values are based on the following standard conditions:*
   a) *343 mm larger dimension for bending and shear and 292 mm larger dimension for tension and compression parallel to grain;*
   b) *dry-service conditions; and*
   c) *standard-term duration of load.*
6) *The designer is strongly advised to check availability of species, grade, and sizes before specifying. See Clause A.6.2.1.2.*

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Table 6.8
Specified strengths and modulus of elasticity for machine stress-rated lumber, 38 mm wide by all depths, MPa
(See Clauses 6.3.2, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, and A.6.5.6 and Table 8.2.)

Grade	Bending, f_b	Modulus of elasticity, E	Tension parallel to grain, f_t		Compression	
			64–184 mm	> 184 mm*	Parallel to grain, f_c	Perpendicular to grain, f_{cp}†
1200F_b -1.2E	17.4	8300	6.7	—	15.1	5.3
1350F_b -1.3E	19.5	9000	8.4	—	16.9	5.3
1450F_b -1.3E	21.0	9000	9.0	—	17.3	5.3
1500F_b -1.4E	21.7	9700	10.1	—	17.5	5.3
1650F_b -1.5E	23.9	10 300	11.4	—	18.1	5.3
1800F_b -1.6E	26.1	11 000	13.2	—	18.7	5.3
1950F_b -1.7E	28.2	11 700	15.4	—	19.3	5.3
2100F_b -1.8E	30.4	12 400	17.7	—	19.9	6.5
2250F_b -1.9E	32.6	13 100	19.6	—	20.5	6.5
2400F_b -2.0E	34.7	13 800	21.6	—	21.1	6.5
2550F_b -2.1E	36.9	14 500	23.0	—	21.7	6.5
2700F_b -2.2E	39.1	15 200	24.1	—	22.3	6.5
2850F_b -2.3E	41.3	15 900	25.8	—	22.9	6.5
3000F_b -2.4E	43.4	16 500	26.9	—	23.5	6.5
The following MSR grades provide a modulus of elasticity with higher corresponding strengths. For these MSR grades, qualification and daily quality control for tensile strength are required.						
1400F_b -1.2E	20.3	8300	9.0	9.0	17.1	5.3
1600F_b -1.4E	23.2	9700	10.7	10.7	17.9	5.3
1650F_b -1.3E	23.9	9000	11.4	11.4	18.1	5.3
1800F_b -1.5E	26.1	10 300	14.6	14.6	18.7	5.3

(Continued)

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Table 6.8 (Concluded)

Grade	Bending, f_b	Modulus of elasticity, E	Tension parallel to grain, f_t		Compression	
			64–184 mm	> 184 mm*	Parallel to grain, f_c	Perpendicular to grain, f_{cp}†
2000F_b -1.6E	29.0	11 000	14.6	14.6	19.5	5.3
2250F_b -1.7E	32.6	11 700	19.6	19.6	20.5	5.3
2250F_b -1.8E	32.6	12 400	19.6	19.6	20.5	6.5
2400F_b -1.8E	34.7	12 400	21.6	21.6	21.1	6.5

** The tension design values for narrow depths may be assigned to these sizes, provided that the lumber is subject to the appropriate level of qualification and daily quality control testing for tension strength, as specified in NLGA SPS 2.*

† Compression perpendicular to grain values are for S-P-F MSR (all grades) and Hem-Fir MSR lumber with E grade of 10 300 MPa or higher. For other species or grades, use corresponding values for visually stress-graded lumber taken from Table 6.4 for the appropriate group.

Notes:

1) *Tabulated values are based on standard-term duration of load and dry-service conditions.*
2) *The size factor K_Z for MSR lumber shall be 1.00, except that K_{Zv} shall be in accordance with Table 6.13, K_{Zcp} shall be determined in accordance with Clause 6.5.6.4, and K_{Zc} shall be calculated in accordance with Clause 6.5.5.2.4.*
3) *The designer is strongly advised to check availability of species, grade, and sizes before specifying. See Clause A.6.2.1.2.*

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Table 6.9
Specified strengths and modulus of elasticity for machine evaluated lumber, 38 mm wide by all depths, MPa
(See Clauses 6.3.2, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, A.6.5.5.3.6, and A.6.5.6.)

Grade	Bending, f_b	Modulus of elasticity, E	Tension parallel to grain, f_t	Compression	
				Parallel to grain, f_c	Perpendicular to grain, f_{cp}*
M-10	20.3	8300	9.0	17.1	5.3
M-11	22.4	10 300	9.5	17.7	5.3
M-12	23.2	11 000	9.5	17.9	5.3
M-13	23.2	9700	10.7	17.9	5.3
M-14	26.1	11 700	11.2	18.7	5.3
M-15	26.1	10 300	12.3	18.7	5.3
M-18	29.0	12 400	13.5	19.5	6.5
M-19	29.0	11 000	14.6	19.5	5.3
M-21	33.3	13 100	15.7	20.7	6.5
M-22	34.0	11 700	16.8	20.9	5.3
M-23	34.7	12 400	21.3	21.1	6.5
M-24	39.1	13 100	20.2	22.3	6.5
M-25	39.8	15 200	22.4	22.5	6.5
M-26	40.6	13 800	20.2	22.7	6.5

** Compression perpendicular to grain values are for S-P-F MEL (all grades) and Hem-Fir MEL lumber with E grade of 10 300 MPa or higher. For other species or grades, use corresponding values for visually stress-graded lumber taken from Table 6.4 for the appropriate group.*

Notes:

1) *Tabulated values are based on standard-term duration of load and dry-service conditions.*
2) *The size factor K_Z for MEL lumber shall be 1.00, except that K_{Zv} shall be in accordance with Table 6.13, K_{Zcp} shall be determined in accordance with Clause 6.5.6.4, and K_{Zc} shall be calculated in accordance with Clause 6.5.5.2.4.*
3) *The designer is strongly advised to check availability of species, grade, and sizes before specifying. See Clause A.6.2.1.2.*

6.4 Modification factors

6.4.1 Load-duration factor, K_D

The specified strength of lumber shall be multiplied by a load-duration factor, K_D, in accordance with Clause 5.3.2.

6.4.2 Service-condition factor, K_S

The specified strength and modulus of elasticity of lumber shall be multiplied by a service-condition factor, K_S, in accordance with Table 6.10.

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6.4.3 Treatment factor, K_T

6.4.3.1 General

The specified strength and modulus of elasticity of lumber shall be multiplied by a treatment factor, K_T, in accordance with Table 6.11.

6.4.3.2 Other treatments

For lumber treated with fire-retardant or other strength-reducing chemicals, strength and stiffness capacities shall be based on the documented results of tests that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 4.3.2.

Note: *The effects of fire-retardant treatments can vary depending on manufacturing materials and processes.*

6.4.4 System factor, K_H

6.4.4.1 Case 1

Specified strengths for sawn lumber members in a system consisting of three or more essentially parallel members spaced not more than 610 mm apart and so arranged that they mutually support the applied load may be multiplied by the system factor for Case 1 given in Table 6.12.

Note: *Case 1 applies to systems of closely spaced structural components such as light-frame trusses, composite building components, and glued-laminated timbers. Case 1 may also apply to some conventional joist and rafter systems where the framing details do not meet the requirements of Clause 6.4.4.2.*

6.4.4.2 Case 2

Specified strengths for sawn lumber used in a system of solid joists, rafters, or studs meeting the requirements of Clause 6.4.4.1 may be multiplied by the system factor for Case 2 given in Table 6.12, provided that the following additional conditions are met:

a) the joists, rafters, or studs are sheathed with plywood, waferboard, or OSB of minimum 9.5 mm thickness, or with 17 mm minimum thickness lumber in combination with panel covering such as underlayment or with wood finish flooring; and

b) the sheathing or subfloor is attached to the members to provide a minimum stiffness equivalent to that provided by 2 in common nails at 150 mm centres at edges of sheathing panels, and 300 mm centres elsewhere.

Tabulated Case 2 system factors shall be applied to single-member section properties and shall not be used in conjunction with augmented section properties based on analysis of partial composite action between lumber and sheathing.

Note: *Case 2 applies to systems such as conventional light-frame wood floor, roof, and wall systems using dimension lumber framing members and minimum required sheathing and connections.*

6.4.4.3 Built-up beams

For lumber in built-up beams consisting of two or more individual members of the same depth that are fastened or glued together so that the beam will deflect as a unit, specified strengths may be multiplied by the system factor, K_H, given in Table 6.12.

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6.4.5 Size factor, K_Z

Δ **6.4.5.1 Visually stress-graded lumber**

Except as provided in Clause 6.4.5.2, specified strengths of visually stress-graded lumber shall be multiplied by a size factor, K_Z, in accordance with Table 6.13.

6.4.5.2 Light framing grades

The size factor, K_Z, for light-framing grades shall be 1.00, except that K_{Zc} shall be calculated in accordance with Clause 6.5.5.2.4, and K_{Zcp} may be determined in accordance with Clause 6.5.6.4.

6.4.5.3 Machine stress-rated (MSR) and machine evaluated lumber (MEL)

The size factor, K_Z, for machine stress-rated lumber and machine evaluated lumber shall be 1.00, except that K_{Zv} shall be in accordance with Table 6.13, K_{Zc} shall be calculated in accordance with Clause 6.5.5.2.4, and K_{Zcp} may be determined in accordance with Clause 6.5.6.4.

Table 6.10
Service-condition factors, K_S
(See Clauses 6.4.2, 10.5.2, 10.5.3, 12.4.4.4, and 12.4.4.5.)

K_S	Property	Dry-service conditions	Wet-service conditions: sawn lumber, piling, and poles of least dimension	
			89 mm or less	Over 89 mm
K_{Sb}	Bending	1.00	0.84	1.00
K_{Sf}	Fracture shear	1.00	0.70	0.70
K_{Sv}	Longitudinal shear	1.00	0.96	1.00
K_{Sc}	Compression parallel to grain	1.00	0.69	0.91
K_{Scp}	Compression perpendicular to grain	1.00	0.67	0.67
K_{St}	Tension parallel to grain	1.00	0.84	1.00
K_{SE}	Modulus of elasticity	1.00	0.94	1.00

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Table 6.11
Treatment factor, K_T
(See Clause 6.4.3.1.)

Product	Dry-service conditions	Wet-service conditions
Untreated lumber	1.00	1.00
Preservative-treated non-incised lumber	1.00	1.00
Preservative-treated incised lumber of thickness 89 mm or less		
Modulus of elasticity	0.90	0.95
Other properties	0.75	0.85
Fire-retardant-treated lumber	See Clause 6.4.3.2 for effects of fire-retardant treatment	

Table 6.12
System factors, K_H
(See Clauses 6.4.4.1–6.4.4.3.)

		Case 2†		
Property	Case 1*	Visually graded	MSR	Built-up beams
Bending	1.10	1.40	1.20	1.10
Longitudinal shear	1.10	1.40	1.20	1.10
Compression parallel to grain	1.10	1.10	1.10	1.00
Tension parallel to grain	1.10	—	—	1.00
All other properties	1.00	1.00	1.00	1.00

* *See Clause 6.4.4.1 for conditions applying to Case 1.*
† *See Clause 6.4.4.2 for conditions applying to Case 2.*

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Table 6.13
Size factor, K_Z, for visually stress-graded lumber
(See Clauses 6.4.5.1, 6.4.5.3, 10.5.3, and 16.2.3.1.1 and Tables 6.8 and 6.9.)

	Bending and shear, K_{Zb}, K_{Zv}			Tension parallel to grain, K_{Zt}	Compression perpendicular to grain, K_{Zcp}	Compression parallel to grain, K_{Zc}	All other properties
	Smaller dimension, mm						
Larger dimension, mm	38–64	89–102	114 or more	All	All	All	All
38	1.7	—	—	1.5	See Clause 6.5.6.4	Value computed using formula in Clause 6.5.5.2.4	1.0
64	1.7	—	—	1.5			1.0
89	1.7	1.7	—	1.5			1.0
114	1.5	1.6	1.3	1.4			1.0
140	1.4	1.5	1.3	1.3			1.0
184–191	1.2	1.3	1.3	1.2			1.0
235–241	1.1	1.2	1.2	1.1			1.0
286–292	1.0	1.1	1.1	1.0			1.0
337–343	0.9	1.0	1.0	0.9			1.0
387 or larger	0.8	0.9	0.9	0.8			1.0

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6.5 Strengths and resistances

6.5.1 General

Clause 6.5 specifies design data and methods that apply to sawn lumber of rectangular cross-section.

6.5.2 Sizes

6.5.2.1 Standard dry size

Except as provided in Clause 6.5.2.2, the standard dry size rounded to the nearest millimetre (net dimension) of lumber shall be used.

6.5.2.2 Green size

In conjunction with Tables 6.6 and 6.7, green manufactured sizes may be used for all service conditions.

Notes:

1) *In developing specified strengths in this Standard, variables of moisture content and shrinkage, and their relationship to strength and stiffness, have been taken into account. Standard sizes and net dimensions of structural lumber and timbers are given in CSA O141.*
2) *Sizes rounded to the nearest millimetre are given in Table A.3.*

6.5.3 Bending moment resistance

6.5.3.1 General

The factored bending moment resistance, M_r, of sawn lumber members shall be calculated as follows:

$M_r = \phi F_b S K_{Zb} K_L$

where

ϕ = 0.9

F_b = $f_b (K_D K_H K_{Sb} K_T)$

where

f_b = specified strength in bending, MPa (see Tables 6.4 to 6.9)

S = section modulus, mm^3

K_{Zb} = size factor in bending (see Clause 6.4.5)

K_L = lateral-stability factor (see Clause 6.5.3.2)

6.5.3.2 Lateral-stability factor, K_L

6.5.3.2.1 Bending members with depth-to-width ratio ≤ 2.5:1

For laterally-unsupported bending members, the lateral-stability factor, K_L, may be taken as unity, provided that the maximum depth-to-width ratio of the member does not exceed 2.5:1.

Δ ##### 6.5.3.2.2 Bending members with depth-to-width ratio > 2.5:1

For bending members, other than built-up beams, with a maximum depth-to-width ratio greater than 2.5:1, lateral support shall be provided at points of bearing to prevent lateral displacement and rotation and K_L shall be determined in accordance with Clause 7.5.6.4, except as otherwise permitted in Clause 6.5.3.2.3.

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6.5.3.2.3 Prescriptive lateral support provisions

The lateral-stability factor, K_L, may be taken as unity when lateral support is provided at points of bearing to prevent lateral displacement and rotation, provided that the maximum depth-to-width ratio of the member does not exceed the following values:

a) 4:1 if no additional intermediate support is provided;
b) 5:1 if the member is held in line by purlins or tie rods;
c) 6.5:1 if the compressive edge is held in line by direct connection of decking or joists spaced not more than 610 mm apart;
d) 7.5:1 if the compressive edge is held in line by direct connection of decking or joists spaced not more than 610 mm apart and bridging or blocking is installed at intervals not exceeding eight times the depth of the member; or
e) 9:1 if both edges are held in line.

Δ #### 6.5.3.2.4 Built-up beams

For built-up beams consisting of two or more individual members of the same depth, the maximum ratio permitted in Clause 6.5.3.2.1 for laterally-unsupported members may be based on the total width of the beam, provided that the individual members are fastened together securely at intervals not exceeding four times the depth. For other conditions, lateral support shall be provided at points of bearing to prevent lateral displacement and rotation and K_L shall be determined in accordance with Clause 7.5.6.4.

6.5.4 Shear resistance

6.5.4.1 General

All members shall be verified for shear resistance in accordance with Clause 6.5.4.3. Members with notches on tension side shall also be verified for fracture shear resistance at the notch in accordance with Clause 6.5.4.4.

6.5.4.2 Loads near supports

6.5.4.2.1 Loads for shear resistance

In Clause 6.5.4.3, the effect of all loads acting within a distance from a support equal to the depth of the member may be omitted from consideration.

6.5.4.2.2 Loads for fracture shear resistance at notch

In Clause 6.5.4.4.1, the effect of all loads acting on the member shall be taken into account, including the loads acting within a distance from a support equal to the depth of the member.

6.5.4.3 Shear resistance

The factored shear resistance, V_r, shall not be less than the maximum factored shear force, V_f, and shall be calculated as follows:

$$V_r = \phi F_v \frac{2A_n}{3} K_{Zv}$$

where

ϕ = 0.9

F_v = $f_v\,(K_D\,K_H\,K_{Sv}\,K_T)$

where

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f_v = specified strength in shear, MPa (see Clause 6.3)

A_n = net cross-sectional area, mm² (see Clause 5.3.8)

K_{Zv} = size factor in shear (see Clause 6.4.5)

6.5.4.4 Fracture shear resistance at notch

6.5.4.4.1 General

The factored fracture shear resistance at a notch on the tension side at a support, F_r, shall not be less than the maximum factored shear force, V_f (see Clause 6.5.4.4.3), and shall be calculated as follows:

$F_r = \phi\, F_f\, A_g\, K_N$

where

ϕ = 0.9

F_f = $f_f (K_D\, K_H\, K_{Sf}\, K_T)$

where

f_f = specified fracture shear strength at notch, MPa

= 0.50 for all sawn lumber members

A_g = $b \times d$ = gross cross-sectional area, mm²

K_N = notch factor (see Clause 6.5.4.4.2)

6.5.4.4.2 Notch factor, K_N

The notch factor for members of rectangular cross-sections shall be calculated as follows:

$$K_N = \left[0.006d\left(1.6\left(\frac{1}{\alpha} - 1\right) + \eta^2\left(\frac{1}{\alpha^3} - 1\right)\right)\right]^{-\frac{1}{2}}$$

where

d = depth of cross-section, mm

α = $1 - (d_n/d)$

where

d_n = depth of notch measured normal to the member axis in accordance with Figure 6.1, mm, which shall not exceed $0.25d$

η = e/d

where

e = length of notch measured parallel to the member axis, mm, from the centre of the nearest support to the re-entrant corner of notch (see Figure 6.1). For a member notched over an end support, the length of support may be taken as the lesser of minimum required bearing length (see Clause 6.5.6) or the actual bearing length. For a continuous member, the length of support equals the actual bearing length.

Note: *Values of $K_N\sqrt{d}$ for selected combinations of α and η are given in Table 6.14.*

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Figure 6.1
Determination of length and depth of notch
(See Clause 6.5.4.4.2 and 7.5.7.5.2.)

Legend:

d_n = depth of notch

e = length of notch

Table 6.14
Values of $K_N\sqrt{d}$
(See Clause 6.5.4.4.2.)

	α				
η	0.75	0.80	0.85	0.90	0.95
0.15	17.2	19.9	23.7	29.9	43.5
0.20	16.8	19.5	23.3	29.4	42.8
0.25	16.4	19.0	22.8	28.8	42.0
0.30	15.9	18.5	22.2	28.1	41.0
0.35	15.4	18.0	21.5	27.3	39.9
0.40	14.9	17.4	20.9	26.5	38.8
0.45	14.3	16.8	20.2	25.7	37.6
0.50	13.8	16.2	19.5	24.8	36.4
0.60	12.7	15.0	18.1	23.1	34.0

(Continued)

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Table 6.14 (Concluded)

	α				
η	0.75	0.80	0.85	0.90	0.95
0.70	11.8	13.9	16.8	21.5	31.7
0.80	10.9	12.8	15.6	20.0	29.6
0.90	10.1	11.9	14.5	18.7	27.6
1.00	9.36	11.1	13.5	17.4	25.8
1.20	8.15	9.70	11.8	15.3	22.7
1.40	7.20	8.57	10.5	13.6	20.2
1.60	6.42	7.66	9.39	12.1	18.1
1.80	5.79	6.91	8.48	11.0	16.4
2.00	5.26	6.29	7.72	10.0	14.9

Notes:
1) $\alpha = 1 - d_n/d$; $\eta = e/d$
2) *Interpolation may be applied for intermediate values of α and η.*

6.5.4.4.3 Shear force at notches

In the calculation of the fracture shear resistance at a notch near a support, the associated applied force is the factored shear force in the member at the support. The shear force shall be calculated using the component of the force normal to the member axis.

Note: *Consideration of the shear resistance at notch concerns avoidance of fracture at a re-entrant corner of a notch and does not negate the need to ensure that the residual cross-section at a notch can resist the factored shear force.*

6.5.5 Compressive resistance parallel to grain

6.5.5.1 Effective length, L_e

Unless otherwise noted in this Standard, the effective length $L_e = K_e L$ shall be used in determining the slenderness ratio of compression members.

Note: *Recommended effective-length factors, K_e, for compression members are given in Table A.4.*

6.5.5.2 Simple compression members

6.5.5.2.1 General

The factored compressive resistance parallel to grain for sawn lumber, P_r, shall be checked for both axes.

Δ #### 6.5.5.2.2 Slenderness ratio, C_C

The slenderness ratio, C_C, of simple compression members of constant rectangular section shall not exceed 50 and shall be calculated for both axes as follows:

$$C_C = \frac{\text{effective length associated with width}}{\text{member width}}$$

$$C_C = \frac{\text{effective length associated with depth}}{\text{member depth}}$$

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6.5.5.2.3 Eccentrically applied loads

Bending moments due to eccentrically applied axial loads shall be considered in accordance with Clause 6.5.9.

6.5.5.2.4 Factored compressive resistance under axial load

The factored compressive resistance parallel to grain, P_r, shall be calculated as follows:

$P_r = \phi F_C A K_{Zc} K_C$

where

ϕ = 0.8

F_C = $f_c (K_D K_H K_{Sc} K_T)$

where

f_c = specified strength in compression parallel to grain, MPa (see Tables 6.4 to 6.9)

K_{Zc} = $6.3 (dL)^{-0.13} \leq 1.3$

where

d = dimension in direction of buckling (depth or width), mm

L = length of member, mm

Note: *K_{Zc} given above is conservative for members that have a non-uniform compressive force along its length. Refer to Clause A.6.5.5.2.4 for a design procedure involving non-uniform compressive forces.*

Δ #### 6.5.5.2.5 Slenderness factor, K_C

For both axes, the slenderness factor, K_C, shall be calculated as follows:

$$K_C = \left[1.0 + \frac{F_C K_{Zc} C_C^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

where

E_{05} = $0.82E$ for MSR lumber, MPa

= $0.75E$ for MEL lumber, MPa

= as specified in Tables 6.4 to 6.7 for visually stress-graded lumber

6.5.5.3 Spaced compression members

Spaced compression members shall be designed using the specified strengths and adjustment factors for sawn lumber.

Note: *Spaced compression members may be designed in accordance with the provisions of Clause A.6.5.5.3.*

6.5.5.4 Built-up compression members

6.5.5.4.1 General

Built-up rectangular compression members shall consist of two to five individual members of at least 38 mm thickness joined with nails or bolts, or bolts and split ring connectors. The factored compressive resistance of built-up compression members may be evaluated in accordance with Clauses 6.5.5.4.2 to 6.5.5.4.4, provided that the minimum values of end distance, edge distance, and spacing for fasteners conform to the appropriate requirements in Clause 12 and the maximum value of end distance does not exceed 1.2 times the minimum value. The factored compressive resistance of the built-up compression member may be taken as the greater of the values calculated in accordance with Clause 6.5.5.4.2,

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6.5.5.4.3, or 6.5.5.4.4, or the combined factored resistance of the individual pieces taken as independent members.

Note: *Slenderness ratios are calculated according to Clause 6.5.5.2.2 using the overall dimensions of the composite member or the dimensions of the individual pieces, as appropriate.*

6.5.5.4.2 Nailed built-up compression members

The factored compressive resistance of a built-up compression member fastened together with nails or spikes may be taken as 60% of the compressive strength of a solid member of equivalent gross cross-sectional dimensions designed in accordance with Clause 6.5.5.2, provided that the following requirements are satisfied:

a) spacing of nails along the member length shall not exceed six times the thickness of the thinnest piece and spacing perpendicular to the member length shall not exceed 20 times the nail diameter;
b) all nails shall penetrate through at least 3/4 of the thickness of the last individual piece and nails shall be driven alternately from either face of the built-up member along the length; and
c) when the individual pieces of the built-up member are wider than three times their thickness, there shall be at least two rows of nails across the member width.

6.5.5.4.3 Bolted built-up compression members

The factored compressive resistance of a built-up compression member fastened together by minimum 6 mm (1/4 in) diameter bolts may be taken as 75% of the compressive strength of a solid member of equivalent gross cross-sectional dimensions designed in accordance with Clause 6.5.5.2, provided that the following requirements are satisfied:

a) spacing of bolts along the member length shall not exceed six times the thickness of the thinnest piece and spacing perpendicular to the member length shall not exceed ten times the bolt diameter; and
b) when the individual pieces of the built-up member are wider than three times their thickness, there shall be at least two rows of bolts across the member width.

6.5.5.4.4 Split-ring-connected built-up compression members

The factored compressive resistance of a built-up compression member fastened together at intervals not exceeding six times the thickness of the thinnest piece by minimum 12 mm (1/2 in) diameter bolts and 63 mm (2-1/2 in) split-ring connectors may be calculated as having 80% of the compressive strength of a solid member of equivalent gross cross-sectional dimensions designed in accordance with Clause 6.5.5.2.

6.5.5.4.5 Built-up compression members as simple compression members

Except for spaced compression members, the factored compressive resistance of built-up compression members not meeting the requirements of Clauses 6.5.5.4.1 to 6.5.5.4.4 shall be taken as the combined factored compressive strength of the individual pieces considered as independent members.

6.5.5.4.6 Strong axis buckling

The strength reduction factors given in Clauses 6.5.5.4.2 to 6.5.5.4.4 may be omitted for buckling in the direction of the strong axis of the laminations.

6.5.5.5 Stud walls

When stud walls are adequately sheathed on at least one side, as in light-frame construction, the dimension of the stud normal to the sheathing may be used in calculating the slenderness ratio.

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6.5.6 Compressive resistance perpendicular to grain

Note: *See Clause A.6.5.6.*

6.5.6.1 General

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 6.5.6.2 and 6.5.6.3.

Note: *See Figure 6.2.*

Figure 6.2
Compressive resistance perpendicular to grain
(See Clauses 6.5.6.1, 6.5.6.3.1, 7.5.9.1, 7.5.9.3.1, 8.4.7.3.1, and 15.3.3.7.3.)

6.5.6.2 Effect of all applied loads

The factored compressive resistance perpendicular to grain, Q_r, under the effect of all factored applied loads shall be calculated as follows:

$Q_r = \phi\, F_{cp}\, A_b\, K_B\, K_{Zcp}$

where

ϕ = 0.8

F_{cp} = $f_{cp}\,(K_D\, K_{Scp}\, K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain, MPa (see Tables 6.4 to 6.9)

A_b = bearing area, mm^2

K_B = length-of-bearing factor (see Clause 6.5.6.5)

K_{Zcp} = size factor for bearing (see Clause 6.5.6.4)

6.5.6.3 Effect of loads applied near a support

6.5.6.3.1 Factored compressive resistance perpendicular to grain

The factored compressive resistance perpendicular to grain, Q'_r, under the effect of only those factored loads applied within a distance from the centre of the support equal to the depth of the member shall be calculated as follows:

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$Q_r' = (2/3)\ \phi\ F_{cp}\ A_b'\ K_B\ K_{Zcp}$

where

ϕ = 0.8

F_{cp} = $f_{cp}\ (K_D\ K_{Scp}\ K_T)$

A_b' = average bearing area, mm^2 (see Clause 6.5.6.3.2)

Note: *See Figure 6.2.*

Δ **6.5.6.3.2 Unequal bearing areas on opposite surfaces of a member**

Where unequal bearing areas are used on opposite surfaces (top and bottom) of a member, the average bearing area, A_b', shall be calculated as follows:

$$A_b' = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \leq 1.5b(L_{b1})$$

where

b = average bearing width (perpendicular to grain), mm

L_{b1} = lesser bearing length, mm

L_{b2} = larger bearing length, mm

Note: *Where a compression member bears on a continuously supported bearing plate, A_b' may be calculated as $1.5b(L_{b1})$.*

6.5.6.4 Size factor for bearing, K_{Zcp}

When the width of a member (dimension perpendicular to the direction of the load) is greater than the depth of the member (dimension parallel to the direction of the load), the specified strength in compression perpendicular to grain may be multiplied by a size factor for bearing, K_{Zcp}, in accordance with Table 6.15.

Table 6.15
Size factor for bearing, K_{Zcp}
(See Clause 6.5.6.4.)

Ratio of member width to member depth*	K_{Zcp}
1.0 or less	1.00
2.0 or more	1.15

* *Interpolation applies for intermediate ratios.*

6.5.6.5 Length-of-bearing factor, K_B

When lengths of bearing or diameters of washers are less than 150 mm, specified strengths in compression perpendicular to grain may be multiplied by a length-of-bearing factor, K_B, in accordance with Table 6.16, provided that

a) no part of the bearing area is less than 75 mm from the end of the members; and

b) bearing areas do not occur in positions of high bending stresses.

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Table 6.16
Length-of-bearing factor, K_B
(See Clause 6.5.6.5.)

Bearing length (parallel to grain) or washer diameter, mm	K_B
12.5 and less	1.75
25.0	1.38
38.0	1.25
50.0	1.19
75.0	1.13
100.0	1.10
150.0 or more	1.00

Note: *Alternatively, K_B may be calculated as follows:*

K_B = $(L_b + 9.525)/L_b$ *for* $L_b < 150$ *mm*

K_B = 1.00 *for* $L_b \geq 150$ *mm*

where

L_b = *bearing length measured parallel to grain or washer diameter, mm*

Δ **6.5.7 Compressive resistance at an angle to grain**

The factored compressive resistance at an angle to grain, N_r, shall be calculated as follows:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

P_r = factored compressive resistance parallel to grain, N (see Clause 6.5.5.2.4, assuming K_C = 1.00)

Q_r = factored compressive resistance perpendicular to grain, N (see Clause 6.5.6.2)

θ = angle between direction of grain and direction of load, degrees

6.5.8 Tensile resistance parallel to grain

The factored tensile resistance, T_r, parallel to grain shall be calculated as follows:

$T_r = \phi\, F_t\, A_n\, K_{Zt}$

where

ϕ = 0.9

F_t = $f_t\,(K_D\, K_H\, K_{St}\, K_T)$

where

f_t = specified strength in tension parallel to grain, MPa (see Tables 6.4 to 6.9)

A_n = net cross-sectional area, mm^2 (see Clause 5.3.8)

K_{Zt} = size factor in tension (see Clause 6.4.5)

6.5.9 Resistance to combined bending and axial load

Members subject to combined bending and compressive or tensile axial loads shall be designed to satisfy the following appropriate interaction equation:

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$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] \leq 1$$

or

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1$$

where

P_f = factored compressive axial load

P_r = factored compressive load resistance parallel to grain calculated in accordance with Clause 6.5.5

M_f = factored bending moment

M_r = factored bending moment resistance calculated in accordance with Clause 6.5.3

P_E = Euler buckling load in the plane of the applied moment

$$= \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$$

where

E_{05} = modulus of elasticity for design of compression members, MPa

I = moment of inertia in the plane of the applied moment, mm^4

L_e = effective length in the plane of the applied moment, mm

= $K_e L$

where

K_e = the effective-length factor given in Clause A.6.5.5.1

T_f = factored tensile axial load

T_r = factored tensile load resistance parallel to grain calculated in accordance with Clause 6.5.8

Note: *See Clause A.6.5.9 for a more detailed interaction formula for combined bending and compressive loads.*

6.5.10 Decking

6.5.10.1 General

To utilize continuity in the design of decking, the conditions specified in Clauses 6.5.10.2 to 6.5.10.5 shall apply.

6.5.10.2 Plank decking

6.5.10.2.1 Connection requirements

Material shall be 75 mm or more in width and shall be tongued and grooved or splined. Planks 38 mm or less in thickness shall be nailed to the supporting members with nails not shorter than twice the thickness of the plank and not less than 63 mm (2-1/2 in). Planks thicker than 38 mm shall be toe-nailed to the supporting members with one 127 mm (5 in) toe-nail and face-nailed with one or more nails not less than 152 mm (6 in) in length. Planks 140 mm or less in width shall be nailed with two nails to each support. Planks more than 140 mm in width shall be nailed with three nails to each support.

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6.5.10.2.2 Butt joints

In bridges, each plank shall extend over at least one support. In roofs and floors, planks not extending over at least one support in any span may be used if they are

a) double tongue-and-groove planks more than 38 mm in thickness;
b) flanked by planks that rest on both supports of that span; and
c) separated by at least six planks in that span, each of which extends over at least one support.

6.5.10.3 Nail-laminated decking

6.5.10.3.1 Connection requirements

Materials shall be 38 mm or more in thickness, 64 mm or more in width, laid on edge, and spiked together. Nails used to spike the laminations together shall be at least 4 in long for 38 mm thickness laminations and 6 in long for 64 mm thickness laminations. Decking 140 mm or less in depth shall be spiked together with a staggered single row of nails at intervals of not more than 450 mm in the row. One nail shall be placed not more than 100 mm from the end of each lamination. Decking more than 140 mm in depth shall be spiked together with a staggered double row of nails at intervals of not more than 450 mm in each row. Two nails shall be placed not more than 100 mm from the end of each lamination. Each lamination shall be toe-nailed to each support with not less than 4 in nails.

6.5.10.3.2 Butt joints

In bridges, each lamination shall extend over at least one support. In roofs and floors, laminations not extending over at least one support in any span shall be flanked by laminations that rest on both supports of that span, and shall be separated by at least six laminations in that span, each of which extends over at least one support.

6.5.10.4 Deflection calculations

For uniform design loads, decking deflections for the laying patterns described in Table 6.18 shall be calculated using the formulas given in Table 6.18. For other loading conditions or laying patterns, deflections shall be calculated using recognized engineering formulas.

6.5.10.5 Bending

Bending moments for plank decking laid in a controlled, random pattern, as described in Table 6.18, shall be calculated based on simple span moments. For other deck patterns, bending moments shall be calculated based on recognized engineering formulas.

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Table 6.18
Laying patterns and deflection formulas for decking
(See Clauses 6.5.10.4 and 6.5.10.5.)

Pattern	Description	Deflection*
Simple span	All pieces bear on two supports only	$\Delta_1 = \frac{5w\ell^4}{384E_S I}$
Controlled random	Decking continuous for three or more spans End joints staggered in adjacent planks not less than 610 mm Joints in same general line separated by at least two intervening courses End joints in first half of end spans avoided Requirements of Clause 6.5.10.2.2 or 6.5.10.3.2 shall be met	$\Delta_2 = 0.77\Delta_1$
Continuous over two spans	All pieces bear on three supports	$\Delta_3 = 0.42\Delta_1$

* *where*

Δ = *deflection, mm*

w = *uniformly distributed specified load, kN/m²*

ℓ = *span, mm*

E_S = *modulus of elasticity for stiffness calculations, MPa (see Clause 5.4.1)*

I = *moment of inertia of the decking, mm⁴ per m of width*

6.5.11 Permanent wood foundations

6.5.11.1 General

All lumber and plywood in permanent wood foundations shall be treated with a preservative in accordance with the CAN/CSA-O80 Series, except where exempted from treatment by CAN/CSA-S406.

The provisions for design of permanent wood foundations are predicated on the use of lumber and plywood identified by a certification mark on the material that confirms that treatment, where required, has been carried out by a plant certified under CSA O322.

Note: *CSA O322 outlines procedures for the certification of treatment plants and for the identification of wood materials pressure treated for use in permanent wood foundations. See Clause A.6.5.11 for more details about wood foundations.*

6.5.11.2 Wall footings

In the design of wall footings for permanent wood foundations, the specified strength in bending perpendicular to grain for wood footing plates that are wider than the bottom plates shall not exceed one-third the factored shear resistance. A granular layer, under footings, may be assumed capable of distributing the load transferred by the footing to the undisturbed soil at an angle of not more than 30° to the vertical.

6.5.12 Specific truss applications

Note: *See Clause A.6.5.12.*

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6.5.12.1 Scope

The design methods specified in Clause 6.5.12 apply only to fully triangulated light-gauge metal-plate-connected wood roof trusses that meet the following conditions:

a) spacing not to exceed 610 mm;
b) clear span between inside face of supports not to exceed 12.20 m;
c) total truss length between outermost panel points not to exceed 18.0 m, with no single cantilever length exceeding 25% of the adjacent clear span; and
d) top chord pitch not less than 2 in 12.

Clause 6.5.12 does not apply to girder, bowstring, semi-circular, or attic trusses (which have non-triangulated sections), or to flat or floor trusses.

Notes:

1) *The provisions of Clause 6.5.12 are predicated on the determination of load effects on members and connections in accordance with recognized methods of analysis such as those in* Truss Plate Institute of Canada (TPIC).
2) *The provisions of Clause 6.5.12 are predicated on truss fabrication and erection in accordance with recognized practices such as those referenced by* TPIC.

6.5.12.2 General

Except as modified in Clauses 6.5.12.3 to 6.5.12.5, truss member design shall be in accordance with the sawn lumber provisions of Clauses 6.5.1 to 6.5.9. Truss plate design shall be in accordance with Clause 12.8.

6.5.12.3 Compressive resistance parallel to grain

6.5.12.3.1 Effective length, L_e

Unless otherwise required, the effective length, $L_e = K_e L_{pa}$, shall be used in determining the slenderness ratio for truss compression members, where

L_e = effective length of truss compression member

K_e = effective-length factor for truss compression member

L_{pa} = actual length of member between adjacent panel points

Note: *Effective-length factors for compression members and conditions under which these factors apply can be found in* TPIC.

6.5.12.3.2 Chord member with a splice

A compression chord member containing a metal-plate-connected splice may be considered continuous for a specific load case if the splice is located within ±10% of the panel length from an inflection point.

Δ #### 6.5.12.3.3 Size factor, K_{Zc}

The member length, L, used in Clause 6.5.5.2.4 to compute the size factor, K_{Zc}, shall be the greater of the panel length or one-half the chord length between pitch breaks.

6.5.12.4 Compressive resistance perpendicular to grain

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 6.5.6.2 and 6.5.6.3.1.

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The requirements of Clause 6.5.6.3.1 may be met by providing bearing reinforcement against the effects of concentrated bearing loads acting near a support.

Note: *Bearing reinforcing details using light-gauge metal plates that conform to Clause 12.8.1 can be found in* TPIC.

6.5.12.5 Resistance to combined bending and axial load

Members subject to combined bending and compressive or tensile axial load shall be designed to satisfy the appropriate interaction equation:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{K_M M_r} \leq 1.0$$

or

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where

P_f = factored compressive axial load

P_r = factored compressive load resistance parallel to grain calculated in accordance with Clauses 6.5.5 and 6.5.12.3

M_f = factored bending moment (unless mid-panel deflection of truss chords is limited to criteria such as are specified in *TPIC*, moments shall be multiplied by $[1/(1-P_f/P_E)]$, where P_E is defined in Clause 6.5.9

K_M = bending capacity modification factor as defined in Table 6.19

M_r = factored bending moment resistance calculated in accordance with Clause 6.5.3

T_f = factored tensile axial load

T_r = factored tensile load resistance parallel to grain calculated in accordance with Clause 6.5.8

Δ

Table 6.19
Bending capacity modification factor, K_M, for specific truss applications
(See Clauses 6.5.12.5 and A.6.5.12.5.)

K_M*	Applicable condition
$\left[1.31 + 0.12\left(\frac{M_1}{M_2}\right)\right] \bullet \left(\frac{L_{pa}}{d}\right)^{-\frac{1}{6}} \leq 1.3$	Compression chord members continuous over one or more panel points, and where $1.0 < \frac{M_1}{M_2} \leq 3.0$
$\left[2.20 - 0.53\left(\frac{M_1}{M_2}\right) - 0.64\left(\frac{M_1}{M_2}\right)^2 + 0.41\left(\frac{M_1}{M_2}\right)^3\right] \bullet \left(\frac{L_{pa}}{d}\right)^{-\frac{1}{6}} \leq 1.3$	Compression chord members continuous over one or more panel points, and where $-1.0 \leq \frac{M_1}{M_2} \leq 1.0$
$1.67\left(\frac{L_{pa}}{d}\right)^{-\frac{1}{6}} \leq 1.3$	All other compression chord members

* *where*

(Continued)

Table 6.19 (Concluded)

M_1 = *maximum bending moment between panel points, N•mm*

M_2 = *maximum of the two panel point bending moments, N•mm*

L_{pa} = *actual length of the member between adjacent panel points, mm*

d = *depth of the member between adjacent panel points, mm*

Note: *The sign of the bending moments, M_1 and M_2, is retained in determining K_M. The factored bending moment, M_f, used in Clause 6.5.12.5 is the larger of the absolute values of M_1 and M_2.*

6.6 Serviceability limit states

The design of sawn lumber members for serviceability limit states shall be in accordance with Clauses 5.1.3 and 5.4.

7 Glued-laminated timber (glulam)

7.1 Scope

The design data and methods specified in Clause 7 apply only to glued-laminated timber manufactured in accordance with CAN/CSA-O122.

7.2 Materials

7.2.1 Stress grades

Design in accordance with Clause 7 is based on the use of the stress grades of glued-laminated timber given in Table 7.1.

Note: *Part 4 of the* National Building Code of Canada *requires that glued-laminated timber be fabricated in plants conforming to CSA O177. A list of certified manufacturers can be obtained from the certifying agency or agencies providing certification service.*

Table 7.1
Glued-laminated timber stress grades
(See Clause 7.2.1.)

	Wood species		
Primary application	**Douglas Fir-Larch**	**Spruce-Lodgepole Pine-Jack Pine**	**Hem-Fir and Douglas Fir-Larch**
Bending members	20f-E, 24f-E 20f-EX, 24f-EX	20f-E 20f-EX	24f-E 24f-EX
Compression members	16c-E	12c-E	
Tension members	18t-E	14t-E	

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7.2.2 Appearance grades

7.2.2.1 General

Except as noted in Clause 7.2.2.2, appearance grades as defined in CAN/CSA-O122 do not affect the specified strength.

7.2.2.2 Textured finishes

Some manufacturers offer a variety of textured finishes. Designers should check the availability of textured finishes before specifying.

Such finishes can change the finished sizes and tolerances given in CAN/CSA-O122. Depending on the degree of texturing, the designer might need to compensate for any resulting reduction of cross-section and/or specified strength of the member.

7.3 Specified strengths and moduli of elasticity

The specified strengths and moduli of elasticity for glued-laminated timber are given in Table 7.2.

Table 7.2
Specified strengths and moduli of elasticity for glued-laminated timber, MPa
(See Clauses 7.3, 7.5.6.4.4, 7.5.6.5.1, 7.5.6.6.1, 7.5.6.6.2, 7.5.7.3, 7.5.7.4, 7.5.8.5, 7.5.8.6, 7.5.9.2, 7.5.9.3.1, 7.5.11, 10.5.3–10.5.5, 10.6.3.1, 10.6.3.6, 10.6.3.7, 12.4.4.4, 12.4.4.5, 16.2.3.1.2, 16.2.3.1.3, 16.2.3.4, A.6.5.5.3.6, A.12.7.3.3.1, A.12.7.3.3.2, and B.2.2.)

	Douglas Fir-Larch					
	24f-E	**24f-EX**	**20f-E**	**20f-EX**	**18t-E**	**16c-E**
Bending moment (pos.), f_b	30.6	30.6	25.6	25.6	24.3	14.0
Bending moment (neg.), f_b	23.0	30.6	19.2	25.6	24.3	14.0
Longitudinal shear, f_v	2.0	2.0	2.0	2.0	2.0	2.0
Compression parallel, f_c	30.2*	30.2*	30.2*	30.2*	30.2	30.2
Compression parallel combined with bending, f_{cb}	30.2*	30.2	30.2*	30.2	30.2	30.2
Compression perpendicular, f_{cp}	7.0	7.0	7.0	7.0	7.0	7.0
Compression face bearing						
Tension face bearing	7.0	7.0	7.0	7.0	7.0	7.0
Tension net section, f_{tn} (see Clause 7.5.11)	20.4*	20.4	20.4*	20.4	23.0	20.4
Tension gross section, f_{tg}	15.3*	15.3	15.3*	15.3	17.9	15.3
Tension perpendicular to grain, f_{tp}	0.83	0.83	0.83	0.83	0.83	0.83
Modulus of elasticity, E	12 800	12 800	12 400	12 400	13 800	12 400

(Continued)

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Table 7.2 (Concluded)

	Spruce-Lodgepole Pine-Jack Pine				Hem Fir and Douglas Fir-Larch	
	20f-E	20f-EX	14t-E	12c-E	24f-E	24-EX
Bending moment (pos.), f_b	25.6	25.6	24.3	9.8	30.6	30.6
Bending moment (neg.), f_b	19.2	25.6	24.3	9.8	23.0	30.6
Longitudinal shear, f_v	1.75	1.75	1.75	1.75	1.75	1.75
Compression parallel, f_c	25.2*	25.2*	25.2	25.2	—	—
Compression parallel combined with bending, f_{cb}	25.2*	25.2	25.2	25.2	—	—
Compression perpendicular, f_{cp}	5.8	5.8	5.8	5.8	4.6	7.0
Compression face bearing						
Tension face bearing	5.8	5.8	5.8	5.8	7.0	7.0
Tension net section, f_{tn} (see Clause 7.5.11)	17.0*	17.0	17.9	17.0	20.4*	20.4
Tension gross section, f_{tg}	12.7*	12.7	13.4	12.7	15.3*	15.3
Tension perpendicular to grain, f_{tp}	0.51	0.51	0.51	0.51	0.83	0.83
Modulus of elasticity, E	10 300	10 300	10 700	9 700	13 100	13 100

* *The use of this stress grade for this primary application is not recommended.*

Notes:

1) *Designers should check the availability of grades before specifying.*
2) *Tabulated values are based on the following standard conditions:*
   a) *dry-service conditions; and*
   b) *standard-term duration of load.*

7.4 Modification factors

7.4.1 Load-duration factor, K_D

The specified strength of glued-laminated timber shall be multiplied by a load-duration factor, K_D, in accordance with Clause 5.3.2.

7.4.2 Service-condition factor, K_S

7.4.2.1 General

The specified strength and modulus of elasticity of glued-laminated timber shall be multiplied by service-condition factor, K_S, in accordance with Table 7.3.

7.4.2.2 Semi-wet-service conditions

Where glued-laminated timber members that could be exposed to free moisture are adequately protected, an intermediate value of K_S between 1.00 and that listed in Table 7.3 may be used.

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7.4.3 Treatment factor, K_T

Δ **7.4.3.1 General**

For preservative-treated, non-incised glued-laminated timber, the treatment factor, K_T, may be taken as unity.

7.4.3.2 Other treatments

For glued-laminated timber treated with fire-retardant or other potentially strength-reducing chemicals, strength and stiffness capacities shall be based on documented results of tests that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 4.3.2. Glued-laminated members shall not be treated with water-borne chemicals after gluing.

7.4.4 System factor, K_H

The specified strength for glued-laminated timber members in a system consisting of three or more essentially parallel members spaced not more than 610 mm apart and so arranged that they mutually support the applied load may be multiplied by a system factor, K_H, equal to 1.00 for tension parallel to grain, shear strength at locations with compression side notches, and fracture shear strength at a notch, and 1.10 for all other strength properties.

Table 7.3
Service-condition factors, K_S
(See Clauses 7.4.2.1, 7.4.2.2, 10.5.2, 10.5.3, 12.4.4.4, and 12.4.4.5.)

		Glued-laminated timber	
K_S	Property	Dry-service conditions	Wet-service conditions
K_{Sb}	Bending	1.00	0.80
K_{Sf}	Fracture shear	1.00	0.85
K_{Sv}	Longitudinal shear	1.00	0.87
K_{Sc}	Compression parallel to grain	1.00	0.75
K_{Scp}	Compression perpendicular to grain	1.00	0.67
K_{St}	Tension parallel to grain	1.00	0.75
K_{Stp}	Tension perpendicular to grain	1.00	0.85
K_{SE}	Modulus of elasticity	1.00	0.90

7.5 Strengths and resistances

7.5.1 Scope

Clause 7.5 provides design information and design formulas for glued-laminated timber members of rectangular cross-section.

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7.5.2 Orientation

The design data for bending members specified in Clause 7.5 apply to horizontally laminated members, the wide faces of whose laminations are normal to the direction of load.

7.5.3 Vertically glued-laminated beams

Vertically glued-laminated timber beams, where the narrow faces of laminations are normal to the direction of load, shall be designed as a built-up system of sawn lumber members of No. 2 grade, in accordance with Clause 6.4.4.3.

7.5.4 Net section

The net section, obtained by deducting from the gross cross-section the area of all material removed by boring, grooving, dapping, notching, or other means, shall be checked by calculating the resistance of a member. The net section shall not be less than 75% of the gross section.

7.5.5 Size

For design purposes, the actual dry size rounded to the nearest millimetre (net dimension) shall be used for both dry- and wet-service conditions.

Note: *Standard sizes rounded to the nearest millimetre are given in Clause A.7.5.5.*

7.5.6 Bending moment resistance

7.5.6.1 Members of constant cross-section

The factored bending moment resistance of glued-laminated timber members of constant cross-section shall be determined in accordance with Clause 7.5.6.5.

7.5.6.2 Curved and/or double-tapered members

In addition to meeting the requirements of Clause 7.5.6.5, the factored bending moment resistance of rectangular curved and/or tapered glued-laminated timber members shall not exceed the value determined in accordance with Clause 7.5.6.6.

7.5.6.3 Lateral-stability conditions

7.5.6.3.1 General

For laterally-unsupported glued-laminated timber bending members, the lateral-stability factor, K_L, may be taken as unity, provided that the maximum depth-to-width ratio of the member does not exceed 2.5:1. If the ratio is greater than 2.5:1, lateral support shall be provided at points of bearing to prevent lateral displacement and rotation and K_L shall be determined in accordance with Clause 7.5.6.4, except as otherwise permitted in Clause 7.5.6.3.2.

7.5.6.3.2 Combined bending and axial loads

In the case of glued-laminated members of rectangular cross-section subjected to combined bending and axial loads, the provisions of Clause 6.5.3.2.3 may be applied.

7.5.6.3.3 Built-up beams

For built-up beams consisting of two or more individual members of the same depth, the maximum ratio permitted in Clause 7.5.6.3.1 for laterally-unsupported members may be based on the total width

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of the beam, provided that the individual members are fastened together securely at intervals not exceeding four times the depth.

7.5.6.4 Lateral-stability factor, K_L

7.5.6.4.1 Unsupported length, ℓ_u

When no additional intermediate support is provided, the unsupported length, ℓ_u, shall be the distance between points of bearing or the length of the cantilever. When intermediate support is provided by purlins so connected that they prevent lateral displacement of the compressive edge of the bending member, the unsupported length shall be taken as the maximum purlin spacing, a (see Table 7.4).

7.5.6.4.2 Prevention of lateral displacement

When the compressive edge of the bending member is supported throughout its length to prevent lateral displacement, the unsupported length may be taken as zero. For decking to provide such support, it shall be fastened securely to the bending member and adjacent framing to provide a rigid diaphragm.

7.5.6.4.3 Slenderness ratio, C_B

The slenderness ratio of a bending member shall not exceed 50 and shall be calculated as follows:

$$C_B = \sqrt{\frac{L_e d}{b^2}}$$

where

L_e = effective length, mm, from Table 7.4

Table 7.4
Effective length, L_e, for bending members
(See Clauses 7.5.6.4.1 and 7.5.6.4.3.)

	Intermediate support	
Type of load	**Yes**	**No**
Beams		
Any loading	$1.92a$	$1.92\ell_u$
Uniformly distributed load	$1.92a$	$1.92\ell_u$
Concentrated load at centre	$1.11a$	$1.61\ell_u$
Concentrated loads at 1/3 points	$1.68a$	—
Concentrated loads at 1/4 points	$1.54a$	—
Concentrated loads at 1/5 points	$1.68a$	—
Concentrated loads at 1/6 points	$1.73a$	—
Concentrated loads at 1/7 points	$1.78a$	—
Concentrated loads at 1/8 points	$1.84a$	—
Cantilevers		
Any loading	—	$1.92\ell_u$

(Continued)

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Table 7.4 (Concluded)

Type of load	Intermediate support	
	Yes	No
Uniformly distributed load	—	$1.23\ell_u$
Concentrated load at free end	—	$1.69\ell_u$

Note: *ℓ_u and a are as defined in Clause 7.5.6.4.1.*

Δ **7.5.6.4.4 Calculation of lateral-stability factor, K_L**

The lateral-stability factor, K_L, shall be calculated as follows:

a) when C_B does not exceed 10:

$K_L = 1.0$

b) when C_B is greater than 10 but does not exceed C_K:

$$K_L = 1 - \frac{1}{3}\left(\frac{C_B}{C_K}\right)^4$$

where

$$C_K = \sqrt{\frac{0.97EK_{SE}K_T}{F_b}}$$

where

E = specified modulus of elasticity, MPa (see Table 7.2)

c) when C_B is greater than C_K but does not exceed 50:

$$K_L = \frac{0.65EK_{SE}K_T}{C_B^2F_bK_X}$$

where

$F_b = f_b\,(K_D\,K_H\,K_{Sb}\,K_T)$

where

f_b = specified strength in bending, MPa (see Table 7.2)

K_X = curvature factor (see Clause 7.5.6.5.2)

7.5.6.5 Moment resistance

7.5.6.5.1 General

Except as provided for in Clauses 7.5.6.5.3 and 7.5.6.6, the factored bending moment resistance, M_r, of glued-laminated timber members shall be taken as the lesser of M_{r1} or M_{r2} as follows:

$M_{r1} = \phi\,F_b\,SK_x\,K_{Zbg}$

$M_{r2} = \phi\,F_b\,SK_x\,K_L$

where

ϕ = 0.9

F_b = $f_b\,(K_D\,K_H\,K_{Sb}\,K_T)$

where

f_b = specified strength in bending, MPa (see Table 7.2)

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K_X = curvature factor (see Clause 7.5.6.5.2)

$K_{Zbg} = \left(\frac{130}{b}\right)^{\frac{1}{10}}\left(\frac{610}{d}\right)^{\frac{1}{10}}\left(\frac{9100}{L}\right)^{\frac{1}{10}} \leq 1.3$

where

b = beam width (for single-piece laminations) or the width of widest piece (for multiple-piece laminations), mm

d = beam depth, mm

L = length of beam segment between points of zero moment, mm

K_L = lateral-stability factor (see Clause 7.5.6.4)

Note: *For beams with one or more points of inflection (i.e., multi-span beams and cantilevered beams), the size factor is calculated for each beam segment. The moment resistance for each beam segment, as modified by the appropriate size factor, is then compared to the maximum factored moment within that segment.*

7.5.6.5.2 Curvature factor, K_X

For the curved portion only of glued-laminated timber members, the specified strength in bending shall be multiplied by the curvature factor, K_X, calculated as follows:

$$K_X = 1 - 2000\left(\frac{t}{R}\right)^2$$

where

t = lamination thickness, mm

R = radius of curvature of the innermost lamination, mm

The minimum radius of curvature permitted for a given thickness of lamination shall meet the requirements of CAN/CSA-O122 (see Table A.7).

Note: *K_X = 1.0 for straight members and the straight portion of curved members.*

7.5.6.5.3 Curved double-tapered members

At the apex of curved double-tapered members (pitched cambered beams), the factored bending moment resistance, M_r, shall be taken as the value obtained from the formula in Clause 7.5.6.5.1 divided by the factor [1 + 2.7 tanα], where α = the slope of upper surface of member (roof slope) in degrees.

7.5.6.6 Radial resistance

Notes:

1) *Radial stresses occur in curved and/or double-tapered glued-laminated members and can limit their factored bending moment resistance.*
2) *When the bending moment tends to decrease curvature (increase the radius), the corresponding radial stress is tension perpendicular to grain. When the bending moment tends to increase curvature (decrease the radius), the corresponding radial stress is compression perpendicular to grain.*

7.5.6.6.1 General

The factored bending moment resistance as governed by tension perpendicular to grain, M_r, shall be calculated using the following formula, but shall not be greater than the value determined in accordance with Clauses 7.5.6.5.1 and 7.5.6.5.3. For double-tapered members, the additional requirement of Clause 7.5.6.6.2 shall also be satisfied.

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$$M_r = \phi F_{tp} \frac{2A}{3} R K_{Ztp}$$

where

ϕ = 0.9

F_{tp} = f_{tp} (K_D K_H K_{Stp} K_T)

where

f_{tp} = specified strength in tension perpendicular to grain, MPa (see Table 7.2)

R = radius of curvature at centreline of member, mm

K_{Ztp} = size factor in tension perpendicular to grain (see Table 7.5)

Δ

Table 7.5
Size factor, K_{Ztp}, for tension perpendicular to grain*
(See Clauses 7.5.6.6.1 and 7.5.6.6.2.)

	Loading	
Member	**Uniformly distributed**	**All other**
Constant depth, curved	$\frac{24}{(AR\beta)^{0.2}}$	$\frac{20}{(AR\beta)^{0.2}}$
Double-tapered, curved	$\frac{35}{(AR\beta)^{0.2}}$	$\frac{22}{(AR\beta)^{0.2}}$
Double-tapered, straight	$\frac{36}{(Ad)^{0.2}}$	$\frac{23}{(Ad)^{0.2}}$

* *where*

A = *maximum cross-sectional area of member, mm²*

R = *radius of curvature at centreline of member, mm*

β = *enclosed angle in radians (1 radian = 57.3°)*

Notes:

1) *For curved double-tapered members, β is measured between points of tangency.*
2) *For curved uniform section members, β is measured between the points where the factored bending moment is 85% of the maximum factored bending moment.*

7.5.6.6.2 Double-tapered members

For double-tapered members subjected to radial tension perpendicular to grain, the following additional requirement shall be satisfied:

$$M_r = \phi F_{tp} S K_{Ztp} K_R$$

where

ϕ = 0.9

F_{tp} = f_{tp} (K_D K_H K_{Stp} K_T)

where

f_{tp} = specified strength in tension perpendicular to grain, MPa (see Table 7.2)

S = section modulus at apex, mm³

K_{Ztp} = size factor in tension perpendicular to grain (see Table 7.5)

K_R = radial-stress factor (see Clause 7.5.6.6.3)

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7.5.6.6.3 Radial-stress factor, K_R

The radial-stress factor, K_R, shall be calculated as follows:

$$K_R = \left[A + B\left(\frac{d}{R}\right) + C\left(\frac{d}{R}\right)^2\right]^{-1}$$

where

A, B, C = constants specified in Table 7.6

d = maximum depth at apex, mm

R = radius of curvature at centreline of member, mm

Table 7.6
Values of constants for determination of radial stress in double-tapered curved members
(See Clause 7.5.6.6.3.)

Angle α*	*A*	*B*	*C*
2°30′	0.01	0.17	0.13
5°	0.02	0.13	0.19
7°30′	0.03	0.09	0.22
10°	0.04	0.08	0.21
15°	0.06	0.06	0.17
20°	0.09	0.06	0.14
25°	0.12	0.06	0.12
30°	0.16	0.06	0.11

* *See Figure 7.1.*

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Figure 7.1
Double-tapered member
(See Table 7.6.)

a) Pitched cambered beam

b) Arch member

Legend:

R = radius of curvature at centreline of member

d = maximum depth at apex

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Figure 7.1 (Concluded)

α = slope of upper surface of member

β = enclosed angle in radians

7.5.7 Shear resistance

Δ **7.5.7.1 General**

Members without notches shall be checked for shear resistance in accordance with Clause 7.5.7.3. Members with notches on the compression side shall be checked for shear resistance in accordance with Clause 7.5.7.4. Members with notches on the tension side shall also be verified for shear resistance at locations other than end notches in accordance with Clause 7.5.7.5.1 and for fracture shear resistance at notch in accordance with Clause 7.5.7.5.2.

7.5.7.2 Loads near supports

7.5.7.2.1 Loads for shear resistance

In Item b) of Clause 7.5.7.3, and in Clauses 7.5.7.4 and 7.5.7.5.1, the effect of loads acting within a distance from a support equal to the depth of the member may be omitted from consideration.

7.5.7.2.2 Loads for fracture shear resistance at notch

In Clause 7.5.7.3 a) and Clause 7.5.7.5.2, the effect of all loads acting on the member shall be considered, including the loads acting within a distance from the support equal to the depth of the member.

7.5.7.3 Shear resistance at locations other than end notches

The factored shear resistance of glued-laminated timber members shall be determined as follows:

a) For beams of any volume, the total factored load, W_f, acting normal to a member shall not exceed the total factored shear resistance, W_r, calculated as follows:

$$W_r = \phi F_v 0.48 A_g C_V Z^{-0.18} \geq W_f$$

Note: *As an alternative for beams less than 2.0 m³ in volume, the factored shear resistance may be calculated using the equation in Item b).*

b) For members other than beams, the factored shear resistance, V_r, shall not be less than the maximum factored shear force, V_f, and shall be calculated as follows:

$$V_r = \phi F_v \frac{2A_g}{3}$$

where

ϕ = 0.9

F_v = $f_v (K_D K_H K_{Sv} K_T)$

where

f_v = specified strength in shear, MPa (see Table 7.2)

A_g = $b \times d$ = gross cross-sectional area of member, mm²

C_V = shear load coefficient (see Clause 7.5.7.6)

Z = beam volume, m³

Note: *The shear resistance requirements of this Clause are additional to those applicable to notched members (see Clauses 7.5.7.4 and 7.5.7.5).*

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7.5.7.4 Shear resistance at locations with compression side notches

The factored shear resistance, V_r, shall not be less than the maximum factored shear force, V_f, and shall be calculated as follows:

a) for $e_c > (d-d_n)$: $V_r = \phi F_v \frac{2A_n}{3}$

b) for $e_c < (d-d_n)$: $V_r = \phi F_v \frac{2A_g}{3}\left(1 - \frac{d_n e_c}{d(d-d_n)}\right)$

where

ϕ = 0.9

F_v = f_v (K_D K_H K_{Sv} K_T)

where

f_v = specified strength in shear, MPa (see Table 7.2)

A_n = $b(d - d_n)$ = net cross-sectional area of member, mm² (see Clause 7.5.4)

A_g = $b \times d$ = gross cross-sectional area of member, mm²

b = member width, mm

d = member depth, mm

d_n = notch depth, mm (which shall not exceed 0.25d)

e_c = notch length, mm, from inner edge of closest support to farthest edge of notch

7.5.7.5 Shear resistance at locations with tension side end notches

7.5.7.5.1 Shear resistance of residual member above notch

Tension side notches not exceeding 0.25d may be within a distance 'd' from the inner edge of the closest support to the farthest edge of the notch without a reduction in shear resistance as calculated in accordance with Clause 7.5.7.3 using the gross cross-sectional area of member, A_g. Members with tension side notches beyond a distance 'd' from the inner edge of the closest support to the farthest edge of the notch shall be designed using the net section, A_n, in accordance with Clause 5.3.8.1.

Δ #### 7.5.7.5.2 Fracture shear resistance at notch

The factored fracture shear resistance at a notch on the tension side at a support, F_r, shall not be less than the maximum factored shear force, V_f, and shall be calculated as follows:

$F_r = \phi F_f A_g K_N$

where

ϕ = 0.9

F_f = f_f (K_D K_H K_{Sf} K_T)

where

f_f = specified fracture shear strength at a notch, MPa

= 2.5 $b_{eff}^{-0.2}$ or 0.9 MPa, whichever is greater

where

b_{eff} = effective lamination width (mm)

= beam width (for single-piece laminations) or the width of widest piece (for multiple-piece laminations)

A_g = $b \times d$ = gross cross-sectional area, mm²

K_N = notch factor

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$$= \left[0.006d\left(1.6\left(\frac{1}{\alpha}-1\right)+\eta^2\left(\frac{1}{\alpha^3}-1\right)\right)\right]^{-\frac{1}{2}}$$

where

d = member depth (unreduced), mm

α = $1 - (d_n/d)$

where

d_n = notch depth, mm (see Figure 6.1, which shall not exceed $0.25d$)

η = e/d

where

e = notch length, mm (see Figure 6.1)

Δ 7.5.7.6 Shear-load coefficient, C_V

For any load condition not specified in Tables 7.7 to 7.12, the shear-load coefficient, C_V, for simple span, continuous, or cantilevered beams of constant depth may be determined using the following procedure (the principle of superposition of loads does not apply):

a) Construct the shear force diagram for the beam. If the beam is under moving concentrated loads, construct the diagram of the maximum shear forces occurring along the full length of the beam without regard to sign convention. (Positive and negative maximum shear forces both show positive.)

b) Divide the total beam length, L, into n segments of variable lengths, ℓ_a, such that within each segment there are neither abrupt changes nor changes from negative to positive in the shear force.

c) For each segment determine

i) V_A = factored shear force at beginning of segment, N;

ii) V_B = factored shear force at end of segment, N; and

iii) V_C = factored shear force at centre of segment, N

and calculate the factor G as follows:

$$G = \ell_a\,[V_A^5 + V_B^5 + 4V_C^5]$$

d) Determine the coefficient, C_V, as follows:

i) for stationary loads:

$$C_V = 1.825W_f\left(\frac{L}{\Sigma G}\right)^{0.2}$$

where

W_f = the total of all factored loads applied to the beam, N

ii) for moving loads:

$$C_V = 1.825W_f\left(\frac{L}{\Sigma G}\right)^{0.2}$$

where

W_f = the total of all factored moving loads and all factored distributed loads applied to the beam, N

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Table 7.7
Shear-load coefficient, C_V, for simple span beams
(See Clause 7.5.7.6.)

Number of equal loads equally and symmetrically spaced	r*			
	0.0	0.5	2.0	10.0 and over
1	3.69	3.34	2.92	2.46
2	3.69	3.37	3.01	2.67
3	3.69	3.41	3.12	2.84
4	3.69	3.45	3.21	2.97
5	3.69	3.48	3.28	3.08
6	3.69	3.51	3.34	3.16

$$^{*}r = \frac{\text{total of concentrated loads}}{\text{total of uniform loads}}$$

Table 7.8
Shear-load coefficient, C_V, for distributed loads
(See Clause 7.5.7.6.)

Type of loading	P_{min}/P_{max}					
	0.0	0.2	0.4	0.6	0.8	1.0
	3.40	3.55	3.63	3.67	3.69	3.69

Table 7.9
Shear-load coefficient, C_V, for cantilevered beams
(See Clause 7.5.7.6.)

Beam type and loading	L_1/L_2	r*			
		0.00	0.50	2.00	10.0 and over
	0.05	3.91	5.64	4.06	2.73
	0.10	4.13	5.19	3.07	2.08
	0.20	4.55	4.36	2.53	1.75
	0.30	4.88	3.83	2.31	1.62

(Continued)

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Table 7.9 (Concluded)

Beam type and loading	L_1/L_2	r* 0.00	0.50	2.00	10.0 and over
	0.05	4.13	6.19	7.13	4.86
	0.10	4.58	6.72	5.42	3.72
	0.20	5.50	6.90	4.49	3.17
	0.30	6.40	6.31	4.10	2.97

$L = L_2 + 2L_1$

$$^{*}r = \frac{\text{total of concentrated loads}}{\text{total of uniform loads}}$$

Table 7.10
Shear-load coefficient, C_V, for 2-span continuous beams
(See Clause 7.5.7.6.)

Loading case†	L_1/L	r* 0.00	0.50	2.00	10.0 and over
	0.2	4.09	3.04	2.35	2.01
	0.3	5.10	3.48	2.57	2.15
	0.4	6.09	3.96	2.82	2.32
	0.5	6.66	4.42	3.07	2.50

$$^{*}r = \frac{\text{total of concentrated loads}}{\text{total of uniform loads}}$$

† *The specified values correspond to the worst position for the concentrated loads.*

Table 7.11
Shear-load coefficient, C_V, for tapered beams — Uniformly distributed loads
(See Clause 7.5.7.6.)

Beam case	L/d_0*	m 0.20	0.10	0.05	0.00	−0.04	−0.08
	10	4.95	4.99	4.68	3.97	—	—
	20	4.46	4.63	4.54	3.69	—	—
	30	4.33	4.44	4.47	3.58	—	—

$$m = \frac{(d_1 - d_0)}{L}$$

(Continued)

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Table 7.11 (Concluded)

Beam case	L/d_0*	m					
		0.20	0.10	0.05	0.00	−0.04	−0.08
	10	4.40	4.49	4.32	3.97	3.58	2.42
	15	4.09	4.26	4.16	3.77	3.13	0.93
	20	3.95	4.11	4.08	3.69	2.40	0.23
	30	3.83	3.93	3.99	3.58	0.91	—

$m = \frac{2(d_1 - d_0)}{L}$

* *For calculating the ratio L/d_0, L and d_0 shall be in the same units.*

Table 7.12
Shear load coefficient, C_V, for moving loads

(See Clause 7.5.7.6.)

Type of concentrated loads	Span, L, mm	r*				
		0.5	1.5	3.0	10.0	100.0
	10 000	2.65	2.12	1.90	1.71	1.62
	20 000	2.65	2.12	1.90	1.71	1.62
	30 000	2.65	2.12	1.90	1.71	1.62
	40 000	2.65	2.12	1.90	1.71	1.62
	10 000	2.87	2.38	2.17	1.98	1.90
	20 000	2.76	2.24	2.03	1.83	1.75
	30 000	2.72	2.20	1.98	1.79	1.70
	40 000	2.70	2.18	1.96	1.77	1.68
	10 000	3.93	4.05	4.11	4.17	4.20
	20 000	3.39	3.13	2.99	2.86	2.80
	30 000	3.14	2.75	2.57	2.40	2.32
	40 000	3.03	2.59	2.40	2.21	2.13
	10 000	3.78	3.76	3.74	3.72	3.70
	20 000	3.40	3.13	3.00	2.86	2.79
	30 000	3.20	2.84	2.67	2.50	2.42
	40 000	3.08	2.66	2.47	2.29	2.21
	10 000	3.78	3.76	3.73	3.70	3.67
	20 000	3.24	2.90	2.73	2.58	2.50
	30 000	3.05	2.63	2.43	2.25	2.17
	40 000	2.96	2.51	2.31	2.12	2.03

$*r = \frac{\text{total of concentrated loads}}{\text{total of uniform loads}}$

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7.5.8 Compressive resistance parallel to grain

7.5.8.1 Effective length, L_e

Unless otherwise noted in this Standard, the effective length, $L_e = K_eL$, shall be used in determining the slenderness ratio of compression members.

Note: *Recommended effective-length factors, K_e, for compression members are given in Table A.4.*

Δ #### 7.5.8.2 Slenderness ratio, C_C

The slenderness ratio, C_C, of simple compression members of constant rectangular section shall not exceed 50 and shall be calculated as the greater of

$$C_C = \frac{\text{effective length associated with width}}{\text{member width}}$$

or

$$C_c = \frac{\text{effective length associated with depth}}{\text{member depth}}$$

7.5.8.3 Variable rectangular cross-section

Tapered rectangular compression members shall be designed for an effective width or depth equal to the minimum width or depth plus 0.45 times the difference between the maximum and minimum widths or depths. The factored compressive resistance determined in this manner shall not exceed the factored resistance based on the minimum dimensions in conjunction with a slenderness factor $K_C = 1.00$.

7.5.8.4 Eccentrically applied loads

Bending moments due to eccentrically applied axial loads shall be considered in accordance with Clause 7.5.12.

7.5.8.5 Factored compressive resistance under axial load

The factored compressive resistance parallel to grain, P_r, shall be calculated as follows:

$P_r = \phi\ F_c\ A\ K_{Zcg}\ K_C$

where

ϕ = 0.8

F_c = $f_c\ (K_D\ K_H\ K_{Sc}\ K_T)$

where

f_c = specified strength in compression parallel to grain, MPa (see Table 7.2)

A = cross-sectional area, mm^2

K_{Zcg} = $0.68(Z)^{-0.13} \leq 1.0$

where

Z = member volume, m^3

K_C = slenderness factor (see Clause 7.5.8.6)

Δ #### 7.5.8.6 Slenderness factor, K_C

The slenderness factor, K_C, shall be calculated as follows:

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$$K_c = \left[1.0 + \frac{F_c K_{Zcg} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

where

E_{05} = $0.87E$

where

E = specified modulus of elasticity, MPA (see Table 7.2)

7.5.8.7 Spaced compression members

Spaced compression members shall be designed using the specified strengths and modification factors appropriate for glued-laminated timber.

Note: *Spaced compression members may be designed in accordance with the provisions of Clause A.6.5.5.3.*

7.5.8.8 Built-up compression members

Built-up compression members shall be designed in accordance with Clause 6.5.5.4 using the specified strengths and modification factors appropriate for glued-laminated timber.

7.5.9 Compressive resistance perpendicular to grain (bearing)

7.5.9.1 General

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 7.5.9.2 and 7.5.9.3.

Note: *See Figure 6.2.*

7.5.9.2 Effect of all applied loads

The factored compressive resistance perpendicular to grain, Q_r, under the effect of all applied loads shall be calculated as follows:

$$Q_r = \phi\, F_{cp}\, A_b\, K_B\, K_{Zcp}$$

where

ϕ = 0.8

F_{cp} = $f_{cp}\,(K_D\, K_{Scp}\, K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain, MPa (see Table 7.2)

A_b = bearing area, mm^2

K_B = length-of-bearing factor (see Clause 6.5.6.5)

K_{Zcp} = size factor for bearing, where the depth is the thickness of lamination (see Clause 6.5.6.4)

7.5.9.3 Effect of loads applied near a support

7.5.9.3.1 Factored compressive resistance perpendicular to grain

The factored compressive resistance perpendicular to grain, Q_r' under the effect of only those applied loads acting within a distance from the centre of the support equal to the depth of the member shall be calculated as follows:

$$Q_r' = (2/3)\, \phi\, F_{cp}\, A'_b\, K_B\, K_{Zcp}$$

where

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ϕ = 0.8

F_{cp} = f_{cp} (K_D K_{Scp} K_T)

where

f_{cp} = f-E grade value from Table 7.2 for compression face bearing strength, MPa

A_b' = average bearing area, mm² (see Clause 7.5.9.3.2)

Note: *See Figure 6.2.*

7.5.9.3.2 Unequal bearing areas on opposite surfaces of a member

Where unequal bearing areas are used on opposite surfaces (top and bottom) of a member, the average bearing area, A'_b, shall be calculated as follows:

$$A_b' = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \leq 1.5b(L_{b1})$$

where

b = average bearing width (perpendicular to grain), mm

L_{b1} = lesser bearing length, mm

L_{b2} = larger bearing length, mm

7.5.10 Compressive resistance at an angle to grain

The factored compressive resistance at an angle to grain shall be calculated in accordance with Clause 6.5.7 using the appropriate specified strengths and modification factors for glued-laminated timber.

7.5.11 Tensile resistance parallel to grain

The factored tensile resistance parallel to grain, T_r, shall not be less than the maximum factored tensile force, T_f, and shall be calculated as the lesser of

$T_r = \phi\ F_{tn}\ A_n$

or

$T_r = \phi\ F_{tg}\ A_g$

where

ϕ = 0.9

F_{tn} = f_{tn} (K_D K_H K_{St} K_T)

where

f_{tn} = specified strength in tension parallel to grain at net section, MPa (see Table 7.2)

A_n = net cross-sectional area, mm²

F_{tg} = f_{tg} (K_D K_H K_{St} K_T)

where

f_{tg} = specified strength in tension parallel to grain at gross section, MPa (see Table 7.2)

A_g = gross cross-sectional area, mm²

Δ

7.5.12 Resistance to combined bending and axial load

Members subject to combined bending and compressive or tensile axial loads shall be designed to satisfy the appropriate interaction equation:

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$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] \leq 1$$

or

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1$$

where

P_f = factored compressive axial load

P_r = factored compressive load resistance parallel to grain calculated in accordance with Clause 7.5.8.5 using $F_{cb} = f_{cb}\,(K_D\,K_H\,K_{Sc}\,K_T)$

M_f = factored bending moment

M_r = factored bending moment resistance calculated in accordance with Clause 7.5.6.5.1

T_f = factored tensile axial load

T_r = factored tensile load resistance parallel to grain calculated in accordance with Clause 7.5.11

P_E = Euler buckling load in the plane of the applied moment

$= \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$

where

E_{05} = modulus of elasticity for design of compression members

E_{05} = 0.87E, MPa

where

E = Specified modulus of elasticity, MPa

I = moment of inertia in the plane of the applied moment, mm^4

L_e = effective-length in the plane of the applied moment

$= K_e L$

where

K_e = the effective-length factor given in Clause A.6.5.5.1

Note: *See Clause A.6.5.9 for a more detailed interaction formula for combined bending and compressive loads.*

7.6 Serviceability limit states

The design of glued-laminated timber lumber members for serviceability limit states shall be in accordance with Clauses 5.1.3 and 5.4.

8 Cross-laminated timber (CLT)

8.1 Scope

The design data and methods specified in Clause 8 apply only to panels of the primary and custom CLT grades, in layups as defined in Clause 8.2, manufactured and certified in accordance with ANSI/APA PRG 320. Panels with alternative CLT layups shall be designed in accordance with Clause 4.3.2.

Note: *The provisions contained within Clause 8 are intended for use of CLT in compression and flatwise bending applications.*

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8.2 Materials

8.2.1 General

Design of CLT panels in accordance with this Standard is based on the use of the primary and custom CLT grades and layups approved by the certification organization.

Note: *Designers should check the availability of CLT grades and layups before specifying. A list of certified manufacturers can be obtained from the certification organization.*

8.2.2 CLT layups

In this Standard, the CLT layup shall be a balanced combination of orthogonal layers where all laminations oriented in the same direction are made of sawn lumber of the same grade and species combination.

Note: *CLT panels may be designed with adjacent layers oriented in the same direction using the section properties provided by the product manufacturer (see Clause 8.4.3.2).*

8.2.3 Primary CLT grades

The primary CLT grades shall be as specified in Table 8.1. Custom CLT grades shall be specified by the product manufacturer.

Table 8.1
Primary CLT grades
(See Clause 8.2.3.)

Stress grade	Species combinations and grades of laminations
E1	1950 F_b-1.7E Spruce-Pine-Fir MSR lumber in all longitudinal layers and No. 3/Stud Spruce-Pine-Fir lumber in all transverse layers
E2	1650 F_b-1.5E Douglas Fir-Larch MSR lumber in all longitudinal layers and No. 3/Stud Douglas Fir-Larch lumber in all transverse layers
E3	1200 F_b-1.2E Northern Species MSR lumber in all longitudinal layers and No. 3/Stud Northern Species lumber in all transverse layers
V1	No. 1/No. 2 Douglas Fir-Larch lumber in all longitudinal layers and No. 3/Stud Douglas Fir-Larch lumber in all transverse layers
V2	No. 1/No. 2 Spruce-Pine-Fir lumber in all longitudinal layers and No. 3/Stud Spruce-Pine-Fir lumber in all transverse layers

8.2.4 Specified strengths and moduli of elasticity of laminations

Specified strength values and moduli of elasticity for the laminations in longitudinal and transverse layers of the primary CLT grades are given in Table 8.2. Those properties for custom CLT grades shall be as published by the product manufacturer.

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Table 8.2
Specified strengths and moduli of elasticity of laminations in primary CLT grades, MPa
(See Clause 8.2.4.)

Stress grade	Longitudinal layers						Transverse layers					
	f_b	E	f_t	f_c	f_s	f_{cp}	f_b	E	f_t	f_c	f_s	f_{cp}
E1	28.2	11 700	15.4	19.3	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3
E2	23.9	10 300	11.4	18.1	0.63	7.0	4.6	10 000	2.1	7.3	0.63	7.0
E3	17.4	8300	6.7	15.1	0.43	3.5	4.5	6500	2.0	5.2	0.43	3.5
V1	10.0	11 000	5.8	14.0	0.63	7.0	4.6	10 000	2.1	7.3	0.63	7.0
V2	11.8	9500	5.5	11.5	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3

Notes:

1) *Tabulated values are based on the following standard conditions:*
   a) *dry-service conditions; and*
   b) *standard-term duration of load.*
2) *The specified values are taken from Table 6.8 for MSR lumber and Table 6.4 for visually stress-graded lumber. The specified strength in rolling shear, f_s, is taken as approximately 1/3 of the specified strength in shear, f_v, for the corresponding species combination. See Figure 8.1 for clarification of rolling shear.*
3) *The transverse modulus of elasticity, $E_\perp$, may be estimated as E/30.*
4) *The shear modulus, G, may be estimated as E/16.*
5) *The rolling shear modulus, $G_\perp$, may be estimated as G/10. See Figure 8.1 for clarification of rolling shear.*
6) *The modulus of elasticity for design of compression members, E_{05}, shall be taken from Table 6.4 for visually stress-graded lumber and 0.82E for MSR lumber.*

Figure 8.1
Rolling shear in CLT
(See Table 8.2.)

8.2.5 Appearance grades

Appearance grades as defined in ANSI/APA PRG 320 do not affect the structural performance of CLT panels.

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8.3 Modification factors

8.3.1 Load-duration factor, K_D

The specified strength of CLT shall be multiplied by a load-duration factor, K_D, in accordance with Clause 5.3.2.

8.3.2 Service-condition factors, K_S

CLT shall only be used in dry-service conditions for all service-condition factors, K_S = 1.0.

Note: *CLT structures may be used in wet-service conditions only if specifically permitted by the manufacturer based on documented test data in accordance with Clause 4.3.2 and if approved by the certification organization.*

8.3.3 Treatment factor, K_T

For CLT treated with fire-retardant or other potentially strength-reducing chemicals, strength and stiffness shall be based on documented results of tests that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 5.3.4; otherwise, K_T shall be equal to 1.0. CLT shall not be treated with water-borne preservatives after gluing.

8.3.4 System factor, K_H

K_H shall be equal to 1.0 for all strength properties of CLT.

8.4 Strengths and resistances

8.4.1 Scope

Clause 8.4 provides design formulas and information for CLT panels of constant width. Factored resistances of CLT panels shall be calculated in accordance with Clause 8.4 using the specified strengths and moduli of elasticity of laminations (see Clause 8.2.4), modification factors (see Clause 8.3), and section properties determined in accordance with Clause 8.4.3.2 or as provided by the product manufacturer.

8.4.2 Sizes

For design purposes, the actual dry sizes of the panel rounded to the nearest millimetre (net dimension) shall be used. Depending on the layup, the actual sizes and tolerances of CLT panels may deviate from those specified in ANSI/APA PRG 320 and shall be accounted for in calculations.

8.4.3 Flatwise bending moment resistance

8.4.3.1 General

The factored flatwise bending moment resistance, $M_{r,f}$, of a CLT panels shall be calculated as follows:

a) in the major strength direction [see Figure 8.2 a)]:

$M_{r,f,0} = \phi\ F_b\ S_{eff,f,0}\ K_{rb,0}$

where

ϕ = 0.9

F_b = $f_b\ (K_D\ K_H\ K_{Sb}\ K_T)$

where

f_b = specified bending strength of laminations in the longitudinal layers, MPa (see Clause 8.2.4)

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$$S_{eff,f,0} = \frac{(EI)_{eff,f,0}}{E}\frac{2}{h}$$

where

$(EI)_{eff,f,0}$ = effective flatwise bending stiffness of the panel in the major strength direction, N•mm² (see Clause 8.4.3.2)

E = specified modulus of elasticity of laminations in the longitudinal layers, MPa [see Clause 8.2.4 and Figure 8.2 a)]

h = thickness of the panel, mm [see Figure 8.2 a)]

$K_{rb,0} = 0.85$

b) in the minor strength direction [see Figure 8.2 b)]:

$M_{r,f,90} = \phi\ F_b\ S_{eff,f,90}\ K_{rb,90}$

where

ϕ = 0.9

F_b = $f_b\ (K_D\ K_H\ K_{Sb}\ K_T)$

where

f_b = specified bending strength of laminations in the transverse layers, MPa (see Clause 8.2.4)

$$S_{eff,f,90} = \frac{(EI)_{eff,f,90}}{E}\frac{2}{h_{90}}$$

where

$(EI)_{eff,f,90}$ = effective flatwise bending stiffness of the panel in the minor strength direction, N•mm² (see Clause 8.4.3.2)

E = specified modulus of elasticity of laminations in the transverse layers, MPa [see Clause 8.2.4 and Figure 8.2 b)]

h_{90} = thickness of the panel without the outer longitudinal layers, mm [see Figure 8.2 b)]

$K_{rb,90} = 1.0$

Δ 8.4.3.2 Effective stiffness in flatwise bending

Formulas given in this Clause shall apply to the section properties of CLT panels with alternating orthogonal layers only. The section properties of CLT panels with adjacent layers oriented in the same direction shall be provided by the product manufacturer.

Effective flatwise bending stiffness, $(EI)_{eff,f}$, and effective shear stiffness in flatwise bending , $(GA)_{eff,f}$, of a CLT panel with alternating orthogonal layers shall be calculated as follows:

a) in the major strength direction [see Figure 8.2 a)]:

$$(EI)_{eff,f,0} = \sum_{i=1}^{n} E_i\, b_0 \frac{t_i^3}{12} + \sum_{i=1}^{n} E_i\, b_0\, t_i\, z_i^2$$

$$(GA)_{eff,f,0} = \frac{\left(h - \frac{t_1}{2} - \frac{t_n}{2}\right)^2}{\left[\left(\frac{t_1}{2G_1\, b_0}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i\, b_0}\right) + \left(\frac{t_n}{2G_n\, b_0}\right)\right]}$$

where

n = number of layers in the panel

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E_i = modulus of elasticity of laminations in the ith layer, MPa

= E, for laminations in the longitudinal layers, MPa

= $E\perp$, for laminations in the transverse layers, MPa

b_0 = width of the panel measured perpendicular to the major strength direction [see Figure 8.2 a)], mm

t_i = thickness of laminations in the ith layer, mm

z_i = distance between the centre point of the ith layer and the neutral axis, mm [see Figure 8.2 a)]

h = thickness of the panel [see Figure 8.2 a)]

G_i = shear modulus of laminations in the ith layer, MPa

= G, for laminations in the longitudinal layers, MPa

= $G\perp$, for laminations in the transverse layers, MPa

b) in the minor strength direction [see Figure 8.2 b)]:

$$(EI)_{eff,f,90} = \sum_{i=2}^{n-1} E_i\ b_{90}\frac{t_i^3}{12} + \sum_{i=2}^{n-1} E_i\ b_{90}\ t_i z_i^2$$

$$(GA)_{eff,f,90} = \frac{\left(h - \frac{t_1}{2} - \frac{t_n}{2}\right)^2}{\left[\left(\frac{t_1}{2G_1\ b_{90}}\right) + \left(\sum_{i=2}^{n-1}\frac{t_i}{G_i\ b_{90}}\right) + \left(\frac{t_n}{2G_n\ b_{90}}\right)\right]}$$

where

n = number of layers in the panel

E_i = modulus of elasticity of laminations in the ith layer, MPa

= E, for laminations in the transverse layers, MPa

= $E\perp$, for laminations in the longitudinal layers, MPa

b_{90} = width of the panel measured perpendicular to the minor strength direction, mm [see Figure 8.2 b)]

t_i = thickness of laminations in the ith layer, mm

z_i = distance between the centre point of the ith layer and the neutral axis, mm [see Figure 8.2 a)]

h = thickness of the panel, mm [see Figure 8.2 b)]

G_i = shear modulus of laminations in the ith layer, MPa

= G, for laminations in the transverse layers, MPa

= $G\perp$, for laminations in the longitudinal layers, MPa

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Δ

Figure 8.2
Properties of a primary five-layer CLT panel
(See Clauses 8.4.3.1, 8.4.3.2, and 8.4.4.2.)

a) Properties for the major strength direction

b) Properties for the minor strength direction

8.4.4 Flatwise shear resistance

8.4.4.1 Loads near supports

In Clause 8.4.4.2, the effect of loads acting within a distance from a support equal to the thickness of the panel may be omitted from consideration.

8.4.4.2 Factored flatwise shear resistance

The factored flatwise shear resistance, $V_{r,f}$, of a CLT panel shall be calculated as follows:

a) in the major strength direction:

$$V_{r,f,0} = \phi F_s \frac{2A_{g,0}}{3}$$

where

ϕ = 0.9

F_s = $f_s\,(K_D\,K_H\,K_{Sv}\,K_T)$

where

f_s = specified strength in rolling shear of laminations in the transverse layers, MPa (see Clause 8.2.4)

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$A_{g,0}$ = gross cross-sectional area of the panel measured perpendicular to the major strength direction, mm² [see Figure 8.2 a)]

= $b_0 h$

b) in the minor strength direction:

$$V_{r,f,90} = \phi F_s \frac{2A_{g,90}}{3}$$

where

ϕ = 0.9

F_s = $f_s (K_D K_H K_{Sv} K_T)$

where

f_s = specified strength in rolling shear of laminations in the longitudinal layers, MPa (see Clause 8.2.4)

$A_{g,90}$ = gross cross-sectional area of the panel measured perpendicular to the minor strength direction, mm², excluding the outermost longitudinal layers [see Figure 8.2 b)]

= $b_{90} h_{90}$

Note: *For a CLT panel with multiple longitudinal outermost layers, all these layers are excluded from the gross cross-sectional area, $A_{g,90}$.*

8.4.5 Compressive resistance under axial load

8.4.5.1 General

Only the layers with laminations oriented parallel to the applied axial load shall be assumed to carry that load.

8.4.5.2 Effective length, L_e

The effective length, $L_e = K_e L$, shall be used in determining the slenderness ratio of CLT panels. Recommended effective-length factors, K_e, for compression members are given in Table A.4.

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8.4.5.3 Slenderness ratio, C_C

The slenderness ratio, C_C, of CLT panels of constant rectangular cross-section shall not exceed 43 (i.e., $L_e/r_{eff} \leq 150$) and shall be calculated as follows:

$$C_C = \frac{L_e}{\sqrt{12}\, r_{eff}}$$

where

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}}$$

where

I_{eff} = effective out-of-plane moment of inertia of the panel accounting only for the layers with laminations oriented parallel to the applied axial load, mm⁴

A_{eff} = effective cross-sectional area of the panel accounting only for the layers with laminations oriented parallel to the applied axial load, mm²

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8.4.5.4 Eccentrically applied loads

Bending moments due to eccentrically applied axial loads shall be considered in accordance with Clause 8.4.6.

8.4.5.5 Factored compressive resistance under axial load

The factored compressive resistance of CLT panels under axial load shall be calculated as follows:

$P_r = \phi\, F_c\, A_{eff}\, K_{Zc}\, K_C$

where

ϕ = 0.8

F_c = $f_c\,(K_D\, K_H\, K_{Sc}\, K_T)$

where

f_c = specified strength in compression parallel to grain of the laminations oriented parallel to the axial load, MPa (see Clause 8.2.4)

$$K_{Zc} = 6.3\left(\sqrt{12}\, r_{eff}\, L\right)^{-0.13} \leq 1.3$$

where

L = height of the panel, mm

$$K_C = \left[1.0 + \frac{F_c\, K_{Zc}\, C_c^3}{35 E_{05}(K_{SE}\, K_T)}\right]^{-1}$$

where

E_{05} = modulus of elasticity for design of compression members, only for the laminations oriented parallel to the applied axial load, MPa (see Clause 8.2.4)

Δ **8.4.6 Resistance to combined bending and axial compressive load**

CLT panels subject to combined flatwise bending and compressive axial load shall be designed to satisfy the interaction equation:

$$\frac{P_f}{P_r} + \frac{M_f}{M_{r,f}}\left[\frac{1}{1 - \frac{P_f}{P_{E,v}}}\right] \leq 1$$

where

P_f = factored compressive axial load

P_r = factored compressive resistance under axial load, calculated in accordance with Clause 8.4.5.5

M_f = factored bending moment

$M_{r,f}$ = factored flatwise bending moment resistance, calculated in accordance with Clause 8.4.3

$P_{E,v}$ = Euler buckling load in the plane of the applied bending moment adjusted for shear deformation

$$= \frac{P_E}{1 + \frac{P_E}{(GA)_{eff,f}}}$$

where

P_E = Euler buckling load in the plane of the applied bending moment in accordance with Clause 7.5.12 where E_{05} and I_{eff} are determined accounting only for the layers with laminations oriented parallel to the axial load (see Clauses 8.2.4 and 8.4.5.3)

$(GA)_{eff,f}$ = effective shear stiffness in flatwise bending accounting for all layers, N (see Clause 8.4.3.2)

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8.4.7 Compressive resistance perpendicular to the face of the panel (bearing)

8.4.7.1 General

Factored bearing forces shall not exceed the factored compressive resistance of CLT panels perpendicular to grain as determined in accordance with Clauses 8.4.7.2 and 8.4.7.3.

8.4.7.2 Effect of all applied loads

The factored compressive resistance perpendicular to grain, Q_r, of CLT panels under the effect of all applied loads shall be calculated as follows:

$Q_r = \phi\ F_{cp}\ A_b\ K_B\ K_{Zcp}$

where

ϕ = 0.8

F_{cp} = $f_{cp}\ (K_D\ K_{Scp}\ K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain of laminations in the outer layers (see Clause 8.2.4), MPa

A_b = bearing area, mm^2

K_B = length-of-bearing factor (see Clause 6.5.6.5)

K_{Zcp} = size factor for bearing

= 1.0

8.4.7.3 Effect of loads applied near a support

Δ #### 8.4.7.3.1 Factored compressive resistance perpendicular to grain

The factored compressive resistance perpendicular to grain, Q'_r, of CLT panels under the effect of only those applied loads acting within a distance from the centre of the support equal to the depth of the panel shall be calculated as follows:

$Q'_r = (2/3)\phi\ F_{cp} A'_b\ K_B\ K_{Zcp}$

where

ϕ = 0.8

F_{cp} = $f_{cp}\ (K_D\ K_{Scp}\ K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain of laminations in the outer layers (see Clause 8.2.4), MPa

A'_b = average bearing area, mm^2 (see Clause 8.4.7.3.2)

Note: *See Figure 6.2.*

8.4.7.3.2 Unequal bearing areas on opposite surfaces of a panel

In cases where unequal bearing areas are used on opposite surfaces (top and bottom) of a panel, the average bearing area, A'_b, shall be calculated as follows:

$$A'_b = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \leq 1.5b(L_{b1})$$

where

b = average bearing width, mm

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L_{b1} = lesser bearing length, mm

L_{b2} = larger bearing length, mm

8.4.8 Compressive resistance at an angle to the face of a panel

The factored compressive resistance at an angle to the face of CLT panel shall be calculated in accordance with Clause 6.5.7 using the factored resistance values calculated in Clause 8.4.5 with $K_{Zc} = K_C = 1.0$ and Clause 8.4.7.

8.5 Serviceability limit states

8.5.1 General

The design of CLT panels for serviceability limit states shall be in accordance with Clauses 5.1.3 and 5.4.

8.5.2 Deflection of CLT panels

The maximum deflection under a specified load acting perpendicular to the plane of the panel shall be calculated as a sum of the deflections due to moment and shear using the effective flatwise bending stiffness $(EI)_{eff,f}$, and the effective shear stiffness in flatwise bending, $(GA)_{eff,f}$, as defined in Clause 8.4.3.2, with consideration for creep effects.

Note: *A method for calculating deflection under static uniform or concentrated load is provided in Clause A.8.5.2.*

8.5.3 Vibration performance of CLT floors

A method for calculating vibration-controlled spans for CLT floors is provided in Clause A.8.5.3.

9 Structural panels

9.1 Scope

The design data, methods, and construction requirements specified in Clause 9 apply to the materials specified in Clause 9.2.

Design methods and construction requirements for glued building components manufactured using structural panels are provided in Clause 10.

9.2 Materials

9.2.1 Plywood

Clauses 9.3 to 9.5 apply to standard constructions of regular grades of unsanded

a) Douglas fir plywood manufactured and identified in accordance with CSA O121; and
b) Canadian softwood plywood manufactured and identified in accordance with CSA O151.

9.2.2 OSB

Clauses 9.3 to 9.5 also apply to OSB panels that are qualified and identified in accordance with CSA O325, which pertains to wood-based panel products designed and manufactured for protected construction uses such as roof sheathing, wall sheathing, and floor sheathing in light frame construction applications. For applications where OSB is primary load-bearing element, other tests and criteria shall be considered.

Note: *Construction sheathing OSB is distinguished from other mat-formed panels by a mark showing the direction of face alignment. See Clause A.9.2.2 for additional information.*

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9.2.3 Adhesives for stress joints

Adhesives for stress joints for structural panels shall meet the requirements of CSA O112.7.

9.3 Specified capacities

9.3.1 Plywood

The specified capacities for minimum veneer layup for each plywood thickness are given for Douglas fir plywood in Table 9.1 and for Canadian softwood plywood in Table 9.2.

9.3.2 OSB

The specified capacities for construction sheathing OSB conforming to CSA O325 are given in Table 9.3.

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Table 9.1
Specified strength, stiffness, and rigidity capacities for standard constructions of regular grades of unsanded Douglas fir plywood (DFP)
(See Clauses 9.3.1, 9.5.2, 9.5.3, 9.5.4.1, 9.5.4.2, 9.5.5.1, 9.5.5.2, 9.5.6–9.5.8, 10.5.2, 10.5.4–10.5.6, 10.6.2, 10.6.3.1, 10.6.3.6, 10.6.3.7, 11.6.2.2, 11.6.3.2, 11.7.1.2, 11.7.2, 16.2.5, and A.11.7.1. See also Table A.1 and Figure A.4.)

Nominal thickness, mm	No. of plies	Bending, m_p, N•mm/mm		Axial tension, t_p, N/mm		Axial compression, p_p, N/mm		Shear-through-thickness, V_p, N/mm	Planar shear: Bending, v_{psb}, N/mm		Planar shear: Shear in-plane, V_{pf}, MPa	
		Orientation of applied force relative to face grain										
		0°	90°	0°	90°	0°	90°	0° and 90°	0°	90°	0°	90°
7.5	3	180	38	97	23	130	40	20	3.7	1.2	0.72	0.72
9.5	3	270	51	97	27	130	46	24	3.9	1.3	0.55	0.72
12.5	3	520	110	170	38	210	66	34	6.3	1.9	0.72	0.72
	4*	420	130	97	55	130	96	30	5.5	2.8	0.55	0.72
	5	560	200	130	71	170	79	30	7.3	3.7	0.72	0.72
15.5	4	610	230	110	72	140	130	37	6.6	3.6	0.55	0.72
	5*	770	280	130	71	170	79	36	9.4	4.9	0.72	0.72
	6	730	310	130	71	170	79	36	6.9	4.1	0.55	0.55
18.5	5	980	460	150	100	190	120	43	9.0	5.0	0.55	0.55
	6*	930	430	130	71	170	79	43	8.5	5.1	0.55	0.55
	7	1100	450	160	110	210	120	43	9.7	7.1	0.72	0.72
20.5	5	1200	740	180	130	230	150	47	10.0	5.7	0.55	0.55
	6	1100	550	130	71	170	79	47	9.5	5.8	0.55	0.55
	7	1200	560	160	110	210	120	47	11.0	8.5	0.72	0.72
	8	1100	560	160	110	210	120	47	8.3	6.4	0.55	0.55
22.5	6	1500	790	230	110	300	130	52	15.0	7.0	0.72	0.55
	7*	1300	640	170	110	210	130	51	12.0	9.8	0.72	0.72

(Continued)

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Table 9.1 (Concluded)

Nominal thickness, mm	No. of plies	Bending, m_p, N•mm/mm		Axial tension, t_p, N/mm		Axial compression, p_p, N/mm		Shear-through-thickness, V_p, N/mm	Planar shear: Bending, v_{psb}, N/mm		Planar shear: Shear in-plane, V_{pf}, MPa	
		Orientation of applied force relative to face grain										
		0°	90°	0°	90°	0°	90°	0° and 90°	0°	90°	0°	90°
	8	1400	580	160	110	210	120	51	9.3	7.2	0.55	0.55
	9	1500	730	200	140	250	160	51	12.0	8.8	0.72	0.72
25.5	7	1700	950	210	160	270	180	57	13.0	11.0	0.72	0.72
	8*	1600	730	160	110	210	120	57	11.0	8.8	0.55	0.55
	9	1700	860	200	140	250	160	57	14.0	10.0	0.72	0.72
	10	1700	800	200	140	250	160	57	11.0	7.8	0.55	0.55
28.5	8	2000	1100	190	140	250	160	63	11.0	10.0	0.55	0.55
	9*	2000	1000	200	140	250	160	63	16.0	12.0	0.72	0.72
	10	2000	940	200	140	250	160	63	12.0	9.2	0.55	0.55
	11	2100	1200	230	180	300	200	63	15.0	12.0	0.72	0.72
31.5	8	2700	1600	240	190	320	210	71	13.0	11.0	0.55	0.55
	9	2400	1500	230	190	300	210	69	17.0	13.0	0.72	0.72
	10*	2200	1100	200	140	250	160	69	13.0	10.0	0.55	0.55
	11	2400	1400	230	180	300	200	69	16.0	14.0	0.72	0.72
	12	2400	1200	230	180	300	200	69	13.0	10.0	0.55	0.55

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Table 9.1 (Continued)

Nominal thickness, mm	No. of plies	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Orientation of applied force relative to face grain				
		0°	90°	0°	90°	0° and 90°
7.5	3	440 000	17 000	70 000	24 000	4600
9.5	3*	840 000	27 000	70 000	28 000	5500
12.5	3	2 100 000	79 000	120 000	39 000	7800
	4*	1 700 000	190 000	70 000	57 000	6900
	5	1 700 000	350 000	94 000	47 000	6900
15.5	4	3 100 000	430 000	77 000	75 000	8500
	5*	3 000 000	630 000	94 000	47 000	8400
	6	3 000 000	760 000	94 000	47 000	8400
18.5	5	4 600 000	1 300 000	110 000	69 000	9800
	6*	4 600 000	1 300 000	94 000	47 000	9800
	7	4 900 000	1 400 000	120 000	71 000	9800
20.5	5	6 300 000	2 600 000	130 000	89 000	11 000
	6	5 800 000	1 900 000	94 000	47 000	11 000
	7	6 200 000	2 000 000	120 000	71 000	11 000
	8	6 100 000	2 100 000	120 000	71 000	11 000
22.5	6	8 400 000	3 200 000	160 000	75 000	12 000
	7*	7 600 000	2 500 000	120 000	75 000	12 000
	8	8 000 000	2 500 000	120 000	71 000	12 000
	9	8 300 000	3 100 000	140 000	95 000	12 000
25.5	7	11 000 000	4 300 000	150 000	110 000	13 000

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Table 9.1 (Continued)

Nominal thickness, mm	No. of plies	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Orientation of applied force relative to face grain				
		0°	90°	0°	90°	0° and 90°
	8*	11 000 000	3 700 000	120 000	71 000	13 000
	9	11 000 000	4 100 000	140 000	95 000	13 000
	10	11 000 000	4 100 000	140 000	95 000	13 000
28.5	8	15 000 000	6 500 000	140 000	95 000	15 000
	9*	14 000 000	5 700 000	140 000	95 000	15 000
	10	15 000 000	5 600 000	140 000	95 000	15 000
	11	15 000 000	6 400 000	160 000	120 000	15 000
31.5	8	22 000 000	10 000 000	180 000	120 000	16 000
	9	19 000 000	9 400 000	170 000	120 000	16 000
	10*	18 000 000	7 400 000	140 000	95 000	16 000
	11	19 000 000	8 500 000	160 000	120 000	16 000
	12	20 000 000	8 200 000	160 000	120 000	16 000

* *Most commonly available product.*

Notes:

1) *For specified stiffness in edgewise bending, use axial stiffness values.*
2) *Tabulated values are based on dry-service conditions and standard-term duration of load.*
3) *The specified strength in bearing (normal to plane of panel), qp, is 4.5 MPa.*
4) *See Figure 9.1 for clarification of shear orientations.*

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Table 9.2
Specified strength, stiffness, and rigidity capacities for standard constructions of regular grades of unsanded Canadian softwood plywood (CSP)

(See Clauses 9.3.1, 9.5.2, 9.5.3, 9.5.4.1, 9.5.4.2, 9.5.5.1, 9.5.5.2, 9.5.6–9.5.8, 10.5.2, 10.5.4–10.5.6, 10.6.2, 10.6.3.1, 10.6.3.6, 10.6.3.7, 11.6.2.2, 11.6.3.2, 11.7.1.2, 11.7.2, 16.2.5, and A.11.7.1. See also Table A.1 and Figure A.4.)

Nominal thickness, mm	No. of plies	Bending, m_p, N•mm/mm		Axial tension, t_p, N/mm		Axial compression, p_p, N/mm		Shear-through-thickness, v_p, N/mm	Planar shear: Bending, v_{psb}, N/mm		Planar shear: Shear in-plane, v_{pf}, MPa	
		Orientation of applied force relative to face grain										
		0°	90°	0°	90°	0°	90°	0° and 90°	0°	90°	0°	90°
7.5	3	190	38	83	23	93	40	18	3.7	1.2	0.72	0.72
9.5	3*	270	51	83	27	93	46	23	3.9	1.3	0.55	0.72
12.5	3	470	110	120	38	140	66	30	6.3	1.9	0.72	0.72
	4*	420	130	83	55	93	96	30	5.3	2.8	0.55	0.72
	5	450	200	120	71	130	79	30	7.3	3.7	0.72	0.72
15.5	4	600	230	89	72	99	130	38	6.6	3.6	0.55	0.72
	5*	600	280	120	71	130	79	38	9.1	4.9	0.72	0.72
	6	580	310	120	71	130	79	38	6.9	4.1	0.55	0.55
18.5	5	770	460	120	100	140	120	46	8.7	5.0	0.55	0.55
	6*	740	430	120	71	130	79	46	8.3	5.1	0.55	0.55
	7	840	450	150	110	170	120	46	9.7	7.1	0.72	0.72
20.5	5	900	740	150	130	170	150	51	9.9	5.7	0.55	0.55
	6	840	550	120	71	130	79	51	9.3	5.8	0.55	0.55
	7	960	560	150	110	170	120	51	11.0	8.5	0.72	0.72
	8	900	560	150	110	170	120	51	8.3	6.4	0.55	0.55

(Continued)

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Table 9.2 (Concluded)

Nominal thickness, mm	No. of plies	Bending, m_p, N•mm/mm		Axial tension, t_p, N/mm		Axial compression, p_p, N/mm		Shear-through-thickness, v_p, N/mm	Planar shear: Bending, v_{psb}, N/mm		Planar shear: Shear in-plane, v_{pf}, MPa	
		Orientation of applied force relative to face grain										
		0°	90°	0°	90°	0°	90°	0° and 90°	0°	90°	0°	90°
22.5	6	1000	720	200	100	220	120	54	14.0	6.8	0.72	0.55
	7*	1000	580	150	110	170	120	54	12.0	9.5	0.72	0.72
	8	1100	560	150	110	170	120	54	9.0	6.9	0.55	0.55
	9	1200	730	190	140	210	160	54	12.0	8.8	0.72	0.72
25.5	7	1300	880	180	150	200	160	61	13.0	10.0	0.72	0.72
	8*	1300	690	150	110	170	120	61	10.0	8.4	0.55	0.55
	9	1400	810	190	140	210	160	61	13.0	9.7	0.72	0.72
	10	1400	920	190	140	210	160	61	10.0	7.8	0.55	0.55
28.5	8	1500	950	160	130	180	140	68	11.0	9.7	0.55	0.55
	9*	1500	970	190	140	210	160	68	15.0	11.0	0.72	0.72
	10	1600	890	190	140	210	160	68	12.0	8.7	0.55	0.55
	11	1700	1100	230	180	250	200	68	14.0	12.0	0.72	0.72
31.5	8	1800	1400	200	170	230	190	76	12.0	11.0	0.55	0.55
	9	1800	1300	200	170	230	190	76	17.0	13.0	0.72	0.72
	10*	1800	1000	190	140	210	160	76	13.0	10.0	0.55	0.55
	11	1900	1300	230	180	250	200	76	16.0	13.0	0.72	0.72
	12	1900	1200	230	180	250	200	76	12.0	9.9	0.55	0.55

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Table 9.2 (Continued)

Nominal thickness, mm	No. of plies	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Orientation of applied force relative to face grain				
		0°	90°	0°	90°	0° and 90°
7.5	3	340 000	17 000	55 000	24 000	3400
9.5	3*	610 000	27 000	55 000	28 000	4300
12.5	3	1 400 000	79 000	81 000	39 000	5700
	4*	1 300 000	190 000	55 000	57 000	5700
	5	1 400 000	350 000	79 000	47 000	5700
15.5	4	2 300 000	430 000	59 000	75 000	7100
	5*	2 300 000	630 000	79 000	47 000	7100
	6	2 400 000	760 000	79 000	47 000	7100
18.5	5	3 600 000	1 300 000	83 000	69 000	8600
	6*	3 600 000	1 300 000	79 000	47 000	8600
	7	3 900 000	1 400 000	100 000	71 000	8600
20.5	5	4 600 000	2 600 000	100 000	89 000	9500
	6	4 600 000	1 900 000	79 000	47 000	9500
	7	4 900 000	2 000 000	100 000	71 000	9500
	8	4 800 000	2 100 000	100 000	71 000	9500
22.5	6	5 600 000	2 800 000	130 000	69 000	10 000
	7*	5 700 000	2 200 000	100 000	71 000	10 000
	8	6 000 000	2 300 000	100 000	71 000	10 000
	9	6 400 000	3 100 000	130 000	95 000	10 000
25.5	7	7 900 000	3 900 000	120 000	98 000	12 000

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Table 9.2 (Continued)

Nominal thickness, mm	No. of plies	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Orientation of applied force relative to face grain				
		0°	90°	0°	90°	0° and 90°
	8*	8 000 000	3 400 000	100 000	71 000	12 000
	9	8 400 000	3 700 000	130 000	95 000	12 000
	10	8 800 000	4 400 000	130 000	95 000	12 000
28.5	8	10 000 000	5 400 000	110 000	85 000	13 000
	9*	11 000 000	5 200 000	130 000	95 000	13 000
	10	11 000 000	5 100 000	130 000	95 000	13 000
	11	12 000 000	6 100 000	150 000	120 000	13 000
31.5	8	14 000 000	8 800 000	140 000	110 000	14 000
	9	14 000 000	7 900 000	140 000	110 000	14 000
	10*	14 000 000	6 700 000	130 000	95 000	14 000
	11	15 000 000	7 700 000	150 000	120 000	14 000
	12	15 000 000	7 500 000	150 000	120 000	14 000

* *Most commonly available product.*

Notes:

1) *For specified stiffness in edgewise bending, use axial stiffness values.*
2) *The tabulated values are based on dry-service conditions and standard-term duration of load.*
3) *The specified strength in bearing (normal to plane of panel), q_p, is 4.5 MPa.*
4) *See Figure 9.1 for clarification of shear orientations.*

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Table 9.3
Specified strength, stiffness, and rigidity capacities for construction sheathing OSB

(See Clauses 9.3.2, 9.5.2, 9.5.3, 9.5.4.1, 9.5.4.2, 9.5.5.1, 9.5.5.2, 9.5.6–9.5.8, 10.5.2, 10.5.4–10.5.6, 10.6.2, 10.6.3.1, 10.6.3.6, 10.6.3.7, 11.6.2.2, 11.6.3.2, 11.7.1.2, 11.7.2, 16.2.5, A.9.2.2, and A.11.7.1. See also Table A.1 and Figure A.4.)

Panel mark (CSA O325)	Minimum nominal thickness, mm	Bending, m_p, N•mm/mm		Axial tension, t_p, N/mm		Axial compression, p_p, N/mm		Shear-through-thickness, v_p, N/mm	Planar shear: Bending, v_{psb}, N/mm		Planar shear: Shear in-plane, v_{pf}, MPa	
		Capacities relative to major axis*										
		0°	90°	0°	90°	0°	90°	0° and 90°	0°	90°	0°	90°
W24	9.5	NA	NA	NA	NA	62	50	42	NA	NA	NA	NA
2R24	9.5	180	57	53	18	62	54	42	3.8	2.4	0.60	0.38
W24	11.0	NA	NA	NA	NA	71	54	46	NA	NA	NA	NA
1R24/ 2F16	11.0	240	68	60	30	71	54	46	4.4	2.4	0.60	0.33
2R32/ 2F16	12.0	270	100	65	38	77	67	50	4.8	3.0	0.60	0.38
2R40/ 2F20	15.0	460	160	67	48	92	87	55	6.1	3.8	0.61	0.38
2R48/ 2F24	18.0	630	240	92	59	110	94	60	7.8	4.4	0.65	0.37
1F16	15.0	310	100	60	43	87	78	47	5.2	3.3	0.52	0.33
1F20	15.0	360	150	67	48	92	87	54	6.1	3.9	0.61	0.39
1F24	18.0	480	230	77	59	110	94	59	7.8	4.5	0.65	0.37
1F32	22.0	640	400	92	75	140	130	64	9.2	6.4	0.63	0.44
1F48	28.5	1200	720	130	110	180	150	85	14.0	10.0	0.73	0.55

(Continued)

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Table 9.3 (Concluded)

Panel mark (CSA O325)	Minimum nominal thickness, mm	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Capacities relative to major axis*				
		0°	90°	0°	90°	0° and 90°
W24	9.5	NA	NA	44 000	27 000	9000
2R24	9.5	560 000	100 000	44 000	33 000	10 000
W24	11.0	NA	NA	48 000	35 000	11 000
1R24/2F16	11.0	730 000	140 000	48 000	36 000	11 000
2R32/2F16	12.0	1 100 000	220 000	55 000	36 000	11 000
2R40/2F20	15.0	2 100 000	500 000	66 000	38 000	12 000
2R48/2F24	18.0	3 800 000	820 000	77 000	44 000	13 000
1F16	15.0	1 400 000	300 000	56 000	36 000	11 000
1F20	15.0	2 000 000	360 000	56 000	38 000	11 000
1F24	18.0	2 800 000	720 000	75 000	44 000	12 000
1F32	22.0	6 100 000	2 100 000	99 000	55 000	15 000
1F48	28.5	11 100 000	4 400 000	108 000	61 000	20 000

** Orientation of applied force relative to panel's major strength direction.*

Notes:

1) *For specified stiffness in edgewise bending, use axial stiffness values.*
2) *The tabulated values are based on dry-service conditions and standard-term duration of load.*
3) *The specified strength in bearing (normal to plane of panel), q_p, is 4.2 MPa.*
4) *See Figure 9.1 for clarification of shear orientations.*

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Figure 9.1
Shear orientation in panel products
(See Tables 9.1, 9.2, and 9.3.)

Shear-through-thickness, v_p, N/mm

Planar shear due to in-plane forces in a gusset or splice plate, v_{pf}, MPa

Planar shear due to bending, v_{psb}, N/mm

9.4 Modification factors

9.4.1 Load-duration factor, K_D

9.4.1.1 General

Except as specified in Clause 9.4.1.2, the specified strength capacity values for structural panels shall be multiplied by a load-duration factor, K_D, as specified in Table 5.1.

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9.4.1.2 Long-term loads

For OSB used in structures subject to long-term loads greater than 50% of design capacity that are protected from direct exposure to moisture but exposed to intermittent high temperature and/or humidity conditions, the load-duration factor shall be 0.45.

9.4.2 Service-condition factor, K_S

The specified strength capacity values for structural panels shall be multiplied by a service-condition factor, K_S, as specified in Table 9.4.

Note: *OSB is specified for use under dry-service conditions only.*

Table 9.4
Service-condition factor, K_S
(See Clauses 9.4.2, 10.5.2, 11.6.2.2, and 11.6.3.2.)

	Plywood		OSB
	Service conditions		
Property to be modified	**Dry**	**Wet**	**Dry**
Specified strength capacity	1.0	0.80	1.0
Specified stiffness and rigidity capacities	1.0	0.85	1.0

9.4.3 Treatment factor, K_T

For preservative-treated plywood, the treatment factor, K_T, shall be equal to 1.0. For other preservative-treated structural panels and structural panels treated with fire-retardant or other potentially strength-reducing chemicals, strength and stiffness capacities shall be based on the documented results of tests considering the effects of time, temperature, and moisture content in accordance with Clause 4.3.2.

9.4.4 Stress-joint factor, X_J

9.4.4.1 Scarf joints

Where scarf joints transmit forces from one panel to another, the specified strength capacity shall be multiplied by a stress-joint factor, X_J, as specified in Table 9.5. Construction requirements for scarf joints are given in Clause 10.3.3.

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Table 9.5
Stress-joint factor, X_J, for scarf joints
(See Clauses 9.4.4.1 and 10.4.2.)

Panel type	Slope of scarf	Tension, bending	Compression	Shear
Plywood	1 in 12	0.85	1.00	1.00
	1 in 10	0.80	1.00	1.00
	1 in 8	0.75	1.00	1.00
	1 in 5	0.60	1.00	—
OSB	1 in 6	0.80	0.80	0.80
	1 in 5	0.70	0.70	0.70
	1 in 4	0.60	0.60	0.60

9.4.4.2 Butt joints

Where structural panels are used as glued splice plates in butt joints to transmit forces from one panel to another or from one lumber piece to another, the specified strength of splice plates shall be multiplied by a stress-joint factor, X_J, as specified in Table 9.6. Construction requirements for butt joints are given in Clause 10.3.4.

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Table 9.6
Stress-joint factor, X_J, for butt joints
(See Clauses 9.4.4.2, 10.3.4.4, 10.3.4.5, 10.4.3.1, and 10.4.3.2.)

		One side			Both sides		
Main/side member	**Nominal thickness of panel splice plate, mm**	**Minimum length of splice plate, mm**	**Tension**	**Compression, shear**	**Minimum length of splice plate, mm**	**Tension**	**Compression, shear**
Plywood/ plywood	7.5	200	0.67	1.00	200	0.85	1.00
	9.5	300	0.67	1.00	300	0.85	1.00
	12.5	350	0.67	1.00	350	0.85	1.00
	15.5–20.5	400	0.50	1.00	400	0.85	1.00
OSB/OSB	9.5–11.0	200	0.40	0.40	200	0.90	0.90
		650*	0.70	0.70	—	—	—
	12.5–18.5	200	0.30	0.30	200	0.85	0.85
		800*	0.50	0.50	—	—	—
Lumber/OSB	9.5–11.0	200	0.60	0.60	200	0.90	0.90
		250*	0.75	0.75	—	—	—
	12.5–18.5	200	0.40	0.40	200	0.85	0.85
		300*	0.55	0.55	—	—	—

* *For intermediate OSB splice plate lengths, the stress joint factor shall be obtained by linear interpolation.*

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9.4.5 End-use factor, K_F, for permanent wood foundations

For plywood in permanent wood foundations supported at intervals not exceeding 815 mm, the end-use factor for panel bending and planar shear, K_F, shall be 1.15. For all other properties, K_F shall be 1.0.

9.5 Strengths and resistances

9.5.1 Stress orientation

Structural panels are orthotropic materials. The specified strength capacities used in calculations shall be those for the orientation of the major axis of the panel intended in the design.

9.5.2 Flatwise bending

The factored flatwise bending moment resistance, M_r, of a structural panel shall be calculated as follows:

$M_r = \phi\, M_p\, b_p$

where

ϕ = 0.95

M_p = $m_p\,(K_D\, K_S\, K_T\, K_F)$ for plywood

= $m_p\,(K_D\, K_S\, K_T)$ for OSB

where

m_p = specified strength capacity in bending, N•mm/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

b_p = width of panel, mm

9.5.3 Edgewise bending

The factored edgewise bending moment resistance, M_r, of a structural panel that is adequately braced to prevent lateral buckling shall be calculated as follows:

$$M_r = \phi T_p \frac{d_p^2}{6}$$

where

ϕ = 0.95

T_p = $t_p\,(K_D\, K_S\, K_T)$

where

t_p = specified strength capacity in tension, N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

d_p = depth of panel in plane of bending, mm

9.5.4 Planar shear

9.5.4.1 Planar shear due to bending

The factored resistance in planar shear, V_{rp}, for structural panels subjected to bending shall be calculated as follows:

$V_{rp} = \phi\, V_{psb}\, b_p$

where

ϕ = 0.95

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V_{psb} = v_{psb} (K_D K_S K_T K_F) for plywood

= v_{psb} (K_D K_S K_T) for OSB

where

v_{psb} = specified strength capacity in planar shear (due to bending), N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

9.5.4.2 Planar shear in structural panel splice or gusset plate

The factored resistance in planar shear, V_{rp}, developed by a glued structural panel splice or gusset plate, or by the splice plates at a structural panel butt joint, shall be calculated as follows:

$V_{rp} = \phi\ V_{pf}\ A_c$

where

ϕ = 0.95

V_{pf} = v_{pf} (K_D K_S K_T)

where

v_{pf} = specified strength capacity in planar shear (due to in-plane forces), MPa (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

A_c = contact area of splice or gusset plate on one side of joint, mm^2

9.5.5 Shear-through-thickness of structural panel

9.5.5.1 Shear due to edgewise bending of structural panel

The factored resistance in shear, V_r, through the thickness of a structural panel due to edgewise bending shall be calculated as follows:

$$V_r = \phi\ V_p\ \frac{2d_p}{3}$$

where

ϕ = 0.95

V_p = v_p (K_D K_S K_T)

where

v_p = specified strength capacity in shear-through-thickness, N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

9.5.5.2 Shear-through-thickness in structural panel splice or gusset plate

The factored shear-through-thickness resistance, V_r, developed by a structural panel splice or gusset plate shall be calculated as follows:

$V_r = \phi\ V_p\ L_G$

where

ϕ = 0.95

V_p = v_p (K_D K_S K_T)

where

v_p = specified strength capacity in shear-through-thickness, N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

L_G = length of splice or gusset plate subjected to shear, mm

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9.5.6 Compression parallel to panel edge

The factored compressive resistance, P_r, parallel to a laterally supported panel edge shall be calculated as follows:

$P_r = \phi\ P_p\ b_p$

where

ϕ = 0.95

P_p = $p_p\ (K_D\ K_S\ K_T)$

where

p_p = specified strength capacity in axial compression, N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

9.5.7 Tension parallel to panel edge

The factored tensile resistance, T_r, parallel to a panel edge shall be calculated as follows:

$T_r = \phi\ T_p\ b_n$

where

ϕ = 0.95 for all plywood thicknesses and numbers of plies except 3- and 4-ply layups stressed perpendicular to face grain

= 0.60 for 3- and 4-ply plywood layups stressed perpendicular to face grain

= 0.95 for OSB

T_p = $t_p\ (K_D\ K_S\ K_T)$

where

t_p = specified strength capacity in axial tension, N/mm (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

b_n = net width of panel after cutting of holes, etc., mm

9.5.8 Compressive resistance perpendicular to face (bearing)

The factored bearing resistance, Q_r, normal to the plane of panel shall be calculated as follows:

$Q_r = \phi\ Q_p\ A_b$

where

ϕ = 0.95

Q_p = $q_p\ (K_D\ K_S\ K_T)$

where

q_p = specified strength capacity in bearing normal to plane of panel, MPa (see Tables 9.1 and 9.2 for plywood and Table 9.3 for OSB)

A_b = bearing area, mm^2

10 Composite building components

10.1 Scope

Design data, methods, and construction requirements specified in Clause 10 apply to glued composite building components using plywood, OSB, and lumber or glued-laminated timber specified in this Standard. The design requirements of Clause 10 apply if the glue joints are made in strict compliance

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with the requirements of the manufacturer of the adhesive used in fabricating the composite building components.

Note: *Clause 10 does not apply to proprietary structural wood products.*

10.2 Materials

10.2.1 General

Clauses 10.3 to 10.6 apply to the materials specified in Clause 6 for lumber, Clause 7 for glued-laminated timber, Clause 9 for plywood and OSB, and Clause 10.2.2 for adhesives.

10.2.2 Adhesives for structural components

Adhesives for the assembly of structural components shall meet the requirements of CSA O112.7, CSA O112.9, or CSA O112.10.

10.3 Construction requirements for stress joints

10.3.1 Types of stress joints

Joints transmitting forces from one panel to another may be either scarf joints or butt joints.

10.3.2 Adhesives for stress joints

Adhesives for the assembly of stress joints shall meet the requirements of Clause 10.2.2.

10.3.3 Scarf joints

10.3.3.1 Scarf joints in shear

The slope of plywood scarf joints shall not be steeper than 1:8. The slope of OSB scarf joints shall not be steeper than 1:4.

10.3.3.2 Scarf joints in tension, compression, or bending

The slope of plywood scarf joints shall not be steeper than 1:5. The slope of OSB scarf joints shall not be steeper than 1:4.

10.3.4 Butt joints

10.3.4.1 Splice plates for butt joints

Butt joints shall be backed on one or both sides of the panel by a panel splice plate of a type and grade at least equal to the panel being spliced. The splice plate shall be centred on the joints and glued to both panels meeting at the joint. The splice plate shall be oriented with its major axis perpendicular to the joint.

10.3.4.2 Splice plate thickness

Splice plates shall have a minimum thickness equal to that of the panel being spliced.

10.3.4.3 Butt joints in shear

Splice plates stressed in shear shall have a length in the direction perpendicular to the joint equal to 12 times the thickness of the butt-jointed panel and shall have a width equal to the full depth or width of the panel between framing members.

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10.3.4.4 Butt joints in tension

Splice plates stressed in tension shall have a minimum length as specified in Table 9.6.

10.3.4.5 Butt joints in compression

Splice plates stressed in compression may have a length as specified in Table 9.6. For plywood joints, if shorter lengths are used, the strength of the joint shall be reduced as specified in Clause 10.4.3.2.

10.4 Stress-joint factor, X_J

10.4.1 Joint requirements

The stress-joint factors, X_J, specified in Clauses 10.4.2 and 10.4.3 apply to glued plywood and OSB stress joints fabricated in accordance with Clause 10.3.

10.4.2 Scarf joints

The stress-joint factor for scarf joints across the major axis of plywood or OSB stressed in tension, compression, or shear-through-thickness shall be as specified in Table 9.5.

10.4.3 Butt joints

10.4.3.1 General

For the length of splice plates perpendicular to the joint, the stress-joint factors for butt joints across the major axis of plywood or OSB stressed in tension, compression, or shear-through-thickness shall be as specified in Table 9.6.

10.4.3.2 Butt joints with short plywood plates in compression

For plywood butt joints with splice plates shorter than the minimum length specified in Table 9.6, the stress-joint factor for compression shall be reduced in direct proportion to such reduction in length.

10.5 Plywood- and OSB-web beams

10.5.1 General

A panel-web beam shall have one or more plywood or OSB webs glued or nailed at upper and lower edges to sawn lumber or glued-laminated timber flanges, with lumber stiffeners at intervals along the web to prevent buckling (see Figure 10.1).

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Figure 10.1
Panel web beam dimensions, mm
(See Clauses 10.5.1, 10.5.3, and 10.5.6.)

10.5.2 Effective stiffness

The effective stiffness, $(EI)_e$, of a panel-web beam shall be calculated as follows:

$$(EI)_e = \left(\sum B_a\right) K_S \frac{(c_t^3 + c_c^3)}{3} + (EI)_f K_{SE}$$

where

(ΣB_a) = sum of axial stiffness of panel webs, N/mm (see Tables 9.1 to 9.3)

K_S = service-condition factor for web material (see Table 9.4)

$(EI)_f$ = stiffness of flanges with respect to neutral axis of composite section, N•mm^2

K_{SE} = service-condition factor for modulus of elasticity of flange (see Table 6.10 for sawn lumber and Table 7.3 for glulam)

Δ **10.5.3 Bending moment resistance**

The factored bending moment resistance, M_r, of a panel-web beam shall be the lesser of the factored resistance of the tension or compression flanges, calculated as follows:

a) compression flange:

$$M_r = \phi F_c K_{Zc} \frac{(EI)_e}{E K_{SE} c_c}$$

where

ϕ = 0.8 for sawn lumber

= 0.9 for glued-laminated timber

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F_c = f_c (K_D K_{Sc} K_T K_H)

where

f_c = specified strength of flange in compression, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

K_H = system factor (see Clause 6.4.4 for sawn lumber and Clause 7.4.4 for glulam)

K_{Zc} = size factor for compression for sawn lumber (see Clause 6.5.5.2.4)

E = modulus of elasticity of flange, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

K_{SE} = service-condition factor for modulus of elasticity of flange (see Table 6.10 for sawn lumber and Table 7.3 for glulam)

c_c = distance from neutral axis to compression face (see Figure 10.1)

b) tension flange:

$$M_r = \phi F_t K_{Zt} \frac{(EI)_e}{E K_{SE} c_t}$$

where

ϕ = 0.9

F_t = f_t (K_D K_{St} K_T K_H)

where

f_t = specified strength of flange in tension, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

K_{Zt} = size factor for tension for sawn lumber (see Table 6.13)

E = modulus of elasticity of flange, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

c_t = distance from neutral axis to tension face (see Figure 10.1)

Δ **10.5.4 Web shear-through-thickness**

The factored shear resistance, V_r, of the web of a panel-web beam at its neutral axis shall be calculated as follows:

$$V_r = \phi V_p X_J \frac{(EI)_e}{E K_{SE} Q_f + 0.5(\Sigma B_a) K_S c_w^2}$$

where

ϕ = 0.95

V_p = (Σv_p) (K_D K_S K_T)

where

Σv_p = sum of specified strengths of all panel webs in shear-through-thickness, N/mm (see Tables 9.1 to 9.3)

X_J = stress-joint factor (see Clause 10.4)

E = modulus of elasticity of flange, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

Q_f = moment of area of flange about neutral axis, mm³

ΣB_a = sum of specified axial stiffness for all panel webs, N/mm (see Tables 9.1 to 9.3)

c_w = greatest distance from neutral axis to outer edge of web, mm

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10.5.5 Flange-web shear

The factored shear resistance, V_{rp}, of the glued area between the flange and web of a panel-web beam shall be the lesser of the shear capacities of the web or flange components, calculated as follows:

$$V_{rp} = \phi V_g \left(\Sigma b_g X_v \right) \frac{(EI)_e}{E K_{SE} Q_f}$$

where

Σb_g = sum of contact widths between flange and web

E = modulus of elasticity of flange, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

Q_f = moment of area of flange about neutral axis, mm³

a) for web:

ϕ = 0.95

V_g = v_{pf} (K_D K_S K_T)

where

v_{pf} = specified strength in planar shear, MPa (see Tables 9.1 to 9.3)

X_v = shear-modification factor (see Figure 10.2)

b) for flange:

ϕ = 0.9

V_g = f_v (K_D K_{Sv} K_T)

where

f_v = specified strength in shear, MPa (see Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.2 for glulam)

X_v = 2.00

10.5.6 Deflection calculation

Deflection shall be calculated as the sum of the deflections due to moment, using the effective stiffness, $(EI)e$, determined in accordance with Clause 10.5.2, and due to shear, Δ_s, calculated as follows:

$$\Delta_s = \frac{B_a M h^2 X_s}{B_v (EI)_e}$$

where

B_a = specified axial stiffness of the web, N/mm (see Tables 9.1 to 9.3)

M = maximum bending moment due to specified loads, N•mm

h = height of web beam, mm (see Figure 10.1)

X_s = section-shear coefficient (see Figure 10.3)

B_v = specified shear-through-thickness rigidity of the web, N/mm (see Tables 9.1 to 9.3)

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Δ

Figure 10.2
Shear-modification factor, X_v
(See Clauses 10.5.5 and 10.6.3.7.)

Notes:

1) *b_g, a_{min}, and a_{max} are given in millimetres.*
2) *The following requirements shall apply to the determination of X_v:*
   a) *stressed skin panels:*
      i) *at an inside web, $a_{min} = a_{max}$; and*
      ii) *at an outside web:*
         i) *a_{min} = the overhang at the edge; and*
         ii) *a_{max} = one-half the clear spacing between the outside web and the adjacent web;*
   b) *panel web beams: $a_{min} = 0$; and*
   c) *for all other cases (splice plates, etc.), the unmodified specified strength capacity in planar shear shall be used.*

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Δ

Figure 10.3
Section-shear coefficient, X_s
(See Clause 10.5.6.)

Notes:

1) *The section-shear coefficient, X_s, is a geometrical property of a beam section that depends on the shape of the cross-section and arises because of non-uniform distribution of shearing stresses across the section. It can be derived using fundamental engineering theory and evaluated for any beam geometry as follows:*

$$X_s = \frac{1}{Ih^2}\int_{y=0}^{y=h}\frac{Q^2 dy}{b_x}$$

where

I = moment of inertia, mm^4

h = overall beam depth, mm

Q = first moment of beam, mm^3

b_x = width of beam carrying the shear associated with Q, mm

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Figure 10.3 (Concluded)

2) *This Figure is valid only for box and I-beams symmetrical about two axes.*

10.5.7 Lateral stability of panel-web beams

Lateral stability of a panel-web beam shall be determined by considering the flange as a column that tends to deflect sideways between points of support or by application of one of the following rules:

a) If the ratio of the moment of inertia of the cross-section about the neutral axis to the moment of inertia about the axis perpendicular to the neutral axis does not exceed 5:1, lateral support shall not be required.
b) If the ratio of the moments of inertia is greater than 5:1 but does not exceed 10:1, the ends of the beam shall be held in position at the bottom flange at supports.
c) If the ratio of the moments of inertia is greater than 10:1 but does not exceed 20:1, the beam shall be held in line at the ends.
d) If the ratio of the moments of inertia is greater than 20:1 but does not exceed 30:1, one edge shall be held in line.
e) If the ratio of the moments of inertia is greater than 30:1 but does not exceed 40:1, the beam shall be restrained by bridging or other bracing at intervals of not more than 2400 mm.
f) If the ratio of the moments of inertia is greater than 40:1, the compression flanges shall be fully restrained.

10.5.8 Stiffeners

Load distribution stiffeners shall be provided at reaction points and at the location of heavy concentrated loads. The stiffeners shall be securely fastened to the webs, bear on the inner surfaces of the top and bottom flanges, and be made as wide as the flanges. Their dimensions parallel to the span shall be adequate to support the applied concentrated loads or reactions. The cross-sectional area of a load-bearing stiffener shall be such that the factored resistance of the flange material perpendicular to grain is not less than the concentrated load or the reaction due to the factored loads.

10.5.9 Web stabilizers

Web-stabilizing stiffeners shall be provided as necessary to prevent buckling of the webs.

10.6 Stressed skin panels

10.6.1 General

A stressed skin panel shall have continuous or spliced longitudinal web members and continuous or spliced panel flanges on one or both panel faces, with the flanges glued to the web members (see Figure 10.4).

10.6.2 Effective stiffness

The effective stiffness, $(EI)_e$, of a stressed skin panel shall be calculated as follows:

$$(EI)_e = (EI)_w K_{SE} + b_f (B_{at} y_t^2 + B_{ac} y_c^2) K_S$$

where

$(EI)_w$ = stiffness of lumber webs, N•mm^2

B_{at} = specified axial stiffness of tension flange, N/mm (B_a for appropriate panel thickness in Tables 9.1 to 9.3)

B_{ac} = specified axial stiffness of compression flange, N/mm (see Tables 9.1 to 9.3)

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b_f, y_t, y_c = panel dimensions, mm, in accordance with Figure 10.4

Figure 10.4
Stressed skin panel dimensions
(See Clauses 10.6.1, 10.6.2, 10.6.3.4, and 10.6.3.6.)

10.6.3 Bending moment resistance

Δ 10.6.3.1 Bending along stressed skin panel span

The factored bending moment resistance along the direction of the webs of a stressed skin panel shall be the least of the factored resistances of the tension or compression flanges or the web, calculated as follows:

a) tension flange:

$$M_r = \phi T_p X_J X_G \frac{(EI)_e}{B_a K_S c_t}$$

where

ϕ = 0.95

T_p = t_p (K_D K_S K_T)

where

t_p = specified strength capacity of flange in axial tension, N/mm (see Tables 9.1 to 9.3)

X_J = stress-joint factor (see Clause 10.4)

X_G = panel geometry reduction factor (see Clause 10.6.3.2)

B_a = specified axial stiffness of the flange, N/mm (see Tables 9.1 to 9.3)

b) compression flange:

$$M_r = \phi P_p X_J X_G \frac{(EI)_e}{B_a K_S c_c}$$

where

ϕ = 0.95

P_p = p_p (K_D K_S K_T)

where

p_p = specified strength capacity of flange in axial compression, N/mm (see Tables 9.1 to 9.3)

c) web:

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$$M_r = \phi F_b K_{Zb} K_L X_G \frac{(EI)_e}{EK_{SE} c_w}$$

where

ϕ = 0.9

F_b = f_b (K_D K_{Sb} K_T K_H)

where

f_b = specified strength in bending of webs, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

E = modulus of elasticity of web, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

c_w = greatest distance from neutral axis to outer edge of web, mm

10.6.3.2 Panel geometry reduction factor

The panel geometry reduction factor, X_G, shall be calculated as follows:

$$X_G = 1 - 4.8\left(\frac{s}{\ell_p}\right)^2$$

where

s = clear spacing between longitudinal web members, mm

ℓ_p = span of stressed skin panel, mm

Note: *This formula accounts for shear lag and is valid for values of s/ℓ_p ranging from 0.05 to 0.25.*

10.6.3.3 Bending perpendicular to panel span

The factored bending resistance of the compression flange between web members shall be calculated using Clause 9.5.2.

10.6.3.4 Buckling of compression flange

The compression flange of a stressed skin panel shall be designed according to principles of engineering mechanics to prevent elastic buckling failure. If a detailed analysis is not made, such a condition shall be assumed to be met if

a) $s \leq 50h_c$ for panels with their major axis parallel to the span (ℓ_p); or

b) $s \leq 40h_c$ for panels with their major axis perpendicular to the span (ℓ_p)
where s and h_c are as shown in Figure 10.4.

Note: *See Clause A.10.6.3.4 for additional information on buckling of compression flanges and a detailed analysis of stressed skin panels with a plywood compression flange.*

10.6.3.5 Shear in-plane of plies

The factored planar shear resistance of the compression flange in a stressed skin panel shall be calculated in accordance with Clause 9.5.4.1.

10.6.3.6 Shear resistance

The factored shear resistance at the neutral plane of a stressed skin panel, V_r, shall be calculated as follows:

$$V_r = \phi F_v K_{Zv} \frac{(EI)_e \Sigma b_g}{EK_{SE} \Sigma Q_w + B_a K_s b_f y}$$

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where

ϕ = 0.9

F_v = f_v (K_D K_{Sv} K_T K_H)

where

f_v = specified strength in shear of webs, MPa (see Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.2 for glulam)

K_{Zv} = size factor in shear (see Clause 6.4.5)

b_g = contact width between flange and web (see Figure 10.4)

E = modulus of elasticity of web, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

ΣQ_w = sum of moments of area of all webs about neutral plane, mm^3

B_a = specified axial stiffness of the flange, N/mm (see Tables 9.1 to 9.3)

b_f = width of flange, mm

y = the greater value of y_t or y_c, mm

10.6.3.7 Flange-web shear

The factored shear resistance, V_{rp}, of the glued area between the flange and the web of a stressed skin panel shall be taken as the lesser of the shear capacities based on flange or web components, calculated as follows:

$$V_{rp} = \phi V_g \frac{(EI)_e \Sigma\left(b_g X_v\right)}{B_a K_S b_p\, y}$$

where

B_a = specified axial stiffness of the web, N/mm (see Tables 9.1 to 9.3)

y = the greater value of y_t or y_c, mm

a) for flange:

ϕ = 0.95

V_g = v_{pf} (K_D K_S K_T)

where

v_{pf} = specified strength capacity in planar shear of the web, MPa (see Tables 9.1 to 9.3)

X_v = shear-modification factor (see Figure 10.2)

b) for web:

ϕ = 0.9

V_g = f_v (K_D K_S K_T)

where

f_v = specified strength in shear, MPa (see Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.2 for glulam)

X_v = 2.00

10.6.3.8 Deflection

The deflection of stressed skin panels shall be calculated using the effective stiffness, $(EI)_e$, determined in accordance with Clause 10.6.2, multiplied by the panel geometry reduction factor, X_G, determined in accordance with Clause 10.6.3.2.

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11 Lateral-load-resisting systems

11.1 Scope

Clause 11 covers the design of structural systems and assemblies subjected to lateral loads (e.g., wind, seismic, and earth pressure loads).

11.2 Materials

11.2.1 General

Clause 11 applies to the structures made with the materials specified in Clause 6 for lumber, Clause 7 for glued-laminated timber, Clause 8 for CLT, Clause 9 for structural panels, Clause 12 for connections, and Clause 15.3 for proprietary SCL that has been qualified for a specific grade for use in shearwalls with full-scale cyclic testing.

11.2.2 Additional materials

Clauses 11.3 to 11.8 also apply to gypsum wallboard conforming to Type X (fire-rated) in ASTM C1396/C1396M.

11.3 Design of light-frame shearwalls and diaphragms

11.3.1 General

Shearwalls and diaphragms constructed in accordance with requirements specified in Clause 11.3 may be used to resist lateral forces if they meet the following design requirements:

a) The factored shear resistance determined in accordance with Clauses 11.4 and 11.6.2 for shearwalls and Clause 11.6.3 for diaphragms shall exceed the factored lateral design forces.
b) Shearwalls and diaphragms shall be designed to resist in-plane bending moments resulting from the application of lateral design forces in accordance with Clause 11.6.4.
c) Boundary frame members shall be designed to resist axial forces resulting from the application of lateral design forces. Boundary frame members in single-layer diagonally sheathed shearwalls and diaphragms shall be designed in accordance with Clause 11.6.
d) All boundary members, chords, and struts of shearwalls and diaphragms shall be designed and detailed to transmit the induced axial forces.
e) Connections and anchorages capable of resisting the forces to be transferred shall be provided between the shearwall or diaphragm and the attached components and designed in accordance with Clause 12.
f) Where applicable, seismic design requirements shall be considered in accordance with Clause 11.8.

11.3.2 Shearwalls and diaphragms using wood structural panels

11.3.2.1 Framing

Framing members shall be at least 38 mm thick in shearwalls and diaphragms. For diaphragms with multiple rows of fasteners, framing members shall be at least 64 mm thick and 64 mm wide at boundaries and adjoining panel edges. Boundary members shall be adequately interconnected at corner intersections and member joints shall be spliced adequately.

11.3.2.2 Sheathing

Shearwalls and diaphragms using plywood or OSB shall be constructed with panels not less than 7.5 mm in thickness and 1200 × 2400 mm in plane dimensions, except near boundaries and changes in framing,

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where up to two short or narrow panels may be used. For construction sheathing OSB, the product specification shall also include a panel mark identifying an end-use rating. Table A.9 provides an equivalence between nominal thicknesses of OSB panel and panel marks. For panels that also act as exterior siding, the minimum panel thickness shall be 9.5 mm.

Sheathing panels for blocked shearwalls and diaphragms shall be arranged as indicated in Figures 11.1 and 11.2, respectively. All panel edges shall bear and be attached to the framing members or blocking except for unblocked assemblies where the blocking between framing members is omitted as indicated in Figures 11.5 and 11.6. A gap of not less than 2 mm shall be left between adjacent panel edges. Panels less than 300 mm wide shall be blocked.

Figure 11.1
Configurations of blocked shearwalls
(See Clause 11.3.2.2.)

Figure 11.2
Configurations of blocked diaphragms
(See Clause 11.3.2.2.)

	Cases 1 and 3: Continuous panel joints perpendicular to framing	Cases 2 and 4: Continuous panel joints parallel to framing	Cases 5 and 6: Continuous panel joints perpendicular and parallel to framing
Long panel direction perpendicular to supports			
Long panel direction parallel to supports			

Source: *Table 4.2A of ANSI/AWC SDPWS.*

11.3.2.3 Sheathing fastening

Fasteners shall be placed not less than 9 mm from the framing edge and panel edge and shall be spaced not less than 50 mm and not more than 150 mm along all edges of each panel. Fastener spacing along all intermediate framing members shall not be greater than 300 mm. Fasteners shall be firmly driven into framing members but shall not be over-driven into sheathing more than 15% of the panel thickness.

Where the fastening in a shearwall is a double-shear connection with a panel in the middle and framing members as side members, the framing members and mid-panel shall be connected using fasteners with penetration meeting the required lengths given in Figure 12.17, and with the same spacing at all locations. There shall be a minimum of 3 mm gap between adjacent panels. For studs where panels meet, additional fasteners not subjected to double shear shall be installed to prevent detachment of the studs (see Figure 11.3).

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Figure 11.3
Fastening for mid-panel shearwalls
(See Clause 11.3.2.3.)

In diaphragms with multiple rows of fasteners, the fasteners in adjacent rows shall be staggered: the spacing of fasteners along the edges of each panel shall not be less than 64 mm within the row and not less than 9 mm between the rows.

11.3.2.4 Additional construction requirements

The following are additional requirements for shearwalls and diaphragms using wood structural panels:

a) Where the fastener spacing at panel edges is 50 mm, framing at adjoining panel edges shall be 64 mm or thicker (or two 38-mm-thick framing members connected to transfer the factored shear force), and the fasteners shall be staggered.

b) Where the fasteners of 3.66 mm in diameter penetrate more than 41 mm into framing and are spaced 75 mm or less, framing at adjoining panel edges shall be 64 mm or thicker (or two 38-mm-thick framing members connected to transfer the factored shear force), and the fasteners shall be staggered.

c) For shearwalls where panels are applied on both faces of a wall and fastener spacing is less than 150 mm on either side, framing shall be 64 mm or thicker (or two 38-mm-thick framing members connected to transfer the factored shear force), and fasteners on each side shall be staggered.

11.3.3 Shearwalls using gypsum wallboard

11.3.3.1 Sheathing

Shearwalls using gypsum wallboard shall be constructed with panels not less than 1200 × 2400 mm, except near boundaries and changes in framing, where up to two short or narrow panels may be used. The application of gypsum wallboard shall be restricted to platform frame construction where the height of a storey does not exceed 3.6 m.

Notes:

1) *There should exist a balanced spatial distribution of the gypsum wallboard and wood structural panels resisting shear in each direction in a storey.*
2) *See Clause 11.8.8 for seismic design requirements for shearwalls using gypsum wallboard.*
3) *See Clause 11.8.9 for seismic design requirements for load-bearing walls constructed with gypsum wallboard only.*

11.3.3.2 Sheathing fastening

In gypsum wallboard application nails or screws shall be placed not less than 9 mm from the panel edge.

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11.3.4 Shearwalls and diaphragms using diagonal lumber sheathing

11.3.4.1 General

Shearwalls and diaphragms described in Clauses 11.3.4.2 and 11.3.4.3 shall be constructed using a single layer of diagonally sheathed lumber (see Clause 11.3.4.2) or a double layer of diagonally sheathed lumber (see Clause 11.3.4.3). More than two layers of diagonal lumber sheathing shall not be permitted.

11.3.4.2 Single-layer diagonal sheathing

Single-layer diagonal sheathing shall be made up of 19-mm or 38-mm boards laid at an angle of approximately 45° to supports. The sheathing shall be fastened to each intermediate framing member with at least two common nails (d_F = 3.25 mm or d_F = 3.33 mm) for 19 × 140 mm boards and three common nails (d_F = 3.25 mm or d_F = 3.33 mm) for 19 × 184 mm or wider boards. One additional nail shall be used in each board at the boundary framing members. End joints in adjacent rows of boards shall be staggered by at least one stud or joist space. Joints on the same support shall be separated by at least two rows of boards. Shearwalls and diaphragms constructed with a 38-mm-thick diagonal sheathing using common nails (d_F = 4.06 mm) may be used at the same shear values and in the same locations as for 19-mm boards fastened with common nails (d_F = 3.25 mm or d_F = 3.33 mm), if there are no splices in adjacent boards on the same support and the supports are not less than 89 mm in depth or 64 mm in thickness.

11.3.4.3 Double-layer diagonal sheathing

Double-layer diagonal sheathing shall meet the requirements of Clause 11.3.4.2 and consist of two layers of diagonal boards 19 mm in thickness for each layer and at 90° to each other on the same face of the supporting members. The nails used for fastening the outer layer shall have a minimum penetration of $8d_F$ into the framing members. Nails in the inner and outer layers shall be staggered along the framing members.

11.4 Design considerations for light-frame shearwalls

11.4.1 Shearwalls with openings

11.4.1.1 General

Only fully sheathed segments that meet the requirements for maximum aspect ratio limits in Clause 11.4.1.2 shall be considered in the analysis of shearwalls with openings. The contribution of sheathing above and below openings shall not be included in the calculation of shearwall resistance.

11.4.1.2 Shearwall segment aspect ratio

The maximum aspect ratio (height-to-length ratio) of a blocked shearwall segment shall be 3.5:1. The maximum aspect ratio of an unblocked shearwall segment shall be 2:1. The height is defined as the height from the underside of the bottom shearwall plate to the topside of the top shearwall plate within a storey.

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11.4.2 Shearwalls with dissimilar materials

11.4.2.1 Standard method

Except as permitted by Clause 11.4.2.2, shearwalls constructed with dissimilar materials, thicknesses, or fastener spacings along the length of the shearwall shall be analyzed as the sum of the different shearwall segments.

11.4.2.2 Alternative method

Shearwalls constructed with dissimilar materials, thicknesses, or fastener spacing along the length of the shearwall may be analyzed as a shearwall of uniform construction, provided that the least value of the shearwall segment resistance, determined in accordance with Clause 11.6.2, is applied over the entire shearwall length.

11.4.3 Shearwalls with multiple layers

11.4.3.1 Shearwalls with two layers of sheathing panels on one side

The factored shear resistance for a shearwall with two layers of the same or different sheathing panels applied to one side shall be determined by the first (inside) layer of panels. Where the first layer is gypsum wallboard with thickness not exceeding 15.9 mm and the second layer is a structural panel, the specified shear resistance of the shearwall may be calculated by assuming that the second layer is applied directly to framing and using the actual fastener penetration into the framing member, if minimum fastener penetration in the framing is satisfied.

11.4.3.2 Shearwalls with sheathing panels on two sides

The factored shear resistance of a shearwall shall be cumulative when sheathing panels of the same or different materials are applied on each side.

Notes:

1) *See Table 11.5 for the appropriate seismic design factors for shearwalls with different sheathing materials.*
2) *See Clause 11.8.8 for seismic design requirements for shearwalls using gypsum wallboard.*

11.4.4 Overturning restraint

11.4.4.1 Shearwall segments with hold-downs

Where the factored dead loads do not prevent shearwall overturning, hold-downs [see Figure 11.4 a)] shall be designed to resist the factored uplift forces and transfer the forces through a continuous load path to the foundation.

11.4.4.2 Shearwall segments without hold-downs

Where the factored dead loads do not prevent shearwall overturning and hold-downs are not used, anchorage [see Figure 11.4 b)] on the bottom plate within 300 mm from each end of the shearwall segment shall transfer the factored uplift force, R_{ij} (see Figure 11.7), to the supporting structure (i.e., to the top plate of the shearwall below or to the foundation).

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Figure 11.4
Examples of hold-downs and anchorages
(See Clauses 11.4.4.1 and 11.4.4.2.)

a) Hold-downs

b) Anchorages

Notes:

1) *These are examples only. Other types of hold-downs and anchorages may be used.*
2) *Hold-downs as specified in Clause 11.4.4.1 provide a continuous direct load path, typically between upper-storey shearwall chords and lower-storey chords, beams, or foundations.*
3) *Anchorages as specified in Clause 11.4.4.2 typically transfer loads from the sill plate of the upper-storey shearwall segment to the lower-storey top plates, beams, or foundations.*

11.4.5 Shearwall anchorage

The anchor bolt connections that resist the lateral forces shall be designed in accordance with Clause 12.

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11.4.6 Concrete or masonry wall anchorage

11.4.6.1 Anchorage design

Where wood roofs and floors are used to provide lateral support to concrete and masonry walls, they shall be anchored to those walls. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be designed to resist the factored lateral force induced by the wall, but not less than 3 kN/m of the wall.

11.4.6.2 Anchorage details

Anchorage to concrete or masonry walls shall not be accomplished by use of toe-nails or nails subject to withdrawal, nor shall wood ledgers be designed to resist tensile stresses perpendicular to grain.

11.5 Strength-adjustment factors for light-frame shearwalls and diaphragms

Δ

11.5.1 Fastener spacing factor, J_s

The shear strength of shearwalls and diaphragms built with wood structural panel and lumber framing shall be multiplied by the fastener spacing factor, J_s, given as follows:

J_s = 1.0 for $s \geq 150$ mm

= $1 - \left(\frac{150 - s}{150}\right)^{4.2}$ for 50 mm $\leq s <$ 150 mm

where

s = fastener spacing along panel edges, mm

Note: *For proprietary framing material, appropriate adjustment factors can be obtained from the product manufacturer (i.e., proprietary design literature and Product Evaluation Reports in the CCMC* Registry of Product Evaluations*).*

11.5.2 Fastener row factor for diaphragms, J_f

The shear strength of diaphragms sheathed with wood structural panels shall be multiplied by the fastener row factor, J_f, given in Table 11.1, to account for the number of rows of fasteners attaching the sheathing to framing in blocked diaphragms.

Table 11.1
Fastener row factor, J_f, for diaphragms
(See Clause 11.5.2.)

No. of rows	Minimum thickness of framing member, mm	J_f
1	38	0.89
	64*	1.00
2	64*	1.78
	89†	2.00
3	89†	2.67

* *Or two 38-mm-thick members connected to transfer the factored shear force.*
† *Or three 38-mm-thick members connected to transfer the factored shear force.*

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11.5.3 Factor for unblocked diaphragms, J_{ud}

The shear strength of unblocked diaphragms sheathed with wood structural panels shall be calculated by multiplying the specified shear strength of a blocked diaphragm with one row of sheathing fasteners spaced at 150 mm along panel edges and 300 mm along intermediate framing members by the factor, J_{ud}, given in Table 11.2.

Table 11.2
Factor, J_{ud}, for unblocked diaphragms
(See Clause 11.5.3.)

Configuration case*	J_{ud}
1	0.89
2, 3, 4, 5, and 6	0.67

** See Figure 11.5 for configurations of unblocked diaphragms.*

Figure 11.5
Configurations of unblocked diaphragms
(See Figure 11.2 and Table 11.2.)

Source: *Table 4.2C of ANSI/AWC SDPWS.*

11.5.4 Factor for unblocked shearwalls, J_{us}

The shear strength of unblocked shearwalls sheathed with wood structural panels shall be calculated by multiplying the specified shear strength of a blocked shearwall with sheathing fasteners spaced at

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150 mm along panel edges and 300 mm along intermediate framing members by the factor, J_{us}, given in Table 11.3. The maximum height, H, of unblocked shearwalls shall be 4.88 m.

Table 11.3
Factor, J_{us}, for unblocked shearwalls
(See Clauses 11.5.4 and 11.7.1.3.)

Fastener spacing at supported edges, mm	Fastener spacing at intermediate studs, mm	Stud spacing, mm			
		300	400	500	600
150	150	1.0	0.8	0.6	0.5
150	300	0.8	0.6	0.5	0.4

Note: *See Figure 11.6 for configurations of unblocked shearwalls.*

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Figure 11.6
Configurations for unblocked shearwalls
(See Table 11.3.)

11.5.5 Hold-down-effect factor for shearwall segments, J_{hd}

11.5.5.1 Shearwall segments with hold-downs

For shearwall segments with hold-downs that are designed to resist all the factored uplift forces due to overturning, J_{hd} shall equal 1.0.

11.5.5.2 Conditions for shearwall segments with J_{hd} < 1.0

The conditions for calculating J_{hd} in Clauses 11.5.5.3 to 11.5.5.5 shall be as follows:

a) the maximum fastener diameter shall be 3.33 mm and the minimum fastener spacing shall be 100 mm;

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b) the maximum specified shear strength, including both sides of a shearwall where applicable, shall be 10.3 kN/m; and
c) the maximum height of the shearwall segment shall be less than 3.6 m.

Δ 11.5.5.3 Shearwall segments without hold-downs

For shearwall segments
a) without hold-downs at either end;
b) meeting the requirements of Clauses 11.4.4.2 and 11.5.5.2; and
c) with P_{ij} not less than zero,

J_{hd} shall be calculated as follows:

$$J_{hd} = \sqrt{1 + 2\frac{P_{ij}}{V_{hd}} + \left(\frac{H_s}{L_s}\right)^2} - \frac{H_s}{L_s} \leq 1.0$$

where

P_{ij} = factored uplift restraint force for storey i at the bottom of the end stud of a shearwall segment j, kN

V_{hd} = factored shear resistance of the shearwall segment calculated with J_{hd} equal to 1.0, kN

H_s = height of shearwall segment, m

L_s = length of shearwall segment, m

Note: *See Figure 11.7.*

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Figure 11.7
Multi-storey shearwall force diagrams
(See Clauses 11.4.4.2 and 11.5.5.3.)

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Legend:

q_i = factored storey *i* dead load to resist overturning (includes only dead weight from current storey sill plate to next storey floor or roof), kN/m

P_{ij} = factored uplift restraint force at story *i* and segment *j*, kN

R_{ij} = factored uplift force at storey *i* and segment *j*, kN

F_{ij} = factored shear load at storey *i* on the shearwall segment *j*, kN

= $F_i \frac{V_{rs}}{\Sigma V_{rs}}$

where

F_i = total applied factored shear load on the shearwall at storey *i*, kN

V_{rs} = factored shear resistance of the shearwall segment calculated in accordance with Clause 11.6.1, kN

ΣV_{rs} = sum of factored shear resistances of all segments in a shearwall, kN

11.5.5.4 Shearwall segments with hold-downs only at the bottom of the segment

For a lower-storey shearwall segment with hold-downs at the bottom of the shearwall segment and without hold-downs at the top, and with *P* less than zero,

$$J_{hd} = \frac{V_{hd} + P}{V_{hd}} \leq 1.0$$

where

V_{hd} = factored shear resistance of the shearwall segment calculated with J_{hd} equal to 1.0, kN

P = factored uplift restraint force at the top of the end stud of a shearwall segment, kN

11.5.5.5 Shearwall segments with hold-downs only on one end

For a shearwall segment with hold-downs only on one end of the segment, the following shall apply:

a) J_{hd} shall be determined in accordance with Clause 11.5.5.1 if designed to resist all uplift forces due to overturning;

b) J_{hd} shall be determined in accordance with Clause 11.5.5.3 if there is no hold-down on the tension end of the segment; and

c) J_{hd} shall be determined in accordance with Clause 11.5.5.4 if there is a hold-down only at the bottom of the tension end of a lower-storey segment.

11.6 Strength and resistance of light-frame shearwalls and diaphragms

11.6.1 General

11.6.1.1 Standard methods

Provisions in Clauses 11.6 to 11.8 apply to light-frame shearwalls and diaphragms constructed with shear panels consisting of wood structural panels, gypsum wallboard, or diagonal lumber sheathing fastened to framing members using nails of diameter 3.76 mm or smaller, unless otherwise stated in these Clauses.

11.6.1.2 Alternative methods

Factored shear resistance of light-frame shearwalls and diaphragms may be derived from full-scale testing, provided that the tests are conducted in accordance with recognized test standards.

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11.6.2 Shear resistance of shearwalls

11.6.2.1 General

The factored shear resistance, V_r, of shearwalls constructed with wood structural panels, gypsum wallboard, or diagonal lumber sheathing in accordance with Clauses 11.3.2, 11.3.3, or 11.3.4, respectively, shall be calculated as follows:

$V_r = \Sigma V_{rs}$

where

V_{rs} = factored shear resistance of each shearwall segment along the length of the shearwall, kN, in accordance with Clauses 11.6.2.2, 11.6.2.3, or 11.6.2.4, respectively.

Δ 11.6.2.2 Shearwall segments with wood structural panels

The factored shear resistance, V_{rs}, of a shearwall segment with wood structural panels shall be taken as the smaller shear resistance value determined in Items a) and b) as follows:

a) shear resistance governed by sheathing-to-framing connection strength resistance:

$V_{rs} = \phi\ v_d\ J_D\ n_s\ J_{us}\ J_s\ J_{hd}\ L_s$

where

ϕ = 0.8

v_d = N_u/s, kN/m

where

N_u = lateral strength resistance of sheathing-to-framing connections along panel edges, per shear plane per fastener, N (see Clause 12.9.3.2)
For single-shear fastening shearwalls (shearwalls with plywood or OSB panels fastened to framing with single shear connection), the embedment strength of the structural panel in Clause 12.9.3.2, f_1, may be calculated using:
$f_1 = 51(1 - 0.1d_F)$, MPa

s = fastener spacing along panel edges, mm (see Clause 11.3.4)

J_D = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.3.1)

n_S = number of shear planes in sheathing-to-framing connections (see Clause 11.3.2.3)

J_{us} = factor for unblocked shearwalls (see Clause 11.5.4)

J_s = fastener spacing adjustment factor (see Clause 11.5.1)

J_{hd} = hold-down-effect factor for shearwall segment (see Clause 11.5.5)

L_s = length of shearwall segment parallel to the direction of load, m

b) shear resistance governed by sheathing panel buckling:

$V_{rs} = \phi\ v_{pb}\ K_D\ K_S\ K_T\ L_s$

where

ϕ = 0.8

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb}\frac{\pi^2 t^2}{3000b}\left(B_{a,0}B_{a,90}{}^3\right)^{\frac{1}{4}}$$

where

K_{pb} = panel buckling factor

$$= 1.7\left(\eta + 1\right)\exp\left(\frac{-\alpha}{0.05\eta + 0.75}\right) + (0.5\eta + 0.8)$$

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where

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

$$\eta = \frac{2B_v}{\sqrt{B_{a,0}B_{a,90}}}$$

where

a = larger dimension of panel, mm

b = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, N/mm (see Tables 9.1 to 9.3)

$B_{a,90}$ = axial stiffness of panel 90° orientation, N/mm (see Tables 9.1 to 9.3)

B_v = shear-through-thickness rigidity, N/mm (see Tables 9.1 to 9.3)

t = panel thickness, mm

K_S = service-condition factor (see Table 9.4)

11.6.2.3 Shearwall segments with gypsum wallboard sheathing

The factored shear resistance, V_{rs}, of a shearwall segment with gypsum wallboard sheathing shall be calculated as follows:

$$V_{rs} = \phi v_d J_{hd} L_s$$

where

ϕ = 0.7

v_d = specified shear strength for shearwall segment sheathed with gypsum wallboard, kN/m (see Table 11.4)

J_{hd} = hold-down-effect factor for shearwall segment (see Clause 11.5.5)

L_s = length of shearwall segment parallel to the direction of load, m

Table 11.4
Specified shear strength, v_d, for gypsum wallboard shearwalls, kN/m
(See Clause 11.6.2.3.)

Minimum nominal panel thickness, mm	Minimum nail and screw penetration in framing, mm	Wall construction	Panels applied directly to framing		
			Nail and screw spacing at panel edges, mm		
			200	150	100
12.5	19	unblocked	1.2	1.4	1.6
12.5	19	blocked	1.4	1.7	2.1
15.9	19	unblocked	1.5	1.7	2.1
15.9	19	blocked	1.7	2.2	2.5

Notes:

1) *The tabulated values are based on dry-service conditions and are applicable to wood framing of all species. Values for unblocked walls are given for 400 mm stud spacing and shall be reduced by 20% for 500 mm stud*

(Continued)

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Table 11.4 (Concluded)

spacing and 40% for 600 mm stud spacing. The values for blocked walls are applicable to stud spacings from 400 to 600 mm.

2) *Gypsum wallboard shall only be considered effective in resisting loads of short-term duration. Gypsum wallboard shall not be used in wet-service conditions.*
3) *The tabulated values apply when gypsum wallboard is applied to framing with nails that meet the requirements of CSA B111, gypsum board application nails-ring thread, and ASTM F1667.*
4) *The tabulated values apply when gypsum wallboard is applied to framing with wallboard screws that meet the requirements of ASTM C1002, Type W.*
5) *Nails and screws shall be spaced at maximum 300 mm on centre along intermediate framing members.*

Δ 11.6.2.4 Shearwall segments with diagonal lumber sheathing

The factored shear resistance, V_{rs}, of a shearwall segment with diagonal lumber sheathing shall be calculated as follows:

$V_{rs} = \phi\, v_d\, J_D\, n_L\, L_s$

where

ϕ = 0.8

$v_d = \frac{2}{3}\frac{N_u}{\sqrt{2}s}$

where

N_u = lateral strength resistance of sheathing-to-framing connections along the boundary frame members of single-layer sheathing or the inner layer of double-layer sheathing, per fastener, N (see Clauses 11.3.4.2, , and 12.9.3)

s = fastener spacing evenly distributed along boundary frame members, but not less than 65 mm (see Clause 11.3.4.2)

J_D = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.3.1)

n_L = number of sheathing layers

= 1 for single-layer lumber sheathing

= 2 for double-layer lumber sheathing

L_s = length of shearwall segment parallel to the direction of load, m

11.6.3 Shear resistance of diaphragms

11.6.3.1 General

The factored shear resistance of diaphragms constructed with wood structural panels in accordance with Clause 11.3.2 shall be calculated in accordance with Clause 11.6.3.2. The factored shear resistance of diaphragms constructed with diagonal lumber sheathing in accordance with Clause 11.3.4 shall be calculated in accordance with Clause 11.6.3.3.

Δ 11.6.3.2 Diaphragms with wood structural panels

The factored shear resistance, V_{rs}, of a diaphragm with wood structural panels shall be taken as the smaller value determined as follows:

a) shear resistance governed by sheathing-to-framing connection strength resistance:

$V_{rs} = \phi\, v_d\, J_D\, J_s\, J_f\, J_{ud}\, L_D$

where

ϕ = 0.8

v_d = N_u/s, kN/m

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where

N_u = lateral strength resistance of sheathing-to-framing connections along panel edges, per fastener, N (see Clause 12.9.3.2 for nails)
For diaphragms with plywood or OSB panels fastened to framing, the embedment strength of the structural panel in Clause 12.9.3.2, f_1, may be calculated using:
$f_1 = 51(1 - 0.1d_F)$, MPa

s = fastener spacing along the edges, mm

J_D = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.3.1)

J_s = fastener spacing factor (see Clause 11.5.1)

J_f = fastener row factor for blocked diaphragms (see Clause 11.5.2)

J_{ud} = factor for unblocked diaphragms (see Clause 11.5.3)

L_D = dimension of diaphragm parallel to the direction of load, m

b) shear resistance governed by sheathing panel buckling:

$V_{rs} = \phi\, v_{pb}\, K_D\, K_s\, K_T\, L_D$

where

ϕ = 0.8

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb}\frac{\pi^2 t^2}{3000b}\left(B_{a,0}B_{a,90}{}^3\right)^{\frac{1}{4}}$$

where

K_{pb} = panel buckling factor

$$= 1.7\left(\eta + 1\right)\exp\left(\frac{-\alpha}{0.05\eta + 0.75}\right) + (0.5\eta + 0.8)$$

where

$$\eta = \frac{2B_v}{\sqrt{B_{a,0}B_{a,90}}}$$

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

where

a = larger dimension of panel, mm

b = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, N/mm (see Tables 9.1 to 9.3)

$B_{a,90}$ = axial stiffness of panel 90° orientation, N/mm (see Tables 9.1 to 9.3)

B_v = shear-through-thickness rigidity, N/mm (see Tables 9.1 to 9.3)

t = panel thickness, mm

K_S = service-condition factor (see Table 9.4)

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Δ 11.6.3.3 Diaphragms with diagonal lumber sheathing

The factored shear resistance, V_{rs}, of a diaphragm with diagonal lumber sheathing shall be calculated as follows:

$V_{rs} = \phi\, v_d\, J_D\, n_L\, L_D$

where

ϕ = 0.8

$v_d = \frac{2}{3}\frac{N_u}{\sqrt{2}s}$

where

N_u = lateral strength resistance of sheathing-to-framing connections along the boundary frame members of single-layer sheathing or the inner layer of double-layer sheathing, per fastener, N (see Clauses 11.3.4.2, 11.3.4.3, and 12.9.3)

s = fastener spacing evenly distributed along boundary frame members, but not less than 65 mm (see Clause 11.4.3.2)

J_D = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.3.1)

n_L = number of sheathing layers

= 1 for single-layer lumber sheathing

= 2 for double-layer lumber sheathing

L_D = dimension of diaphragm parallel to the direction of load, m

11.6.4 Moment resistance of shearwalls and diaphragms

11.6.4.1 General

Except as provided in Clause 11.6.4.2, the factored moment resistance of shearwalls and diaphragms shall be calculated as follows:

$M_r = P_r\, h$

where

P_r = factored axial tension and compression resistance of the elements resisting chord forces (with due allowance being made for joints), N

h = centre-to-centre distance between moment-resisting elements, mm

= centre-to-centre distance between diaphragm chords in the design of diaphragms, mm

= centre-to-centre distance between stud chords in shearwall segments designed with hold-downs at both ends of the shearwall segment, mm

= length of shearwall segment minus 300 mm for shearwall segments designed without hold-downs at both ends of the segment, mm

11.6.4.2 Moment resistance for shearwall segments without hold-downs

For shearwall segments without hold-downs, a moment resistance calculation for the design of the tension chords shall not be required.

11.6.5 Boundary members in shearwalls and diaphragms with single-layer diagonal sheathing

Note: *See the CWC* Commentary on CSA O86.

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11.6.5.1 General

Single-layer diagonal sheathing produces a load component acting perpendicular to the boundary frame members in the plane of the shear panel. Boundary frame members in single-layer diagonally sheathed shearwalls and diaphragms shall be designed to resist the

a) factored bending moment, M_f, caused by the normal load component, in combination with the boundary member axial forces; and
b) factored mutual separation force, P_c, at corner intersections.

11.6.5.2 Shearwalls

M_f and P_c shall be calculated for shearwall segments as follows:

$$M_f = \frac{F_{ij} H_s^2}{8L_s}$$

$$P_c = \frac{F_{ij} H_s}{2L_s}$$

where

F_{ij} = factored shear force applied to the shearwall segment

H_s = height of the shearwall segment

L_s = length of the shearwall segment

11.6.5.3 Diaphragms

M_f and P_c shall be calculated for diaphragms as follows:

$$M_f = \frac{F_d L_D}{8}$$

$$P_c = \frac{F_d}{2}$$

where

F_d = factored diaphragm shear force at the boundary member

L_D = dimension of diaphragm parallel to direction of factored load

11.7 Deflection of light-frame shearwalls and diaphragms

11.7.1 Deflection of shearwalls

11.7.1.1 General

Deflection of single-storey blocked and unblocked shearwall segments may be calculated in accordance with Clauses 11.7.1.2 and 11.7.1.3 respectively.

In the calculation of deflection of multi-storey shearwalls, multi-storey effects shall be considered.

Note: *See Clause A.11.7.1 for additional information on multi-storey effects.*

Δ #### 11.7.1.2 Blocked shearwall segments

The static deflection at the top of the wall, Δ_{sw}, mm, of a blocked shearwall segment with wood structural panels and gypsum wallboard constructed in accordance with Clauses 11.3.2 to 11.3.3 respectively, may be calculated as follows:

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$$\Delta_{sw} = \frac{2vH_s^3}{3EAL_s} + \frac{vH_s}{B_v} + 0.0025H_s e_n + \frac{H_s}{L_s} d_a$$

where

v = maximum shear due to specified loads at the top of the wall, N/mm

H_s = height of shearwall segment, mm

E = modulus of elasticity of vertical boundary framing members, N/mm^2

A = cross-sectional area of the boundary member, mm^2

L_s = length of shearwall segment, mm

B_v = shear-through-thickness rigidity of the sheathing, N/mm (see Tables 9.1 to 9.3 for wood structural panels. For gypsum wallboard, B_v may be taken as 7000 N/mm)

e_n = sheathing-to-framing connection deformation, mm (see Clause A.11.7)

d_a = total vertical elongation of the wall overturning restraint system (including fastener slip, hold-down device elongation, anchor, or rod elongation) at the induced shear load

For shearwall segments without hold-downs, the total vertical elongation, d_a, may be calculated as follows:

$$d_a = 2.5 d_F K_m \left[\frac{\left(vH_s - P_{ij}\right)\frac{s_n}{L_s}}{n_u} \right]^{1.7}$$

Note: *This equation is an approximation.*

where

d_F = diameter of sheathing fastener, mm

K_m = service-creep factor (see Table A.23)

v = maximum shear due to specified load at the top of the wall, N/mm

H_s = height of shearwall segment, mm

P_{ij} = specified uplift restraint force for storey i at the bottom of the end stud of a shearwall segment j, N

s_n = sheathing fastener spacing along panel edge, mm

L_s = length of shearwall segment, mm

n_u = unit lateral strength resistance per shear plane of sheathing-to-framing connection, N (see Clause 12.9.3.1)

11.7.1.3 Unblocked shearwall segments

The deflection of an unblocked shearwall segment with wood structural panels, Δ_{ub}, may be calculated as follows:

$$\Delta_{ub} = \frac{\Delta_{sw}}{J_{us}}$$

where

Δ_{sw} = deflection of a blocked shearwall segment with fasteners spaced at 150 mm on centre around panel edges and 300 mm on centre along intermediate framing members, calculated in accordance with Clause 11.7.1.2

J_{us} = adjustment factor for unblocked shearwalls (see Table 11.3)

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Δ 11.7.2 Deflection of diaphragms

The lateral static deflection at mid-span, Δ_d, mm, of a simply supported blocked diaphragm with wood structural panels constructed in accordance with Clause 11.3.2 may be calculated as follows:

$$\Delta_d = \frac{5vL^3}{96EAL_D} + \frac{vL}{4B_v} + 0.00061Le_n + \frac{\Sigma(\Delta_c x)}{2L_D}$$

where

v = maximum shear due to specified loads in the direction under consideration, N/mm

L = dimension of diaphragm perpendicular to the direction of load, mm

E = modulus of elasticity of the chord, N/mm^2

A = cross-sectional area of the chord, mm^2

L_D = dimension of diaphragm parallel to the direction of load, mm

B_v = shear-through-thickness rigidity of the sheathing, N/mm (see Tables 9.1 to 9.3)

e_n = sheathing-to-framing connection deformation, mm (see Clause A.11.7)

$\Sigma(\Delta_c x)$ = sum of the individual chord-splice slip values, Δ_c, on both sides of the diaphragm, each multiplied by its distance x to the nearest support

11.8 Seismic design considerations for light-frame shearwalls and diaphragms

Δ 11.8.1 General

Seismic design forces for shearwalls and diaphragms shall be determined using the appropriate procedures specified in the *National Building Code of Canada*. The corresponding ductility-related seismic-force-modification factor, R_d, and system overstrength related force modification factor, R_o, for shearwalls are given in Table 11.5. The sheathing-to-framing connections shall be designed to yield in mode c), d), e), or g) shown in Clause 12.9.3.2 for nails to ensure ductility in the shearwall and diaphragm.

Table 11.5
Seismic design factors for shearwalls
(See Clauses 11.4.3.2 and 11.8.1.)

Construction	R_d	R_o
Shearwalls with nailed wood structural panels	3	1.7
Shearwalls: wood structural and gypsum panels in combination	2	1.7

Clauses 11.8.2 to 11.8.6 apply to structures where the Seismic Category of building is SC3 or SC4. Clause 11.8.7 applies to structures where the Seismic Category of building is SC1 or SC2.

Note: *The Seismic Category of buildings is determined in accordance with the* National Building Code of Canada, *Subclause 4.1.8.5 on the basis of $I_E S(0.2)$ and $I_E S(1.0)$ irrespective of the fundamental lateral period of the building T_a.*

11.8.2 Shearwall hold-downs and shear transfer connections

Anchor bolts and interstorey connections resisting seismic shear forces and hold-downs resisting seismic uplift forces of a shearwall or a shearwall segment shall be designed for the forces that are at least 20% greater than the effects of the seismic load.

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11.8.3 Over-capacity of wood light-frame seismic force resisting system (SFRS)

11.8.3.1 Storey over-capacity coefficient, C_i

The over-capacity coefficient, C_i, for each storey, i, for each horizontal direction, shall be greater than or equal to 1.0 and shall be calculated as follows:

$$C_i = \frac{V_{ri}}{V_{fi}}$$

where

V_{ri} = factored resistance of the wood light-frame SFRS in each storey, i

V_{fi} = factored seismic shear load for storey i obtained using $R_d R_o$ coefficients for the wood light-frame SFRS in accordance with the *National Building Code of Canada*

11.8.3.2 Ratio of second storey to first storey over-capacity coefficients

For structures that have three or more storeys, the vertical SFRSs shall be designed so that the ratio of the over-capacity coefficient of the second storey, C_2, and the over-capacity coefficient of the first storey, C_1, is as follows:

$$0.9 < \frac{C_2}{C_1} \leq 1.2$$

11.8.4 Light-frame diaphragms supported on light-frame shearwalls

11.8.4.1 General

Light-frame diaphragms shall be designed to resist seismic forces determined in accordance with Clause 11.8.4.2.

11.8.4.2 Seismic design forces for diaphragms

The seismic design force for a diaphragm at each storey, i, for each horizontal direction, shall be calculated as follows:

$$V_{Di} = C_{Di} F_i$$

where

V_{Di} = seismic design force for a diaphragm at each storey, i

C_{Di} = over-capacity coefficient at storey i for each horizontal direction

= lesser of C_i or 1.2

F_i = factored seismic force at each storey, i, calculated using $R_d R_o$ for the light-frame shearwalls

11.8.5 Light-frame diaphragms in buildings with SFRSs other than light-frame shearwalls

11.8.5.1 General

Clause 11.8.5 applies to buildings that

a) use reinforced concrete, masonry, steel, or wood-based structural systems other than light-frame shearwalls for supporting vertical SFRSs; and

b) have light-frame diaphragms.

Vertical SFRSs shall be designed in accordance with appropriate CSA material standards.

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11.8.5.2 Seismic design forces for diaphragms

11.8.5.2.1 Diaphragms designed to yield

Light-frame diaphragms intended to exhibit ductile behaviour and to yield before the supporting SFRS shall be designed to resist seismic forces determined using the R_d R_o factors for the vertical SFRS. Such seismic design loads, however, shall not be less than loads determined using R_d R_o = 2.0.

11.8.5.2.2 Diaphragms designed not to yield

Light-frame diaphragms intended not to yield before the supporting SFRS shall be designed to resist a seismic force, V_{Di}, calculated as follows:

V_{Di} = 1.2 C_iF_i (for wood SFRS other than shearwalls sheathed with wood structural panels)

V_{Di} = γ_iF_i (for non-wood SFRS)

where

V_{Di} = seismic design force on the diaphragm at storey *i*

C_i = over-capacity coefficient at storey *i* equal to the value of C_i determined in accordance with Clause 11.8.3.1

F_i = factored seismic force at storey *i* calculated using the R_d R_o for the SFRS

γ_i = over-strength coefficient applied at level *i* for the vertical SFRS, determined using principles of capacity-based design in accordance with the applicable CSA material standard

Note: *V_{Di} may be taken as not greater than F_i determined using R_d R_o = 1.3.*

11.8.6 Design of force transfer elements

Diaphragm chords, splice joints in structural members and connections of the diaphragm around openings, and other force-transfer elements shall be designed for seismic loads that are at least 20% greater than the seismic design force on the diaphragm V_{Di}.

Connections and drag struts that are transferring shear forces between the segments of the vertical SFRS and the diaphragm shall be designed for seismic forces that are at least 20% greater than the shear force that is being transferred.

Parts of the diaphragm around wall offsets shall be designed for seismic loads that are at least 20% greater than the seismic design force of the offset SFRS.

Note: *Forces used for the design of force transfer elements may be taken as not greater than the forces determined using R_d R_o = 1.3.*

11.8.7 Structures in low seismic zones

11.8.7.1 General

Clauses 11.8.7.2 and 11.8.7.3 apply to structures where the Seismic Category of building is SC1 or SC2 determined in accordance with the *National Building Code of Canada*, Subclause 4.1.8.5.

11.8.7.2 Seismic design forces for diaphragms

Seismic design forces for diaphragms shall be determined in accordance with Clauses 11.8.4 and 11.8.5, with the over-capacity coefficient, C_i, taken as 1.0.

11.8.7.3 Design of force-transfer elements

Force-transfer elements shall be designed in accordance with Clause 11.8.6.

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11.8.8 Requirements for shearwalls using gypsum wallboard

For seismic design, gypsum wallboard shall be used in combination with wood structural panels. The shear force resisted by gypsum wallboard shall be equal to or less than the percentage of storey shear forces specified in Table 11.6. If the maximum percentages are exceeded, R_d shall be equal to 1.0 and R_o shall be equal to 1.0. Gypsum wallboard shall not be considered to provide lateral resistance when the interstorey drift ratio exceeds 1%. For buildings that are higher than 4 storeys, the contribution of the gypsum wallboard shall not be accounted for in seismic resistance.

Table 11.6
Maximum percentage of total seismic shear forces resisted by gypsum wallboard in a storey
(See Clause 11.8.8.)

	Percentage of shear forces			
Storey	**4-storey building**	**3-storey building**	**2-storey building**	**1-storey building**
4th	80	—	—	—
3rd	60	80	—	—
2nd	40	60	80	—
1st	40	40	60	80

11.8.9 Load-bearing walls constructed with gypsum wallboard only

When the interstorey drift due to seismic loading exceeds 1% on any floor, all load-bearing stud walls within the storey which rely on the gypsum wallboard sheathing to prevent weak axis buckling shall be designed to resist the factored axial load combination with seismic load in accordance with the applicable building code, and assuming the gypsum wallboard is providing no lateral support. Alternatively a secondary blocking system shall be used to provide lateral support to the studs.

11.9 Design of CLT shearwalls and diaphragms

11.9.1 Scope

11.9.1.1 Standard systems

Clause 11.9 shall apply to platform-type CLT construction where

a) the lateral-load-resisting system consists of CLT shearwalls; and
b) CLT walls are placed on
   i) a platform of CLT floor panels; or
   ii) a concrete podium or concrete foundation at the lowest level.

11.9.1.2 Alternative systems

Alternative systems shall be designed in accordance with Clause 4.3.2 of this Standard and the *National Building Code of Canada*, Article 4.1.8.

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11.9.2 Strength and resistance

11.9.2.1 Lateral resistance of CLT shearwalls

The lateral resistance of CLT shearwalls shall be governed by the resistance of connections between the shearwalls and the foundation or floor, and connections between the individual panels, calculated using methods of mechanics.

11.9.2.2 Lateral resistance of CLT diaphragms

The lateral resistance of CLT diaphragms shall be governed by the resistance of connections between the diaphragms and the supporting structure and connections between the individual panels, calculated using methods of mechanics.

11.9.2.3 Shearwall resistance to overturning

Where the factored dead loads do not prevent shearwall overturning, hold-downs shall be used to transfer the forces through a continuous load path.

11.9.2.4 Compressive resistance at ends of CLT shearwalls

The compressive resistance at the ends of shearwalls shall be calculated in accordance with Clauses 8.4.5 and 8.4.6 and shall be greater than or equal to the overturning forces, calculated using methods of mechanics.

11.9.3 Seismic design considerations for CLT structures

11.9.3.1 General

Except as specified in Clause 11.9.3.2.2, a SFRS consisting of moderately ductile CLT shearwalls, platform-type construction, as defined in the *National Building Code*, shall be designed to resist forces calculated using R_d = 2.0 and R_o = 1.5, where energy is dissipated in rocking motion of shearwall segments through designated connections in accordance with Clause 11.9.3.3.1 following the capacity design principles described in Clause 11.9.3.4. Shearwall segments shall have an aspect ratio as specified in Clause 11.9.3.5.2.

11.9.3.2 Restrictions and limitations

11.9.3.2.1 Irregularity restrictions

CLT platform-type SFRS constructed with more than 4 storeys, and where the Seismic Category of building is SC3 or SC4, shall be designed in accordance with the provisions of the *National Building Code*, Article 4.1.8.7 and shall not contain irregularities prohibited in the *National Building Code*, Article 4.1.8.10.

Note: *The Seismic Category of buildings is determined in accordance with the* National Building Code of Canada*, Subclause 4.1.8.5 on the basis of $I_E S(0.2)$ and $I_E S(1.0)$ irrespective of the fundamental lateral period of the building T_a.*

11.9.3.2.2 Shear segments with aspect ratios less than 2:1

SFRS consisting of CLT shearwall segments with aspect ratios less than 2:1 as specified in Clause 11.9.3.5.2 and that are considered part of the SFRS shall be considered limited ductility CLT shearwalls, platform type construction, as defined in the *National Building Code*, and shall be designed for forces calculated using $R_d R_o \leq 1.3$, provided energy is dissipated through designated connections in accordance with Clause 11.9.3.3.1 following the capacity design principles described in Clause 11.9.3.4.

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11.9.3.2.3 Height limitations

11.9.3.2.3.1 Height limit for Seismic Categories SC1, SC2, and SC3

The height of the CLT SFRS shall not exceed 30 m where the Seismic Category determined in accordance with the *National Building Code* is SC1, SC2, or SC3.

11.9.3.2.3.2 Height limit for Seismic Category SC4

The height of the CLT SLRS shall not exceed 20 m where the Seismic Category determined in accordance with the *National Building Code* is SC4.

11.9.3.3 CLT shearwall connections

11.9.3.3.1 Energy-dissipative connections

Energy-dissipative connections shall satisfy all the following requirements:

a) Connections shall be designed so that a yielding mode governs the resistance.
b) Connections shall be at least moderately ductile in the directions associated with the rocking rigid body motions of CLT panels.
c) Connections shall possess sufficient deformation capacity under the force and displacement demands that are induced in them to allow for the CLT panels to develop rocking motion.

11.9.3.3.2 Non-dissipative connections

Non-dissipative connections shall be capacity-protected by designing them to remain elastic under the force and displacement demands that are induced in them when the energy-dissipative connections reach the 95th percentile of their ultimate resistance or target displacement in accordance with engineering principles of equilibrium and displacement compatibility. Alternatively, the seismic design force may be determined using $R_d R_o$ = 1.3.

11.9.3.3.3 Overturning restraint devices

11.9.3.3.3.1 General

Where shearwall overturning restraint devices are required, discrete hold-downs or continuous steel rods shall be designed to transfer the forces in accordance with Clause 11.9.3.3.3.2 or 11.9.3.3.3.3, respectively.

11.9.3.3.3.2 Discrete hold-downs

The factored resistance of discrete hold-downs shall be at least 20% greater than the forces developed in them when the connections between CLT segments reach their nominal resistance.

11.9.3.3.3.3 Continuous steel rods

Continuous steel rods used as hold-downs shall be anchored at each floor level and designed in accordance with CSA S16 to remain elastic while allowing for the expected uplift due to overturning of the shearwalls.

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11.9.3.4 Capacity design principles

11.9.3.4.1 Energy-dissipative connections

Energy-dissipative connections shall be designed in accordance with the requirements in Clause 11.9.3.3.1 to ensure that all principle inelastic deformations and principle energy dissipation occurs in

a) connections between vertical joints of adjacent shearwall segments; and
b) shear connections of shearwalls to foundations, and shearwalls to floors beneath, in uplift only.

11.9.3.4.2 CLT panels

CLT panels that are part of the SFRS shall be capacity-protected by designing them for seismic forces that are developed when energy-dissipative connections in shearwalls reach the 95th percentile of their ultimate resistance. Alternatively, the seismic design force may be determined using $R_d R_o \leq 1.3$.

Note: *The in-plane shear resistance of the CLT panels is provided by the product manufacturer.*

11.9.3.5 CLT shearwalls

11.9.3.5.1 Minimum shearwall thickness

CLT shearwalls shall have a minimum thickness of 87 mm.

11.9.3.5.2 Permitted aspect ratio of shearwall segments

Except as permitted in Clause 11.9.3.2.2, all CLT shearwall segments considered part of the SFRS shall have an aspect ratio of not less than 2:1 and not greater than 4:1.

11.9.3.5.3 Panel aspect ratios less than 2:1

Except as permitted in Clause 11.9.3.2.2, CLT panels with aspect ratios less than 2:1 may be divided and re-joined with energy-dissipative connections in accordance with Clause 11.9.3.3.1 to form shearwall segments in accordance with Clause 11.9.3.5.2.

11.9.3.5.4 Openings in shearwalls

Parts of shearwalls interrupted by openings shall not be considered part of the SFRS.

11.9.3.6 Deflections of CLT shearwalls

11.9.3.6.1 General

Deflections of shearwalls shall be determined using established methods of mechanics. Calculations shall account for the main sources of shearwall deformations, such as panel rocking, connection stiffness, and deformation of supports.

Note: *Deflections determined in this Clause are used to confirm displacement compatibility of the energy-dissipative and non-dissipative connections in Clause 11.9.3.3.*

11.9.3.6.2 Limitation on flexural deformations

CLT shearwalls shall be designed so that the lateral deflection due to flexural deformations shall not exceed 30% of the total lateral deflection at the top of the building.

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11.9.3.7 CLT diaphragms

11.9.3.7.1 CLT panels and connections

The CLT panels in the diaphragms shall be capacity protected in accordance with Clause 11.9.3.4.2. The shear connections that connect the diaphragms to the walls beneath and the connections between adjacent diaphragm panels shall be non-dissipative and designed in accordance with Clause 11.9.3.3.2.

11.9.3.7.2 Compressive resistance perpendicular to the face of the CLT panels (bearing)

The compressive resistance perpendicular to the face of the diaphragm at locations where vertical overturning forces are transferred from the walls shall be calculated in accordance with Clause 8.4.7 and shall be greater than or equal to the overturning forces calculated using methods of mechanics.

Note: *See the CWC* Commentary on CSA O86 *for further information.*

11.9.3.7.3 Force-transfer elements

Diaphragm chords, struts, and other force-transfer elements, including those around openings, shall be capacity protected. They shall remain elastic under the force and displacement demands that are induced in them when the energy-dissipative connections that are connecting them to the seismic-force-resisting system reach the 95th percentile of their ultimate resistance or the design target displacement.

Note: *The seismic design force is permitted to be taken as not greater than the force determined using* $R_d R_o = 1.3$.

11.9.4 Members not considered part of the SFRS

11.9.4.1 General

Clause 11.9.4 applies to all members of the structure not considered part of the SFRS unless the building is located where the Seismic Category is SC1 or SC2.

11.9.4.2 Non-structural elements

Elements not required to resist either gravity or lateral loading shall be considered non-structural and are not required to be detailed for the effect of seismic loads provided that

a) the effects of non-structural elements are accounted for in the design of structural elements if the non-structural elements increase the seismically induced deformation of, or the forces applied to, a structural element; and
b) the non-structural elements are attached to the building in accordance with the *National Building Code of Canada*, Article 4.1.8.18.

11.9.4.3 Gravity-load-resisting systems

All structural elements that are not considered part of the SFRS shall have the resistance and displacement capability to support their gravity loads while undergoing seismically-induced deformations calculated in accordance with the *National Building Code*, Article 4.1.8.13.

Note: *Gravity-load-resisting members not part of the seismic-force-resisting system may be designed to behave linearly or non-linearly.*

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12 Connections

12.1 Scope

Clause 12 specifies criteria for the engineering design and appraisal of connections using split-ring and shear-plate connectors, bolts, dowels, drift pins, lag screws, wood screws, timber rivets, truss plates, nails, spikes, and joist hangers.

Note: *The lateral resistance values for bolts, dowels, drift pins, lag screws, nails, spikes, and wood screws are based on the relative density of the wood material. Relative density values are given in Table A.11.*

12.2 General requirements

12.2.1 All connections

12.2.1.1 General

The requirements in Clause 12 related to fasteners in the side grain of the members shall apply equally to those in the panel face of CLT, unless specified otherwise. The angle of loading shall be measured relative to the grain in wood members or in the outer laminations on the panel face of CLT. Special provisions for fasteners installed in the end grain and in the panel edge (narrow face) of CLT shall be applied.

Note: *Connection details where shrinkage of the wood can lead to excessive tension perpendicular to grain stress should be avoided.*

12.2.1.2 Connection assembly

Connections shall be assembled so that the surfaces are brought into close contact.

12.2.1.3 Minimum end distance

The tables in Clause 12 are predicated on the requirement that the projecting end of a member shall not be trimmed or otherwise altered in a manner that reduces the specified minimum end distance.

12.2.1.4 Conditions conducive to corrosion

Under severe conditions conducive to corrosion, connection design shall provide protection against corrosion.

12.2.1.5 Connections made using hardwoods

Connections made using hard maple, soft maple, elm, beech, black oak, white oak, or birch may be assigned the same resistances as the Douglas Fir-Larch species combination.

12.2.1.6 Shear depth at connections

Where a fastener or group of fasteners exerts a shear force on a member, the factored shear resistance of the member, calculated in accordance with Clauses 6.5.4 for sawn lumber and Clause 7.5.7 for glulam, shall be based on the shear depth d_e shown in Figure 12.1, instead of the full depth d. The shear depth d_e is the distance, measured perpendicular to the axis of the member, from the extremity of the fastener or group of fasteners to the loaded edge of the member.

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Figure 12.1
Shear depth
(See Clause 12.2.1.6.)

12.2.1.7 Modification factors

12.2.1.7.1 Load-duration factor, K_D

The load-duration factor, K_D, for connections shall be as specified in Clause 5.3.2.

12.2.1.7.2 Service-condition factors, K_{SF}

The service-condition factors for connections, K_{SF}, shall be as specified in Table 12.1.

12.2.1.7.3 Treatment factor, K_T

For connections containing members treated with fire-retardant or other strength-reducing chemicals, the strength resistance of connections shall be based on the documented results of tests that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 4.3.2.

Note: *The effects of fire-retardant treatments can vary depending on manufacturing materials and processes.*

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Table 12.1
Service-condition factors, K_{SF}, for connections
(See Clauses 12.2.1.7.2, 12.3.2, and A.12.6.5.4.)

Service conditions	Moisture content of wood when connection is fabricated: Dry (≤ 19%) — Dry	Dry (≤ 19%) — Wet	Green (> 19%) — Dry	Green (> 19%) — Wet	Connection detail	Angle of load to grain
Timber rivets						
Lateral loads	1.00	0.80	0.90	0.80	All	All
Withdrawal loads	1.00	*	0.60	*	—	—
Split-ring and shear-plate connectors, and truss plates	1.00	0.67	0.80	0.67	All	All
Bolts, dowels, drift pins, and lag screws†	1.00	0.67	1.00	0.67	A	All
	1.00	0.67	1.00	0.67	B	0°
	1.00	0.67	0.40	0.27	B	90°
	1.00	0.67	0.40	0.27	C	All
Nails, spikes, and wood screws						
Lateral loads	1.00	0.67	0.80	0.67	All	All
Withdrawal loads	1.00	0.67	0.40	0.40	All	90°

Legend:

A = a single fastener or single row parallel to grain with steel splice plates

B = a single row parallel to grain with wood splice plates, two rows parallel to grain not more than 127 mm apart with a common wood splice plate, or multiple rows with separate wood or steel splice plates for each row

C = all other arrangements

* *No data available for this condition.*

† *In calculations of the lateral resistance of bolts and dowels, K_{SF} shall be applied to yielding (see Clause 12.4.4.3) and perpendicular-to-grain splitting (see Clause 12.4.4.7) failure modes. For failure modes involving shear and tension parallel to grain, the corresponding service-condition factors, K_{Sv} and K_{St}, shall be applied (see Clauses 12.4.4.4 to 12.4.4.6).*

Note: *See the CWC* Commentary on CSA O86 *for further information on connection details.*

12.2.2 Split-ring and shear-plate connectors, bolts, dowels, and lag screws

Note: *Clause 12.2.2 provides general requirements for connections with split-ring and shear-plate connectors, bolts, dowels, and lag screws. Specific requirements are provided in Clause 12.3 for connections with split-ring and shear-plate connectors, in Clause 12.4 for connections with bolts and dowels, and in Clause 12.6 for connections with lag screws.*

12.2.2.1 Inspection and tightening

Structures that have been assembled with unseasoned or partially seasoned wood members shall be inspected regularly at intervals not exceeding six months until it becomes apparent that further wood

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shrinkage will not be appreciable. At each inspection, the fasteners shall be tightened to bring the faces of the connected members into close contact without deformation.

12.2.2.2 Grooves, daps, and holes

12.2.2.2.1 Grooves, daps, and holes in wood members

Grooves, daps, and holes in wood members shall be fabricated and oriented accurately in the contacting faces. Bolt holes in wood members shall be accurately aligned and drilled not less than 1.0 mm and not more than 2.0 mm larger than the bolt diameter. Pre-drilled holes in connections with dowels shall have a diameter not greater than the dowel diameter. Lag-screw holes shall be drilled in accordance with Clause 12.6.2.5.

12.2.2.2.2 Holes in steel plates

Holes in steel plates shall be accurately placed to line up with holes in the adjoining wood and shall not be more than 2 mm larger than the diameter of the fastener.

12.2.2.3 Group of fasteners

12.2.2.3.1 General

A group of fasteners consists of one or more rows of fasteners of the same type and size arranged symmetrically with respect to the axis of the load (see Clauses 12.3.3, 12.4.3, and 12.6.2).

12.2.2.3.2 Row of fasteners

A row of fasteners consists of one or more fasteners of the same type and size aligned with the direction of the load.

12.2.2.3.3 Multiple rows of fasteners

When fasteners in adjacent rows are staggered and the spacing between adjacent rows is less than one-quarter the spacing between the closest fasteners in adjacent rows measured parallel to the rows, the adjacent rows shall be considered as one row to determine the resistance of the group. For a group of fasteners having an even number of rows, this principle shall apply to each pair. For a group of fasteners having an odd number of rows, the more conservative interpretation shall apply.

12.2.2.3.4 Group action factor, J_G

The group action factor, J_G, for split-ring, shear-plate, and lag-screw connections shall be as specified in Tables 12.2 and 12.3.

12.2.2.4 Washers

12.2.2.4.1 General

A standard cut washer or its equivalent (see Table 12.4), or a metal strap of the same thickness as the washer, shall be placed between the wood and the fastener head and between the wood and the nut.

12.2.2.4.2 Steel plates

When a bolt or lag-screw head or nut bears directly on a steel plate, washers may be omitted.

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12.2.2.4.3 Washers for axially loaded fasteners

Bolts or lag screws in axial tension or with a calculated tension component shall be provided with steel plate washers, standard ogee washers, or malleable iron washers under heads and nuts. The area of these washers shall be such that the bearing stress on the wood under the washer does not exceed the factored resistance in compression perpendicular to grain. If steel washers are used, the thickness shall not be less than one-tenth the diameter or one-tenth the length of the longer side of the washer.

12.2.2.5 Net section

12.2.2.5.1 General

The net section of wood members in connections made using bolts, dowels, lag screws, and split-ring and shear-plate connectors shall be checked in accordance with Clause 5.3.8.

12.2.2.5.2 Staggered bolts, dowels, and lag screws

For a connection under parallel-to-grain loading, staggered adjacent bolts, dowels, or lag screws shall be considered to be placed at the critical section unless their spacing centre-to-centre parallel to grain is more than $8d_F$.

12.2.2.5.3 Split-ring and shear-plate connectors

For connections using timber split-ring and shear-plate connectors, the area deducted from the gross section shall include the projected area of that portion of the connectors within the member and that portion of the bolt hole not within the connector projected area, located at the critical plane. Where connectors are staggered, adjacent connectors shall be considered as occurring in the same critical transverse plane unless their spacing parallel to grain is more than two connector diameters.

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Table 12.2
Group action factor, J_G, for split-ring, shear-plate, and lag-screw connections with wood side plates
(See Clauses 12.2.2.3.4, 12.3.6, 12.6.5.1.1, and A.12.6.5.4.)

Area ratio*	The lesser of A_m† or A_s‡	No. of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
0.5	< 8000	1.00	0.92	0.84	0.76	0.68	0.61	0.55	0.49	0.43	0.38	0.34
	8001–12 000	1.00	0.95	0.88	0.82	0.75	0.68	0.62	0.57	0.52	0.48	0.43
	12 001–18 000	1.00	0.97	0.93	0.88	0.82	0.77	0.71	0.67	0.63	0.59	0.55
	18 001–26 000	1.00	0.98	0.96	0.92	0.87	0.83	0.79	0.75	0.71	0.69	0.66
	26 001–42 000	1.00	1.00	0.97	0.94	0.90	0.86	0.83	0.79	0.76	0.74	0.72
	> 42 000	1.00	1.00	0.98	0.95	0.91	0.88	0.85	0.82	0.80	0.78	0.76
1.0	< 8000	1.00	0.97	0.92	0.85	0.78	0.71	0.65	0.59	0.54	0.49	0.44
	8001–12 000	1.00	0.98	0.94	0.89	0.84	0.78	0.72	0.66	0.61	0.56	0.51
	12 001–18 000	1.00	1.00	0.97	0.93	0.89	0.85	0.80	0.76	0.72	0.68	0.64
	18 001–26 000	1.00	1.00	0.99	0.96	0.92	0.89	0.85	0.83	0.80	0.78	0.75
	26 001–42 000	1.00	1.00	1.00	0.97	0.94	0.91	0.88	0.85	0.84	0.82	0.80
	> 42 000	1.00	1.00	1.00	0.99	0.96	0.93	0.91	0.88	0.87	0.86	0.85

* *Area ratio = the lesser of A_m/A_s or A_s/A_m*
† *A_m = gross cross-sectional area of main member, mm²*
‡ *A_s = sum of gross cross-sectional areas of side members, mm²*
Note: *For area ratios between 0.5 and 1.0, interpolate between tabulated values. For area ratios less than 0.5, extrapolate from tabulated values.*

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Table 12.3
Group action factor, J_G, for split-ring, shear-plate, and lag-screw connections with steel side plates
(See Clauses 12.2.2.3.4, 12.3.6, 12.6.5.1.1, and A.12.6.5.4.)

Area ratio*	A_m	No. of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
2–12	16 000–26 000	1.00	0.94	0.87	0.80	0.73	0.67	0.61	0.56	0.51	0.46	0.42
	26 001–42 000	1.00	0.96	0.92	0.87	0.81	0.75	0.70	0.66	0.62	0.58	0.55
	42 001–76 000	1.00	0.98	0.95	0.91	0.87	0.82	0.78	0.75	0.72	0.69	0.66
	76 001–130 000	1.00	0.99	0.97	0.95	0.92	0.89	0.86	0.84	0.81	0.79	0.78
12–18	26 001–42 000	1.00	0.98	0.94	0.90	0.85	0.80	0.75	0.70	0.67	0.62	0.58
	42 001–76 000	1.00	0.99	0.96	0.93	0.90	0.86	0.82	0.79	0.75	0.72	0.69
	76 001–130 000	1.00	1.00	0.98	0.95	0.94	0.92	0.89	0.86	0.83	0.80	0.78
	> 130 000	1.00	1.00	1.00	0.98	0.97	0.95	0.93	0.91	0.90	0.88	0.87
18–24	26 001–42 000	1.00	1.00	0.96	0.93	0.89	0.84	0.79	0.74	0.69	0.64	0.59
	42 001–76 000	1.00	1.00	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.76	0.73
	76 001–130 000	1.00	1.00	0.99	0.98	0.96	0.94	0.92	0.90	0.88	0.86	0.85
	> 130 000	1.00	1.00	1.00	1.00	0.98	0.96	0.95	0.93	0.92	0.92	0.91
24–30	26 001–42 000	1.00	0.98	0.94	0.90	0.85	0.80	0.74	0.69	0.65	0.61	0.58
	42 001–76 000	1.00	0.99	0.97	0.93	0.90	0.86	0.82	0.79	0.76	0.73	0.71
	76 001–130 000	1.00	1.00	0.98	0.96	0.94	0.92	0.89	0.87	0.85	0.83	0.81
	> 130 000	1.00	1.00	0.99	0.98	0.97	0.95	0.93	0.92	0.90	0.89	0.89
30–35	26 001–42 000	1.00	0.96	0.92	0.86	0.80	0.74	0.68	0.64	0.60	0.57	0.55
	42 001–76 000	1.00	0.98	0.95	0.90	0.86	0.81	0.76	0.72	0.68	0.65	0.62
	76 001–130 000	1.00	0.99	0.97	0.95	0.92	0.88	0.85	0.82	0.80	0.78	0.77
	> 130 000	1.00	1.00	0.98	0.97	0.95	0.93	0.90	0.89	0.87	0.86	0.85
35–42	26 001–42 000	1.00	0.95	0.89	0.82	0.75	0.69	0.63	0.58	0.53	0.49	0.46
	42 001–76 000	1.00	0.97	0.93	0.88	0.82	0.77	0.71	0.67	0.63	0.59	0.56

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Table 12.3 (Concluded)

Area ratio*	A_m	No. of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
	76 001–130 000	1.00	0.98	0.96	0.93	0.89	0.85	0.81	0.78	0.76	0.73	0.71
	> 130 000	1.00	0.99	0.98	0.96	0.93	0.90	0.87	0.84	0.82	0.80	0.78

* *Area ratio* = A_m/A_s

where

A_m = *gross cross-sectional area of main member, mm²*

A_s = *sum of gross cross-sectional area of steel side plates, mm²*

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Table 12.4
Minimum washer sizes for connections with bolts, lag screws, and split-ring and shear-plate connectors

(See Clause 12.2.2.4.1.)

Washer type	Use	Bolts or lag screws: Diameter of fastener, d_F, mm	Bolts or lag screws: Outside dimension, d_o, mm	Bolts or lag screws: Thickness, t, mm	Split-ring and shear-plate connectors: 2-1/2 in split ring with 1/2 in bolt: t, mm	2-1/2 in split ring with 1/2 in bolt: d_o, mm	4 in split ring with 3/4 in bolt: t, mm	4 in split ring with 3/4 in bolt: d_o, mm	2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt*: t, mm	2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt*: d_o, mm
Standard cut (steel)	For bolts and lag screws only; too thin to resist any tensile loads	12.7	35	3	Cut washers shall not be used with connectors	—	—	—	—	—
		15.9	45	4	—	—	—	—	—	—
		19.1	50	4	—	—	—	—	—	—
		22.2	60	4	—	—	—	—	—	—
		25.4	65	4	—	—	—	—	—	—
Square plate (steel)	For connector or tensile load	12.7	65	5	3.2	50	4.8	75	6.4	75
		15.9	70	6	—	—	—	—	—	—
		19.1	75	6	—	—	—	—	—	—
		22.2	85	9	—	—	—	—	—	—
		25.4	90	8	—	—	—	—	—	—
Round plate (steel)	For any use, unless tensile loading develops enough stress to crush wood	12.7	65	5	3.2	50	4.8	75	6.4	75
		19.1	75	6	—	—	—	—	—	—
		22.2	85	8	—	—	—	—	—	—
Ogee (cast iron)	Thicker and wider than normal or malleable iron washers (for increased stiffness	12.7	65	13	3.2	55	4.8	75	6.4	75
		15.9	75	16	—	—	—	—	—	—
		19.1	90	19	—	—	—	—	—	—

(Continued)

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Table 12.4 (Concluded)

Washer type	Use	Bolts or lag screws: Diameter of fastener, d_F, mm	Bolts or lag screws: Outside dimension, d_o, mm	Bolts or lag screws: Thickness, t, mm	Split-ring and shear-plate connectors: 2-1/2 in split ring with 1/2 in bolt: t, mm	2-1/2 in split ring with 1/2 in bolt: d_o, mm	4 in split ring with 3/4 in bolt: t, mm	4 in split ring with 3/4 in bolt: d_o, mm	2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt*: t, mm	2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt*: d_o, mm
	and bearing strength)	22.2	100	22	—	—	—	—	—	—
		25.4	100	25	—	—	—	—	—	—
Malleable iron	Wider than normal washers (for added bearing strength)	12.7	65	6	3.2	55	4.8	75	6.4	75
		15.9	70	8	—	—	—	—	—	—
		19.1	75	11	—	—	—	—	—	—
		22.2	90	11	—	—	—	—	—	—
		25.4	100	13	—	—	—	—	—	—

* *For 4 in shear plates used with 7/8 in bolts, d_o is 90 mm.*

Note: *Square or round plate bevelled washers can be necessary when bolts project at an angle to the wood.*

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12.3 Split-ring and shear-plate connectors

12.3.1 General

12.3.1.1 Connector unit

For specifying connection resistance in this Standard, a connector unit shall consist of one of the following, in any connection of any number of members:

a) one split ring with a bolt or a lag screw;
b) one shear plate with a bolt or a lag screw, used with a steel strap or plate in a wood-to-metal connection; or
c) two shear plates used back-to-back in the contact faces of a wood-to-wood connection with a bolt or a lag screw.

12.3.1.2 Split-ring connectors

The tabulated resistances and design methods for split-ring connectors specified in Clause 12.3 are for split rings that have dimensions in accordance with Table 12.5A and are manufactured from hot-rolled carbon steel, SAE 1010, meeting the requirements of the *SAE Handbook*. Each ring shall form a closed true circle with the principal axis of the cross-section of the ring metal parallel to the geometric axis of the ring. The ring shall be beveled from the central portion toward the edges to a thickness less than that at midsection so that it will fit snugly in a precut groove; alternatively, another means that achieves equivalent performance may be used. The ring shall be cut through in one place in its circumference to form a tongue and slot.

12.3.1.3 Shear-plate connectors

The tabulated resistances and design methods are for shear-plate connectors specified in Clause 12.3 are for shear plates that have dimensions in accordance with Table 12.5B and meet the requirements of Item a) or b):

a) Pressed steel type: pressed steel shear plates manufactured from hot-rolled carbon steel, SAE 1010, meeting the requirements of the *SAE Handbook*. Each plate shall be a true circle with a flange around the edge, extending at right angles to the face of the plate from one face only. The plate portion shall have a central hole and two small perforations on diametrically opposite sides of the hole, each midway from the centre and circumference.
b) Malleable iron type: malleable iron shear plates manufactured in accordance with ASTM A47, Grade 32510 (or ASTM A47M, Grade 22010). Each casting shall consist of a perforated round plate with a flange extending at right angles to the face of the plate and projecting from one face only. The plate portion shall have a central bolt hole, reamed to size where required, with an integral hub concentric to the bolt hole and extending from the same face as the flange.

Table 12.5A
Split ring dimensions, mm
(See Clauses 12.3.1.2 and 12.3.1.3.)

Dimensions	2-1/2 in	4 in
Inside diameter at centre when closed	63.5	101.6
Thickness of steel at centre	4.1	4.9
Depth of steel	19.0	25.4

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Table 12.5B
Shear plate dimensions, mm
(See Clauses 12.3.1.2 and 12.3.1.3.)

Dimensions	2-5/8 in	4 in	
		3/4 in bolt	7/8 in bolt
Diameter of plate	66.5	102.1	102.1
Diameter of bolt hole	20.6	20.6	23.9
Thickness of plate	4.3	5.1	5.1
Depth of flange	10.7	15.7	15.7

12.3.1.4 Connector grooves

Connector grooves shall have the dimensions specified in Tables 12.6A and 12.6B for split rings and shear plates respectively.

Table 12.6A
Groove dimension for split rings, mm
(See Clause 12.3.1.4.)

Dimensions	2-1/2 in split ring	4 in split ring
Bolt hole diameter	14.3	20.6
Inside diameter	65.0	103.6
Width	4.6	5.3
Depth	9.8	13.1

Table 12.6B
Groove dimension for shear plates, mm
(See Clause 12.3.1.4.)

Dimensions	2-5/8 in shear plate	4 in shear plate	
		3/4 in bolt	7/8 in bolt
	A = 66.8	102.4	102.4
	B = not applicable	39.4	39.4
	C = 20.6	20.6	23.8
	D = not applicable	24.6	24.6
	E = 4.5	5.3	5.3
	F = 11.4	16.3	16.3
	G = 6.4	5.6	5.6

(Continued)

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Table 12.6B (Concluded)

Dimensions	2-5/8 in shear plate	4 in shear plate	
		3/4 in bolt	7/8 in bolt
	H = not applicable	12.7	12.7
	I = 57.2	88.6	88.6

12.3.2 Service-condition factors, K_{SF}

The service-condition factors, K_{SF}, specified in Table 12.1 are based on the moisture content within a depth of 20 mm from the connected surface.

12.3.3 Configuration factors, J_C

Connectors installed at any edge distance, end distance, or spacing less than the minimum for which a tabulated value appears in the appropriate columns of Tables 12.7 to 12.9 shall not be considered to provide resistance. Configuration factors, J_C, for edge distance, end distance, and spacing shall be separately determined and the lowest factor so determined for any one connector shall be applied to all connectors resisting a common force in a connection (see Figures 12.2 and 12.3).

Table 12.7
Factor J_C for connector edge distance
(See Clauses 12.3.3 and 12.3.6.)

Edge distance, mm	2-1/2 in split ring or 2-5/8 in shear plate			4 in split ring or shear plate		
	θ = 15°	θ = 30°	θ = 45°–90°	θ = 15°	θ = 30°	θ = 45°–90°
45	0.94	0.88	0.83	—	—	—
50	0.97	0.91	0.87	—	—	—
55	1.00	0.94	0.90	—	—	—
60	1.00	0.98	0.93	—	—	—
65	1.00	1.00	0.97	—	—	—
70	1.00	1.00	1.00	0.93	0.88	0.83
75	1.00	1.00	1.00	0.97	0.91	0.86
80	1.00	1.00	1.00	1.00	0.94	0.89
85	1.00	1.00	1.00	1.00	0.97	0.93
90	1.00	1.00	1.00	1.00	1.00	0.96

(Continued)

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Table 12.7 (Concluded)

Edge distance, mm	2-1/2 in split ring or 2-5/8 in shear plate			4 in split ring or shear plate		
	θ = 15°	θ = 30°	θ = 45°–90°	θ = 15°	θ = 30°	θ = 45°–90°
95	1.00	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	1.00	1.00	1.00

Notes:

1) *At an angle of load to grain of θ = 0°, the minimum edge distance for the connector size gives a value of J_C = 1.00. For intermediate values of θ, linear interpolation may be used.*
2) *The values of J_C apply to loaded edge distance only. The minimum edge distance for loaded or unloaded edge is 40 mm for 2-1/2 in split rings and 2-5/8 in shear plates, and 65 mm for 4 in split rings and shear plates.*

Table 12.8
Factor J_C for connector end distance

(See Clauses 12.3.3 and 12.3.6.)

End distance, mm		Tension	
For members ≥ 130 mm thick	For members < 130 mm thick	2-1/2 in split ring or 2-5/8 in shear plate	4 in split ring or shear plate
		θ = 0°–90°	θ = 0°–90°
70	105	0.62	—
75	115	0.65	—
80	120	0.68	—
85	130	0.70	—
90	135	0.73	0.63
95	145	0.76	0.65
100	150	0.78	0.67
105	160	0.81	0.69
110	165	0.84	0.71
115	175	0.86	0.73
120	180	0.89	0.75
125	190	0.92	0.77
130	195	0.94	0.79
135	205	0.97	0.82
140	210	1.00	0.84

(Continued)

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Table 12.8 (Concluded)

End distance, mm		Tension	
		2-1/2 in split ring or 2-5/8 in shear plate	4 in split ring or shear plate
For members ≥ 130 mm thick	For members < 130 mm thick	θ = 0°–90°	θ = 0°–90°
145	220	1.00	0.86
150	225	1.00	0.88
155	235	1.00	0.90
160	240	1.00	0.92
165	250	1.00	0.94
170	255	1.00	0.96
175	265	1.00	0.98
180	270	1.00	1.00

Notes:

1) *The values of θ are for angle of load to grain.*
2) *For connectors loaded in compression, J_C = 1.00. The minimum end distances for connectors loaded in compression are the values specified in this Table for connectors loaded in tension.*

Table 12.9
Connector spacing, mm, for J_C between 0.75 and 1.0

(See Clauses 12.3.3 and 12.3.6.)

Angle of load to grain, θ°	Angle of connector row to grain, β°	Minimum spacing between connectors, mm			
		2-1/2 in split rings and 2-5/8 in shear plates		4 in split rings and 4 in shear plates	
		J_C = 0.75	J_C = 1.00	J_C = 0.75	J_C = 1.00
0	0	90	170	125	230
	15	90	160	125	215
	30	90	135	125	185
	45	90	110	125	155
	60	90	100	125	140
	75	90	90	125	130
	90	90	90	125	125
15	0	90	150	125	205
	15	90	145	125	195
	30	90	130	125	180
	45	90	115	125	160
	60	90	105	125	145
	75	90	100	125	135

(Continued)

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Table 12.9 (Concluded)

Angle of load to grain, $\theta°$	Angle of connector row to grain, $\beta°$	Minimum spacing between connectors, mm			
		2-1/2 in split rings and 2-5/8 in shear plates		4 in split rings and 4 in shear plates	
		J_C = 0.75	J_C = 1.00	J_C = 0.75	J_C = 1.00
	90	90	95	125	135
30	0	90	130	125	180
	15	90	125	125	175
	30	90	120	125	165
	45	90	110	125	155
	60	90	105	125	145
	75	90	100	125	145
	90	90	100	125	140
45	0	90	110	125	150
	15	90	110	125	150
	30	90	110	125	150
	45	90	110	125	150
	60	90	105	125	145
	75	90	105	125	145
	90	90	105	125	145
60–90	0	90	90	125	125
	15	90	90	125	125
	30	90	90	125	125
	45	90	100	125	135
	60	90	100	125	145
	75	90	105	125	150
	90	90	110	125	150

Note: *Values of J_C between 0.75 and 1.00 for intermediate connector spacings may be obtained by linear interpolation.*

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Figure 12.2
End distance for member with sloping end cut
(See Clauses 3.1 and 12.3.3.)

Legend:

a = end distance

d_F = diameter of split ring or shear plate

Figure 12.3
End distance, edge distance, and spacing of connectors
(See Clauses 3.1 and 12.3.3.)

Legend:

a = end distance

e_p = unloaded edge distance

e_Q = loaded edge distance

S = spacing

12.3.4 Thickness factor, J_T

For connectors installed in dimension lumber, thickness factor, J_T, specified in Table 12.10 shall be applied. Connectors installed in lumber of a thickness less than the minimum specified in Table 12.10 for the connector type and use shall not be considered to provide resistance.

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Table 12.10
Thickness factor for connector, J_T
(See Clauses 12.3.4 and 12.3.6.)

Connector type and size	No. of faces of a piece containing connectors on a bolt	Thickness of piece, mm	J_T
2-1/2 in split ring	1	38	1.00
		25	0.85
	2	51	1.00
		38	0.80
4 in split ring	1	38	1.00
		25	0.65
	2	76	1.00
		64	0.95
		51	0.80
		38	0.65
2-5/8 in shear plate	1	64	1.00
		51	0.95
		38	0.95
	2	64	1.00
		51	0.95
		38	0.75
4 in shear plate	1	44	1.00
		38	0.85
	2	89	1.00
		76	0.95
		64	0.85
		51	0.75
		44	0.65

12.3.5 Connectors used with lag screws

When lag screws instead of bolts are used with connectors, the resistance shall vary uniformly with penetration into the point-side member, from the full resistance for one connector unit with bolt for standard penetration to 0.75 times the full resistance for one connector unit with bolt for minimum penetration. Penetration shall be as specified in Table 12.11 and shall not be less than the minimum value.

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Table 12.11
Penetration factor, J_P, for split-ring and shear-plate connectors used with lag screws
(See Clauses 12.3.5 and 12.3.6.)

		Penetration of lag screw into point-side member (times nominal diameter of lag screws)				
		Species combination				
Connector	**Penetration**	**Douglas Fir-Larch**	**Hem-Fir**	**Spruce-Pine-Fir**	**Northern Species**	J_P
2-1/2 in split ring,	Standard	8	10	10	11	1.00
4 in split ring or 4 in shear plate*	Minimum	3.5	4	4	4.5	0.75
2-5/8 in shear plate*	Standard	5	7	7	8	1.00
	Minimum	3.5	4	4	8	0.75

* *When steel side plates are used with shear plates, use J_P = 1.0.*

Note: *For intermediate penetrations, linear interpolation may be used for values of J_P between 0.75 and 1.00.*

12.3.6 Lateral resistance

The factored lateral strength resistance of a split-ring or shear-plate connection, P_r, Q_r, or N_r, determined using the equations specified in this Clause, shall be greater than or equal to the effect of the factored loads. The factored strength resistance per shear-plate unit shall not exceed the values specified in Table 12.14.

a) for parallel-to-grain loading:
$P_r = \phi\, P_u\, n_F\, J_F$

b) for perpendicular-to-grain loading:
$Q_r = \phi\, Q_u\, n_F\, J_F$

c) for loads at angle θ to grain:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

ϕ = 0.6

P_u = $p_u\,(K_D\, K_{SF}\, K_T)$

where

p_u = lateral strength resistance parallel to grain, kN (see Table 12.12)

n_F = number of connector units in the connection

$J_F = J_G\, J_C\, J_T\, J_O\, J_P$

where

J_G = group action factor (see Clause 12.2.2.3.4)

J_C = minimum configuration factor (see Clause 12.3.3)

J_T = thickness factor (see Table 12.10)

J_O = factor for connector orientation in grain

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= 1.00 for side grain installation

= 0.67 for end grain and all other installations

J_P = penetration factor for lag screw (see Clause 12.3.5)

$Q_u = q_u (K_D K_{SF} K_T)$

where

q_u = lateral strength resistance perpendicular to grain, kN (see Table 12.13)

Table 12.12
Lateral strength resistance parallel to grain, p_u, of connector unit, kN
(See Clause 12.3.6.)

	Split ring		Shear plate	
Species combination	**2-1/2 in**	**4 in**	**2-5/8 in**	**4 in**
Douglas Fir-Larch	31	55	27	49
Hem-Fir	27	49	24	44
Spruce-Pine-Fir	23	45	23	42
Northern Species	21	42	22	40

Notes:

1) *The values for 4 in shear plates are for plates with 3/4 in bolts. For plates with 7/8 in bolts, resistances may be increased by 25%.*

2) *Where wood side plates are used with 4 in shear plates, resistances are 90% of the tabulated resistances.*

Table 12.13
Lateral strength resistance perpendicular to grain, q_u, of connector unit, kN
(See Clause 12.3.6.)

	Split ring		Shear plate	
Species combination	**2-1/2 in**	**4 in**	**2-5/8 in**	**4 in**
Douglas Fir-Larch	22	42	23	35
Hem-Fir	18	35	19	28
Spruce-Pine-Fir	17	31	17	26
Northern Species	15	28	15	24

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Table 12.14
Maximum factored strength resistance per shear plate unit, kN
(See Clause 12.3.6.)

Type of load	2-5/8 in shear plate	4 in shear plate	
		3/4 in bolt	7/8 in bolt
Washers provided — no bearing on the threaded portion of the bolt	18	32	43
When bearing can occur on the threaded portion of the bolt	16	28	38

12.4 Bolts and dowels

12.4.1 General

The design requirements specified in Clause 12.4 apply to metallic bolts and dowels whose material properties are referenced by a Canadian material design standard.

12.4.2 Materials

12.4.2.1 Wood members

The resistance of wood members in a connection shall be determined in accordance with Clause 12.4.4. The bearing length of fasteners in wood members, t_i, shall be the lesser of the thinnest wood member or the smallest bearing length shown in Figure 12.4.

Note: *To avoid eccentricity in loading, side members should be of the same thickness.*

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Figure 12.4
Bearing length of fasteners in wood members
(See Clause 12.4.2.1.)

12.4.2.2 Metal plates

When a connection consists of a mixture of wood members and metal plates, only metal plates with engineering design specifications in accordance with a Canadian structural design or material standard shall be used.

12.4.2.3 Concrete or masonry

When a connection consists of a wood member attached to concrete or masonry, the connection shall be designed as a two-member connection in accordance with the equations in Clause 12.4.4. The wood member shall be considered the side member. The concrete or masonry member shall be assumed to have a thickness equal to the penetration of the bolt in the concrete or masonry. The concrete or masonry shall be designed to resist the applied loads.

12.4.3 Placement of bolts and dowels in connections

12.4.3.1 Load component parallel to grain

In a group of fasteners where the load component is acting parallel to the grain, including those in the panel face of CLT, the minimum spacing of fasteners shall be as specified in Table 12.15 (see Figure 12.5).

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Table 12.15
Placement of bolts and dowels in a connection loaded parallel to grain
(See Clause 12.4.3.1.)

Symbol	Dimension*	Minimum spacing
S_P	Spacing parallel to grain	$4d_F$
S_Q	Spacing perpendicular to grain	$3d_F$
a	Unloaded end distance parallel to grain	Maximum {$4d_F$, 50 mm}
a_L	Loaded end distance parallel to grain	Maximum {$5d_F$, 50 mm}
e_P	Unloaded edge distance perpendicular to grain	Maximum {$1.5d_F$, 1/2 S_Q}

* *d_F = nominal diameter of fastener.*

Figure 12.5
Placement of bolts and dowels in a connection loaded parallel to grain
(See Clauses 12.4.3.1 and 12.4.4.4.)

a) Member in tension

b) Member in compression

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12.4.3.2 Load component perpendicular to grain

In a group of fasteners where the load component is acting perpendicular to the grain, including those in the panel face of CLT, the minimum spacing of fasteners shall be as specified in Table 12.16 (see Figure 12.6).

Δ

Table 12.16
Placement of bolts and dowels in a connection loaded perpendicular to grain
(See Clause 12.4.3.2.)

Symbol	Dimension*	Minimum spacing
S_P	Spacing parallel to grain	$3d_F$
S_Q	Spacing perpendicular to grain	$3d_F$
a	End distance parallel to grain	Maximum {$4d_F$ or 50 mm}
e_P	Unloaded edge distance perpendicular to grain	$1.5d_F$
e_Q	Loaded edge distance perpendicular to grain	$4d_F$

* d_F = *nominal diameter of fastener.*

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Δ

Figure 12.6
Placement of bolts and dowels in a connection loaded perpendicular to grain
(See Clause 12.4.3.2.)

a) Connection at member end

b) Connection away from member end

12.4.3.3 Placement of bolts and dowels in the panel edge of CLT

In a group of fasteners installed in the panel edge of CLT, the minimum spacing of fasteners shall be as specified in Table 12.17 (see Figure 12.7).

Table 12.17
Placement of bolts and dowels in the panel edge of CLT
(See Clause 12.4.3.3.)

Symbol	Dimension*	Minimum spacing
S_R	Spacing parallel to the load direction	$4d_F$
S_C	Spacing perpendicular to the load direction	$3d_F$
a	Unloaded end distance	Maximum {$4d_F$, 50 mm}
a_L	Loaded end distance	Maximum {$5d_F$, 50 mm}

(Continued)

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Table 12.17 (Concluded)

Symbol	Dimension*	Minimum spacing
e_P	Unloaded edge distance	$1.5d_F$
e_Q	Loaded edge distance	$5d_F$

* d_F = *nominal diameter of fastener.*

Figure 12.7
Placement of fasteners in panel edge of CLT
(See Clause 12.4.3.3.)

12.4.3.4 Maximum distance perpendicular to grain

Except for connections in CLT, a single steel splice plate shall not be used for rows of bolts or dowels when the distance between the two outer rows exceeds 125 mm.

12.4.4 Lateral resistance

12.4.4.1 Potential failure modes

Connections shall be designed to resist all possible yielding and brittle failure modes associated with the fasteners and material used as shown in Figure 12.8.

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Figure 12.8
Potential failure modes
(See Clauses 12.4.4.1 and 12.4.4.5.)

Yielding

a) All loading directions

b) Parallel-to-grain

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Q_f

Splitting

c) Perpendicular-to-grain

Note: *A yielding failure can occur if the force is applied parallel to grain, perpendicular to grain, or at an angle to grain.*

Δ 12.4.4.2 Requirements

Connections shall be designed in accordance with the following requirements:

a) $N_f \leq N_r$

where

N_f = factored load on the connection

N_r = factored lateral yielding resistance (see Clause 12.4.4.3.1)

b) $P_f \leq P_r$

where

P_f = factored load parallel to grain

P_r = factored resistance parallel to grain

= the lesser of PR_{rT}, PG_{rT}, or TN_{rT}

where

PR_{rT} = factored row shear resistance (see Clause 12.4.4.4)

PG_{rT} = factored group tear-out resistance (see Clause 12.4.4.5)

TN_{rT} = factored net tension resistance (see Clause 12.4.4.6.1)

c) $Q_f \leq Q_r$

where

Q_f = factored load perpendicular to grain

Q_r = QS_{rT}

where

QS_{rT} = factored splitting resistance (see Clause 12.4.4.7)

d) for loading at an angle to grain, θ:

$$N_f \leq \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

θ = angle between the applied load and the grain

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12.4.4.3 Yielding resistance

12.4.4.3.1 General

The factored lateral yielding resistance, N_r, of a group of fasteners in a connection is a function of the number of shear planes and shall be calculated as follows:

$N_r = \phi_y \, n_u \, n_s \, n_F$

where

ϕ_y = resistance factor for yielding failures

= 0.8

n_u = unit lateral yielding resistance per shear plane, N (see Clause 12.4.4.3.2)

n_s = number of shear planes in the connection

n_F = number of fasteners in the connection

Δ **12.4.4.3.2 Unit lateral yielding resistance**

The unit lateral yielding resistance per shear plane, n_u, shall be taken as the smallest value calculated in accordance with Items a) to g). For two-member connections, only Items a), b), and d) to g) shall be considered valid. For three-member connections, only Items a), c), d), and g) shall be considered valid. For a connection with four or more members, the resistance shall be determined by assuming that each member is part of a series of three-member connections.

a) $f_1 d_F t_1$

b) $f_2 d_F t_2$

c) $\frac{1}{2} f_2 d_F t_2$

d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

f) $f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$

g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}}$

where

d_F = nominal diameter of fastener, mm

t_1, t_2 = bearing length of fastener in members 1 and 2 in accordance with Clause 12.4.2.1, mm

f_1, f_2 = embedment strength of members 1 and 2 calculated in accordance with Clause 12.4.4.3.3, where member 1 is the side member, MPa

f_y = yield strength of fastener in bending in accordance with Clause 12.4.4.3.3.3, MPa

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12.4.4.3.3 Embedment strength

Δ **12.4.4.3.3.1 Wood members**

For fasteners installed in the side grain, the embedment strength $f_{i\theta}$, in MPa, shall be calculated as follows:

$$f_{i\theta} = \frac{f_{ip} f_{iQ}}{f_{ip} \sin^2 \theta + f_{iQ} \cos^2 \theta} (K_D K_{SF} K_T)$$

where

$f_{i\theta}$ = embedment strength of member i for a fastener bearing at angle θ relative to the grain

f_{iP} = embedment strength for a fastener bearing parallel to grain ($\theta = 0°$)

= $50\ G\ (1–0.01 d_F)\ J_X$

where

G = mean relative density (see Table A.11)

J_X = adjustment factor for connections in CLT

= 0.9 for CLT

= 1.0 for other wood members

f_{iQ} = embedment strength for a fastener bearing perpendicular to grain ($\theta = 90°$)

= $22\ G\ (1– 0.01 d_F)$

θ = angle of bearing relative to the grain

K_D = load-duration factor in accordance with Clause 5.3.2

K_{SF} = service-condition factor in accordance with Clause 12.2.1.7.2

K_T = treatment factor in accordance with Clause 12.2.1.7.3

For fasteners installed in the panel edge of CLT, the embedment strength in any loading direction, $f_{i\theta}$, shall not be greater than 0.6 f_{iQ} ($K_D\ K_S\ K_T$) of the fasteners installed perpendicular to the panel face.

Note: *The values of $f_{i\theta}$ are applicable to the entire dowel-bearing length of the fastener.*

12.4.4.3.3.2 Non-wood materials

The embedment strength, f_i, of non-wood materials shall be calculated as follows:

a) for steel:

$f_i = K_{sp}\ (\phi_{steel}/\phi_y)\ f_u$

where

K_{sp} = 3.0 for mild steel referenced in CSA S16

= 2.25 for cold-formed light-gauge steel referenced in CSA S136

f_u = specified minimum tensile strength of steel, MPa

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g.,*

a) *f_u = 400 MPa for ASTM A36/A36M steel;*

b) *f_u = 450 MPa for CSA G40.21 steel, Grades 300W and 350W; and*

c) *f_u = 310 MPa for cold-formed light-gauge steel, Grade SS 230.*

ϕ_{steel} = resistance factor for steel plates in connections with bolts and dowels

= 0.8 for mild steel referenced in CSA S16

= 0.5 for cold-formed light-gauge steel referenced in CSA S136

ϕ_y = resistance factor for yielding failures in wood members in connections with bolts and dowels

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= 0.8

b) for concrete or masonry:
f_i = 125 MPa

12.4.4.3.3.3 Dowel or bolt yield strength

The yield strength, f_y, of a dowel or bolt shall be taken as follows:

a) for ASTM A307 and SAE J429 Grade 2 fasteners:
f_y = 310 MPa; or

b) for other CSA- or ASTM-compliant fasteners:

$$f_y = \frac{f_{ym} + f_{um}}{2}$$

where

f_{ym} = specified yield strength obtained from applicable material standards, MPa

f_{um} = specified tensile strength obtained from applicable material standards, MPa

12.4.4.4 Row shear resistance

The total factored shear resistance, PR_{rT}, of a connection shall be calculated as the sum of the factored row shear resistances of the wood members resisting the load, as follows:

$$PR_{rT} = \Sigma\ (PR_{r\,i})$$

The total factored row shear resistance of fasteners in a wood member *i* shall be calculated as follows:

$$PR_{ri} = \phi_w\ PR_{ij\,min}\ n_R$$

where

ϕ_w = resistance factor for brittle failures

= 0.7

$PR_{ij\,min}$ = minimum row shear resistance of any row in the connection from PR_{i1} to PR_{inR}

n_R = number of fastener rows

PR_{ij} = shear resistance of fastener row *j* in member *i*, N

= $1.2\ f_v\ (K_D\ K_{Sv}\ K_T)\ K_{ls}\ t\ n_c\ a_{cr\,i}$

where

f_v = specified longitudinal shear strength of member *i*, MPa

= value specified in Tables 6.4, 6.6, 6.7, or 7.2

= 0.6 × value specified in Table 6.5

K_{ls} = factor for member loaded surfaces (see Figure 12.9)

= 0.65 for side member

= 1.0 for internal member

t = thickness of member *i*, mm

n_c = number of fasteners in row *j* of member *i*

$a_{cr\,i}$ = minimum of a_L and S_P for row *j* of member *i*, mm (see Figure 12.5)

K_{Sv} = service-condition factor for longitudinal shear (see Tables 6.10 and 7.3)

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The row shear resistance of each member shall be checked separately. For CLT, the row shear failure is not required to be considered.

Figure 12.9
Factor for member loaded surfaces, K_{ls}
(See Clause 12.4.4.4.)

Δ **12.4.4.5 Group tear-out resistance**

The total factored group tear-out resistance, PG_{rT}, of a connection shall be calculated as the sum of the factored group tear-out resistance of the wood members resisting the load, as follows:

$$PG_{rT} = \Sigma\,(PG_{ri})$$

The factored group tear-out resistance of wood member *i*, in N, shall be calculated as follows:

$$PG_{ri} = \phi_w[(PR_{i\,1} + PR_{inR})/2 + f_{ti}(K_D\,K_{St}\,K_T)\,A_{PGi}]$$

where

ϕ_w = resistance factor for brittle failures
= 0.7

$PR_{i\,1}$ = shear resistance of wood member i along row 1 *i* bounding the fastener group, N
= $1.2\,f_{vi}\,(K_D\,K_{Sv}\,K_T)\,K_{ls}\,t\,n_{c1}\,a_{cr1}$

PR_{inR} = shear resistance along row n_R of member *i* bounding the fastener group, N
= $1.2\,f_{vi}\,(K_D\,K_{Sv}\,K_T)\,K_{ls}\,t\,n_{cnR}\,a_{crnR}$

f_{ti} = specified strength in tension parallel to grain of member *i*, MPa
= value specified in Table 6.4, 6.6, or 6.7
= value for net section specified in Table 7.2
= 0.65 × values specified in Table 6.5

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K_{St} = service-condition factor for tension parallel to grain (see Tables 6.10 and 7.3)

K_{Sv} = service-condition factor for longitudinal shear (see Tables 6.10 and 7.3)

A_{PGi} = critical perpendicular net area between rows 1 and n_R of member i, mm²

The group tear-out resistance of each member in tension shall be checked separately. For CLT, group tear-out failure is not required to be considered. Group tear-out failure is not required to be considered if the load is applied in such a way that the member is in compression (see Figure 12.8).

12.4.4.6 Net tension resistance

12.4.4.6.1 General

The total factored net tension resistance, T_{NrT}, of wood members loaded parallel to grain at a group of fasteners shall be calculated as follows:

$$T_{NrT} = \Sigma T_{Nr\,i}$$

$T_{Nr\,i}$ of member i at a group of fasteners shall be determined using Clause 6.5.8 for sawn lumber and Clause 7.5.11 for glulam.

12.4.4.6.2 Net section of wood member at the connection

The cross-sectional area deducted from the gross cross-section shall not be greater than 25% of the member gross area in accordance with Clause 5.3.8.

12.4.4.7 Splitting resistance

The total factored splitting resistance, QS_{rT}, of a connection shall be calculated as the sum of the splitting resistances of the wood members resisting the load as follows:

$$QS_{rT} = \Sigma QS_{ri}$$

The factored splitting resistance of wood member i, in N, shall be calculated as follows:

$$QS_{ri} = \phi_w QS_i\,(K_D\,K_{SF}\,K_T)$$

where

ϕ_w = resistance factor for brittle failures

= 0.7

$$QS_i = 14t\sqrt{\frac{d_e}{1-\frac{d_e}{d}}}$$

where

t = member thickness, mm

d_e = effective depth of member, mm

= $d - e_p$

where

d = depth of member, mm

e_p = unloaded edge distance, mm

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The splitting resistance is not required to be considered for fasteners installed in the panel face of CLT and loaded perpendicular to the panel face, but it shall be checked for fasteners installed in the panel edge of CLT.

Note: *See Clause 12.2.1.6 for member shear resistance based on effective depth.*

12.4.5 Axial resistance

Bolts shall be designed to resist the load component parallel to their axis. Washers or plates shall be designed to resist the factored load component parallel to the axis of the bolt. Dowels shall not be designed to resist axial loads.

12.4.6 Combined lateral and axial resistance

The resistance of bolts subjected to lateral and axial loading shall be verified in accordance with the applicable Canadian material design standard.

12.5 Drift pins

12.5.1 General

The design requirements specified in Clause 12.5 apply to cribwork connections with drift pins with a diameter of 16 to 25 mm and meeting the requirements of CSA G40.21 or ASTM A307.

12.5.2 Pre-drilled holes

Holes in timber shall be pre-drilled not less than 0.8 mm and not more than 1 mm smaller than the drift-pin diameter.

12.5.3 Drift-pin points

The leading end of drift pins shall be chisel-pointed, conically tapered, and hemispherical or otherwise shaped to permit driving into pre-drilled holes with minimum damage to the wood.

12.5.4 Axial tension

Drift-pin connections shall be used only where gravity or mechanical restraint prevents axial tensile stress in the drift pins.

12.5.5 Size and placement of drift pins in connections

Δ **12.5.5.1 Drift-pin length**

Drift-pin length shall be equal to the sum of the depths of two superimposed members to be connected, less 15 mm. Each drift pin shall be considered to give one shear plane. Figure 12.10 shows a typical drift-pin connection in timber cribwork.

12.5.5.2 Drift-pin diameter

The drift-pin diameter shall not be greater than one-tenth of the width of the timbers to be connected.

12.5.5.3 End and edge distances

End and edge distances shall be at least 2.5 times the pin diameter.

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12.5.5.4 Spacing

Spacing between pins parallel and perpendicular to the grain shall be at least four times the pin diameter.

Δ 12.5.6 Lateral resistance

The factored lateral strength resistance, P_r, Q_r, or N_r, of a drift-pin connection shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

a) for parallel-to-grain loading, P_r shall be the lesser of 0.6 $PR_{r\,T}$, 0.6 $PG_{r\,T}$, or 0.6 $T_{Nr\,T}$;

b) for perpendicular-to-grain loading, Q_r shall be 0.6 $QS_{r\,T}$; and

c) for loads at angle θ to grain, N_r shall be as follows:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

$PR_{r\,T}$ = factored row shear resistance (see Clause 12.4.4.4)

$PG_{r\,T}$ = factored group tear-out resistance (see Clause 12.4.4.5)

$T_{Nr\,T}$ = factored net tension resistance (see Clause 12.4.4.6)

$QS_{r\,T}$ = factored splitting resistance (see Clause 12.4.4.7)

At any shear plane between two overlapping timbers, as shown in Figure 12.10, only two pins shall be counted as resisting the shear force.

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Figure 12.10
Placement of drift pins
(See Clauses 12.5.5.1 and 12.5.6.)

12.6 Lag screws

12.6.1 General

12.6.1.1 Materials

The design requirements specified in Clause 12.6 apply to two-member connections with lag screws that meet the requirements of ASME B18.2.1 and are made of steel that meets or exceeds the properties of ASTM A307 Grade A or SAE J429 Grade 1.

Note: *For the use of alternate dowel-type self-drilling fasteners, see the CWC Commentary on CSA O86.*

12.6.1.2 Connection assembly

The threaded portion of the lag screw shall be inserted by turning with a wrench or other similar tool, not by driving. Care shall be exercised to avoid over-tightening the lag screws during installation.

Note: *If over-tightened, the lag screw will not have withdrawal resistance specified in this Standard.*

12.6.1.3 Facilitating connection assembly

Soap or other non-petroleum-based lubricant may be used on the lag screws or in the lead hole to facilitate insertion and prevent damage to the lag screw.

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12.6.1.4 Net section of wood member at the connection

The cross-sectional area deducted from the gross cross-section shall not be greater than 25% of the member gross area in accordance with Clause 5.3.8.

12.6.2 Placement of lag screws in connections

12.6.2.1 Load component parallel to grain

In a group of fasteners where the load component is acting parallel to the grain, including those in the panel face of CLT, the minimum spacing of fasteners shall be as specified in Table 12.18 [see Figure 12.11 a)].

Δ

Table 12.18
Placement of lag screws in a connection loaded parallel to grain
(See Clause 12.6.2.1.)

Symbol	Dimension*	Minimum spacing
S_P	Spacing parallel to grain	$4d_F$
S_Q	Spacing perpendicular to grain	$2d_F$
a	Unloaded end distance parallel to grain	Maximum {$4d_F$, 50 mm}
a_L	Loaded end distance parallel to grain	Maximum {$7d_F$, 50 mm}
e	Unloaded edge distance perpendicular to grain	Maximum {$1.5d_F$, or 1/2 S_Q}

Note: *d_F = nominal diameter of lag screw.*

12.6.2.2 Load component perpendicular to grain

In a group of fasteners where the load component is acting perpendicular to the grain, including those in the panel face of CLT, the minimum spacing of fasteners shall be as specified in Table 12.19 [see Figure 12.11 b)].

Table 12.19
Placement of lag screws in a connection loaded perpendicular to grain
(See Clause 12.6.2.2.)

Symbol	Dimension*	Minimum spacing
S_P	Spacing parallel to grain	
	if $t = 2d_F$	$2.5d_F$
	if $t \geq 6d_F$	$5d_F$
	if $2d_F < t < 6d_F$	by linear interpolation
S_Q	Spacing perpendicular to grain	$\max\begin{cases} 3d_F \\ S_P \text{ of the attached member} \end{cases}$
a	Unloaded end distance parallel to grain	Maximum {$4d_F$, 50 mm}

(Continued)

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Table 12.19 (Concluded)

Symbol	Dimension*	Minimum spacing
e_P	Unloaded edge distance perpendicular to grain	$1.5d_F$
e_Q	Loaded edge distance perpendicular to grain	$4d_F$

Note: *d_F = nominal diameter of lag screw.*

12.6.2.3 Placement of lag screws in the panel edge of CLT

For lag screws in the panel edge of CLT, the minimum fastener spacing, as specified in Clause 12.4.3.3, shall be used.

12.6.2.4 Maximum distance perpendicular to grain

Except for connections in CLT, a single steel splice plate shall not be used for rows of lag screws when the distance between the two outer rows exceeds 125 mm.

12.6.2.5 Lag screw holes

Lag screw holes in wood members shall consist of the clearance hole and lead hole meeting the following requirements:

a) The clearance hole for the unthreaded portion of the shank shall have the same diameter, d_F, and the same depth as the penetration length of the unthreaded shank.

b) The lead hole for the threaded portion of the shank shall have a diameter equal to $0.65d_F$ to $0.85d_F$ for dense hardwoods, $0.60d_F$ to $0.75d_F$ for Douglas Fir-Larch, and $0.40d_F$ to $0.70d_F$ for less dense wood. The larger lead-hole diameter in each range shall apply to lag screws of the greater diameters. The length of the lead hole shall fit the threaded portion of the lag screw.

Note: *Lead holes for the threaded portion are not required for 10 mm and smaller diameter lag screws loaded primarily in withdrawal in wood with relative density, $G \leq 0.5$, if the fasteners are placed so as to prevent unusual splitting.*

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Δ

Figure 12.11
Placement of lag screws in connections
(See Clauses 12.6.2.1 and 12.6.2.2.)

a) Load parallel to grain

b) Load perpendicular to grain

12.6.3 Penetration length of lag screws

12.6.3.1 Penetration length

For determining the penetration length of a lag screw into a member, the length of the tapered tip of the threaded shank shall not be included.

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12.6.3.2 Maximum penetration length

Unless the tensile resistance of the lag screw at the net (root) section is verified (see Clause 12.6.6.2), the maximum length of threaded shank penetration into the point-side member (L_t in Clause 12.6.6.1) used to determine withdrawal resistance shall not exceed the following values:

a) for Douglas Fir-Larch: $9d_F$;
b) for Hem-Fir: $10d_F$;
c) for Spruce-Pine-Fir and Northern Species: $11d_F$; and
d) for denser species: determined based on the tensile resistance of fasteners (see Clause 12.6.6.2).

Note: *For lag screws loaded laterally, the same limits on the maximum length of penetration into main member (t_2 in Clause 12.6.5.1.2) may be used, because further increasing the length of a lag screw does not increase either the strength or stiffness of the connection.*

12.6.3.3 Minimum penetration length

For lag screws loaded laterally, the minimum length of penetration into the point-side member (t_2 in Clause 12.6.5.1.2) shall be not less than $5d_F$.

12.6.4 Side members

12.6.4.1 Wood side members

The thickness of wood side members shall be at least $2d_F$.

12.6.4.2 Steel side plates

The stresses induced in the steel side plate and at the bearing of the fastener on the plate shall not exceed the resistance of the steel used.

12.6.5 Lateral resistance

12.6.5.1 Side grain

12.6.5.1.1 General

The factored lateral strength resistance, P_r, Q_r, or N_r, of a two-member lag-screw connection shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

a) for parallel-to-grain loading:
$P_r = \phi\, P_u\, n_F\, J_G\, J_{PL}$
b) for perpendicular-to-grain loading:
$Q_r = \phi\, Q_u\, n_F\, J_G\, J_{PL}$
c) for loads at angle θ to grain:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

ϕ = 0.6

P_u = $p_u\,(K_D\, K_{SF}\, K_T)$

p_u = unit lateral strength resistance for parallel-to-grain loading, N (see Clause 12.6.5.1.2)

n_F = number of fasteners in the connection

J_G = group-action factor (see Tables 12.2 and 12.3)

J_{PL} = factor for reduced penetration

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= 0.625 for penetration of $5d_F$ and 1.0 for $8d_F$ (for intermediate values, interpolate linearly)

Q_u = q_u (K_D K_{SF} K_T)

q_u = unit lateral strength resistance for perpendicular-to-grain loading, N (see Clause 12.6.5.1.2)

Δ 12.6.5.1.2 Unit lateral strength resistance

The unit lateral strength resistances, p_u or q_u, shall be taken as the smallest value determined as follows:

a) $f_1 d_F t_1$

b) $f_2 d_F t_2$

c) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

e) $f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$

f) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}}$

where

d_F = nominal diameter of lag screw, mm

t_1 = thickness of side member, mm (see Clause 12.6.4)

t_2 = length of penetration into main member, mm (see Clause 12.6.3)

f_1 = embedment strength of side member, MPa

f_2 = embedment strength of wood main member, MPa

For wood side members:

f_1 = $f_2 = 50G\ (1–0.01d_F)\ J_x$ for parallel-to-grain loading

= $f_2 = 22G\ (1–0.01d_F)$ for perpendicular-to-grain loading

where

G = mean relative density (see Table A.11)

J_x = 0.90 for CLT

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= 1.0 for other wood members

For steel side plates:

f_1 = K_{sp} (ϕ_{steel} / ϕ_{wood})f_u

where

K_{sp} = 3.0 for mild steel referenced in CSA S16

= 2.25 for cold-formed light-gauge steel referenced in CSA S136

f_u = specified minimum tensile strength of steel

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g., for*

a) *f_u = 400 MPa for ASTM A36/A36M steel;*

b) *f_u = 450 MPa for CSA G40.21 steel, Grades 300W and 350W; and*

c) *f_u = 310 MPa for cold-formed light-gauge steel, Grade SS 230.*

ϕ_{steel} = resistance factor for steel plates in connections with lag screws

= 0.8 for mild steel referenced in CSA S16

= 0.5 for cold-formed light-gauge steel referenced in CSA S136

ϕ_{wood} = resistance factor for wood members in connections with lag screws

= 0.6

f_y = yield strength of fastener in bending, MPa

= 310 MPa for lag screws meeting SAE J429 Grade 1

12.6.5.2 End grain

The lateral resistance of lag screws inserted parallel to grain in the end grain of the main member shall not be greater than two-thirds of the lateral side grain resistance for perpendicular to grain if wood side plates are used. If steel side plates are used, the lateral resistance shall not be greater than one-half of the lateral side grain resistance for perpendicular-to-grain loading in the main member.

12.6.5.3 Panel edge of CLT

The lateral resistance of lag screws inserted in panel edge of CLT shall not be greater than two-thirds of the lateral resistance for perpendicular to grain of laminations in the panel face if wood side plates are used, or one-half if steel side plates are used.

12.6.5.4 Connection deformation

Where required for design, the lateral deformation of lag screw connections may be estimated in accordance with Clause A.12.6.5.4.

12.6.6 Withdrawal resistance

Δ 12.6.6.1 General

The factored withdrawal resistance, P_{rw}, of a lag screw connection shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

$P_{rw} = \phi\ Y_w\ L_t\ n_F\ J_E$

where

ϕ = 0.6

Y_w = y_w ($K_D\ K_{SF}\ K_T$)

where

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y_w = specified withdrawal resistance per millimetre of threaded shank penetration into main member, N/mm

= $59 d_F^{0.82} G^{1.77} J_X$

where

d_F = nominal diameter of lag screw, mm

G = mean relative density of main member (see Table A.11)

J_X = 0.9 for CLT

= 1.0 in all other cases

L_t = length of threaded shank penetration into main member, mm (see Clause 12.6.3)

n_F = number of fasteners in the connection

J_E = end grain factor for lag screws

= 0.75 in end grain

= 0.67 in panel edge of CLT

= 1.00 in all other cases

Note: *The use of lag screws in end grain should be avoided.*

12.6.6.2 Tensile resistance of the lag screw

The tensile resistance of the lag screw at the net (root) section shall not be exceeded. The specified tensile strength of the lag screw, f_u, shall be a minimum of 410 MPa.

12.7 Timber rivets

Note: *Timber rivets are also referred to as "glulam rivets" or "rivets" in the following clauses.*

12.7.1 General

12.7.1.1 Timber rivets with steel side plates

The design methods and tabulated resistances specified in Clause 12.7 are for timber rivets with steel side plates that meet the following requirements:

a) rivets are manufactured to have the following properties:
   i) hardness: Rockwell C32-39;
   ii) ultimate tensile strength: 1000 MPa, minimum; and
   iii) finish: hot-dip galvanized;

b) steel side plates meet the requirements of CSA G40.21 or ASTM A36/A36M; and

c) rivets and steel plates have dimensions as shown in Figure 12.12.

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Figure 12.12
Steel side plates for timber rivets
(See Clauses 12.7.1.1, 12.7.3.3, and A.12.7.3.3.1 and Table 12.21.)

Notes:
1) *Hole diameter: 6.4 mm minimum to 7.0 mm maximum.*
2) *Tolerance in location of holes: 3 mm maximum in any direction.*
3) *Orient wide face of rivets parallel to grain, regardless of plate orientation.*
4) *All dimensions are prior to galvanizing, in millimetres.*

12.7.1.2 Steel side plates for wet-service conditions

For wet-service conditions, steel side plates shall be hot-dip galvanized.

12.7.1.3 Wood members

The design criteria in Clause 12.7 for timber rivet connections shall only apply to timber rivets that meet the requirements of Clause 12.7.1.1 loaded in single shear or in withdrawal, with steel side plates, on Douglas Fir-Larch or Spruce-Lodgepole Pine-Jack Pine glued-laminated timber manufactured in accordance with CAN/CSA-O122, or on sawn lumber of 64 mm minimum thickness.

12.7.1.4 Cross-section of steel side plates

Steel side plates shall have a cross-section designed to resist tension and compression forces, as well as buckling at critical sections, but shall not be less than 3.2 mm thick.

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12.7.1.5 Installation of timber rivets

Each rivet shall be placed with its major cross-sectional dimension aligned parallel to grain. The design criteria in Clause 12.7 are based on rivets driven through circular holes in the side plates until the conical heads are firmly seated, but rivets shall not be driven flush.

Note: *Rivets at the perimeter of the group should be driven first. Successive rivets should be driven in a spiral pattern from the outside to the centre of the group.*

12.7.2 Placement of timber rivets in connections

12.7.2.1 Minimum spacing

The minimum spacing of timber rivets shall be 15 mm perpendicular to the grain and 25 mm parallel to the grain.

12.7.2.2 Minimum end and edge distances

Minimum end and edge distances (see Figure 12.13) shall be as specified in Table 12.20.

Table 12.20
Minimum end and edge distances for timber rivet connections
(See Clause 12.7.2.2 and Figure 12.13.)

	Minimum end distance, mm		Minimum edge distance, mm	
No. of rivet rows, n_R	Load parallel to grain, a_L	Load perpendicular to grain, a	Unloaded edge, e_P	Loaded edge, e_Q
1–2	75	50	25	50
3–8	75	75	25	50
9–10	100	80	25	50
11–12	125	100	25	50
13–14	150	120	25	50
15–16	175	140	25	50
17 and greater	200	160	25	50

12.7.2.3 Penetration of timber rivets

The maximum penetration of any rivet shall be 70% of the thickness of the wood member, whether driven on two faces or one face only. Except as permitted by Clause 12.7.2.4 for connections with rivets driven from opposite faces of a wood member, the rivet length shall be such that the points do not overlap.

12.7.2.4 Overlapping of timber rivets

For connections where rivets are driven from opposite faces of a wood member such that their points overlap, the minimum spacing requirements of Clause 12.7.2.1 shall apply to the distance between the rivets at their points. The total lateral resistance of the connection shall be calculated in accordance with Clause 12.7.3, considering the connection to be a one-sided timber rivet connection, with

a) the number of rivets associated with the one plate equaling the total number of rivets at the connection; and

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b) S_p and S_Q determined as the distances between the rivets at their points.

12.7.2.5 Maximum distance perpendicular to grain

For wet fabrication conditions in sawn lumber, the maximum distance perpendicular to grain over which a timber rivet group spans shall not exceed 200 mm.

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Figure 12.13
End and edge distances for timber rivet connections
(See Clauses 12.7.2.2 and A.12.7.3.3 .)

Load parallel to grain

Load perpendicular to grain

Load at angle to grain

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12.7.3 Lateral resistance

12.7.3.1 General

A timber rivet joint (one steel side plate and the rivets associated with it) in side grain shall be designed so that the factored lateral strength resistance of the joint is greater than or equal to the effect of the factored loads on the joint.

Note: *The resistance of timber rivet joints loaded laterally is governed by either the ductile failure of the rivets or the brittle failure of the wood.*

12.7.3.2 Factored lateral strength resistance parallel to grain

For loading parallel to grain, the factored lateral strength resistance, P_r, of a timber rivet joint shall be calculated as follows:

$P_r = \phi P_u H$

where

ϕ = 0.6

P_u = unit lateral strength resistance parallel to grain, kN (see Clause 12.7.3.3)

H = material factor

= 1.00 for Douglas Fir-Larch glulam

= 0.80 for Spruce-Lodgepole Pine-Jack Pine glulam

= 0.50 for Douglas Fir-Larch sawn timber

= 0.45 for Hem-Fir sawn timber

= 0.40 for Spruce-Pine-Fir sawn timber

= 0.35 for Northern Species sawn timber

12.7.3.3 Unit lateral strength resistance parallel to grain

The unit lateral strength resistance parallel to grain, P_u, per joint, shall be the lesser of P_y or P_w, calculated as follows:

$P_y = (1.09 L_p^{0.32} n_R n_C) J_Y (K_{SF} K_T)$ for rivet resistance

$P_w = p_w (K_D K_{SF} K_T)$ for wood resistance

where

L_p = length of penetration, mm (see Figure 12.12)

= (overall rivet length) – (plate thickness) – 3.2

n_R = number of rows of rivets parallel to the direction of load

n_C = number of rivets per row

J_Y = side plate factor

= 1.00 for a side plate thickness of 6.4 mm and more

= 0.90 for a side plate thickness of 4.7 mm or more but less than 6.4 mm

= 0.80 for a side plate thickness of 3.2 mm or more but less than 4.7 mm

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p_w = specified unit lateral strength resistance per joint parallel to grain, kN (see Table 12.21), using wood member thickness for the member dimension in Table 12.21 for connections with steel plates on opposite sides and using twice the wood member thickness for the member dimension in Table 12.21 for connections having only one plate

Note: *As an alternative, p_w may be calculated in accordance with Clause A.12.7.3.3.1.*

12.7.3.4 Factored lateral strength resistance perpendicular to grain

For loading perpendicular to grain, the factored lateral strength resistance, Q_r, of a timber rivet joint shall be calculated as follows:

$Q_r = \phi\ Q_u\ H$

where

ϕ = 0.6

Q_u = unit lateral strength resistance perpendicular to grain, kN (see Clause 12.7.3.5)

H = material factor (see Clause 12.7.3.2)

12.7.3.5 Unit lateral strength resistance perpendicular to grain

The unit lateral strength resistance perpendicular to grain, Q_u, per joint, shall be the lesser of Q_y or Q_w, calculated as follows:

$Q_y = (0.62 L_p^{0.32}\ n_R\ n_C)\ J_Y\ (K_{SF}\ K_T)$ for rivet capacity

$Q_w = (q_w\ L_p^{0.8} C_t)(K_D\ K_{SF}\ K_T)$ for wood capacity

where L_p, n_R, n_C, J_Y are as specified in Clause 12.7.3.3

q_w = specified unit strength lateral resistance per joint perpendicular to grain, kN (see Table 12.22)

C_t = timber rivet joint configuration factor (see Table 12.23)

Note: *As an alternative, q_w and C_t may be calculated in accordance with Clause A.12.7.3.3.2.*

12.7.3.6 Factored lateral strength resistance at an angle to grain

For loading at an angle to the grain, θ, the factored lateral strength resistance, N_r, of a timber rivet joint shall be calculated as follows:

$$N_r = \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

P_r = factored lateral strength resistance parallel to grain (see Clause 12.7.3.2)

Q_r = factored lateral strength resistance perpendicular to grain (see Clause 12.7.3.4)

12.7.3.7 End grain

When timber rivets are installed in the end grain, the factored lateral strength resistance of the joint shall be 50% of that for rivets installed in the side grain and loaded perpendicular to grain. When timber rivets are installed in the end grain at an intermediate angle, the factored lateral strength resistance may be increased linearly from the value calculated for the end grain, up to 100% of the applicable value for rivets installed in the side grain loaded parallel or perpendicular to grain.

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12.7.4 Withdrawal resistance

12.7.4.1 Load duration

Timber rivets loaded in withdrawal shall only be permitted for dry-service conditions for short-term and standard-term load durations.

12.7.4.2 Withdrawal resistance from the side grain

The factored withdrawal resistance from the side grain, P_{rw}, of a timber rivet joint shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

$P_{rw} = \phi\ Y_w\ L_p\ n_R\ n_C$

where

ϕ = 0.6

Y_w = $y_w\ (K_{SF}\ K_T)$

where

y_w = specified withdrawal resistance per rivet per millimetre of penetration, N/mm

= 13 for glulam

= 7 for sawn timber

L_p, n_R, and n_C are as specified in Clause 12.7.3.3.

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Table 12.21
Values of p_w, kN, parallel to grain for timber rivet joints 40 mm rivets — Spacing: S_p = 25 mm; S_Q = 25 mm

(See Clauses 12.7.3.3 and A.12.7.3.3.1.)

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
80	2	24	56	88	125	160	195	225	260	290	330
	4	35	74	110	155	200	240	270	310	350	390
	6	46	92	135	185	240	280	320	360	410	460
	8	58	108	160	215	270	320	370	410	460	520
	10	68	125	180	245	310	370	410	460	510	580
	12	76	140	205	270	340	400	450	510	570	640
	14	84	155	220	300	370	440	490	560	630	710
	16	88	170	245	320	410	480	530	600	680	770
	18	98	185	270	350	430	510	570	640	730	810
	20	104	205	280	370	460	540	600	680	770	850
130	2	31	62	72	88	112	130	150	180	235	290
	4	44	80	92	108	135	160	180	215	260	310
	6	60	98	112	130	165	190	215	250	310	360
	8	76	116	130	150	190	215	245	280	340	390
	10	88	135	150	175	215	245	270	310	370	440
	12	100	150	170	195	240	270	300	350	410	480
	14	108	170	185	215	260	300	330	380	450	520
	16	114	185	205	235	290	330	360	410	500	580
	18	125	200	225	250	310	350	390	450	540	610
	20	135	220	240	270	330	380	420	490	590	660
175	2	34	58	66	80	104	120	140	170	215	270
	4	50	74	84	100	125	150	170	200	245	290

(Continued)

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Table 12.21 (Concluded)

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
	6	66	90	102	120	150	175	200	235	280	330
	8	84	108	120	140	175	200	225	260	310	360
	10	98	125	140	160	200	225	250	290	350	410
	12	110	140	155	180	220	250	280	330	390	450
	14	120	155	175	200	245	280	310	360	420	490
	16	125	170	190	215	270	300	330	390	460	540
	18	140	185	205	235	290	330	360	420	510	570
	20	150	205	225	250	310	350	390	460	550	620
215 and greater	2	34	56	66	80	102	118	135	165	215	270
	4	50	72	84	98	125	145	165	195	240	290
	6	66	90	100	118	150	170	195	230	280	320
	8	84	106	118	140	170	195	225	260	310	350
	10	98	120	135	155	195	225	250	290	340	400
	12	110	135	155	175	215	250	270	320	380	440
	14	120	155	170	195	240	270	300	350	410	480
	16	125	170	185	210	260	300	330	380	460	530
	18	140	185	200	230	280	320	360	410	500	560
	20	150	200	220	245	300	350	390	450	540	610

(Continued)

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Table 12.21A
40 mm rivets — Spacing: S_p = 40 mm; S_Q = 25 mm

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
80	2	27	64	100	140	185	225	260	300	340	380
	4	40	86	125	175	225	280	320	360	400	450
	6	52	106	155	215	270	330	370	410	470	530
	8	66	125	185	245	310	370	430	470	520	590
	10	76	145	210	280	350	420	480	530	590	660
	12	86	160	235	310	390	460	530	590	650	730
	14	94	180	250	340	420	510	580	640	720	810
	16	100	195	280	370	460	550	620	700	780	880
	18	110	215	310	400	490	590	670	740	830	930
	20	118	235	330	420	520	620	710	790	870	970
130	2	35	66	82	106	145	180	225	290	390	490
	4	50	94	118	150	205	250	310	380	500	590
	6	68	120	150	185	250	310	380	470	610	690
	8	86	150	180	225	300	370	450	540	680	770
	10	100	175	210	260	350	420	510	610	760	860
	12	112	200	240	290	390	480	560	690	840	950
	14	125	225	270	320	430	530	620	760	930	1050
	16	130	250	300	360	430	580	680	830	1010	1140
	18	145	270	320	390	520	630	750	910	1090	1210
	20	150	290	350	420	560	680	810	990	1140	1270
175	2	39	60	76	98	135	170	205	260	360	480
	4	56	88	110	135	190	235	280	350	470	590
	6	76	114	140	175	235	290	350	440	570	710

(Continued)

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Table 12.21A (Concluded)

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
	8	96	140	170	205	280	340	410	500	650	790
	10	110	160	195	240	320	390	470	570	730	910
	12	125	185	225	270	360	440	520	640	810	1010
	14	135	210	250	300	400	490	580	710	900	1110
	16	145	230	280	330	440	540	630	770	990	1230
	18	160	250	300	360	480	580	700	850	1090	1310
	20	170	270	330	390	520	630	760	920	1190	1400
215 and greater	2	39	60	74	96	130	165	205	260	360	480
	4	56	86	108	135	185	230	280	350	460	580
	6	76	112	135	170	235	290	350	430	560	700
	8	96	135	165	205	280	340	410	500	640	780
	10	110	160	190	235	320	380	470	560	710	890
	12	125	180	220	260	360	440	510	630	800	1000
	14	135	205	245	300	400	480	570	700	880	1090
	16	145	225	270	320	430	530	620	760	980	1210
	18	160	250	290	360	470	570	680	830	1080	1290
	20	170	270	320	380	510	620	740	910	1170	1410

(Continued)

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Table 12.21B
40 mm rivets — Spacing: S_p = 25 mm; S_Q = 15 mm

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R				
		2	4	6	8	10
80	2	21	34	48	68	92
	4	31	44	60	86	114
	6	39	54	74	104	140
	8	46	64	88	120	160
	10	54	74	102	140	180
	12	62	84	116	155	205
	14	70	94	130	170	225
	16	78	104	140	190	245
	18	86	114	155	205	260
	20	92	125	165	220	290
130	2	24	34	46	66	86
	4	31	42	60	82	106
	6	39	54	74	98	130
	8	46	64	88	116	150
	10	54	74	100	130	170
	12	62	84	114	145	186
	14	70	94	125	165	205
	16	78	104	140	180	225
	18	86	114	150	195	245
	20	94	125	165	210	260
175 and greater	2	24	34	46	64	84
	4	31	42	60	80	104
	6	39	54	74	96	125

(Continued)

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Table 12.21B (Concluded)

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R				
		2	**4**	**6**	**8**	**10**
	8	46	64	86	114	145
	10	54	74	100	130	165
	12	62	84	112	145	180
	14	70	94	125	160	200
	16	78	104	140	175	220
	18	86	114	150	190	235
	20	94	125	160	205	250

(Continued)

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Table 12.21C
65 mm rivets — Spacing: S_p = 25 mm; S_Q = 25 mm

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
130	2	27	64	98	140	180	220	250	290	330	370
	4	39	84	125	175	225	270	310	350	390	440
	6	52	104	150	210	270	320	360	410	460	520
	8	66	120	180	245	310	360	410	460	520	580
	10	76	140	205	280	350	410	460	520	580	650
	12	86	155	230	310	390	450	510	570	640	720
	14	94	175	250	340	420	500	560	630	710	800
	16	98	195	280	360	460	540	600	680	770	860
	18	110	210	300	390	490	570	650	720	820	920
	20	116	230	320	420	520	610	680	770	860	960
175	2	31	74	114	160	210	250	300	340	380	430
	4	44	96	145	200	260	310	350	400	460	520
	6	60	120	175	240	300	340	390	440	530	600
	8	76	140	210	270	330	370	420	480	560	640
	10	88	165	240	300	360	410	450	510	600	690
	12	100	180	270	320	390	440	480	550	640	740
	14	110	205	290	350	420	470	510	590	680	780
	16	114	225	320	370	450	500	540	620	730	840
	18	125	245	350	400	470	530	580	660	790	870
	20	135	270	370	420	500	560	610	700	830	930

(Continued)

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Table 12.21C (Concluded)

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
215	2	35	82	125	160	205	235	270	320	410	490
	4	50	108	150	180	220	260	290	330	410	480
	6	68	135	170	200	250	280	320	370	440	510
	8	86	160	195	225	280	310	350	390	470	530
	10	100	185	215	245	300	340	370	420	500	580
	12	112	205	240	270	320	370	400	460	540	620
	14	120	225	260	290	350	390	430	490	570	650
	16	130	245	280	310	370	420	450	520	610	710
	18	145	270	290	330	390	440	480	550	660	730
	20	150	280	310	350	420	470	510	590	700	780
265	2	39	90	118	145	185	215	245	290	370	460
	4	56	112	135	160	200	235	260	300	370	440
	6	76	135	160	185	225	260	290	340	400	470
	8	96	155	180	205	250	280	310	360	430	480
	10	110	175	200	225	270	310	340	390	460	530
	12	125	195	215	245	300	330	360	420	490	570
	14	135	215	235	260	320	360	390	450	520	600
	16	145	230	250	280	340	380	410	470	560	650
	18	160	250	270	300	360	400	440	510	600	670
	20	170	260	290	320	380	430	470	540	640	720

(Continued)

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Table 12.21C (Concluded)

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
315 and greater	2	40	88	114	140	180	205	235	280	360	450
	4	60	110	130	155	195	225	250	290	360	420
	6	80	130	155	175	215	245	280	320	380	440
	8	100	150	170	195	240	270	300	340	410	460
	10	116	170	190	215	260	290	320	370	430	500
	12	130	185	205	230	280	320	350	400	470	540
	14	145	205	225	250	300	340	370	430	500	570
	16	150	220	240	270	320	360	390	450	540	620
	18	170	240	260	290	340	380	420	480	580	640
	20	175	250	270	300	360	410	450	520	620	690

(Continued)

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Table 12.21D
65 mm rivets — Spacing: S_p = 40 mm; S_Q = 25 mm

Member dimen-sion, mm*	Rivets per row n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
130	2	31	72	114	160	205	250	300	340	380	430
	4	44	96	145	200	260	310	360	400	450	510
	6	60	120	175	240	310	370	420	470	530	600
	8	74	140	205	280	350	420	480	530	590	670
	10	86	160	235	310	390	480	540	600	660	750
	12	98	180	260	350	440	520	600	660	730	830
	14	108	200	290	390	480	580	650	720	810	910
	16	112	220	320	420	520	620	700	790	880	990
	18	125	240	350	450	560	670	760	840	940	1050
	20	130	260	370	480	590	700	800	890	990	1100
175	2	35	84	130	185	240	300	350	390	440	500
	4	52	112	165	230	300	360	420	470	520	590
	6	68	140	205	280	350	430	490	540	610	690
	8	86	165	240	320	410	490	560	620	690	780
	10	100	190	270	360	460	560	630	690	770	870
	12	114	210	310	410	510	660	700	770	850	960
	14	125	235	330	450	560	670	760	840	940	1060
	16	130	260	370	480	600	730	810	910	1020	1150
	18	145	280	400	520	650	770	880	970	1090	1220
	20	155	310	430	550	680	820	930	1030	1150	1270
215	2	40	94	140	185	250	310	380	440	490	560
	4	58	125	185	230	310	380	450	520	590	660

(Continued)

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Table 12.21D (Concluded)

Member dimen-sion, mm*	Rivets per row n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
	6	76	155	220	270	360	440	530	610	690	780
	8	98	185	260	310	410	490	590	690	770	870
	10	112	210	290	340	450	540	640	770	860	970
	12	125	235	320	380	490	590	690	840	950	1080
	14	140	260	350	410	540	640	750	900	1050	1190
	16	145	290	380	440	570	690	800	960	1140	1290
	18	160	310	400	470	610	730	860	1030	1230	1370
	20	170	340	430	500	650	780	910	1100	1290	1430
265	2	44	94	130	170	230	280	350	440	550	620
	4	64	130	170	210	280	350	410	510	650	740
	6	86	160	205	245	330	400	480	590	760	860
	8	108	190	235	280	370	450	540	650	820	970
	10	125	220	260	310	420	500	590	710	890	1080
	12	140	245	290	340	450	540	630	770	970	1190
	14	155	270	320	370	490	590	690	830	1040	1270
	16	165	290	340	400	530	630	730	880	1130	1380
	18	180	320	370	430	560	670	790	950	1210	1450
	20	190	340	390	460	600	710	840	1020	1300	1550
315 and greater	2	46	92	125	160	220	270	330	420	580	650
	4	68	125	165	200	270	330	400	490	640	780
	6	90	160	195	240	320	390	470	570	730	900
	8	114	185	225	270	360	430	520	620	790	960
	10	130	210	250	300	400	480	570	680	860	1060

(Continued)

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Table 12.21D (Concluded)

Member dimen-sion, mm*	Rivets per row n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
	12	150	235	280	330	430	520	610	740	930	1150
	14	165	260	300	360	470	560	650	790	1000	1230
	16	170	280	330	380	500	600	700	840	1080	1340
	18	190	310	350	410	530	640	750	910	1160	1390
	20	200	330	370	430	570	680	800	970	1240	1490

(Continued)

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Table 12.21E
65 mm rivets — Spacing: S_p = 25 mm; S_Q = 15 mm

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R				
		2	4	6	8	10
130	2	24	36	50	72	100
	4	34	46	66	92	125
	6	42	58	80	110	150
	8	50	68	94	130	175
	10	58	80	110	150	195
	12	66	90	125	165	220
	14	76	102	140	185	240
	16	84	112	150	200	260
	18	92	125	165	220	280
	20	100	135	180	240	310
175 and greater	2	26	36	50	72	100
	4	34	46	66	92	125
	6	42	58	80	110	150
	8	50	68	94	130	170
	10	58	80	110	150	195
	12	66	90	125	165	215
	14	76	102	135	185	240
	16	84	112	150	200	260
	18	92	125	165	220	280
	20	100	135	180	235	310

(Continued)

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Table 12.21F
90 mm rivets — Spacing: S_p = 25 mm; S_Q = 25 mm

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
175	2	28	66	102	145	190	230	270	300	350	390
	4	40	86	130	180	235	280	320	360	410	460
	6	54	108	160	220	280	330	380	420	480	540
	8	68	125	190	250	320	380	430	480	540	610
	10	80	145	215	290	360	430	480	540	600	680
	12	90	165	240	320	400	470	540	600	670	750
	14	98	180	260	350	440	520	580	650	740	830
	16	104	200	290	380	480	560	620	710	800	900
	18	114	220	310	410	510	600	670	760	860	960
	20	120	240	340	430	540	630	710	810	900	1000
215	2	31	72	114	160	210	250	290	330	380	430
	4	44	96	145	200	260	310	350	400	450	510
	6	60	118	175	240	310	370	410	470	530	600
	8	76	140	205	280	350	420	470	530	590	670
	10	88	160	235	320	400	470	530	590	660	750
	12	98	180	260	350	440	520	590	660	730	830
	14	108	200	290	390	480	570	640	720	810	910
	16	114	220	320	420	520	620	690	780	880	990
	18	125	240	340	450	560	660	740	830	950	1050
	20	135	260	370	480	590	690	780	890	990	1100

(Continued)

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Table 12.21F (Concluded)

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
265	2	34	80	125	180	235	280	330	370	420	480
	4	50	106	160	225	290	340	390	450	500	570
	6	66	130	195	270	340	410	460	520	590	670
	8	84	155	230	310	390	470	530	590	660	750
	10	98	180	260	350	440	500	550	610	700	800
	12	110	200	290	390	470	520	550	630	720	820
	14	120	225	320	410	490	540	570	650	740	840
	16	125	245	350	430	500	550	590	660	780	880
	18	140	270	380	440	520	570	610	690	810	890
	20	150	290	410	460	540	590	640	720	850	930
315	2	38	90	140	195	260	310	360	410	470	530
	4	54	118	175	245	320	380	430	490	560	630
	6	74	145	215	300	380	420	470	530	630	720
	8	94	170	250	320	390	430	470	520	610	680
	10	108	200	280	330	400	440	470	530	610	700
	12	120	220	310	340	410	450	480	540	630	720
	14	135	245	320	360	420	470	500	560	650	730
	16	140	270	340	370	440	480	510	580	680	770
	18	155	300	360	390	450	490	530	600	710	780
	20	165	320	370	400	470	510	560	630	740	820

(Continued)

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Table 12.21F (Concluded)

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
365 and greater	2	40	96	150	210	280	330	390	450	510	570
	4	60	125	190	270	340	410	450	520	600	680
	6	80	155	225	290	360	400	440	500	590	670
	8	100	180	250	300	360	400	430	490	570	640
	10	116	205	270	310	370	410	440	490	570	650
	12	130	230	290	320	380	420	450	510	590	670
	14	145	250	310	340	400	440	460	520	600	690
	16	150	280	320	350	410	450	480	540	630	720
	18	165	300	330	360	420	460	500	560	670	740
	20	175	310	350	370	440	480	520	590	700	770

(Continued)

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Table 12.21G
90 mm rivets — Spacing: S_p = 40 mm; S_Q = 25 mm

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
175	2	32	76	118	165	215	270	310	350	390	450
	4	46	100	150	205	270	330	380	420	470	530
	6	62	125	180	250	320	390	440	490	550	620
	8	78	145	215	290	370	440	510	550	620	700
	10	90	170	245	330	410	500	560	620	690	780
	12	102	190	270	370	460	550	630	690	760	860
	14	112	210	300	400	500	600	680	750	840	950
	16	118	230	330	430	540	650	730	820	910	1030
	18	130	250	360	470	580	700	790	870	980	1090
	20	140	280	390	500	610	730	830	930	1030	1150
215	2	35	84	130	180	235	290	340	390	430	490
	4	50	110	165	230	290	360	410	460	520	580
	6	68	135	200	270	350	430	490	540	600	680
	8	86	160	240	320	400	480	560	610	680	770
	10	100	185	270	360	450	550	620	680	760	860
	12	112	205	300	400	500	600	690	760	840	950
	14	120	230	330	440	550	660	750	830	930	1050
	16	130	250	360	480	600	720	800	900	1000	1130
	18	145	280	400	510	640	760	870	960	1080	1200
	20	150	300	420	540	670	810	910	1020	1130	1260

(Continued)

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Table 12.21G (Continued)

Member dimen-sion, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
265	2	39	94	145	205	260	330	380	430	480	550
	4	56	125	185	250	330	400	460	510	580	650
	6	76	150	225	310	390	470	540	600	670	760
	8	96	180	260	350	450	540	620	680	760	860
	10	110	210	300	400	510	610	690	760	850	960
	12	125	230	340	450	560	670	770	850	940	1060
	14	135	260	370	500	610	740	840	930	1030	1170
	16	145	280	410	530	670	800	900	1010	1120	1270
	18	160	310	440	570	710	850	970	1070	1210	1340
	20	170	340	470	600	750	900	1020	1140	1260	1410
315	2	42	102	160	225	290	360	420	470	530	600
	4	62	135	205	280	360	440	510	570	630	720
	6	84	170	245	340	430	520	600	660	740	840
	8	106	200	290	390	490	600	680	750	830	940
	10	120	230	330	420	540	630	740	840	930	1050
	12	140	250	370	440	560	660	760	910	1030	1170
	14	150	280	400	460	590	700	790	950	1140	1290
	16	160	310	420	480	610	720	830	990	1240	1400
	18	175	340	440	500	640	750	870	1040	1320	1480
	20	185	370	460	520	670	790	920	1100	1390	1550

(Continued)

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Table 12.21G (Concluded)

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
365 and greater	2	46	102	175	240	310	390	460	510	580	650
	4	68	145	220	300	390	480	550	610	690	780
	6	90	180	270	350	460	550	640	710	800	910
	8	114	215	310	370	480	560	660	780	900	1020
	10	130	250	340	390	500	590	690	810	1010	1140
	12	150	270	360	410	530	620	710	850	1060	1260
	14	165	310	380	430	550	650	750	890	1100	1340
	16	170	340	400	450	580	680	780	930	1170	1430
	18	190	360	410	470	600	700	820	980	1240	1470
	20	200	380	430	490	630	740	860	1030	1310	1550

(Continued)

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Table 12.21H
90 mm rivets — Spacing: S_p = 25 mm; S_Q = 15 mm

Member dimension, mm*	Rivets per row, n_C	No. of rows, n_R				
		2	4	6	8	10
175 and greater	2	25	36	52	74	100
	4	34	46	66	92	125
	6	42	58	80	112	125
	8	50	70	96	130	175
	10	60	80	110	150	200
	12	68	92	125	170	220
	14	76	102	140	185	245
	16	84	114	155	205	270
	18	94	125	165	225	290
	20	102	135	180	240	310

* *The member dimension is identified as b in Figure 12.12 for connections with steel plates on opposite sides. For connections having only one plate, the member dimension is twice the thickness of the wood member.*

Note: *For intermediate sawn lumber dimensions, interpolation may be used.*

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Δ

Table 12.22
Values of q_w, kN, perpendicular to grain for timber rivet joints — Spacing: S_P = 25 mm
(See Clauses 12.7.3.5 and A.12.7.3.3.1.)

		No. of rows, n_R							
S_Q, mm	Rivets per row, n_C	1	2	3	4	5	6	8	10
15	2	0.57	0.57	0.61	0.61	0.67	0.71	0.84	0.97
	3	0.57	0.57	0.61	0.63	0.66	0.70	0.82	0.93
	4	0.60	0.60	0.65	0.66	0.71	0.74	0.85	0.95
	5	0.63	0.63	0.69	0.70	0.75	0.78	0.89	0.99
	6	0.71	0.71	0.76	0.77	0.81	0.84	0.95	1.06
	7	0.77	0.77	0.82	0.82	0.87	0.89	1.00	1.11
	8	0.86	0.86	0.90	0.90	0.94	0.96	1.07	1.18
	9	0.91	0.91	0.97	0.97	1.01	1.03	1.13	1.25
	10	0.97	0.97	1.05	1.06	1.10	1.12	1.21	1.35
	11	1.05	1.05	1.12	1.13	1.17	1.18	1.28	1.43
	12	1.14	1.14	1.21	1.21	1.24	1.25	1.38	1.52
	13	1.26	1.26	1.29	1.29	1.33	1.33	1.45	1.59
	14	1.42	1.42	1.40	1.37	1.42	1.44	1.54	1.68
	15	1.50	1.50	1.50	1.47	1.50	1.50	1.62	1.78
	16	1.61	1.61	1.62	1.60	1.60	1.58	1.71	1.89
	17	1.73	1.73	1.72	1.69	1.69	1.67	1.79	1.96
	18	1.88	1.88	1.85	1.80	1.80	1.77	1.87	2.04
	20	1.84	1.84	1.91	1.91	1.93	1.93	2.08	2.24
25	2	0.67	0.67	0.70	0.69	0.75	0.79	0.93	1.08
	3	0.66	0.66	0.70	0.72	0.74	0.78	0.91	1.03
	4	0.70	0.70	0.75	0.75	0.80	0.83	0.94	1.06
	5	0.73	0.73	0.80	0.79	0.84	0.87	0.98	1.10
	6	0.82	0.82	0.87	0.86	0.91	0.94	1.05	1.18
	7	0.90	0.90	0.94	0.93	0.97	1.00	1.11	1.23
	8	1.01	1.01	1.04	1.02	1.06	1.08	1.19	1.31
	9	1.06	1.06	1.11	1.10	1.14	1.15	1.26	1.39
	10	1.13	1.13	1.20	1.20	1.24	1.25	1.34	1.50
	11	1.22	1.22	1.29	1.28	1.30	1.32	1.43	1.59
	12	1.33	1.33	1.39	1.37	1.39	1.40	1.53	1.69
	13	1.47	1.47	1.48	1.45	1.48	1.49	1.61	1.77

(Continued)

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Table 12.22 (Concluded)

		No. of rows, n_R							
S_Q, mm	Rivets per row, n_C	1	2	3	4	5	6	8	10
	14	1.65	1.65	1.61	1.55	1.59	1.61	1.71	1.86
	15	1.75	1.75	1.72	1.66	1.68	1.68	1.80	1.97
	16	1.87	1.87	1.86	1.80	1.79	1.77	1.90	2.10
	17	2.02	2.02	1.97	1.91	1.89	1.87	1.98	2.18
	18	2.19	2.19	2.12	2.03	2.01	1.98	2.08	2.27
	20	2.14	2.14	2.19	2.15	2.17	2.16	2.31	2.49
40	2	0.96	0.96	0.98	0.93	0.98	1.03	1.20	1.38
	3	0.95	0.95	0.98	0.96	0.98	1.02	1.16	1.32
	4	1.02	1.02	1.05	1.00	1.05	1.07	1.21	1.36
	5	1.06	1.06	1.11	1.06	1.11	1.13	1.26	1.41
	6	1.19	1.19	1.22	1.16	1.20	1.22	1.35	1.51
	7	1.30	1.30	1.32	1.24	1.28	1.30	1.43	1.58
	8	1.45	1.45	1.45	1.36	1.40	1.40	1.53	1.68
	9	1.53	1.53	1.55	1.47	1.50	1.50	1.61	1.79
	10	1.63	1.63	1.69	1.60	1.63	1.63	1.72	1.93
	11	1.76	1.76	1.80	1.71	1.72	1.71	1.83	2.04
	12	1.92	1.92	1.94	1.83	1.84	1.82	1.96	2.17
	13	2.12	2.12	2.08	1.94	1.96	1.94	2.07	2.27
	14	2.39	2.39	2.25	2.07	2.10	2.09	2.20	2.39
	15	2.53	2.53	2.41	2.22	2.22	2.19	2.31	2.53
	16	2.70	2.70	2.61	2.41	2.36	2.30	2.44	2.69
	17	2.91	2.91	2.76	2.55	2.49	2.43	2.54	2.79
	18	3.17	3.17	2.97	2.72	2.66	2.58	2.66	2.91
	20	3.10	3.10	3.07	2.88	2.86	2.80	2.96	3.19

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Table 12.23
Values of factor C_t
(See Clause 12.7.3.5.)

$e_p/[(n_c-1)s_Q]$	C_t	$e_p/[(n_c-1)s_Q]$	C_t
0.1	5.76	3.2	0.79
0.2	3.19	3.6	0.77
0.3	2.36	4.0	0.76
0.4	2.00	5.0	0.72
0.5	1.77	6.0	0.70
0.6	1.61	7.0	0.68
0.7	1.47	8.0	0.66
0.8	1.36	9.0	0.64
0.9	1.28	10.0	0.63
1.0	1.20	12.0	0.61
1.2	1.10	14.0	0.59
1.4	1.02	16.0	0.57
1.6	0.96	18.0	0.56
1.8	0.92	20.0	0.55
2.0	0.89	25.0	0.53
2.4	0.85	30.0	0.51
2.8	0.81		

12.8 Truss plates

12.8.1 General

12.8.1.1 Materials

The design requirements specified in Clause 12.8 apply to truss plate connections using light-gauge metal plates that

a) depend on extended teeth or nails embedded into the wood to transfer load;
b) meet the requirements of Clause 16.4; and
c) have been tested in accordance with Clause 16.4.2 for the species group with which the plates are to be used.

Notes:

1) *Test results for such plates are listed in the* Registry of Product Evaluations *published by the Canadian Construction Materials Centre at NRC, Ottawa, Ontario.*
2) *The provisions of Clause 12.8 are predicated on truss fabrication and erection in accordance with recognized practices such as those referenced by TPIC.*

12.8.1.2 Limitations

Clause 12.8 does not apply to

a) truss plates under corrosive conditions; and

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b) the use of galvanized truss plates in lumber that has been treated with a fire retardant and is used in wet-service conditions or in locations prone to condensation.

12.8.1.3 Placement of plates in the joints

Connection design shall be based on tight-fitted joints with truss plates placed on opposing faces in such a way that, at each joint, the plates on opposing faces are identical and are placed directly opposite each other.

12.8.1.4 Nail-on plates

In cases where nail-on plates are used, the word "nail" should be read in place of "tooth" in Clause 12.8.

12.8.1.5 Design assumptions

The design criteria for truss plates in Clause 12.8 are based on the following conditions:

a) the plate is prevented from deforming during installation;
b) the teeth are normal to the surface of the lumber;
c) the tooth penetration in connections is not less than that used in the testing specified in Clause 16.4.2; and
d) the lumber beneath the plate complies with Appendix G of *TPIC*.

12.8.1.6 Minimum member thickness

The thickness of members used in connections shall not be less than twice the tooth penetration.

12.8.1.7 Primary axis of a truss plate

The primary axis of a truss plate, in the case of slotted truss plate tooth configurations, shall be parallel to the direction of slots in the plate. For rosette-style configurations, the primary axis shall be that axis of symmetry in which the tensile strength of the truss plate is the greatest.

12.8.2 Design

12.8.2.1 Design requirements

Truss plate connections shall be designed to meet the following requirements:

a) For the strength limit state, the effect of factored loads shall be less than or equal to the factored
   i) ultimate lateral resistance of the teeth;
   ii) tensile resistance of the plates; and
   iii) shear resistance of the plates.
b) For the serviceability limit state, the effect of specified loads shall be less than or equal to the lateral slip resistance of the teeth.

12.8.2.2 Compression loads

Truss plates shall not be considered effective in transferring compression loads at a joint.

12.8.2.3 Design considerations

The design of truss plate connections shall take the following into consideration:

a) species of lumber;
b) orientation of plates relative to the applied load (Figure 12.14, angle ρ);
c) direction of the applied load relative to the grain (Figure 12.14, angle θ); and
d) orientation of plates relative to the applied shear force.

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12.8.2.4 Ultimate lateral resistance and lateral slip resistance of the teeth

The factored ultimate lateral resistance and the lateral slip resistance of the teeth shall be expressed in terms of the surface area of the plates.

12.8.2.5 Surface area

Surface area shall be based on the net area method using the test values, or on the gross area method using 80% of the test values, where the

a) gross area is defined as the total area of a member covered by a truss plate; and
b) net area is defined as the total area of a member covered by a truss plate less the area within a given distance from the edge or end of the member, as shown in Figure 12.15. For net area calculation, the minimum end distance, *a*, measured parallel to grain, shall be the greater of 12 mm or one-half the length of the tooth; the minimum edge distance, *e*, measured perpendicular to grain, shall be the greater of 6 mm or one-quarter the length of the tooth.

12.8.2.6 Tensile and shear resistance of the plates

The factored tensile resistance of the plates shall be expressed in terms of the dimension of the plate measured perpendicular to the line of action of the applied forces. The factored shear resistance shall be expressed in terms of the dimension of the plate measured along the line of action of the shearing forces.

Figure 12.14
Truss plate, load, and grain orientation
(See Clauses 12.8.2.3, 12.8.3.2, and 12.8.4.2.)

Legend:

θ = angle between the load direction and the grain direction

ρ = angle between the load direction and the primary axis of the plate

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Figure 12.15
End and edge distances for truss plates
(See Clause 12.8.2.5.)

Legend:

a = end distance

e = edge distance

12.8.3 Factored resistance of truss plates

12.8.3.1 General

For the strength limit state, the factored resistances of truss plates shall be calculated as follows:

a) for the factored ultimate lateral resistance of the teeth:

$N_r = \phi\ N_u\ J_H$

where

ϕ = 0.9

N_u = n_u (K_D K_{SF} K_T)

where

n_u = ultimate lateral resistance of the teeth, determined in accordance with the manufacturer's product evaluation report (see Clause 12.8.3.2)

J_H = moment factor for heel connection (see Table 12.24)

b) for the factored tensile resistance of the plate:

$T_r = \phi\ t_p$

where

ϕ = 0.6

t_p = tensile resistance of the plate, determined in accordance with the manufacturer's product evaluation report

c) for the factored shear resistance of the plate:

$V_r = \phi\ v_p$

where

ϕ = 0.6

v_p = shear resistance of the plate, determined in accordance with the manufacturer's product evaluation report

The shear resistance values for the tensile mode of failure shall be used where the applied shear forces create tension in the plate. Where the applied forces create compression in the plate, the compressive shear values shall be used. Alternatively, the lower of the two shear resistances for an angle may be used.

Table 12.24
Moment factor for heel joints of pitched trusses, J_H
(See Clause 12.8.3.1.)

Slope of top chord	J_H
< 1/4	0.85
> 1/4 to 1/3	0.8
> 1/3 to 1/2.4	0.75
> 1/2.4 to 1/2.2	0.7
> 1/2.2	0.65

12.8.3.2 Ultimate lateral resistance of the teeth

The ultimate lateral resistance of the teeth shall be calculated as follows:

a) for loads parallel to the primary axis of the plate:

$$n_u = \frac{p_u q_u}{p_u \sin^2\theta + q_u \cos^2\theta}$$

b) for loads perpendicular to the primary axis of the plate:

$$n_u' = \frac{p_u' q_u'}{p_u' \sin^2\theta + q_u' \cos^2\theta}$$

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where

p_u, q_u, p'_u, q'_u = ultimate lateral resistances determined in accordance with the manufacturer's product evaluation report and used with the following values of θ and ρ:

	θ	ρ
p_u	0°	0°
q_u	90°	0°
p'_u	0°	90°
q'_u	90°	90°

Note: *θ and ρ are as shown in Figure 12.14.*

When the primary axis of the plate is oriented at an angle other than parallel or perpendicular to the direction of the load, the resistance value shall be determined by linear interpolation between the values n_u and n'_u.

12.8.4 Lateral slip resistance

12.8.4.1 Adjusted lateral slip resistance of the teeth

For the serviceability limit state, the adjusted lateral slip resistance of the teeth, N_{rs}, shall be calculated as follows:

$N_{rs} = N_s K_{SF}$

where

N_s = lateral slip resistance of the teeth (see Clause 12.8.4.2)

Δ **12.8.4.2 Lateral slip resistance of the teeth**

The lateral slip resistance of the teeth shall be calculated as follows:

a) for loads parallel to the primary axis of the plate:

$$N_s = \frac{p_s q_s}{p_s \sin^2\theta + q_s \cos^2\theta}$$

b) for loads perpendicular to the primary axis of the plate:

$$N'_s = \frac{p'_s q'_s}{p'_s \sin^2\theta + q'_s \cos^2\theta}$$

where

p_s, q_s, p'_s, q'_s = lateral slip resistances determined in accordance with the manufacturer's product evaluation report and used with the following values of θ and ρ:

	θ	ρ
p_s	0°	0°
q_s	90°	0°
p'_s	0°	90°
q'_s	90°	90°

Note: *θ and ρ are as shown in Figure 12.14.*

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When the primary axis of the plate is oriented at an angle other than parallel or perpendicular to the direction of the load, the resistance value shall be determined by linear interpolation between the values N_s and N'_s.

12.9 Nails and spikes

12.9.1 General

The design requirements specified in Clause 12.9 apply only to common round steel wire nails and spikes and common spiral nails spiraled to head as defined in CSA B111 or ASTM F1667, including power-tool driven common nails.

Note: *See Tables A.21 and A.22 for commonly available nail and spike sizes.*

12.9.2 Connection configuration

12.9.2.1 Placement of nails and spikes in side grain

For connections nailed at 10% wood moisture content or greater, the minimum spacing of fasteners in sawn lumber side plates and main members, including those in the panel face of CLT, shall be as specified in Table 12.25. Additional fasteners may be staggered on the intersection of diagonal lines drawn between rows of nails (see Figure 12.16).

Note: *When the moisture content of the wood is expected to be less than 10% at time of fabrication, the spacing should be increased and holes should be predrilled to avoid splitting.*

Table 12.25
Placement of nails and spikes in side grain
(See Clause 12.9.2.1 and Figure 12.16.)

	Minimum spacing	
Dimension*	**Douglas Fir-Larch, Hem-Fir, and Western Red Cedar**	**Spruce-Pine-Fir and Northern Species**
S_P — Spacing parallel to grain	$20d_F$	$16d_F$
S_Q — Spacing perpendicular to grain	$10d_F$	$8d_F$
a — End distance parallel to grain	$15d_F$	$12d_F$
e — Edge distance perpendicular to grain	$5d_F$	$4d_F$

* d_F =*nominal diameter of fastener.*

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Δ

Figure 12.16
Placement of nails and spikes in wood-to-wood connections
(See Clause 12.9.2.1.)

12.9.2.2 Penetration length and member thickness

Penetration length and member thicknesses for wood-to-wood and steel-to-wood connections shall be as shown in Figure 12.17.

12.9.2.3 Placement of nails and spikes in the panel edge of CLT

In a group of fasteners installed in the panel edge of CLT, the minimum spacing of fasteners shall be as specified in Table 12.26 (see Figure 12.7):

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Table 12.26
Placement of nails and spikes in the panel edge of CLT
(See Clause 12.9.2.3 and Figure 12.7.)

Symbol	Dimension*	Minimum spacing
S_R	Spacing parallel to the load direction	$10d_F$
S_C	Spacing perpendicular to the load direction	$4d_F$
a	Unloaded end distance	$7d_F$
a_L	Loaded end distance	$12d_F$
e_P	Unloaded edge distance	$3d_F$
e_Q	Loaded edge distance	$6d_F$

* d_F =*nominal diameter of fastener* .

Figure 12.17
Penetration length and member thickness
(See Clauses 11.3.2.3, 12.9.2.2, and 12.11.2.4.)

Note: d_F = *nominal diameter of fastener.*

12.9.3 Lateral resistance

Δ 12.9.3.1 General

The factored lateral strength resistance of a connection with nails or spikes, N_r, shall be greater than or equal to the effect of the factored loads, and shall be calculated as follows:

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$N_r = \phi\ N_u\ n_F\ n_S\ J_F$

where

ϕ = 0.8

N_u = $n_u\ (K_D\ K_{SF}\ K_T)$

where

n_u = unit lateral strength resistance per shear plane, N (see Clause 12.9.3.2)

n_F = number of fasteners in the connection

n_S = number of shear planes per fastener

$J_F = J_E\ J_A\ J_B\ J_D$

where

J_E = end grain factor

= 0.67 for nailing into end grain

= 1.0 in all other cases

J_A = toe-nailing factor

= 0.83 for toe-nailing, where toe-nails are started at approximately one-third of the nail length from the end of the piece and driven at an angle of about 30° to the grain of the member

= 1.00 for cases other than toe-nailing

J_B = nail clinching factor

= 1.6 for nail clinching on the far side in a two-member connection

= 1.0 if not clinched or in three-or-more member connections

J_D = factor for diaphragm and shearwall construction

= 1.3 for fasteners used in diaphragm and shearwall construction

= 1.0 in all other cases

Δ 12.9.3.2 Unit lateral strength resistance

The unit lateral strength resistance per shear plane, n_u, shall be taken as the smallest value calculated in accordance with Items a) to g). For two-member connections, only Items a), b), and d) to g) shall be considered valid. For three-member connections, only Items a), c), d), and g) shall be considered valid.

a) $f_1 d_F t_1$

b) $f_2 d_F t_2$

c) $\frac{1}{2} f_2 d_F t_2$

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d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

f) $f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$

g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$

where

d_F = nominal diameter of fastener, mm

t_1 = head-side member thickness in two-member connections, mm

= head-side member thickness or length of penetration into point-side member, whichever is less, in three-member connections, mm (see Clause 12.9.2.2)

t_2 = length of penetration into point-side (main) member in two-member connections, mm

= central member thickness in three-member connections, mm (see Clause 12.9.2.2)

f_1 = embedment strength of side member, MPa

f_2 = embedment strength of point-side (main) member for two-member connections for yield modes b) and f), MPa

= embedment strength of central member in three-member connections for yield mode c), MPa

f_3 = embedment strength of point-side wood member for two-member connections for yield modes d), e), and g), MPa

= embedment strength of central member in three-member connections for yield modes d) and g), MPa

For wood members:

f_1 = $f_2 = 50\ G\ (1 - 0.01 d_F)\ J_x$

f_3 = $110\ G^{1.8}\ (1 - 0.01 d_F)\ J_x$

where

G = mean relative density

Note: *Mean relative density values for wood members (lumber, glulam, and CLT) are provided in Table A.11.*

J_x = 0.9 for CLT

= 1.0 for other wood members

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For structural panels:

$f_1 = f_2 = 104\ G\ (1 - 0.1 d_F)$

where

G = 0.49 for DFP

= 0.42 for CSP and OSB

For steel:

$f_1 = f_2 = K_{sp}\ (\phi_{steel}/\phi_{wood})\ f_u$

where

K_{sp} = 3.0 for mild steel referenced in CSA S16

= 2.7 for cold-formed light-gauge steel referenced in CSA S136

ϕ_{steel} = resistance factor for steel members in connections with nails and spikes

= 0.80 for mild steel referenced in CSA S16

= 0.4 for cold-formed light-gauge steel referenced in CSA S136

ϕ_{wood} = resistance factor for wood members in connections with nails and spikes

= 0.8

f_u = specified minimum tensile strength of steel

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g.,*

a) *f_u = 400 MPa for ASTM A36/A36M steel;*

b) *f_u = 450 MPa for CSA G40.21 steel, Grades 300W and 350W; and*

c) *f_u = 310 MPa for cold-formed light-gauge steel, Grade SS 230.*

f_y = yield strength of fastener in bending, MPa

= $50\ (16 - d_F)$

12.9.3.3 Connection deformation

Where required for design, the lateral deformation of wood-to-wood connections with nails and spikes, including structural panel-to-lumber connections, may be estimated in accordance with Clause A.12.9.3.3.

12.9.4 Withdrawal resistance

12.9.4.1 Load duration

Nails and spikes loaded in withdrawal shall only be used for short-term loading.

12.9.4.2 Withdrawal from the side grain

The factored withdrawal resistance of a connection with nails or spikes, P_{rw}, driven into the side grain shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

$P_{rw} = \phi\ Y_w\ L_p\ n_F\ J_A\ J_B$

where

ϕ = 0.6

Y_w = $y_w\ (K_{SF}\ K_T)$

where

y_w = specified withdrawal resistance per millimetre of penetration into point-side member, N/mm

= $16.4 d_F^{0.82} G^{2.2} J_x$

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where

d_F = nominal diameter of fastener, mm

G = mean relative density (see Table A.11)

J_x = 0.9 for CLT

= 1.0 in all other cases

L_p = length of penetration into point-side member, mm

n_F = number of fasteners in the connection

J_A = toe-nailing factor

= 0.67 for toe-nailing

= 1.00 for cases other than toe-nailing

J_B = nail-clinching factor

= 1.6 for nail-clinching in the point-side member of a two-member connection

= 1.0 if not clinched or in three-or-more member connections

12.9.4.3 Withdrawal from the end grain

Nails and spikes driven into the end grain shall not be considered to carry load in withdrawal. Where designs rely on withdrawal resistance of fasteners in the panel edge of CLT, precaution shall be taken to ensure that side grain penetration occurs.

12.10 Joist hangers

12.10.1 General

12.10.1.1 Materials

The design requirements specified in Clause 12.10 apply to proprietary mass-produced metal devices, usually cold-formed from light-gauge steel or welded from steel plate, that are used to transfer loads from joists to headers or beams and have been tested in accordance with Clause 16.5. The joists and headers or beams may be sawn lumber, wood trusses, glued-laminated timber, prefabricated wood I-joists, or SCL.

12.10.1.2 Limitations

The provisions of Clause 12.10 do not apply under the following conditions:

a) corrosive conditions;
b) the use of galvanized joist hangers in lumber that has been treated with a fire retardant and is used in wet-service conditions or conditions prone to condensation;
c) joist hangers connected to headers or beams of other than wood-based materials; and
d) special hangers having a skew in the horizontal or vertical plane (except skewed hangers with level bearing seats).

12.10.1.3 Construction requirements

Joist hangers shall be constructed to meet the following requirements:

a) The height of the hanger shall be at least half the depth of the joist and be capable of providing lateral support for the joist unless the joist is prevented from twisting by other means.

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b) The hanger shall be fastened to both the joist and the header or beam. Where nails or wood screws are used, the size and spacing shall prevent splitting of the wood. Where bolts are used, the spacing shall meet the requirements of Clause 12.4 for bolts and Clause 12.6 for lag screws.
c) Hangers used to support prefabricated wood I-joists that do not require bearing stiffeners shall be high enough to provide lateral stability to the top flange of the joist.
d) Where a prefabricated wood I-joist is the header, backer blocks shall be provided between the web and face mount hangers.
e) Where a prefabricated wood I-joist is supporting a top mount hanger, filler blocks shall be used between the top and bottom flange of the I-joist. The blocks shall be tight to the bottom of the top flange.

12.10.2 Design

12.10.2.1 Factored resistance

The factored resistance, N_r, of joist hangers shall be greater than or equal to the effect of the factored loads, and shall be calculated as follows:

$N_r = \phi N_u$

where

ϕ = 0.6

N_u = $n_u (K_D K_{SF} K_T)$

where

n_u = ultimate resistance of the hanger

K_D = load-duration factor determined in accordance with Clause 5.3.2, except that there shall not be an increase for short-term load duration where the ultimate resistance is determined by the strength of the steel

Note: *In general, proprietary design values are published by the product manufacturer (i.e., proprietary design literature and CCMC Evaluation Reports in the CCMC* Registry of Product Evaluations*, published by the Canadian Construction Materials Centre) with appropriate factors for specific applications.*

12.10.2.2 Shear resistance of the header or beam

For joist hangers attached to the face of a header or beam, the shear resistance of the header or beam shall be checked in accordance with Clause 12.2.1.6.

12.11 Wood screws

12.11.1 General

The design requirements specified in Clause 12.11 apply to connections with wood screws that meet the requirements of ASME B18.6.1. Nominal diameters and minimum design yield strengths are given in Table 12.27.

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Table 12.27
Diameter and minimum yield strength of wood screws
(See Clauses 12.11.1, 12.11.3.2, and 12.11.4.2.)

	Nominal size*			
	6	8	10	12
Nominal diameter, d_F, mm†	3.50	4.16	4.82	5.48
Minimum yield strength, f_y, MPa‡	690	620	550	550

* *For wood screws with a nominal size greater than 12, design in accordance with the requirements for lag screws specified in Clause 12.6.*
† *Outside thread diameter.*
‡ *Linear interpolation for yield strength may be used.*

12.11.2 Connection configuration

Δ **12.11.2.1 Lead holes**

Lead holes at least the length of the threaded portions of shanks shall be provided in a wood screw connection except when the relative density, G, of wood members is less than 0.50. Lead hole requirements are dependent on type of loading, relative density, G, shank diameter, d_S, and root diameter, d_R, and are given in Table 12.28.

Table 12.28
Lead hole diameter requirements
(See Clause 12.11.2.1.)

	Relative density	
Loading	**$0.5 \le G \le 0.6$**	**$G > 0.6$**
Lateral	$0.85d_S$	d_S
Withdrawal	$0.7d_R$	$0.9d_R$

12.11.2.2 Placement of wood screws in side grain

The minimum spacing of fasteners in connections with wood screws shall be in accordance with Clause 12.9.2.1.

12.11.2.3 Placement of wood screws in the panel edge of CLT

In a group of wood screws installed in the panel edge of CLT, the minimum spacing of fasteners shall be in accordance with Clause 12.9.2.3.

12.11.2.4 Penetration length and member thicknesses

Penetration length and member thickness for wood-to-wood and steel-to-wood connections shall be as shown in Figure 12.17.

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12.11.3 Lateral resistance

12.11.3.1 Factored lateral strength resistance

The factored lateral strength resistance of a connection with wood screws, N_r, shall be greater than or equal to the effect of the factored loads and shall be calculated as follows:

$N_r = \phi\ N_u\ n_F\ n_s\ J_A\ J_E$

where

ϕ = 0.8

N_u = $n_u\ (K_D\ K_{SF}\ K_T)$

where

n_u = unit lateral strength resistance per shear plane, N (see Clause 12.11.3.2)

n_F = number of fasteners in the connection

n_s = number of shear planes per fastener

J_A = toe-screwing factor

= 0.83 where screws are started at approximately one-third the screw length from the end of a piece and driven at an angle of about 30° to the grain of the member

= 1.0 in all other cases

J_E = factor for fastening into end grain

= 0.67 in end grain

= 1.0 in all other cases

Δ **12.11.3.2 Unit lateral strength resistance**

The unit lateral strength resistance per shear plane, n_u, shall be taken as the smallest value calculated in accordance with Items a) to g). For two-member connections, only Items a), b), d) to f), and g) are valid. For three-member connections, only Items a), c), d), and g) are valid.

a) $f_1 d_F t_1$

b) $f_2 d_F t_2$

c) $\frac{1}{2} f_2 d_F t_2$

d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

e) $$f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$$

f) $$f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$$

g) $$f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$$

where

d_F = nominal diameter of fastener, mm (see Table 12.27)

t_1 = head-side member thickness in two-member connections, mm

= head-side member thickness or length of penetration into point-side member, whichever is less, in three-member connections, mm (see Clause 12.11.2.4)

t_2 = length of penetration into point-side (main) member in two-member connections, mm

= central member thickness in three-member connections, mm (see Clause 12.11.2.4)

f_1 = embedment strength of side member, MPa

f_2 = embedment strength of point-side (main) member in two-member connections for yield modes b) and f), MPa

= embedment strength of central member in three-member connections for yield mode c), MPa

f_3 = embedment strength of point-side wood member for two-member connections for yield modes d), e), and g), MPa

= embedment strength of central member in three-member connections for yield modes d) and g), MPa

For wood members:

f_1 = $f_2 = 50G\,(1 - 0.01d_F)\,J_x$

f_3 = $110\,G^{1.8}\,(1 - 0.01d_F)\,J_x$

where

G = mean relative density

Note: *Mean relative density values for wood members (lumber, glulam, and CLT) are provided in Table A.11.*

J_x = 0.9 for CLT

= 1.0 for other wood members

For structural panels:

f_1 = $f_2 = 104\,G\,(1 - 0.1d_F)$

where

G = 0.49 for DFP

= 0.42 for CSP and OSB

For steel:

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$f_1 = f_2 = K_{sp}\,(\phi_{steel}/\phi_{wood})\,f_u$

where

K_{sp} = 3.0 for mild steel referenced in CSA S16

= 2.7 for cold-formed light-gauge steel referenced in CSA S136

ϕ_{steel} = resistance factor for steel members in connections with wood screws

= 0.8 for mild steel referenced in CSA S16

= 0.4 for cold-formed light-gauge steel referenced in CSA S136

ϕ_{wood} = resistance factor for wood members in connections with wood screws

= 0.8

f_u = specified minimum tensile strength of steel

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g., for*

a) *f_u = 400 MPa for ASTM A36/A36M steel;*

b) *f_u = 450 MPa for CSA G40.21 steel, Grades 300W and 350W; and*

c) *f_u = 310 MPa for cold-formed light-gauge steel, Grade SS 230.*

f_y = yield strength of fastener in bending, MPa (see Table 12.27)

12.11.3.3 Connection deformation

Where required for design, the lateral deformation of wood-to-wood connections with wood screws, including structural panel-to-lumber connections, may be estimated in accordance with Clause A.12.9.3.3.

12.11.4 Withdrawal resistance

12.11.4.1 General

The factored withdrawal resistance of a connection with wood screws shall be greater than or equal to the effect of the factored loads and shall be taken as the lesser of the factored screw withdrawal resistance of the point-side member (see Clause 12.11.4.2) and the factored head pull-through resistance of the head-side member (see Clause 12.11.4.3).

12.11.4.2 Withdrawal resistance of point-side member

For a two-member connection with wood screws, the factored withdrawal resistance, P_{rw}, in N, of the point-side member shall be calculated as follows:

$P_{rw} = \phi\, Y_w\, L_{pt}\, n_F$

where

ϕ = 0.6

Y_w = $y_w\,(K_D\, K_T\, K_{SF})$

where

y_w = specified withdrawal resistance per millimetre of threaded shank penetration in point-side member, N/mm

= $59 d_F^{0.82} G^{1.77} J_x$

where

d_F = nominal diameter of fastener, mm (see Table 12.27)

G = mean relative density of point-side member (see Table A.11)

J_x = 0.9 for CLT

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= 1.0 in all other cases

L_{pt} = threaded length penetration in the point-side member, mm

n_F = number of fasteners in the connection

For a connection with three members, the threaded length penetration shall be the greater threaded length within any member other than the head-side member.

Δ **12.11.4.3 Head pull-through resistance of head-side member**

The factored head pull-through resistance, P_{pt}, in N, of a connection with wood screws shall be calculated as follows:

a) for connections with steel side plates:
$P_{pt} = 1.5\phi\ t_1\ d_w\ f_u\ n_F$

b) for connections with lumber, glulam, CLT, or structural panel head-side members:
$P_{pt} = 65\phi\ t_1\ n_F\ K_D$

where

ϕ = 0.4

t_1 = thickness of head-side member, mm

d_w = diameter of screw head, mm

f_u = specified minimum tensile strength of steel, MPa

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g., for*

a) *f_u = 400 MPa for ASTM A36/A36M steel;*
b) *f_u = 450 MPa for CSA G40.21 steel, Grades 300W and 350W; and*
c) *f_u = 310 MPa for cold-formed light-gauge steel, Grade SS 230.*

n_F = number of fasteners in the connection

12.11.4.4 Withdrawal from the end grain

Wood screws installed through the end grain shall not be considered to carry load in withdrawal. Where designs rely on withdrawal resistance of screws in the panel edge of CLT, precaution shall be taken to ensure that side grain penetration occurs.

13 Timber piling

13.1 Scope

The design data and methods specified in Clause 13 apply only to the engineering design of piling that meets the requirements of CAN/CSA-O56 as structural members. Calculation of the bearing (supporting) capacity of the soil or rock is not covered by Clause 13.

13.2 Materials

13.2.1 Preservative treatment

Except as specified in Clause 13.2.2, the design data and methods specified in Clause 13 are based on the use of piling pressure treated with preservatives in accordance with the requirements of the CAN/CSA-O80 Series of Standards.

Note: *Species covered by CAN/CSA-O56 are not necessarily suitable for pressure treatment with preservatives. See the CAN/CSA-O80 Series of Standards for lists of species of piles that can be treated with preservatives.*

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13.2.2 Untreated piling

The design data and methods specified in Clause 13 may also be applied to untreated piling used for temporary construction.

13.3 Specified strengths and moduli of elasticity

The specified strengths and moduli of elasticity for round timber piles are given in Table 13.1.

13.4 Modification factors

The modification factors specified in Clause 5.3.2 for duration of load and Clause 6.4.2 for service conditions shall apply to timber piling. Other modification factors shall not apply.

13.5 Strengths and resistances

13.5.1 General

Timber piles may act as end-bearing piles or friction piles and shall be designed to transmit all of the applied loads to supporting soil or rock.

13.5.2 Piles as compression members

Piles shall be considered to act as compression members. Where necessary, piles shall be designed to withstand factored bending moments and factored tensile forces due to uplift or other causes, in accordance with the applicable requirements of Clause 6.

13.5.3 Effective length

When the finished pile projects above ground level and is not secured against buckling by bracing, the effective length shall be governed by the fixity conditions imposed on it by the structure it supports and by the nature of the ground into which it is driven. In firm soil, the lower point of contraflexure may be taken to be at a depth below ground level of about one-tenth of the exposed length. Where the top stratum is soft clay or silt, this point may be taken at about one-half the depth of penetration into this stratum, but not less than one-tenth of the exposed length of the pile. Where a pile is wholly embedded, its carrying capacity is not limited by its strength as a long column. However, where there is a stratum of very soft soils or peat, piles shall be designed in accordance with Clause 13.5.5.

13.5.4 Embedded portion

13.5.4.1 General

That portion of a pile permanently in contact with soil or rock providing lateral support shall be designed in accordance with Clause 6.5.5 using a slenderness factor K_C = 1.00.

13.5.4.2 End-bearing piles

The factored compressive resistance, P_r, of end-bearing piles shall be calculated for an area, A, equal to the minimum cross-sectional area of the pile.

13.5.4.3 Friction piles

The factored compressive resistance, P_r, of friction piles shall be calculated over an area, A, equal to the cross-sectional area of the pile at a point one-third of the length of the embedded portion of the pile from the tip.

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13.5.5 Unembedded portion

The factored compressive resistance, P_r, of that portion of the pile in contact with air, water, or soils that do not provide adequate lateral support shall be calculated in accordance with Clause 6.5.5. The slenderness factor, K_C, shall be calculated using a slenderness ratio, C_C, determined in accordance with Clauses 14.5.2.5 and 14.5.2.6. Piles subject to eccentric or lateral loads shall be designed in accordance with Clause 6.5.9.

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Table 13.1
Specified strengths and moduli of elasticity for round timber piles, MPa
(See Clause 13.3.)

Species	Bending, f_b	Longitudinal shear, f_v	Compression parallel to grain, f_c	Compression perpendicular to grain, f_{cp}	Tension parallel to grain, f_t	Modulus of elasticity	
						E	E_{05}
Douglas fir or western larch	20.1	1.4	18.7	7.7	13.6	11 000	7000
Jack pine	18.1	1.5	15.6	5.2	11.6	7000	5000
Lodgepole or ponderosa pine	14.2	1.0	13.2	5.2	9.7	7000	5000
Red pine	13.6	1.2	11.7	5.2	9.0	7000	5000

Notes:
1) *The values specified in this Table are for dry-service conditions and standard-term duration of load.*
2) *Timber piles using southern yellow pine may be assigned the same resistances as the Douglas Fir-Larch species combination.*

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14 Pole-type construction

14.1 Scope

14.1.1 Round poles

The design data and methods specified in Clause 14 apply only to the engineering design of round poles complying with the physical requirements, other than strength properties, of CAN/CSA-O15 as structural members in pole-type structures. Calculation of the bearing (supporting) capacity of the soil is not covered by Clause 14.

14.1.2 Sawn timbers

Sawn timbers used as poles shall meet the requirements of Clause 6.

14.2 Materials

14.2.1 Preservative treatment

The design data and methods specified in Clause 14 for poles and other wood components that are exposed to soil, moisture, inadequate ventilation, contact with masonry or concrete, or other conditions favourable to decay are based on the assumption that the poles and other wood components are pressure treated with preservatives in accordance with the requirements of the CAN/CSA-O80 Series of Standards.

Note: *Species covered by CAN/CSA-O15 are not necessarily suitable for pressure treatment with preservatives. Species of poles to be treated with preservatives are restricted to the species listed in the CAN/CSA-O80 Series of Standards.*

14.2.2 Short poles

When round pole lengths are shorter than specified in CAN/CSA-O15 but meet all other requirements of CAN/CSA-O15, the same taper and the same minimum circumference at the top, specified in CAN/CSA-O15, shall be used in calculations.

14.3 Specified strengths

The specified strengths for round poles, except eastern white cedar poles, shall be 80% of the values specified in Table 6.6 for select structural grade beams and stringers of the applicable species combination. The specified strengths for round eastern white cedar poles shall be 50% of the values specified in Table 6.6 for select structural grade beams and stringers of the species combination Northern Species.

14.4 Modification factors

The modification factors used for beams and stringers in Clause 6 shall apply to round poles.

14.5 Strengths and resistances

14.5.1 General

Poles shall be designed to transmit all applied factored loads to the soil and shall be suitable for the soil conditions at the site.

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14.5.2 Poles as compression members

14.5.2.1 General

Where necessary, poles shall be designed to withstand factored bending moments and factored tensile forces due to uplift or other causes in accordance with the applicable requirements of Clause 6.

14.5.2.2 Effective length

Effective length shall be established in accordance with Clause 13.5.3.

14.5.2.3 Embedded portion

That portion of a pole permanently in contact with soil or rock providing lateral support shall be designed in accordance with Clause 6.5.5, using a slenderness factor K_C = 1.00.

14.5.2.4 Unembedded portion

The factored compressive resistance, P_r, of the portion of a pole in contact with air, water, or soil that does not provide adequate lateral support shall be calculated in accordance with Clause 6.5.5. The slenderness factor, K_C, shall be calculated using a slenderness ratio, C_C, determined in accordance with Clauses 14.5.2.5 and 14.5.2.6. Poles subject to eccentric or lateral loads shall be designed in accordance with Clause 6.5.9.

14.5.2.5 Constant non-rectangular cross-section

For non-rectangular compression members of constant section, the slenderness ratio, C_C, shall be calculated in accordance with Clause 6.5.5.2.2 substituting the member width or depth with $r\sqrt{12}$ where r is the applicable radius of gyration of the cross-section of the member.

14.5.2.6 Variable circular cross-section

The radius of gyration of round tapered compression members shall be calculated for an effective diameter equal to the minimum diameter plus 0.45 times the difference between the maximum and minimum diameters. The factored compressive resistance determined in this manner shall not exceed the factored resistance based on the minimum diameter in conjunction with a slenderness factor K_C = 1.00.

14.5.3 Poles as bending members

The factored bending moment resistance, M_r, of round members shall be taken as that of a square member having the same cross-sectional area. A tapered round member shall be considered an equivalent square member of variable cross-section.

15 Proprietary structural wood products — Design

15.1 Scope

The design requirements for proprietary structural wood products used in specific applications specified in Clause 15 are provided for the project engineer or building designer. The product design values are usually generated by the manufacturer's engineer in accordance with Clause 16. Unless otherwise

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specified by the product manufacturer, proprietary products shall be used only under dry-service conditions.

Note: *In general, proprietary design values are published by the product manufacturer (i.e., proprietary design literature and Product Evaluation Reports in the CCMC* Registry of Product Evaluations*) with appropriate factors for specific applications. For applications where adjustments to design values are possibly warranted, the designer should seek guidance from the product manufacturer.*

15.2 Prefabricated wood I-joists

15.2.1 General

Prefabricated wood I-joists for use in accordance with Clause 15.2 shall meet the applicable requirements of Clause 16.2.

Note: *The I-joist should bear the mark of the accredited certification organization that is providing the I-joist product certification in accordance with ISO/IEC 17065.*

15.2.2 Modification factors

15.2.2.1 Load-duration factor, K_D

The specified strengths of prefabricated wood I-joists shall be multiplied by a load-duration factor, K_D, as specified in Clause 5.3.2.

15.2.2.2 Service-condition factor, K_S

The prefabricated wood I-joists described in Clause 16.2 shall only be used in dry-service conditions for which all service-condition factors K_S = 1.0.

15.2.2.3 Treatment factor, K_T

The strengths and stiffnesses described in Clause 16 are applicable to untreated prefabricated wood I-joists, with K_T = 1.0.

15.2.2.4 System factor, K_H

The system factor, K_H, shall be taken as 1.0 for all strength properties of prefabricated wood I-joists.

Note: *For additional information, see Appendix X1.4.1 of ASTM D5055.*

15.2.3 Strengths and resistances

15.2.3.1 Bending moment resistance

The factored bending moment resistance, M_r, of prefabricated wood I-joists shall be calculated as follows:

$$M_r = \phi\, M_r' \,(K_D\, K_S\, K_H\, K_T)\, K_L$$

where

ϕ = 0.90

M_r' = adjusted bending moment resistance

K_D = load-duration factor (see Clause 15.2.2.1)

K_S = service-condition factor (see Clause 15.2.2.2)

K_H = system factor (see Clause 15.2.2.4)

K_T = treatment factor (see Clause 15.2.2.3)

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K_L = lateral-stability factor (see Clause 15.2.3.2)

Note: *Factored ultimate resistance values, $\phi M_r'$, are listed in the CCMC* Registry of Product Evaluations.

15.2.3.2 Lateral-stability factor, K_L

The lateral-stability factor, K_L, shall be equal to 1.00 when lateral support is provided at points of bearing, to prevent lateral displacement and rotation, and along all compression edges. Lateral support requirements and lateral-stability factors for other applications such as continuous spans shall be based on analytical and engineering principles, documented test data, or both, that demonstrate the safe use of the product in the intended application.

15.2.3.3 Notches

There shall be no notching nor cutting of the flanges of prefabricated wood I-joists unless such details have been evaluated and shown to be acceptable based on documented test data.

15.2.3.4 Shear resistance

15.2.3.4.1 General

In the calculation of shear resistance, the effect of all loads acting on the member shall be considered, including the loads acting within a distance from a support equal to the depth of the member.

15.2.3.4.2 Factored shear resistance

The factored shear resistance, V_r, of prefabricated wood I-joists shall be calculated as follows:

$V_r = \phi V_r' (K_D K_S K_T)$

where

ϕ = 0.90

V_r' = shear resistance determined in accordance with the manufacturer's product evaluation report

Note: *Factored ultimate resistance values, $\phi V_r'$, are listed in the CCMC* Registry of Product Evaluations.

15.2.3.5 Reaction resistance

15.2.3.5.1 General

In the calculation of reaction resistance, the effect of all loads acting on the member shall be considered, including the loads acting near supports.

15.2.3.5.2 Factored end reaction resistance

The factored end reaction resistance, ER_r, at the specified end bearing length of prefabricated wood I-joists shall be calculated as follows:

$ER_r = \phi ER_r' (K_D K_S K_T)$

where

ϕ = 0.90

ER_r' = end reaction resistance determined in accordance with Clause 16 and published in the manufacturer's product evaluation report

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15.2.3.5.3 Factored intermediate reaction resistance

The factored intermediate reaction resistance, IR_r, at the specified intermediate bearing length of prefabricated wood I-joists shall be calculated as follows:

$IR_r = \phi\ IR_r^{'}\ (K_D\ K_S\ K_T)$

where

ϕ = 0.90

$IR_r^{'}$ = intermediate reaction resistance determined in accordance with Clause 16 and published in the manufacturer's product evaluation report

15.2.3.6 Web openings, bearing length, and web stiffener requirements

Designers shall obtain information regarding web openings, bearing length, and web stiffener requirements from the manufacturers of wood I-joists.

15.2.4 Serviceability limit states

The design of prefabricated wood I-joists for serviceability limit states shall be in accordance with Clauses 5.1.3 and 5.4. Deflection calculations shall be in accordance with the manufacturer's product evaluation report. Calculations shall include bending and shear deflection. Bending and shear stiffness properties shall be applicable to untreated joists used in dry-service conditions only.

15.2.5 Connections for prefabricated wood I-joists

15.2.5.1 Nails

Nailed connections shall be designed in accordance with Clause 12.9.

15.2.5.2 Joist hangers and other framing connectors

The use of joist hangers and other framing connectors with prefabricated wood I-joists shall be based on documented test data.

Note: *Required details of use and attachment are available from the manufacturers.*

15.3 Structural composite lumber

15.3.1 General

SCL for use in accordance with Clause 15.3, including products subjected to secondary processing operations, shall meet the applicable requirements of Clauses 16.3.1 to 16.3.6.

Note: *The SCL should bear the mark of the accredited certification organization that is providing the SCL certification in accordance with ISO/IEC 17065.*

15.3.2 Modification factors

15.3.2.1 Load-duration factor, K_D

The load-duration factor, K_D, specified in Clause 5.3.2 is applicable to the specified strengths of SCL, provided that appropriate testing demonstrating the validity of the factor for use with the applicable product has been conducted.

15.3.2.2 Service-condition factor, K_S

The strengths and stiffness of SCL described in Clause 15.3 are applicable for use in dry-service conditions, with K_S = 1.0. If the products are to be used in other than dry-service conditions,

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consideration shall be given to the choice of adhesive, and the specified strength and stiffness shall be evaluated. This shall include development of appropriate strength reduction factors, based on documented test results, determined in accordance with the manufacturer's product evaluation report.

15.3.2.3 Treatment factor, K_T

The strengths and stiffness described in Clause 15.3 are applicable to untreated SCL, with K_T = 1.0. Treatment adjustments for specified strength and stiffness shall be based on the documented results of tests considering the effects of time, temperature, and moisture content, determined in accordance with the manufacturer's product evaluation report.

15.3.2.4 System factor, K_H

The system factor, K_H, for SCL used in a load-sharing system shall be 1.04. To qualify for this increase, SCL shall be part of a wood-framing system consisting of at least three parallel members joined by transverse load-distributing elements capable of supporting the design load and shall not be spaced more than 610 mm on centre.

15.3.2.5 Size factor in bending, K_{Zb}

The size factor in bending, K_{Zb}, for SCL shall be determined in accordance with the manufacturer's product evaluation report.

15.3.2.6 Size factor in tension, K_{Zt}

The size factor in tension, K_{Zt}, for SCL shall be determined in accordance with the manufacturer's product evaluation report.

15.3.3 Strengths and resistances

15.3.3.1 Bending moment resistance

The factored bending moment resistance, M_r, of SCL shall be calculated as follows:

$M_r = \phi\ F_b\ SK_{Zb}\ K_L$

where

ϕ = 0.90

F_b = $f_b\ (K_D\ K_H\ K_{Sb}\ K_T)$

where

f_b = specified bending strength, MPa, determined in accordance with the manufacturer's product evaluation report

K_{Zb} = size factor in bending (see Clause 15.3.2.5)

K_L = lateral-stability factor (see Clause 15.3.3.2)

15.3.3.2 Lateral-stability factor, K_L

The lateral-stability factor, K_L, for SCL shall be determined in accordance with Clause 6.5.3.2.

15.3.3.3 Notches

SCL products with notches or cuts shall not be used unless such details have been evaluated and shown to be acceptable based on documented test data.

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15.3.3.4 Shear resistance

The factored shear resistance, V_r, of SCL shall be calculated as follows:

$$V_r = \phi F_v \frac{2A}{3} K_{Zv}$$

where

ϕ = 0.90

F_v = $f_v (K_D K_{Sv} K_T)$

where

f_v = specified shear strength of SCL, MPa, determined in accordance with the manufacturer's product evaluation report

A = cross-sectional area of member, mm^2

K_{Zv} = 1.0

15.3.3.5 Compressive resistance parallel to grain

15.3.3.5.1 Effective length, L_e

Unless noted otherwise, the effective length, $L_e = K_e L$, shall be used in determining the slenderness ratio of SCL in compression.

Note: *Recommended effective-length factors for SCL in compression are provided in Table A.4.*

Δ #### 15.3.3.5.2 Slenderness ratio, C_C

The slenderness ratio, C_C, of simple compression members of constant rectangular section shall not exceed 50 and shall be taken as the greater of the following:

$$C_C = \frac{\text{effective length associated with width}}{\text{member width}}$$

or

$$C_C = \frac{\text{effective length associated with depth}}{\text{member depth}}$$

15.3.3.5.3 Factored compressive resistance parallel to grain

The factored compressive resistance parallel to grain, P_r, of SCL shall be calculated as follows:

$$P_r = \phi F_c A K_C K_{Zc}$$

where

ϕ = 0.80

F_c = $f_c (K_D K_{Sc} K_T)$

where

f_c = specified strength in compression parallel to grain, MPa, determined in accordance with the manufacturer's product evaluation report

A = cross-sectional area of member, mm^2

K_C = slenderness factor (see Clause 15.3.3.6)

K_{Zc} = 1.0

Δ #### 15.3.3.6 Slenderness factor, K_C

The slenderness factor, K_C, shall be calculated as follows:

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$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_C^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

where

E_{05} = $0.87E$

where

E = specified modulus of elasticity, MPa, determined in accordance with the manufacturer's product evaluation report

15.3.3.7 Compressive resistance perpendicular to grain (bearing)

15.3.3.7.1 *Maximum loads*

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 15.3.3.7.2 to 15.3.3.7.3.

15.3.3.7.2 Effect of all applied loads

The factored compressive resistance perpendicular to grain, Q_r, of SCL under the effect of all applied loads shall be calculated as follows:

$Q_r = \phi\ F_{cp}\ A_b\ K_B\ K_{Zcp}$

where

ϕ = 0.80

F_{cp} = $f_{cp}\ (K_D\ K_S\ K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain, MPa, determined in accordance with the manufacturer's product evaluation report

A_b = bearing area, mm^2

K_B = length-of-bearing factor (see Clause 6.5.6.5)

K_{Zcp} = size factor for bearing (see Clause 6.5.6.4), except that K_{Zcp} shall be equal to 1.0 in the flatwise (L-X plane, as defined in ASTM D5456) orientation

15.3.3.7.3 Effect of loads applied near a support

The factored compressive resistance perpendicular to grain, Q_r', of SCL under the effect of only those loads acting within a distance from the centre of the support equal to the depth of the member shall be calculated as follows:

$Q_r' = (2/3)\phi\ F_{cp}\ A_b'\ K_B\ K_{Zcp}$

where

ϕ = 0.8

F_{cp} = $f_{cp}\ (K_D\ K_S\ K_T)$

where

f_{cp} = specified strength in compression perpendicular to grain, MPa, determined in accordance with the manufacturer's product evaluation report

A_b' = average bearing area, mm^2 (see Clause 6.5.6.3.2)

Note: *See Figure 6.2.*

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15.3.3.8 Compressive resistance at an angle to grain

The factored compressive resistance of SCL at an angle to grain shall be calculated in accordance with Clause 6.5.7 using the appropriate factored resistances for the proprietary grade of the product.

15.3.3.9 Tensile resistance parallel to grain

The factored tensile resistance parallel to grain, T_r, of SCL shall be calculated as follows:

$T_r = \phi F_t A_n K_{Zt}$

where

ϕ = 0.90

F_t = $f_t (K_D K_{St} K_T)$

where

f_t = specified strength in tension parallel to grain, MPa, determined in accordance with the manufacturer's product evaluation report

A_n = net cross-sectional area, mm^2

K_{Zt} = size factor in tension (see Clause 15.3.2.6)

15.3.3.10 Resistance to combined bending and axial load

Members subject to combined bending and compressive or tensile axial loads shall be designed to satisfy the applicable interaction equation, as follows:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] \leq 1$$

or

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1$$

where

P_f = factored compressive axial load

P_r = factored compressive resistance parallel to grain calculated in accordance with Clause 15.3.3.5.3

M_f = factored bending moment

M_r = factored bending moment resistance calculated in accordance with Clause 15.3.3.1

P_E = Euler buckling load in the plane of the applied moment

$= \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$

where

E_{05} = modulus of elasticity for design of compression members, MPa

I = moment of inertia in the plane of the applied moment, mm^4

L_e = effective length in the plane of the applied moment

$= K_e L$

where

K_e = effective-length factor (see Clause A.6.5.5.1)

T_f = factored tensile axial load

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T_r = factored tensile resistance parallel to grain calculated in accordance with the requirements of Clause 15.3.3.9

Note: *See Clause A.6.5.9 for a more detailed interaction formula for combined bending and compressive loads.*

15.3.4 Serviceability limit states

The design of SCL for serviceability limit states shall be in accordance with Clauses 5.1.3 and 5.4. Deflection shall be calculated in accordance with the manufacturer's material evaluation report. The published bending and shear stiffness properties shall be applicable to untreated SCL used in dry-service conditions only.

15.3.5 Connections for structural composite lumber

15.3.5.1 Joist hangers

The use of joist hangers with proprietary SCL shall meet the requirements of Clause 12.10.

15.3.5.2 Other connections

The design procedures for nails, wood screws, bolts, dowels, lag screws, timber rivets, shear plates, truss plates, and split rings in Clause 12 may be used for the design of connections for proprietary SCL when testing has demonstrated the validity of those procedures for use with that product.

Notes:

1) *Design information for some fasteners, primarily nails and joist hangers, applicable for use with specific proprietary products are listed in the CCMC* Registry of Product Evaluations.
2) *In the absence of design information for specific fasteners in specific proprietary products, connections for these products should be limited to bearing-type arrangements.*

16 Proprietary structural products — Materials and evaluation

16.1 Scope

Clause 16 applies to the derivation of design values for proprietary structural products that comply with the requirements of and applicable standards referenced in Clause 16. The design value derivation methods are directed at manufacturers and manufacturers' engineers to provide assurance that the proprietary design values are consistent with the intent of Part 4 of the *National Building Code of Canada* and with this Standard with respect to strength, serviceability, and reliability.

Note: *In general, proprietary design values are published by the product manufacturer (i.e., proprietary design literature and CCMC Evaluation Reports in the CCMC* Registry of Product Evaluations*), with appropriate factors for specific applications.*

16.2 Prefabricated wood I-joists

16.2.1 General

Except as otherwise specified in Clause 16.2, prefabricated wood I-joists for use in accordance with this Standard shall meet the requirements of, and be evaluated for strength and stiffness in accordance with, ASTM D5055. Characteristic values for design with prefabricated wood I-joists shall be determined in accordance with Clause 16.2.3.5. Prefabricated wood I-joists for use in accordance with this Standard shall meet the applicable requirements of Clause 16.2.

Note: *The I-joist should bear the mark of the accredited certification organization that is providing the I-joist product certification in accordance with ISO/IEC 17065.*

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16.2.2 Materials

16.2.2.1 Flange materials

Clause 16.2 applies to flanges as specified in Clause 6 for sawn lumber or Clause 7 for glued-laminated timber. Lumber not meeting the requirements of Clause 6 and SCL may be used as flange material when such material is qualified by testing as specified in ASTM D5055.

16.2.2.2 Structural panel webs

Webs for prefabricated wood I-joists shall be manufactured from structural panels meeting the requirements of CSA O121, CSA O151, CSA O153 (for exterior bond only), or CSA O325.

16.2.2.3 Adhesives

Prefabricated wood I-joists shall be manufactured using

a) a class of adhesives meeting the requirements of CSA O112.6 or CSA O112.7; or
b) alternative adhesives meeting the requirements of CSA O112.9 or CSA O112.10.

Note: *For additional information on these Standards, see the commentary in the referenced Standards.*

16.2.3 Specified strengths, capacities, and modulus of elasticity

16.2.3.1 Specified strength parallel to grain for flanges

16.2.3.1.1 Specified axial strength, f_a, for flanges — General

The specified axial strength, f_a, for flanges shall be the lesser of

$$f_a = f_t K_{LN} K_{Zt}$$

or

$$f_a = f_c$$

where

f_t = specified strength in tension parallel to grain, MPa (see Clause 16.2.3.1.2)

K_{LN} = length-adjustment factor as specified in ASTM D5055 (see Clause A.16.2.3.1)

K_{Zt} = size factor for tension parallel to grain from Table 6.13 (applicable only to visually stress-graded lumber)

f_c = specified strength in compression parallel to grain, MPa (see Clause 16.2.3.1.3)

16.2.3.1.2 Specified strength in tension parallel to grain, f_t, for flanges

The specified strength in tension parallel to grain, f_t, for flanges shall be determined as follows:

a) When the flange material is sawn lumber meeting the requirements of Clause 6.2 or glued-laminated timber meeting the requirements of Clause 7.2, the specified strength in tension parallel to grain shall be determined from f_t in Clause 6.3 or f_{tg} in Table 7.2, respectively.
b) When the flange material is not as described in Item a), f_t shall be calculated as follows:

   $$f_t = t\, K_r$$

   where

   t = characteristic value for tension parallel to grain, MPa, as specified in Clause 16.2.3.5

   K_r = reliability normalization factor for bending and tension as specified in Table 16.1

Table 16.1
Reliability normalization factor, K_r (applicable to prefabricated wood I-joists and structural composite lumber products only)
(See Clauses 16.2.3.1.2, 16.2.3.1.3, 16.2.3.2, 16.2.3.3.2, 16.2.3.3.3, 16.2.4.1, 16.3.3.2, 16.3.3.3, 16.3.3.4, and 16.3.3.6.)

			Shear			Bearing reactions**
CV_w, % *	Bending and tension†	Compression parallel to grain‡	Prefabricated wood I-joists	Structural composite lumber§		Prefabricated wood I-joists
				Case 1	Case 2	
10	0.88	0.84	0.74	0.59	0.88	0.74
11	0.88	0.84	0.74	0.59	0.88	0.74
12	0.88	0.84	0.74	0.59	0.88	0.74
13	0.88	0.84	0.74	0.59	0.88	0.74
14	0.88	0.84	0.74	0.59	0.88	0.74
15	0.88	0.83	0.74	0.59	0.88	0.74
16	0.87	0.82	0.74	0.58	0.87	0.74
17	0.86	0.80	0.74	0.57	0.86	0.74
18	0.84	0.78	0.73	0.56	0.84	0.73
19	0.82	0.77	0.71	0.55	0.82	0.71
20	0.80	0.75	0.71	0.54	0.80	0.71
21	0.79	0.73	0.69	0.52	0.79	0.69
22	0.77	0.71	0.67	0.51	0.77	0.67
23	0.75	0.70	0.66	0.50	0.75	0.66
24	0.73	0.68	0.65	0.49	0.73	0.65
25	0.72	0.66	0.63	0.48	0.72	0.63

* *CV_w shall be determined in accordance with ASTM D5457.*
† *Applicable to SCL in bending and tension as well as to flanges of prefabricated wood I-joist flanges in compression and in tension.*
‡ *Applicable to SCL in compression parallel to grain.*
§ *SCL shear adjustment shall be based on the shear qualification testing method specified in ASTM D5456, i.e., the shear block tests (Case 1) or the structural-size horizontal shear tests (Case 2).*
** *Applicable to prefabricated I-joist reaction capacity at end bearing supports and intermediate bearing supports.*

16.2.3.1.3 Specified strength in compression parallel to grain, f_c, for flanges

The specified strength in compression parallel to grain, f_c, for flanges shall be determined as follows:

a) When the flange material is sawn lumber meeting the requirements of Clause 6.2 or glued-laminated timber meeting the requirements of Clause 7.2, the specified strength in compression parallel to grain shall be determined from f_c in Clause 6.3 or f_c in Table 7.2, respectively.

b) When the flange material is not as described in Item a), f_c shall be calculated as follows:

$f_c = c\ K_r$

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where

c = characteristic value for compression parallel to grain, MPa, as specified in Clause 16.2.3.5

K_r = reliability normalization factor for bending and tension as specified in Table 16.1

16.2.3.2 Specified shear capacity, V_c

The specified shear capacity, V_c, shall be calculated as follows:

$V_c = v\,K_r$

where

v = characteristic value for shear, N, as specified in Clause 16.2.3.5

K_r = reliability normalization factor for shear for prefabricated wood I-joists as specified in Table 16.1

16.2.3.3 Reaction capacities

16.2.3.3.1 General

The specified end reaction capacity and the specified intermediate reaction capacity shall be calculated in accordance with Clauses 16.2.3.3.2 and 16.2.3.3.3, respectively.

16.2.3.3.2 Specified end reaction capacity, ER_c

The specified end reaction capacity, ER_c, at a specified bearing length shall be calculated as

$ER_c = r_e\,K_r$

where

r_e = characteristic value for end reaction as defined in Clause 16.2.3.5, N

K_r = reliability normalization factor for reaction for prefabricated wood I-joists as specified in Table 16.1

16.2.3.3.3 Specified intermediate reaction capacity, IR_c

The specified intermediate reaction capacity, IR_c, at a specified bearing length shall be calculated as

$IR_c = r_i\,K_r$

where

r_i = characteristic value for intermediate reaction as defined in Clause 16.2.3.5, N

K_r = reliability normalization factor for reaction for prefabricated wood I-joists as specified in Table 16.1

16.2.3.4 Modulus of elasticity, E, of flange material

The modulus of elasticity, E, of flange material shall be determined as follows:

a) When the flange material is sawn lumber meeting the requirements of Clause 6.2 or glued-laminated timber meeting the requirements of Clause 7.2, the modulus of elasticity shall be determined from Clause 6.3 or Table 7.2, respectively.

b) When the flange material is not as described in Item a), the modulus of elasticity shall be the mean value of the modulus of elasticity determined from the test results required by Section 6.6.2.1 of ASTM D5055.

Δ

16.2.3.5 Characteristic values

Characteristic values shall be determined as follows:

a) The maximum characteristic shear value, v, for prefabricated wood I-joists shall be the shear capacity specified in ASTM D5055 and multiplied by 2.37.

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b) The maximum characteristic reaction values for prefabricated wood I-joists shall be the end reaction capacity, r_e, and intermediate reaction capacity, r_I, for a specified bearing length, as defined in ASTM D5055 and multiplied by 2.37.

c) The maximum characteristic value in tension parallel to grain, t, for flanges of prefabricated wood I-joists shall be taken as follows:
   i) sawn lumber: the flange tensile capacity determined in accordance with ASTM D5055 and multiplied by 2.1; and
   ii) SCL: the characteristic value in tension, t_{SCL}, as specified in Clause 16.3.3.6.

d) The maximum characteristic value in compression parallel to grain, c, for flanges of prefabricated wood I-joists shall be calculated as follows:
   i) sawn lumber:
   $c = t\ (f_{c1}/f_{t1})$
   where
   f_{c1} = specified strength in compression parallel to grain as assigned in Clause 6.3 for the same grade, species combination, and size as f_{t1}
   f_{t1} = closest specified strength to f_t in tension parallel to grain as assigned in Clause 6.3 for the species combination and size tested in accordance with ASTM D5055
   ii) SCL: the characteristic value in compression, c, as specified in Clause 16.3.3.4.

e) The maximum characteristic value for moment capacity for prefabricated wood I-joists with sawn lumber or structural composite flanges may be determined in accordance with the empirical method specified in Section 6 of ASTM D5055. Where this method is used, the maximum characteristic value for moment capacity shall be based on the lower 5% tolerance limit with 75% confidence.

16.2.4 Adjusted resistances

16.2.4.1 Adjusted bending moment resistance, M_r'

The adjusted bending moment resistance, M_r', of prefabricated wood I-joists shall be calculated as

$$M_r' = f_a\ A_{net}\ Y$$

or

$$M_r' = M_{cv}\ K_r$$

where

f_a = specified axial strength for flanges, MPa (see Clause 16.2.3.1)

A_{net} = net cross-sectional area of one flange, excluding all areas of web material and rout, mm^2

Y = distance between the flange centroids with the rout removed, mm

M_{cv} = characteristic value for moment capacity, N•m, as specified in Clause 16.2.3.5 e)

K_r = reliability normalization factor for bending and tension (see Table 16.1)

16.2.4.2 Shear resistance

16.2.4.2.1 General

In the calculation of adjusted shear resistance, the effect of all loads acting on the member shall be considered, including the loads acting within a distance from a support equal to the depth of the

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member. Adjustments to the shear design value near the support shall be substantiated by independent testing to the shear capacity criteria specified in ASTM D5055.

16.2.4.2.2 Adjusted shear resistance, V'_r

The adjusted shear resistance, V'_r, of prefabricated wood I-joists shall be calculated as

$V'_r = V_c$

where

V_c = specified shear capacity for a given brand and depth of prefabricated wood I-joist, N, in accordance with Clause 16.2.3.2

16.2.4.3 Adjusted reaction resistances

16.2.4.3.1 General

The adjusted end reaction resistance and the adjusted intermediate reaction resistance shall be calculated in accordance with Clauses 16.2.4.3.2 and 16.2.4.3.3, respectively.

16.2.4.3.2 Adjusted end reaction resistance, ER'_r

The adjusted end reaction resistance, ER'_r, of prefabricated wood I-joists shall be calculated as follows:

$ER'_r = ER_c$

where

ER_c = specified end reaction capacity for a given brand and depth of prefabricated wood I-joist, at a specified bearing length in accordance with Clause 16.2.3.3.2, N

16.2.4.3.3 Adjusted intermediate reaction resistance, IR'_r

The adjusted intermediate reaction resistance, IR'_r, of prefabricated wood I-joists shall be calculated as follows:

$IR'_r = IR_c$

where

IR_c = specified intermediate reaction capacity for a given brand and depth of prefabricated wood I-joist at a specified bearing length in accordance with Clause 16.2.3.3.3, N

16.2.4.4 Web openings, minimum bearing length, and web stiffener details

The requirements for web openings, minimum bearing length, and web stiffener details shall be determined in accordance with ASTM D5055.

16.2.5 Serviceability limit states

Deflection calculations shall include shear deformation.

The flange modulus of elasticity, E_s, for stiffness calculations shall be calculated as follows:

$E_s = E\ (K_{SE}\ K_{TE})$

The effective stiffness, EI_w, of prefabricated wood I-joist web members employing structural panels as specified in Clause 9.3 and meeting the requirements of CSA O121, CSA O151, or CSA O325 shall be calculated as follows:

$$EI_w = (\Sigma B_a)\left(\frac{W_D^3}{12}\right)K_{SE}K_{TE}$$

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The shear-through-thickness rigidity, W_S, of structural panels as specified in Clause 9.3 that are used as web members of prefabricated wood I-joists and meeting the requirements of CSA O121, CSA O151, or CSA O325 shall be calculated as follows:

$W_S = B_v K_{SG} K_{TG}$

where

E = specified modulus of elasticity of flange material (see Clause 16.2.3.4)

K_{SE} = service-condition factor for modulus of elasticity

= 1.0 for dry-service conditions

K_{TE} = treatment factor for modulus of elasticity

= 1.0 for untreated prefabricated wood I-joists (see Clause 15.2.2.3)

B_a = axial stiffness (tension or compression) from Table 9.1, 9.2, or 9.3, N/mm

W_D = overall depth of the structural panel web, mm

B_v = shear-through-thickness rigidity from Table 9.1, 9.2, or 9.3, N/mm

K_{SG} = service-condition factor for shear-through-thickness rigidity

= 1.0 for dry-service conditions

K_{TG} = treatment factor for shear-through-thickness rigidity

= 1.0 for untreated prefabricated wood I-joists

Note: *For axial web stiffness and web shear-through-thickness rigidity for products not covered by Table 9.1, 9.2, or 9.3, refer to appropriate standards or documented test data which can be obtained from the manufacturer of the prefabricated wood I-joists.*

16.3 Structural composite lumber

16.3.1 General

SCL for use in accordance with this Standard shall be manufactured to, and evaluated for characteristic values in accordance with, ASTM D5456, except for laminated veneer bamboo that is outside the scope of this Standard.

SCL for use in accordance with this Standard, including products subjected to secondary processing operations, shall meet the applicable requirements of Clauses 16.3.2 to 16.3.6.

Note: *The SCL should bear the mark of the accredited certification organization that is providing the SCL certification in accordance with ISO/IEC 17065.*

16.3.2 Adhesives and binder systems

16.3.2.1 Adhesives

Adhesives used in the manufacture of SCL shall be

a) specific classes of adhesives conforming to CSA O112.6 or O112.7; or

b) alternative adhesives conforming to CSA O112.9 or O112.10.

CSA O112.10 shall be used only with products intended for dry-service conditions, e.g., wood I-joists.

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16.3.2.2 Binder systems

Binder systems shall demonstrate equivalent performance to the adhesives specified in Clause 16.3.2.1 a).

16.3.3 Specified strengths and modulus of elasticity

16.3.3.1 General

Specified strengths and modulus of elasticity for SCL for use in accordance with this Standard shall be determined in accordance with Clauses 16.3.3.2 to 16.3.3.7.

Δ #### 16.3.3.2 Specified bending strength, f_b

The specified bending strength, f_b, for SCL shall be calculated as follows:

$f_b = F_B K_r$

where

F_B = characteristic value for bending, MPa, determined in accordance with ASTM D5456

K_r = reliability normalization factor for bending and tension (see Table 16.1)

16.3.3.3 Specified shear strength, f_v

The specified shear strength, f_v, for SCL shall be calculated as follows:

$f_v = v_{SCL} K_r$

where

v_{SCL} = characteristic value for shear, MPa, determined in accordance with ASTM D5456

K_r = reliability normalization factor for shear for SCL (see Table 16.1)

16.3.3.4 Specified compression strength parallel to grain, f_c

The specified compression strength parallel to grain, f_c, for SCL shall be calculated as follows:

$f_c = c K_r$

where

c = characteristic value in compression parallel to grain, MPa, as determined in accordance with ASTM D5456

K_r = reliability normalization factor for compression parallel to grain (see Table 16.1)

16.3.3.5 Specified compression strength perpendicular to grain, f_{cp}

The specified compression strength perpendicular to grain, f_{cp}, MPa, for SCL shall not exceed the following:

a) the characteristic value determined in accordance with ASTM D5456 and multiplied by 1.09 in the edgewise (L-Y plane, as defined in ASTM D5456) orientation; and

b) the characteristic value determined in accordance with ASTM D5456 and multiplied by 1.81 in the flatwise (L-X plane, as defined in ASTM D5456) orientation.

16.3.3.6 Specified strength in tension parallel to grain, f_t

The specified strength in tension parallel to grain, f_t, for SCL shall be calculated as follows:

$f_t = t_{SCL} K_r$

where

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t_{SCL} = characteristic value in tension parallel to grain, MPa, as determined in accordance with ASTM D5456

K_r = reliability normalization factor for bending and tension (see Table 16.1)

16.3.3.7 Specified modulus of elasticity, *E*

The specified modulus of elasticity, *E*, for SCL shall be the mean value determined in accordance with ASTM D5456.

16.3.4 Modification factors

16.3.4.1 Size factor in bending, K_{Zb}

The size factor in bending, K_{Zb}, for SCL shall be calculated as follows:

$$K_{Zb} = \left(\frac{d_1}{d}\right)^{\frac{1}{n}}$$

where

d_1 = specified depth on which the published specified strength in bending, f_b, is based

d = depth of application member

n = parameter specified in Section 7.4.1.3 of ASTM D5456

16.3.4.2 Size factor in tension, K_{Zt}

The size factor in tension, K_{Zt}, for SCL shall be calculated as follows:

$$K_{Zt} = \left(\frac{L_1}{L}\right)^{\frac{1}{m}}$$

where

L_1 = base length between test grips, determined in accordance with Section 5.5.2 of ASTM D5456

L = end use length

m = parameter determined in accordance with Annex A1 of ASTM D5456

16.3.5 Serviceability limit states

Deflection calculations shall include shear deformation.

The member modulus of elasticity, E_s, for stiffness calculations shall be calculated as follows:

$E_s = E\ (K_{SE}\ K_{TE})$

where

E = specified modulus of elasticity, MPa, in accordance with Clause 16.3.3.7

K_{SE} = service-condition factor for modulus of elasticity

= 1.0 for dry-service conditions

K_{TE} = treatment factor for modulus of elasticity

= 1.0 for untreated SCL

The shear modulus or modulus of rigidity, G_S, for stiffness calculations shall be calculated as follows:

$G_S = G\ (K_{SG}\ K_{TG})$

where

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G = specified shear modulus or modulus of rigidity, MPa, established by testing or as published in a recognized reference for the SCL wood species

K_{SG} = service-condition factor for shear modulus

= 1.0 for dry-service conditions

K_{TG} = treatment factor for shear modulus

= 1.0 for untreated SCL

16.3.6 Connections for structural composite lumber

The design procedures for nails, bolts, screws, lag screws, timber rivets, shear plates, truss plates, and split rings in Clause 12 may be used for the design of connections for proprietary SCL when testing has demonstrated the validity of those procedures for use with that product.

Note: *Design information for fasteners for use with proprietary products should be added to the CCMC* Registry of Product Evaluations *as test data become available.*

16.4 Truss plates

16.4.1 General

16.4.1.1

The design requirements for truss plate connections specified in Clause 16.4.1 are for light-gauge metal plates that depend on extended teeth or nails embedded into the wood to transfer load and meet the requirements of Clause 16.4.

16.4.1.2

Truss plates shall be manufactured from galvanized sheet steel that conforms to Grade SS230, SS255, SS275, HSLA S340, HSLA S410, HSLA II340, or HSLA II410 of ASTM A653/A653M and has the minimum properties specified in Table 16.2.

Table 16.2
Minimum properties of steels used for truss plates
(See Clause 16.4.1.2.)

	Grade				
	SS230	**SS255**	**SS275**	**HSLA S340 or HSLA II340**	**HSLA S410 or HSLA II410**
Ultimate tensile strength, MPa	310	360	380	410	480
Minimum yield strength, MPa	230	255	275	340	410
Elongation in 50 mm length at failure, %	20	18	16	20	16

Note: *Galvanizing may be carried out before manufacture and should conform to G90 coating class.*

16.4.2 Strength resistance of truss plates

16.4.2.1 Ultimate lateral resistance of teeth

Ultimate lateral resistance values p_u, q_u, p'_u, and q'_u shall be obtained from tests carried out in accordance with CSA S347 and calculated as follows:

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$n_u = 0.8\ R_c\ B\ K_{mu}\ K_{pu}$

where

n_u = ultimate lateral resistance values p_u, q_u, p'_u, or q'_u

R_c = the lesser of the following:

a) $R_u\ (1 - 2.104\ CV)$

b) $0.753\ R_u$

where

R_u = average ultimate lateral resistance from the test data

CV = coefficient of variation of the test data

where

B = $1.44 - 2.18\ CV$, but not greater than 1.1

K_{mu} = moisture response factor for ultimate strength

= 0.83 for sawn lumber to sawn lumber connections

= determined in accordance with CSA S347 for SCL to SCL connections, and sawn lumber to SCL connections

K_{pu} = roller press modification factor for ultimate strength determined in accordance with CSA S347

Note: *Where the moisture response factor for SCL to sawn lumber is less than 0.83, two values of n_u will be calculated for sawn lumber connections, one value for sawn lumber to sawn lumber connections and another value for sawn lumber to SCL connections.*

16.4.2.2 Tensile resistance

The tensile resistance of the plate, t_p, shall be determined parallel and perpendicular to the direction of the applied load and calculated as follows:

$t_p = t_{uL}\ CF$

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where

t_{uL} = characteristic tensile strength from CSA S347, N/mm

CF = correction factor for strength of plate material from CSA S347

16.4.2.3 Shear resistance

The shear resistance of the plate, v_p, shall be determined for specified angles of plate axis to load direction and calculated as follows:

$v_p = v_{uL}\, CF$

where

v_{uL} = characteristic shear strength from CSA S347, N/mm

CF = correction factor for strength of plate material from CSA S347

For all other angles, shear resistance shall be determined by linear interpolation.

16.4.3 Lateral slip resistance

The resistance values p_s, q_s, p_s', and q_s' shall be obtained from tests carried out in accordance with CSA S347 and calculated as follows:

$n_s = (R_s/1.2)\, K_{ps}\, K_{ms}$

where

n_s = lateral slip values p_s, q_s, p_s', or q_s'

R_s = average lateral resistance at 0.8 mm wood-to-wood slip determined in accordance with CSA S347

K_{ps} = roller press modification factor for slip, determined in accordance with CSA S347

K_{ms} = moisture response factor for slip

= 0.83 for sawn lumber to sawn lumber connections

= determined in accordance with CSA S347 for SCL to SCL connections, and sawn lumber to SCL connections

Note: *Where the moisture response factor for sawn lumber to SCL is less than 0.83, two values of n_s will be calculated for sawn lumber connections, one value for sawn lumber to sawn lumber and another value for sawn lumber to SCL.*

16.5 Joist hangers

16.5.1 General

The design requirements specified in Clause 16.5 for joist hangers are for proprietary mass-produced metal devices, usually cold-formed from light-gauge steel or welded from steel plate, that are used to transfer loads from a joist to a header or beam. The joist and header or beam may be sawn lumber, wood trusses, glued-laminated timber, prefabricated wood I-joists, or SCL.

The steel shall have a specified minimum tensile and yield strength and specified dimensional characteristics. Sheet steel shall be hot-dip galvanized.

Note: *Galvanizing may be carried out before manufacturing and should conform to minimum G90 galvanizing class.*

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16.5.2 Testing

The design requirements specified in Clause 16.5 are for joist hangers tested for vertical load capacity in accordance with ASTM D7147. A set of at least three tests (six hangers) shall be conducted for each possible variation of the hanger, wood material, and fasteners, including

a) joist species, size, and type;
b) header species, size, and type;
c) joist hanger size and type; and
d) fastener size, type, and spacing.

For sawn wood and glued-laminated timber joists and headers, the relative density of the material used in testing shall not be greater than 2% above the mean relative density values for the material tested. For manufactured wood products, the relative density shall not be greater than 2% above the average of the population. The moisture content of sawn wood at the time of testing shall be no less than 11%. For wood products that are manufactured, installed, and maintained at or below 15% moisture content, the tests shall be made at a moisture content of no less than 7%. To allow for relaxation effects, at least one week shall elapse between assembly and testing of the specimens.

Note: *Mean relative density values for sawn wood and glued-laminated timber are provided in Table A.11.*

16.5.3 Ultimate resistance of joist hangers

The ultimate resistance of joist hangers shall be obtained from vertical load tests on at least three pairs of hangers conducted in accordance with Clause 16.5.2. The ultimate resistance shall be calculated as follows:

a) for fewer than 10 pairs of hangers, the lesser of
   i) the corrected ultimate load per hanger calculated in accordance with Clause 16.5.4, multiplied by 0.91; or
   ii) the average load per hanger at which the vertical movement between the joist and the header is 3 mm, multiplied by 2.42.
b) for 10 or more pairs of hangers, the lesser of
   i) the corrected ultimate load per hanger calculated in accordance with Clause 16.5.4, multiplied by 1.2; or
   ii) the average load per hanger at which the vertical movement between the joist and the header is 3 mm, multiplied by 2.42.

16.5.4 Corrected ultimate load of joist hangers

16.5.4.1 Single species testing

The corrected ultimate load per hanger obtained from testing on at least three pairs of hangers shall be the lower of

a) $N_{ult}\ CF_w$; and
b) $N_{ult\ steel}\ CF_s$.

16.5.4.2 Multiple species testing

The corrected ultimate load per hanger obtained from testing on at least six pairs of hangers where the only difference in test setup is the species of wood (at least three pairs of hangers per species) shall be the lower of

a) $N_{ult}\ CF_w$ per species; and
b) $N_{ult\ steel}\ CF_s$.

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16.5.4.3 Failure other than hanger steel

For testing where failure of the hanger steel does not occur, the highest ultimate load per assembly in place of the $N_{ult\ steel}$ in Clauses 16.5.4.1 b) and 16.5.4.2 b) shall be used

where

CF_w = $N_{r\ spec}/N_{r\ test} \leq 1.0$

CF_s = $f_{u\ spec}/f_{u\ test} \leq 1.0$

$f_{u\ spec}$ = specified minimum tensile strength of the hanger steel

$f_{u\ test}$ = tensile strength of the hanger steel, measured in accordance with ASTM E8/E8M

$N_{r\ spec}$ = factored lateral resistance of the joint used to attach the hanger to the header calculated per Clause 12 using $f_{u\ spec}$ in place of f_u

$N_{r\ test}$ = factored lateral resistance of the joint used to attach the hanger to the header calculated per Clause 12 using $f_{u\ test}$ in place of f_u

N_{ult} = one-half of the lowest ultimate load per assembly

$N_{ult\ steel}$ = one-half of the lowest ultimate load per assembly associated with hanger steel failure

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Annex A (informative)
Additional information and alternative procedures

Notes:

1) *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*
2) *The clause-numbering scheme in this Annex has been devised to facilitate reference to related clauses in the main body of this Standard.*

A.5 General design

A.5.3.5 System factor, K_H

It is well known that the behaviour of a single member does not represent that of a system such as a floor or a flat roof with several joists or rafters. System behaviour can be accounted for in single-member design by implementing system factors: K_H for strength and K_Δ for serviceability. In this Standard, only K_H has been quantified.

The system factor, K_H, is a function of the parameters that define the mechanical state and physical layout of the system. These parameters are the mean live load and its coefficient of variation, mean modulus of rupture (MOR) and its coefficient of variation, mean modulus of elasticity (MOE) and its coefficient of variation, MOE-MOR correlation, sheathing thickness, and connection stiffness.

A.5.4.2 Elastic deflection of wood light-frame systems under static loads

A.5.4.2.1 Wood frame deflection calculations

Wood frame systems connected with sheathing or cladding on one or both sides deflect less than the joists, rafters, or studs carrying the same loads independently. Traditionally, however, deflection calculations have ignored this interaction and assumed that each framing member in these systems is loaded individually on its tributary area. The deflection criteria that evolved from this approach have provided satisfactory system performance based on calculated single-member deflections.

It is possible to estimate system performance by calculating system factors greater than 1.0 as a ratio of system deflection to single-member deflection. Caution should be taken when implementing system factors in traditional design procedures. Where design procedures incorporate a system factor for deflection, there is a need to consider, before adjusting design procedure criteria, whether the system effects add to enhancements that are already present in traditional wood frame performance.

A.5.4.2.2 Elastic deflection of wood light-frame wall systems under wind load

Typical wood light-frame wall systems designed for a single-member deflection of 1/360 of the span can satisfy the intent of masonry design specifications intended to limit the deflection of steel studs in high-rise buildings to 1/720 of the span.

For example, the actual deflection would be approximately half of the deflection calculated on a single member basis under the following conditions:

a) lumber modulus of elasticity (see Table 6.4) derived from visually stress-graded lumber data;
b) lumber used in a stud wall system (i.e., 38 × 89 mm or 38 × 140 mm) meeting minimum requirements for the Case 2 system factor (see Clause 6.4.4.2);

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c) gypsum wallboard or structural sheathing attached to the inside face of the studs in accordance with minimum building code requirements; and
d) cladding and secondary member wind loading based on the tributary area of a stud in a low-rise building.

A.5.4.5 Vibration performance of wood joisted floors

A.5.4.5.1 Vibration-controlled span — General method

For single-span floor, except as permitted in Clause A.5.4.5.2 where the floor systems are constructed with prefabricated wood I-joists, the maximum span for controlling vibration on wood joisted floor systems that include floor joists and wood structural panel subfloor may be calculated using the following equation:

$$\ell_v = \frac{0.122\left(EI_{eff}\right)^{0.284}}{K_{tss}^{0.14} m_L^{0.15}}$$

where

ℓ_v = vibration-controlled span, m

EI_{eff} = effective composite bending stiffness of the floor system in the joist span direction, N•m² (see Clause A.5.4.5.1.1)

K_{tss} = factor to account for the transverse system stiffness contribution in reducing floor deflection (see Clause A.5.4.5.1.3)

m_L = linear mass of the composite cross section of the floor that accounts for the joist, subfloor, and topping, kg/m (see Clause A.5.4.5.1.2)

Notes:

1) *The above equation applies to single-span floor simply supported at all supports and may be used for multiple-span systems, in which case the longest span is used in the calculation and EI_{eff} may be increased by 20% to account for end-fixity effect. This increase does not apply to floors with a cementitious topping.*
2) *Where lateral bracing elements are present that are structurally fastened to the joists, the calculated span may be increased by 5%, provided the spacing between adjacent lateral bracing elements or the distance between lateral bracing element and the nearest support is not more than 2 m. This span increase is not permitted if the floor is topped with a cementitious layer.*
3) *Where gypsum board is fastened to the underside of the joists directly, the calculated span may be increased by 5%. This span increase is not permitted if the fastening of the gypsum board to the joists is through acoustic fittings or a topping is accounted for in the calculation.*
4) *The span calculated using this Clause is expected to lead to wood joisted floor systems with performance that is consistently acceptable to a large majority of the occupants. Manufacturers of engineered joist products may use other mechanics-based or empirical methods in the development of span charts to provide a different level of user acceptance if their use has been supported by testing, engineering analysis, or performance history, and the development of the alternative method was approved by an appropriate certification body. These manufacturers may also provide further guidance beyond the scope of the method presented in this Annex clause. (See CWC* Commentary on CSA O86.*)*

A.5.4.5.1.1 Effective composite bending stiffness of floor system in joist span direction

The calculation of effective composite bending stiffness of the floor system in the span direction in N•m², EI_{eff}, considers the contribution of the joist, subfloor, and topping. The following equation can be used to estimate the effective bending stiffness of the floor system if composite section approach is used. Figure A.1 provides the definitions of the symbols.

$EI_{eff} = EI_u + \overline{EA}_1 h_1^2 - \bar{A}\bar{y}^2$

where

EI_u = $EI_{joist} + b_1(EI_{s\perp} + EI_c)$, N•m²

where

EI_{joist} = apparent bending stiffness of joist, N•m²

$EI_{s\perp}$ = bending stiffness of 1 m wide subfloor panel in span direction, N•m (see Table A.1)

EI_c = bending stiffness of 1 m wide topping, N•m

= $E_c t_c^3/12$

where

E_c = modulus of elasticity of topping, N/m² (see Table A.2)

t_c = topping thickness, m

$\overline{EA}_1$ = $\frac{b_1 EA_1}{1 + 10\frac{b_1 EA_1}{S_1 L_1^2}}$, N

where

EA_1 = $EA_{s\perp} + EA_c$

where

$EA_{s\perp}$ = axial stiffness of 1 m wide subfloor panel in span direction, N/m (see Table A.1)

EA_c = axial stiffness of 1 m wide topping, N/m (see Table A.2)

b_1 = spacing of joist, m

S_1 = load-slip modulus per unit spacing for subfloor-to-joist connection

= 5×10^6 N/m/m for mechanical joint or when topping is present

= 1×10^8 N/m/m for glued joint

L_1 = width of subfloor panels in the span direction for floor without topping, m

= span for floor with topping, m

$\bar{A}$ = $EA_{joist} + \overline{EA}_1$, N

where

EA_{joist} = axial stiffness of joist, N

$\bar{y}$ = $\frac{h_1 \overline{EA}_1}{\bar{A}}$, m

where

h_1 = $\frac{d}{2} + \frac{EA_{s\perp} t_s / 2 + EA_c(t_s + t_c/2)}{EA_{s\perp} + EA_c}$, m

where

t_s = thickness of subfloor, m

Notes:

1) *Apparent bending stiffness of joist, EI_{joist}, accounts for influence of shear and slips at mechanical joints on stiffness properties of the floor system. See CWC* Commentary *for guidance on calculation of EI_{joist}.*

2) *Mechanical properties and density of subfloor panels and topping materials are provided in Tables A.1 and A.2.*

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Figure A.1
Definitions of cross-sectional dimension parameters
(See Clause A.5.4.5.1.1.)

A.5.4.5.1.2 Linear mass of floor in joist span direction

The linear mass of the floor in the joist span direction, m_L, in kg/m is the summation of all components and shall be calculated as follows:

$m_L = m_J + \rho_s t_s b_1 + \rho_c t_c b_1$

where

m_J = mass per unit length of joist, kg/m

ρ_s = density of subfloor, kg/m³ (see Table A.1)

ρ_c = density of topping, if present, kg/m³ (see Table A.2)

Note: *The mass of gypsum board is not required in the calculation of m_L.*

A.5.4.5.1.3 Transverse system stiffness factor

The transverse system stiffness factor, K_{tss}, accounts for the contribution of the subfloor and topping in reducing static deflection of the floor under a point load and is calculated using the following equation:

$K_{tss} = 0.0294 + 0.536K_1^{0.25} + 0.516K_1^{0.5} + 0.31K_1^{0.75}$

where

$K_1 = \frac{K_j}{K_j + K_L}$

where

$K_j = \frac{EI_{eff}}{\ell^3}$

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K_L = $\frac{0.585 \times \ell \times EI_{s\square}}{b_1^3}$, N/m for floor with a subfloor only

= $\frac{0.585 \times \ell \times \left(EI_{s\square} + EI_c + \frac{EA_c \times EA_{s\square}}{EA_c + EA_{s\square}} \times h_3^2\right)}{b_1^3}$, N/m for floor with subfloor and topping

ℓ = floor span, m

$EI_{s||}$ = unit bending stiffness of subfloor in across-span direction, N•m²/m

$EA_{s||}$ = unit axial stiffness of subfloor in across-span direction, N/m

h_3 = $(t_s + t_c)/2$, m

Δ

Table A.1
Properties of subfloor panels*

(See Clauses A.5.4.5.1.1 and A.5.4.5.1.2 and Table A.2.)

Type	t_s (mm)	$EI_{s\|\|}$ (N•m²/m)	$EI_{s\perp}$ (N•m²/m)	$EA_{s\|\|}$ (N/m)	$EA_{s\perp}$ (N/m)	ρ_s (kg/m³)
OSB	12	1100	220	5.5×10^7	3.6×10^7	600
	15	1400	300	5.6×10^7	3.6×10^7	600
	18	2800	720	7.5×10^7	4.4×10^7	600
	22	6100	2100	9.9×10^7	5.5×10^7	600
	28.5	11 000	4400	10.8×10^7	6.1×10^7	600
DFP	12.5	1700	350	9.4×10^7	4.7×10^7	550
	15.5	3000	630	9.4×10^7	4.7×10^7	550
	18.5	4600	1300	9.4×10^7	4.7×10^7	550
	20.5	5800	1900	9.4×10^7	4.7×10^7	550
	22.5	7600	2500	12.0×10^7	7.5×10^7	550
	25.5	11 000	3700	12.0×10^7	7.1×10^7	550
	28.5	14 000	5700	14.0×10^7	9.5×10^7	550
	31.5	18 000	7400	14.0×10^7	9.5×10^7	550
CSP	12.5	1400	350	7.9×10^7	4.7×10^7	500
	15.5	2300	630	7.9×10^7	4.7×10^7	500
	18.5	3900	1 400	10.0×10^7	7.1×10^7	500
	20.5	4600	1 900	7.9×10^7	4.7×10^7	500
	22.5	5700	2 200	10.0×10^7	7.1×10^7	500
	25.5	8000	3 400	10.0×10^7	7.1×10^7	500
	28.5	11 000	5200	13.0×10^7	9.5×10^7	500
	31.5	14 000	6700	13.0×10^7	9.5×10^7	500

* *The design properties are based on Tables 9.1 to 9.3.*

Note: *"||" stands for 0° orientation of applied force relative to face grain. "⊥" stands for 90° orientation of applied force relative to face grain.*

a) $EI_{s\perp}$ *= bending stiffness of 1 m wide subfloor panel in span direction.*

b) $EA_{s||}$ *= unit axial stiffness of subfloor in across-span direction.*

c) $EA_{s\perp}$ *= axial stiffness of 1 m wide subfloor panel in span direction.*

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Δ

Table A.2
Properties of topping materials
(See Clauses A.5.4.5.1.1 and A.5.4.5.1.2.)

Material	E_c (N/m²)	P_c (kg/m³)
Normal weight concrete	22×10^9	2300
Wood panel overlay*	$12EI_{s\perp}/t_c^3$	Density in Table A.1

* *See Table A.1 for the value of* $EI_{s\perp}$.

A.5.4.5.2 Vibration-controlled span — Prefabricated wood I-joist floors

The vibration-controlled span for an engineered floor system constructed with prefabricated wood I-joists and a wood-based structural sheathing subfloor may be determined using

a) Clause A.5.4.5.1; or
b) good engineering practice such as that provided in CWC *Concluding Report on Development of Design Procedures for Vibration Controlled Spans Using Engineered Wood Members*, with the following conditions:
   i) subfloor thickness shall not be greater than 28.5 mm;
   ii) the modification factor applied to the effective composite bending stiffness of the joist for multiple-span system shall be 1.2; and
   iii) when concrete topping is used, its stiffness contribution shall not be applied.

A.5.4.6 Building movements due to moisture content change

Most buildings can accommodate small amounts of movement due to moisture content change in wood members. However, if not considered during design and construction, differential movements can become visible or even cause structural or serviceability problems. The considerations for differential movements become more critical for higher buildings due to the cumulative effect of movement.

Shrinkage can contribute to overall lateral drift but can be mitigated using shrinkage compensators and materials subject to less dimensional change.

Attention should be paid to the following areas to avoid potential shrinkage and swelling-related problems in the design, where:

a) non-uniform movements could occur;
b) metal connectors are used to support large sawn and glulam timber components;
c) differential movements could cause distress in the finish materials or building envelope; and
d) differential movements could cause distress in plumbing, electrical, and mechanical systems.

Compatibility with other materials should be considered in the design detailing. Assuming similar members are exposed to the same environmental conditions, it is reasonable to assume the same degree of shrinkage or swelling will occur along each load path. Where wood members are exposed to different environmental conditions (e.g., some members are in a conditioned space while others are in an unconditioned space), a more detailed analysis is warranted. Similarly, expected movement in non-wood members, such as contraction or expansion due to temperature changes should be considered.

The shrinkage or swelling of a wood member between the initial and final moisture content may be estimated using the following equation:

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$S = D \times (M_i - M_f) \times c$

where

S = shrinkage or swelling in the dimension being considered (thickness, width, or length), mm

D = actual dimension (thickness, width, or length), mm

M_i = the lesser of the initial moisture content or the fibre saturation point (28%)

M_f = the final moisture content

c = shrinkage coefficient*

For lumber

= 0.002 for shrinkage or swelling perpendicular to the grain

= 0.00005 for shrinkage or swelling parallel to the grain

* *More information on shrinkage coefficients for individual wood species can be found in the CWC* Wood Design Manual*. For other wood products, refer to published literature.*

A.6 Sawn lumber

A.6.2.1.2 Availability of lumber

The availability of large-size and long-length timbers (Beams and Stringers and Posts and Timbers Grades) should always be confirmed with suppliers prior to specifying. Sizes up to 394 × 394 mm are generally available in the Douglas Fir-Larch and Hem-Fir species combinations; however, S-P-F and Northern Species timbers are available only in smaller sizes.

Machine stress-rated lumber and machine evaluated lumber are commonly produced for specialty applications such as light-frame trusses and I-joists. Some designers specify machine graded lumber to take advantage of the higher stiffness and strength properties.

The most commonly available grades of machine stress-rated lumber are 1650f-1.5E, 1800f-1.6E, 2100f-1.8E, and 2400f-2.0E; the most common sizes are 38 × 89 mm and 38 × 140 mm. The availability of other grades and sizes should be confirmed with suppliers prior to specifying.

A.6.5.2 Sizes

Table A.3
Minimum dressed sizes of dimension lumber and timbers*
(See Clause 6.5.2.2.)

	Smaller dimension, mm		Larger dimension, mm	
Item	**Dry**	**Green**	**Dry**	**Green**
Dimension lumber	38	40	38	40
	51	53	64	66
	64	66	89	91
	76	78	114	117
	89	91	140	143

(Continued)

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Table A.3 (Concluded)

Item	Smaller dimension, mm		Larger dimension, mm	
	Dry	Green	Dry	Green
	102	104	184	190
	—	—	235	241
	—	—	286	292
	—	—	337	343
	—	—	387	393
Timbers	—	114	—	114
	—	140	—	140
	—	165	—	165
	—	191	—	191
	—	216	—	216
	—	241	—	241
	—	292	—	292
	—	343	—	343
	—	394	—	394

* *Sizes are rounded to the nearest whole millimetre and are based on CSA O141.*

Notes:

1) *Dry lumber is defined as lumber that has been seasoned or dried to a moisture content of 19% or less.*
2) *Green lumber is defined as lumber having a moisture content more than 19%.*

A.6.5.5.1 Effective-length factor, K_e

Table A.4
Minimum design values of effective-length factor, K_e, for compression members
(See Clauses 6.5.5.1, 7.5.8.1, 8.4.5.2, 15.3.3.5.1, A.6.5.9, A.6.5.11.1.5, and A.6.5.12.3.)

Degree of end restraint of compression member	Effective-length factor, K_e	Symbol
Effectively held in position and restrained against rotation at both ends	0.65	
Effectively held in position at both ends and restrained against rotation at one end	0.80	

(Continued)

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Table A.4 (Concluded)

Degree of end restraint of compression member	Effective-length factor, K_e	Symbol
Effectively held in position at both ends but not restrained against rotation	1.00	
Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position	1.20	
Effectively held in position and restrained against rotation at one end, and at the other partially restrained against rotation but not held in position	1.50	
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position	2.00	
Effectively held in position and restrained against rotation at one end but not held in position or restrained against rotation at the other end	2.00	

Note: *Effective length $L_e = K_eL$, where L is the distance between centres of lateral supports of the compression member in the plane in which buckling is being considered. At a base or cap detail, the distance shall be measured from the outer surface of the base or cap plate. The effective-length factor, K_e, shall not be less than what would be indicated by rational analysis. Where conditions of end restraint cannot be evaluated closely, a conservative value for K_e shall be used.*

A.6.5.5.2.4 Factored compressive resistance parallel to grain

Except for members designed in accordance with Clause 6.5.12, the following procedure may be used to determine the factored compressive resistance parallel to grain of members subject to a compressive stress that is not uniform along its length, either due to concentrated loads or distributed loads:

a) Divide the member into two sections with lengths L_1 and L_2 as follows:
   L_1 = length of member with a maximum factored compressive force P_{f1}, mm
   L_2 = length of member with a maximum factored compressive force P_{f2}, mm
   such that $P_{f1} < P_{f2}$

b) Determine the factored compressive resistance parallel to grain, P_{r1}, of section L_1 in accordance with Clause 6.5.5.2.4 using $L = L_1 + L_2$ in the calculation of K_{Zc}.

c) Determine the factored compressive resistance parallel to grain, P_{r2}, of section L_2 in accordance with Clause 6.5.5.2.4 using the following formula:
   $K_{Zc} = 6.3\ (d\ L')^{-0.13} \leq 1.3$
   where
   d = dimension in direction of buckling (depth or width), mm
   $L' = L_1(P_{f1}/P_{f2})^{7.9} + L_2$, mm

d) Verify that $P_{r1} \geq P_{f1}$, and $P_{r2} \geq P_{f2}$. For combined bending and compression, separately check P_{f1} and P_{f2} using P_{r1} and P_{r2} in Clause 6.5.9.

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A.6.5.5.3 Spaced compression members

A.6.5.5.3.1 General

Spaced compression members shall consist of two or more individual members joined with connectors and having spacer and end blocks as specified in Clauses A.6.5.5.3.2 to A.6.5.5.3.4.

A.6.5.5.3.2 Spacer and end blocks

Spacer and end blocks shall meet the following requirements:

a) End blocks shall be placed so that end and edge distances and spacing, as required by Clause 12 for the size and number of connectors, are maintained in end blocks and in individual members. Connectors shall be placed so that the limits specified in Clause A.6.5.5.3.3, depending on the fixity factor assumed, are met. In compression members of trusses, a panel point that is braced laterally may be considered as the end of the spaced member.

b) A single spacer block shall be located within the middle 10% of the length of the compression members; when so located, connectors shall not be necessary for this block. When more than one spacer block is used, the distance between any two blocks shall not exceed one-half the distance between centres of connectors in the end blocks. When two or more spacer blocks are used, connectors shall meet the requirements of Item a).

c) The thickness of spacer and end blocks shall not be less than that of the individual members of the spaced compression member, except that spacer and end blocks of a thickness between that of the individual members and one-half that thickness may be used, provided that the length of the blocks is made inversely proportional to the thickness in relation to the required length of full-thickness block. Spacer and end block sizes shall be adequate to develop the strength required by Clause 12.

A.6.5.5.3.3 Fixity classes

The end-fixity of spaced compression members shall be classified as one of the following in accordance with Figure A.2:

a) Condition A: the centroid of connectors or of the connector group in the end block shall be within 1/20 of the length, L, from the end of the member.

b) Condition B: the centroid of connectors or of the connector group in the end block shall be between 1/20 and 1/10 of the length, L, from the end of the member.

A.6.5.5.3.4 Connectors in end blocks

The connectors in each pair of contacting surfaces of end blocks and individual members at each end of a spaced compression member shall be at least of a size and number to provide a factored strength resistance, in newtons, equal to the required cross-sectional area in square millimetres of one of the individual members multiplied by the appropriate end block constant specified in Table A.5.

A.6.5.5.3.5 Factored compressive resistance parallel to grain

A.6.5.5.3.5.1 General

The factored compressive resistance, P_r, parallel to grain of a spaced member shall be the smaller value calculated in Clauses A.6.5.5.3.5.2 and A.6.5.5.3.5.5.

Δ #### A.6.5.5.3.5.2 Resistance of spaced compression member

The factored compressive resistance, P_r, parallel to grain of a spaced compression member shall be calculated as follows:

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$P_r = \phi F_c A K_c K_{Zc}$

where

ϕ = resistance factor

= 0.80 for sawn lumber

= 0.90 for glued-laminated timber

F_c = $f_c (K_D K_{Sc} K_T)$

where

Δ f_c = specified strength in compression parallel to grain, MPa (see Tables 6.4 to 6.9 for sawn lumber and Table 7.2 for glulam)

A = total cross-sectional area of members, mm²

K_C = slenderness factor (see Clause A.6.5.5.3.5.4)

K_{Zc} = size factor

= $6.3(dL)^{-0.13} < 1.3$ for sawn lumber

= 1.0 for glued-laminated timber

where

d = member dimension in direction of buckling (depth or width), mm

L = length of member, mm

Δ A.6.5.5.3.5.3 Slenderness ratio, C_C

The slenderness ratio, C_C, of spaced compression members of uniform rectangular section shall not exceed 80 and shall be calculated as follows:

$$C_C = \frac{\text{actual length between points of lateral support}}{\text{least dimension of an individual member}}$$

Δ A.6.5.5.3.5.4 Slenderness factor, K_C

The slenderness factor, K_C, depending on C_C (Clause A.6.5.5.3.5.3), shall be calculated as follows:

a) when C_C does not exceed 10:

$K_C = 1.00$

b) when C_C is greater than 10 but does not exceed C_K:

$$K_c = 1 - \frac{1}{3}\left(\frac{C_C}{C_K}\right)^4$$

where

$$C_K = \sqrt{\frac{0.76E_{05}K_{SE}K_EK_T}{F_c}}$$

where

E_{05} = applicable value from Tables 6.4 to 6.7 for visually graded sawn lumber

= $0.82E$ for machine stress-rated lumber

= $0.87E$ for glued-laminated timber

K_E = end-fixity factor (see Clause A.6.5.5.3.3)

= 2.50 for Condition A

= 3.00 for Condition B

c) when C_C is greater than C_K but does not exceed 80:

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$$K_c = \frac{E_{05}K_{SE}K_EK_T}{KC_C^2F_c}$$

where

k = 1.8 for sawn lumber

= 2.0 for glued-laminated timber

Δ **A.6.5.5.3.5.5 Total resistance of individual members**

The factored resistance of spaced compression members shall be checked against the sum of factored resistances of individual members taken as simple compression members where C_C shall be taken as follows:

$$C_C = \frac{\text{actual length between points of lateral support}}{\text{larger dimension of an individual member}}$$

A.6.5.5.3.6 Combined bending and axial load

When axial compression in spaced compression members is combined with bending stresses, Clause 6.5.9 shall be used only if the bending is in a direction parallel to the larger dimension of the individual member.

Table A.5
End block constants for spaced compression members, MPa
(See Clause A.6.5.5.3.4.)

	Species combination			
C_C*	D Fir-L	Hem-Fir (N)	S-P-F	North Species
0–10	0.00	0.00	0.00	0.00
15	0.38	0.30	0.26	0.23
20	0.79	0.62	0.55	0.47
25	1.2	0.92	0.81	0.72
30	1.5	1.3	1.1	0.96
35	1.9	1.5	1.4	1.2
40	2.2	1.9	1.6	1.5
45	2.6	2.2	1.9	1.6
50	3.0	2.5	2.2	1.9
55	3.4	2.8	2.5	2.2
60–80	3.8	3.0	2.6	2.3

* *Constants for intermediate values of C_C may be obtained by straight-line interpolation.*

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Figure A.2
Spaced compression member (connector joined)
(See Clause A.6.5.5.3.3.)

A.6.5.6 Compression perpendicular to grain

A relationship between mean compressive strength perpendicular to grain and mean oven-dry wood density was introduced to establish a consistent basis for bearing strengths for various products in the 1994 edition of this Standard. It is as follows:

$f_{cp} = 0.9L(2243.8G - 473.8)/M$

where

f_{cp} = specified strength in compression perpendicular to grain, MPa

0.9 = factor applied to obtain a lower tolerance limit

L = conversion factor to limit states design and standard-term load duration
= 1.8125

G = mean oven-dry relative density

M = conversion factor for metric units
= 145.038

Tables 6.8 and 6.9 assign increased compression perpendicular to grain design values to specific grades of Spruce-Pine-Fir or Hem-Fir machine-graded lumber, which have higher mean density than visually stress-graded lumber of the same species (see Table A.11).

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The NLGA *Standard Grading Rules for Canadian Lumber* also includes provisions for mills producing any grade of machine-graded lumber to qualify for other density values based on tests, daily quality control, and marking the qualified density value on the lumber. In these cases the rules provide for compression perpendicular values to be based on the marked density value and the formula specified in this Clause, without the 0.9 tolerance limit.

A.6.5.9 Combined bending and compressive loads

Members subject to combined bending and compressive loads may be designed using the alternative interaction equation:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f + P_f \Delta_L \left[\frac{1}{1 - \frac{P_f}{P_E}}\right]}{M_r} \leq 1$$

where

P_f = factored compressive axial load, N

P_r = factored compressive load resistance parallel to grain, calculated in accordance with Clause 6.5.5, N

M_f = factored bending moment, N•mm

Δ_L = lateral deflection calculated using serviceability load combinations

P_E = Euler buckling load in the plane of the applied moment

$= \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$

where

E_{05} = modulus of elasticity for design of compression members, MPa

I = moment of inertia in the plane of the applied moment, mm^4

L_e = effective length in the plane of the applied moment

$= K_e L$

where

K_e = effective-length factor specified in Table A.4

M_r = factored bending moment resistance, calculated in accordance with Clause 6.5.3, N•mm

A.6.5.11 Permanent wood foundations

A.6.5.11.1 General

Studs for permanent wood foundations may be designed in accordance with recognized engineering methods. When assumed to be laterally supported and when no surcharge exists, the formulas specified in Clauses A.6.5.11.1.5 to A.6.5.11.1.10 give conservative approximations for practical construction. The dimensions used in the formulas are identified in Figure A.3.

A.6.5.11.1.1 Studs

Studs for exterior foundation walls may be designed as members subjected to combined bending and axial compressive loading. Deflection due to lateral and axial loads should not exceed 1/300 of the unsupported height of the stud.

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A.6.5.11.1.2 Sheathing

Sheathing for exterior foundation walls may be designed as simple bending members. The calculated maximum deflection at a point 300 mm above the bottom of the sheathing should not exceed 1/180 of the span of sheathing between studs. The nominal thickness of sheathing should not be less than 12.5 mm.

A.6.5.11.1.3 Floors and connections

Floors and connections between floors and walls shall be designed to withstand loads imposed on them by lateral soil pressure as well as floor loads appropriate for the occupancy.

A.6.5.11.1.4 Unequal pressure distribution

Unequal pressure distribution can result from differing backfill heights on opposite sides of a building, openings in foundation walls, openings in floors at the top of foundation walls, or other causes. Framing members and sheathing shall be designed as diaphragms or other systems to resist loads resulting from unequal pressure distribution.

A.6.5.11.1.5 Combined bending and compressive load effects

Combined bending and compressive load effects may be evaluated using the following formula:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f\left[\frac{1}{1-\frac{P_f}{P_E}}\right]}{M_r} \leq 1$$

where

P_f = factored compressive load on stud, N

P_r = factored compressive resistance calculated in accordance with Clause 6.5.5.2.4, N

M_f = maximum factored bending moment on stud calculated in accordance with Clause A.6.5.11.1.6, N•mm

M_r = factored bending moment resistance calculated in accordance with Clause 6.5.3.1, N•mm

$= \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$

P_E = Euler buckling load in the plane of the applied moment

where

E_{05} = modulus of elasticity for design of compression members, MPa

I = moment of inertia in the plane of the applied moment, mm^4

L_e = effective length in the plane of the applied moment

$= K_e L$

where

K_e = effective-length factor specified in Table A.4

P_E = Euler buckling load in the plane of the applied moment

Note: *A K_D factor of 0.65 applies to the calculation of M_r and a K_D factor of 1.0 applies to the calculation of P_r.*

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A.6.5.11.1.6 Factored bending moment on a stud

A.6.5.11.1.6.1 General

The following formula, which is derived from recognized engineering formulas, may be used to calculate the maximum bending moment on a stud, M_f, due to factored lateral load (see Clause A.6.5.11.1.6.2). At any point, x, above the floor, the factored bending moment due to factored lateral load, M_{fx}, in N•mm, is calculated as follows:

$$M_{fx} = \frac{w_f H^2}{6L}\left[(H-3a)\left(\frac{L-x}{H}\right) - L\left(\frac{H-Ax}{H}\right)^3\right]$$

where

w_f = maximum factored lateral load per stud, N/mm

= maximum factored lateral soil pressure, N/mm^2, times stud spacing, mm

H, L, a, and x = variables shown in Figure A.3, mm

A.6.5.11.1.6.2 Maximum factored bending moment on a stud, M_f

The following formulas may be used to determine the maximum factored bending moment, M_f, and its location, x:

a) for wood sleeper and slab floors:

$$M_f = \frac{w_f H^2}{6L}\left[L - H + \frac{2}{3}\sqrt{\frac{H^3}{3L}}\right]$$

$$x = H - \sqrt{\frac{H^3}{3L}}$$

b) for suspended floors, both the moment between supports and the cantilever moment at the support should be checked using

$$M_f = \frac{w_f H^2}{6L} K_m$$

where

$$K_m = (H-3a)\left[K_1 + \frac{L+a-H}{H}\right] - LK_1^3$$

where K_1 is determined as follows:

i) between supports:

$$K_1 = \sqrt{\frac{H-3a}{3L}}$$

$$x = H - a - H\sqrt{\frac{H-3a}{3L}}$$

ii) at the support:

$$K_1 = \frac{H-a}{H} \text{ and } x = 0$$

Note: *Values of K_m for a range of backfill heights and typical wall dimensions are specified in Table A.6.*

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Table A.6
Moment and deflection coefficients, K_m and K_Δ, for typical values of L and a*
(See Clause A.6.5.11.1.6.2.)

	Slab floor		Suspended floor		
	$K_\Delta \times 10^{15}$		$K_\Delta \times 10^{15}$		
Backfill height, H, mm	At point of maximum moment	At $x = 0.45L$	At point of maximum moment	At $x = 0.45L$	K_m†
400	0.85	2.5	—	—	—
600	3.8	8.4	0	–14	–3800
800	11	20	0	–20	–2300
1000	23	37	0	–22	–1600
1200	43	62	0	–19	–1100
1400	71	94	0	–10	–840
1600	110	130	0	6.3	–660
1800	150	180	0	31	–530
2000	200	230	0	64	–430
2200	260	280	96	110	400
2400	320	340	150	150	430
2600	—	—	200	210	450
2800	—	—	270	260	450
3000	—	—	330	320	450

* *Tabulated coefficients are for the cases where L = 2400 mm, a = 0 mm for a slab floor; and L = 2500 mm, a = 500 mm for a suspended floor.*
† *Values for K_m (last column) apply only to foundations with suspended floors. Tabulated are the greater of the calculated values for K_m at the support (negative numbers) or between supports (positive numbers). K_Δ at the point of maximum moment (fourth column) is zero where moment at the support governs.*

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Figure A.3
Dimensions and loading of foundation studs without surcharge
(See Clauses A.6.5.11.1, A.6.5.11.1.6, A.6.5.11.1.7, A.6.5.11.1.9, and A.6.5.11.1.10.)

Legend:

L = stud length, mm

x = location of maximum moment, mm

H = height of backfill, mm

A.6.5.11.1.7 Deflection of studs, Δ

The deflection of studs, Δ, can be calculated at any point, x, above the basement floor (see Figure A.3), using the following formula:

$$\Delta = \frac{w_f(L-x)}{360EILH}K_\Delta$$

where

w_f = variable shown in Figure A.3, N/mm

K_Δ = $[10H^2(H-3a)(2L-x)x - 3(H-a)^5 + K_2]$

where

K_2 = $\frac{3L}{L-x}(H-a-x)^5$ when $x \le H-a$

= 0 when $x > H-a$

E = modulus of elasticity of stud, MPa

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I = moment of inertia of stud in the plane of the applied moment, mm^4

A.6.5.11.1.8 Maximum deflection of studs

Maximum deflection may be calculated from Clause A.6.5.11.1.7 with $x = 0.45L$ and using specified rather than factored loads to give a good approximation of the theoretical maximum deflection.

A.6.5.11.1.9 Maximum factored shear force, V_f, per stud

The maximum factored shear force, V_f, in N, per stud may be taken as the greatest value calculated from the following formulas, which are derived from recognized engineering formulas:

a) for wood sleeper and slab floors at the bottom of the foundation wall:

$$V_f = \frac{w_f H}{2}\left[\frac{H}{3L} - \left(\frac{H-d}{H}\right)^2\right]$$

b) for suspended floors:

i) just above the suspended floor:

$$V_f = \frac{w_f H}{2}\left[\frac{H-3a}{3L} - \left(\frac{H-a-c}{H}\right)^2\right]$$

ii) just below the suspended floor:

$$V_f = \frac{w_f H}{2}\left[1 - \left(\frac{H-a+c}{H}\right)^2\right]$$

where

w_f = variable shown in Figure A.3, N/mm

d = depth of stud, mm

c = depth of stud + 1/2 of the joist depth, mm

H, L, a = variables shown in Figure A.3, mm

A.6.5.11.1.10 Lateral restraint

A.6.5.11.1.10.1 Lateral restraint at top

The lateral restraint required at the top of the foundation wall may be calculated from the following formulas:

a) for wood sleeper and slab floors:

$$R_T = \frac{w_f H^2}{6L}$$

b) for suspended floors:

$$R_T = \frac{w_f H}{2L}\left(\frac{H}{3} - a\right)$$

where

R_T = inward reaction at the top of the stud, N

w_f = variable shown in Figure A.3, N/mm

H, L, a = variables shown in Figure A.3, mm

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A.6.5.11.1.10.2 Lateral restraint at bottom

The lateral restraint required at the bottom of the foundation wall may be calculated from the following formulas:

a) for wood sleeper and slab floors:

$$R_B = \frac{w_f H}{2L}\left(L - \frac{H}{3}\right)$$

b) for suspended floors:

$$R_B = \frac{w_f H}{2L}\left(L + a - \frac{H}{3}\right)$$

where

R_B = inward reaction at the bottom of the stud, N

w_f = variable shown in Figure A.3, N/mm

H, L, a = variables shown in Figure A.3, mm

A.6.5.12 Specific truss applications

A.6.5.12.1 Scope

Fully triangulated — The modified design procedures of Clause 6.5.12* apply only to sawn lumber used in trusses where all the members form a side of a triangle. In such a system, a bending failure at a panel point of a compression member would not normally result in collapse of the triangle. An attic truss is not a fully triangulated system. Also, the ends of top chords in top-chord-bearing trusses, and top-chord overhangs (i.e., outward extensions of truss chords beyond the panel points) do not fall within the scope of Clause A.6.5.12 and therefore are not considered part of the overall truss length.

** The modified design procedures of Clause 6.5.12 are based on research reported in C. Lum, E. Jones, and B. Hintz (1996).*

Clear span limitation — The 12.2 m clear span limitation is consistent with that for snow loading for *National Building Code of Canada* Part 9 structures (see Subsection 9.4.2). When the 12.2 m span limit is exceeded or when other conditions are in effect, an 80% snow load factor shall be used (i.e., the truss is designed as a Part 4 truss). When the 610 mm spacing limit is exceeded, the 80% snow load factor shall be used and the K_H factor is reduced from 1.1 to 1.0.

Truss configuration — Clause A.6.5.12 limits the applicability of Clause 6.5.12 to roof trusses with slopes that are similar to those trusses tested. Compared to the standard pitched chord trusses evaluated in the testing program, very low pitch (less than 2 in 12) roof and flat trusses have a higher axial force to bending moment ratio and are excluded from coverage in this Standard.

The overall length is a limit that the truss industry has traditionally used to place additional limits on design. The two limits, 12.2 m clear span and the 18.0 m overall length, provide a boundary between traditional residential spans and long span roof trusses. Below the 18.0 m overall truss length limit, the truss may consist of cantilevers or multiple clear spans, or both, provided that no single span exceeds 12.2 m.

Girder, bowstring, semi-circular, attic, flat, and floor trusses are outside the scope of Clause A.6.5.12.

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A.6.5.12.3 Compressive resistance parallel to grain

Effective length — Recommended effective-length factors are specified in Table A.4. TPIC also provides recommendations on the effective length of truss compression members. Factor selection depends on the load distribution and the type of structure.

Compression-chord splices — For strength and deflection calculations, a compression-chord splice may be considered continuous if it is located at an inflection point for the load case being considered. To allow for changes in lumber length, the splice need only be within ±10% of the panel length from the inflection point. This allowance also recognizes that the location of the inflection point will depend on the loading condition.

Size effects in compression — In most cases, the member length for computing K_{Zc} is the panel length; between panel points, the axial force changes little compared with changes at the panel points. In cases where the axial stresses are high relative to the bending stresses, resulting in relatively short panel lengths, and where the axial forces are relatively constant between adjacent panels, the member length for computing K_{Zc} should include several panels. One-half the chord length between pitch breaks has been judged to cover these cases.

Note: *A pitch break is a point along the chord analogue line where the slope of the chord changes.*

A.6.5.12.4 Compressive resistance perpendicular to grain

Bearing reinforcement — Bearing reinforcement consists of applying truss plates to both sides of a member that can be subjected to compression perpendicular to grain stresses through the depth of the member. Designs also need to meet the basic bearing requirement of Clause 6.5.6.1.

Note: *Testing to support this method of improving the bearing resistance is described in Bulmanis, N. S., Latos, H. A., and Keenan F. J. (1983).*

A.6.5.12.5 Resistance to combined bending and axial load

Amplified moments — The proposed design procedures have been written to cover the general case where amplified moments are used. Although moment amplification should not be ignored, some structures can be accurately analyzed without having to use analysis methods that include moment amplification. An analysis of moment amplification in trusses indicated that mid-panel moments were more susceptible to P-delta effects, while panel point moments were less susceptible. Furthermore, the amount of moment amplification calculated is sensitive to the type of structure and the analogue assumptions.

Results from the testing program suggest that the design procedure used by TPIC results in trusses that exhibit satisfactory levels of performance. The TPIC design approach does not include moment amplification; however, the procedure does require that the mid-panel deflection be limited. This, in effect, limits the length of truss panels, even though the chord member can possess the required strength. If the deflection limitations in TPIC are not used to limit panel length, it is possible that the bending capacity modification factor, K_M, from Table 6.19 will not be appropriate.

Bending capacity modification factor, K_M — In addition to being a function of the member's span-to-depth ratio, the K_M factor, calculated from Table 6.19,* also depends on the shape of the moment diagram.

A K_M factor greater than 1 generally indicates a bending moment distribution with one or more inflection points in the member between panel points, and a panel point moment that is higher than the mid-panel moment.

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For cases where the mid-panel moment is higher than the panel point moments or where the loading is such that there are no inflection points in the panel, the K_M factor is generally less than 1.

For other cases where the structural analysis indicates zero moment at the panel points (i.e., $M_2 = 0$), the bending capacity increase will simply be a function of the span-to-depth ratio. This generally occurs at pitch breaks or panel point splices, where, although the bending moments are not necessarily zero, designers traditionally assume a pin connection. However, a fictitious analogue member, such as that used to model a heel joint in a pitched chord truss, is considered to introduce a panel point moment in the heel. Therefore, the top chord of a king post truss can be assumed to be continuous over a panel point at the heel, but not at the peak.

The test trusses and trusses analyzed in the impact study, which are considered typical designs, result in values of K_M up to 1.3. The upper limit on K_M at 1.3 has been introduced to prevent extrapolation to higher values.

* *The K_M factors calculated from Table 6.19 are based on Lau, W., Barrett, J. D., and Lam, F. (1995).*

Interaction equation — The modification to the axial ratio term was introduced to recognize the increase in bending capacity when a brittle material is subjected to axial compression in addition to bending. The K_M factor is used to adjust the bending capacity for various moment configurations in the top chords.

These two changes to the design of compression chords help to explain the satisfactory levels of safety observed in the test trusses.

No increase is permitted for combined tension and bending members, as the testing program was not designed to evaluate the performance of tension chord members.

A.7 Glued-laminated timber (glulam)

A.7.5.5 Standard sizes for glued-laminated timber

The standard dimensions for glued-laminated timber are as follows:

a) widths of 80, 130, 175, 215, 265, 315, and 365 mm; and
b) depths, as calculated for
   i) straight or cambered members, in multiples of 38 mm; and
   ii) members curved to a radius of curvature less than 10 800 mm, in multiples of the required lamination thickness (see Table A.7).

Notes:

1) *Commercially available widths can vary.*
2) *For widths greater than 365 mm, designers should check availability before specifying.*

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Table A.7
Minimum radius of curvature
(See Clauses 7.5.6.5.2 and A.7.5.5.)

Lamination thickness, mm	Minimum radius of curvature, mm	
	Tangent end*	Curved end
Standard		
38	8400	10 800
19	2800	3800
Non-standard		
35	7400	9500
32	6300	8500
29	5600	7300
25	4600	6200
16	2300	3000
13	1800	2200
10	1200	1400
6	800	800

* *Tangent end requires a straight length of finished lamination beyond the tangent point of not less than 32 times the lamination thickness.*

A.8 Cross-laminated timber (CLT)

Δ A.8.5.2 Deflection of CLT panels

The maximum deflection of a CLT panel, Δ_{max}, may be calculated as the sum of deflections under short- and long-term loads as follows:

$$\Delta_{max} = \Delta_{ST} + \Delta_{LT} K_{creep}$$

where

Δ_{ST} = elastic deflection due to short-term and/or standard-term loads, without dead loads in combination

Δ_{LT} = instantaneous elastic deflection due to long-term loads

K_{creep} = creep adjustment factor

= 2.0 for dry-service condition

Deflection under a specified uniformly distributed load, ω, acting perpendicular to the face of a single-span panel may be calculated as a sum of the deflections due to moment and shear effects using the effective flatwise bending stiffness, $(EI)_{eff,f}$, and the effective shear stiffness in flatwise bending, $(GA)_{eff,f}$, as follows:

$$\Delta = \frac{5}{384}\frac{\omega L^4}{(EI)_{eff,f}} + \frac{1}{8}\frac{\omega L^2}{(GA)_{eff,f}}$$

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For a concentrated line load, *P,* located in the middle of a single span CLT panel acting perpendicular to the panel, the deflection may be calculated as follows:

$$\Delta = \frac{1}{48}\frac{PL^3}{(EI)_{eff,f}} + \frac{1}{4}\frac{PL}{(GA)_{eff,f}}$$

Note: *Where the shear deformation component of the total deformation of CLT panel under out-of-plane standard-term loading such as snow and live loads is significant (i.e., in short spans, short span cantilever, etc.) as determined by the designer, the shear deformation under these loads should be increased by 30% to account for time-dependent effect associated with rolling shear.*

Δ A.8.5.3 Vibration performance of CLT floors

The allowable span for the control of floor motions caused by walking on a CLT floor may be calculated using the following equation:

$$\ell_v \leq 0.11\frac{\left(\frac{(EI)_{eff,f}}{10^6}\right)^{0.29}}{m^{0.12}}$$

where

ℓ_v = vibration-controlled span limit, m

$(EI)_{eff,f}$ = effective flatwise bending stiffness for a 1 m wide panel, N•mm^2 (see Clause 8.4.3.2)

m = linear mass of CLT for a 1 m wide panel, kg/m

Note: *The above equation assumes the floor*

a) *has a single span with both ends simply supported; and*

b) *is bare with no topping material.*

For multiple-span floors with a non-structural element that is considered to provide enhanced vibration effect, the calculated vibration-controlled span may be increased by up to 20%, provided it is not greater than 8 m.

Notes:

1) *The calculation procedure given in this Clause for single-span systems may be applied to individual spans in a multiple-span system, with adjustment if appropriate.*

2) *For floors with concrete topping, where the concrete is applied directly to the CLT panel, this equation may be used assuming bare floor construction for calculation purposes, i.e., weight of concrete is ignored in calculating the mass, m, provided the area density of the topping is not greater than twice the bare CLT floor area density.*

3) *The current vibration criteria might not address all occupant performance expectations. CLT manufacturer literature may provide additional span adjustments.*

A.9 Structural panels

A.9.2.2 Construction sheathing OSB

As noted in its preface, CSA O325 does not recommend engineering design values, nor does it suggest methods of calculating such values.

The design values in Table 9.3 for construction sheathing OSB that complies with CSA O325 are consistent with values developed through in-grade testing and supported by various organizations.

Certification organizations shall ensure that reference values for bending strength and bending stiffness of construction sheathing OSB that complies with CSA O325 are also consistent with the design values of this Standard.

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Panel marks consist of an end-use mark followed by a span mark (see Table A.8), e.g., 2F24 or W16. Multiple panel marks may be shown on panels qualified for more than one end use, e.g., 1R24/2F16 and 2R48/2F24/1F24.

Note: *See Table A.9.*

Table A.8
Panel marks for construction sheathing products (CSA O325)
(See Clause A.9.2.2.)

a) Panel marks

	Span marks					
	16	20	24	32	40	48
	Recommended framing member spacing, mm (in)					
End-use mark	400 (16)	500 (20)	600 (24)	800 (32)	1000 (40)	1200 (48)
1F	1F16	1F20	1F24	1F32	*	1F48
2F	2F16	2F20	2F24	*	*	*
1R	1R16	1R20	1R24	1R32	1R40	1R48
2R	2R16	2R20	2R24	2R32	2R40	2R48
W	W16	W20	W24	*	*	*

* *Not covered by CSA O325.*

Note: *The span mark relates to the centre-to-centre spacing of supports (test span) used for qualification testing of construction sheathing products. These spans are based on assumed end use and framing member spacings normally found in light wood-frame construction. The framing member itself shall be designed for the expected loads using recognized engineering procedures.*

b) End-use marks

Assumed end use	For panels marked
Single-layer flooring (combination subfloor/ underlayment)	1F
Subflooring used with panel-type underlayment	2F
Roof sheathing used without edge support	1R
Roof sheathing used with edge support	2R
Wall sheathing	W

* *Panels marked 1 are usually stiffer than panels marked 2.*

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Table A.9
Relationship between panel marks and nominal thickness
(See Clauses 11.3.2.2 and A.9.2.2.)

Panel mark	Nominal thickness, mm											
	7.5	9.5	11	12	12.5	15	15.5	18	18.5	22	25	28.5
2R20, W16	P	—	—	—	—	—	—	—	—	—	—	—
2R24, W24	—	P	A	A	A	—	—	—	—	—	—	—
1R24/2F16	—	—	P	A	A	—	—	—	—	—	—	—
2R32/2F16	—	—	—	P	A	A	A	—	—	—	—	—
2R40/2F20	—	—	—	—	—	P	A	A	A	—	—	—
2R48/2F24	—	—	—	—	—	—	—	P	A	A	—	—
1F16	—	—	—	—	—	P	A	—	—	—	—	—
1F20	—	—	—	—	—	P	A	—	—	—	—	—
1F24	—	—	—	—	—	—	—	P	A	—	—	—
1F32	—	—	—	—	—	—	—	—	—	P	A	—
1F48	—	—	—	—	—	—	—	—	—	—	—	P

Note: *P indicates the most commonly available nominal thickness. A indicates possibly available nominal thicknesses. Availability can be determined by checking with suppliers.*

A.10 Composite building components

A.10.6.3.4 Buckling of plywood compression flange

For stressed skin panels with a plywood compression flange, the factored buckling resistance, expressed as the maximum factored load applied perpendicular to the compression flange, shall be calculated as follows:

$$B_r = \phi X_B \frac{(2h_w + h_t + h_c)(10^9)}{2(s\,\ell_p)^2}$$

where

B_r = factored buckling resistance, kN/m^2

ϕ = 0.95

X_B = buckling coefficient (see Table A.10)

s = clear spacing between longitudinals, mm

ℓ_p = span of stressed skin panel, mm

Note: *This Clause is not applicable to OSB compression flanges. Buckling coefficients have not been developed for OSB.*

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Table A.10
Buckling coefficient, X_B, for the plywood compression flange of stressed skin panels, kN•m
(See Clause A.10.6.3.4.)

	Buckling coefficient, X_B	
Unsanded plywood thickness, mm	**Face grain parallel to span**	**Face grain perpendicular to span**
7.5	23	53
9.5	59	110
12.5	160*	240
15.5	350	470
18.5	610	790
20.5	880	1050

** For 12.5 mm, three-ply plywood and face grain parallel to span, use X_B = 120.*

A.11 Lateral-load-resisting systems

A.11.7 Sheathing-to-framing connection deformation, e_n, for shearwall and diaphragm deflection calculations

For nails used in wood-based sheathing with dry lumber, e_n may be calculated as follows:

$$e_n = \left[\frac{0.013\,vs}{d_F^2}\right]^2$$

where

v = maximum specified shear force per unit length along the diaphragm boundary or top of shearwall, N/mm

s = nail spacing along panel edges, mm

For nails used in wood-based sheathing with green lumber, multiply the calculated dry lumber e_n values by 2.

For fasteners used in gypsum wallboard with dry lumber, e_n may be taken as 0.76 mm.

Δ **A.11.7.1 Deflection of shearwalls in multi-storey buildings**

a) Deflection of blocked shearwall segments:
The interstorey drift, Δ_i^{storey}, at the *i*th level, mm, may be calculated as follows:

$$\Delta_i^{storey} = \Delta_{b,i}^{storey} + \Delta_{s,i}^{storey} + \Delta_{n,i}^{storey} + \Delta_{a,i}^{storey}$$

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The total deflection, Δ_i^{total}, at the *i*th level, mm, may be calculated as follows:

$$\Delta_i^{total} = \sum_{j=1}^{i} \Delta_j^{storey}$$

where

$\Delta_{b,i}^{storey}$ = interstorey drift at the *i*th level due to bending

$\Delta_{s,i}^{storey}$ = interstorey drift at the *i*th level due to panel shear

$\Delta_{n,i}^{storey}$ = interstorey drift at the *i*th level due to nail slip

$\Delta_{a,i}^{storey}$ = interstorey drift at the *i*th level due to vertical elongation of the wall anchorage system

i) Deflection due to bending (see Figure A.4)

$$\Delta_{b,i}^{storey} = \Delta_{b,i} + H_i\left(\Sigma_{j=1}^{i-1}\theta_j\right) = \Delta_{b,i} + H_i(\theta_1 + \theta_2 + \ldots + \theta_{i-1})$$

where

θ_i *(or* θ_j*)* = change in slope over the *i*th (or the *j*th) storey height subjected to bending, as shown in Figure A.5, and may be calculated as follows:

$$\theta_i = \frac{M_i H_i}{(EI)_i} + \frac{V_i H_i^2}{2(EI)_i}$$

$\Delta_{b,i}$ = the horizontal distance from the top of the *i*th storey to the tangent of the deflection curve at the bottom of the *i*th storey, as shown in Figure A.5, and may be calculated as follows:

$$\Delta_{b,i} = \frac{M_i H_i^2}{2(EI)_i} + \frac{V_i H_i^3}{3(EI)_i}$$

where

H_i = interstorey height of the *i*th storey, mm

V_i = shear force in the shearwall at the *i*th storey, N, is calculated as follows:

V_i = $\Sigma_{j=i}^{n} F_j = F_i + F_{i+1} + \ldots + F_n$

M_i = overturning moment at level *i* (at the top of *i*th storey), N•mm, as shown in Figure A.6, is calculated as follows:

$$M_i = \Sigma_{j=i+1}^{n} V_j H_j$$

where

n = total number of stories

F_j = lateral force applied to the *j*th level, N

H_j = interstorey height of the *j*th storey, mm

$(EI)_i$ = bending stiffness of the shearwall at the *i*th storey, N•mm^2, may be calculated as follows:

$$(EI)_i = (EA)_i\left(\frac{L_s}{2}\right)^2 \bullet 2 = \frac{(EA)_i L_s^2}{2}$$

where

E = modulus of elasticity of the vertical member at shearwall segment boundary, N/mm^2

A = cross-sectional area of the vertical member at shearwall boundary, mm^2

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L_s = length of shearwall segment, mm

Note: *For a continuous rod system anchored at each floor level, the effective bending stiffness of the shearwall should be used.*

Figure A.4
Deflection due to bending
[See Clause A.11.7.1 a) i).]

Source: *APEGBC Technical and Practice Bulletin.*

ii) Deflection due to panel shear:

$$\Delta_{s,i}^{storey} = \Delta_{s,i}$$

where

$$\Delta_{s,i} = \frac{v_{f,i} H_i}{B_{v,i}}$$

where

$v_{f,i}$ = $\frac{V_i}{L_s}$ = maximum shear flow due to specified loads at the top of the wall at the *i*th storey, N/mm

$B_{v,i}$ = shear-through-thickness rigidity of the sheathing at the i-th storey, N/mm (see Tables 9.1 to 9.3 for wood structural panel sheathing. For gypsum wallboard, B_v may be taken as 7000 N/mm)

iii) Deflection due to sheathing-to-framing connection deformation:

$$\Delta_{n,i}^{storey} = \Delta_{n,i}$$

where

$\Delta_{n,i}$ = $0.0025 H_i e_{n,i}$

where

$e_{n,i}$ = sheathing-to-framing connection deformation at the i-th storey, mm (see Clause A.11.7.1)

iv) Deflection due to elongation in wall anchorage system (see Figures A.7 and A.8):

$$\Delta_{a,i}^{storey} = H_i\left(\sum_{j=1}^{i}\alpha_j\right) = H_i\alpha_i + H_i\left(\sum_{j=1}^{i-1}\alpha_j\right)$$

where

α_i = $\frac{(d_a)_i}{L_s}$

where

$(d_a)_i$ = total vertical elongation of the wall anchorage system at the i-th storey

Figure A.5
θ_i and $\Delta_{b,i}$
(See Clause A.11.7.1.)

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Figure A.6
Moment diagram
(See Clause A.11.7.1.)

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Figure A.7
Deflection due to vertical elongation of the wall anchorage system
[See Clause A.11.7.1 a) iv).]

Source: *APEGBC Technical and Practice Bulletin.*

Figure A.8
Deflection due to vertical elongation of the wall anchorage system at the *i*th storey
[See Clause A.11.7.1 a) iv).]

b) Deflection of unblocked shearwall segments:
The interstorey drift of an unblocked shearwall segment with wood structural panels at the *i*th storey may be calculated as follows:

$$\Delta_{ub,i}^{storey} = \frac{\Delta_i^{storey}}{J_{us,i}}$$

where

$\Delta_{ub,i}^{storey}$ = interstorey drift of a blocked shearwall segment at the *i*th level

$J_{us,i}$ = adjustment factor for unblocked shearwall segment at the *i*th level

Note: *This Clause is appropriate for a typical light-frame wood shearwall cantilevered from its base assuming the shearwall is stacked for the full height of the building. This is purely a mechanics-based approach to determine the deflection for a cantilevered single wall acting as a component of the building. There could be other factors that affect the behaviour of a wall, which should be accounted for by the designer.*

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A.12 Connections

A.12.1 Scope

Table A.11
Relative density values
(See Clauses 12.1, 12.4.4.3.3.1, 12.6.5.1.2, 12.6.6.1, 12.9.3.2, 12.9.4.2, 12.11.3.2, 12.11.4.2, 16.5.2, A.6.5.6, and A.12.6.5.4.)

Visually stress-graded lumber	Glued-laminated timber	MSR (or MEL) E Grades of S-P-F*	CLT	Mean oven-dry relative density
		13 800–16 500 MPa		0.50
D Fir-Larch	D Fir-Larch, Hem-Fir†		V1	0.49
		12 400–13 100 MPa		0.47
Hem-Fir	Hem-Fir†			0.46
	Spruce-Pine			0.44
Spruce-Pine-Fir		8300–11 700 MPa	V2, E1	0.42
Northern Species			E3	0.35

** For species of MSR or MEL lumber other than S-P-F, use visually stress-graded lumber values.*
† The outer laminations (top and bottom sides) of 24f-EX and the outer tension laminations on (bottom side) of 24f-E "Hem-Fir and Douglas Fir-Larch" glulam consist of Douglas Fir-Larch; for this reason, use the relative density of Douglas Fir-Larch for these glulams where the fasteners do not pass through the inner laminations. For all other cases, use the visually stress-graded lumber Hem-Fir density values.

Notes:

1) *Relative density values specified in this Table are based on a mean oven-dry mass and oven-dry volume. In the connection equations, these values are modified to relate connection resistances to fifth percentile density values on a 15% moisture content volume basis.*
2) *Where resistance of a connection with bolts, dowels, drift pins, lag screws, nails, or wood screws in wood of a given relative density is not available, use the resistance in a lower relative density wood or calculate resistance using the equations specified in this Standard.*

A.12.6.5.4 Lateral deformation of lag screw connections

For a specified load, P, the lateral deformation of a lag screw connection may be calculated as follows:

$$\Delta = \frac{P}{k d_F t_2 n_F}$$

where

Δ = lateral deformation, mm

P = specified load on the connection, N

k = lateral slip resistance of a lag screw, MPa

$= k_P = (5.04G - 0.29) J_Y J_G K_{SF}$ for $\theta = 0°$ (parallel-to-grain loading)

$= k_Q = (5.04G - 0.29) J_Q J_G K_{SF}$ for $\theta = 90°$ (perpendicular-to-grain loading)

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$= k_p\, k_Q / \left(k_P \sin^2\theta + k_Q \cos^2\theta\right)$ for $0° < \theta < 90°$ (angle-to-grain loading)

where

G = mean relative density (see Table A.11)

J_Y = side plate factor

= 1.5 for steel side plates

J_G = group-action factor (see Tables 12.2 and 12.3)

J_Q = perpendicular-to-grain load factor (see Table A.12)

K_{SF} = service-condition factor (see Table 12.1)

d_F = nominal diameter of lag screw, mm

t_2 = length of penetration into main member, mm (see Clause 12.6.3)

n_F = number of fasteners in the connection

Note: *Lag screws loaded laterally tend to reach a point of permanent deformation at about 0.8–1.0 mm. This limit should be avoided to prevent permanent deformation.*

Table A.12
Factor, J_Q, for perpendicular-to-grain loading of lag screws
(See Clause A.12.6.5.4.)

Lag screw nominal diameter, in	J_Q
3/16	1.00
1/4	0.97
5/16	0.85
3/8	0.76
7/16	0.71
1/2	0.65
5/8	0.60
3/4	0.55
7/8	0.52
1	0.50

A.12.7.3.3 Analysis of timber rivet joints — General method for Douglas Fir-Larch glulam

Note: *The design formulas in this Clause are derived from tests on timber rivet (also referred to as "glulam rivet") connections in Douglas Fir-Larch glulam. For connections in other species of glulam or in sawn timber, reduce the resulting strength resistance by the factor H specified in Clause 12.7.3.2.*

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Δ A.12.7.3.3.1 Parallel-to-grain loading

The specified unit lateral strength resistance for wood capacity, p_w, kN, per joint-loaded parallel to grain is equal to the least value of the tensile strength of the timber, P_t, and the shear strength of the timber, P_v, calculated as follows:

a) $P_t = \frac{X_t f_{tn} S_Q (n_R - 1)}{K_T \beta_t E_t}$

where

X_t = adjustment factor for tension parallel to grain

= 1.41

f_{tn} = specified strength in tension parallel to grain at net section, MPa (see Table 7.2)

S_Q = rivet spacing perpendicular to grain, mm (see Figure 12.13)

n_R = number of rows of rivets

K_T = constant depending on n_R and n_C (see Table A.13)

where

n_C = number of rivets per row

β_t = constant depending on S_p, S_Q, and n_R (see Table A.14)

where

S_p = rivet spacing parallel to grain, mm (see Figure 12.13)

E_t = constant depending on b and L_p (see Table A.15)

where

b = member dimension, mm (for connections with steel plates on opposite sides, member dimension is the width of the member; for connections having only one plate, member dimension is twice the width of the member)

b) $P_v = \frac{X_v F_v L_p \left[S_p (n_c - 1) + 50\right]}{K_v \beta_v Y}$

where

X_v = adjustment factor for shear

= 1.48

F_v = f_v (0.15 + 4.35 C_V), MPa

f_v = specified longitudinal shear strength, MPa (see Table 7.2)

C_V = $(\beta_1 + \beta_2 \beta_3)^{-0.2}$

where

$$\beta_1 = \frac{1}{1300} L_p S_p (n_c - 1)\left[1 + 500 \exp^{-0.52(S_Q - 12.5)}\right]$$

$$\beta_2 = 0.5 \times 10^{-16} L_p [(n_c - 1) S_p]^3 \left[(n_R - 1) S_Q\right]^{4.5} \exp^{-\left(\frac{a-50}{100}\right)}$$

$$\beta_3 = \left(\frac{3\mu - 1}{2} + L_p \frac{1 - \mu}{50}\right)^5 \left(1 - \exp^{\left(1.9 - 0.95 \frac{b}{L_p}\right)}\right)^5$$

where

a = end distance (see Figure 12.13), mm

μ = 43.9 $[(n_C - 1)S_p]^{-0.4}$ $[(n_R - 1)S_Q]^{-0.2}$

L_p = length of penetration (see Figure 12.12)

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K_v = constant depending on n_R and n_C (see Table A.16)

β_v = constant depending on S_p and S_Q (see Table A.17)

Y = 90.5 + 5.4 L_p

Note: *For cases where $b \geq 175$ mm, $S_p = 40$ mm, $S_Q = 25$ mm, and $L_p \geq 55$ mm, P_v will be greater than P_t.*

A.12.7.3.3.2 Perpendicular-to-grain loading

In Clause 12.7.3.4, the lateral strength resistance for wood capacity, q_w, kN, per joint loaded perpendicular to grain and the value of C_t may be calculated as follows:

a) $$q_w = \frac{23.3 X_{tp} f_{tp} \left[(n_R - 1)S_p\right]^{0.8}}{K_{tp}\beta_{tp} 10^3 \left[(n_C - 1)S_Q\right]^{0.2}}$$

where

X_{tp} = adjustment factor for tension perpendicular to grain

= 1.45

f_{tp} = specified strength in tension perpendicular to grain, MPa (see Table 7.2)

K_{tp} = constant depending on n_R and n_C (see Table A.18)

β_{tp} = constant depending on S_p and S_Q (see Table A.19)

b) $$C_t = \frac{1}{\beta_D}\left[\frac{e_p}{(n_c - 1)S_Q}\right]^{-0.2}$$

where

β_D = constant depending on $e_p/(n_C - 1)S_Q$ (see Table A.20)

e_p = unloaded edge distance, mm (see Figure 12.13)

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Table A.13
Values of K_t
(See Clause A.12.7.3.3.1.)

Rivets per row, n_C	No. of rows, n_R									
	2	**4**	**6**	**8**	**10**	**12**	**14**	**16**	**18**	**20**
2	0.75	1.16	1.37	1.47	1.51	1.54	1.57	1.61	1.64	1.64
4	0.51	0.88	1.08	1.17	1.22	1.26	1.30	1.35	1.38	1.38
6	0.38	0.71	0.89	0.97	1.02	1.06	1.11	1.16	1.18	1.18
8	0.30	0.60	0.75	0.84	0.89	0.93	0.97	1.02	1.05	1.05
10	0.26	0.52	0.66	0.74	0.79	0.82	0.87	0.91	0.94	0.94
12	0.23	0.47	0.59	0.66	0.71	0.75	0.78	0.82	0.85	0.85
14	0.21	0.42	0.54	0.60	0.65	0.68	0.72	0.75	0.77	0.77
16	0.20	0.38	0.49	0.56	0.60	0.63	0.67	0.69	0.71	0.71
18	0.18	0.35	0.45	0.52	0.56	0.59	0.62	0.65	0.66	0.67
20	0.17	0.32	0.42	0.49	0.53	0.56	0.59	0.61	0.63	0.64
22	0.16	0.30	0.40	0.46	0.51	0.54	0.56	0.58	0.60	0.61
24	0.15	0.29	0.38	0.45	0.49	0.52	0.53	0.55	0.57	0.58
26	0.14	0.28	0.36	0.42	0.46	0.50	0.51	0.52	0.54	0.55

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Table A.14
Values of β_t
(See Clause A.12.7.3.3.1.)

S_p, mm	S_Q, mm	No. of rows, n_R 2	4	6	8	10	12	14	16	18	20
25	12.5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	15.0	1.11	1.06	1.04	1.02	1.02	1.01	1.01	1.01	1.01	1.01
	25.0	1.68	1.36	1.23	1.14	1.09	1.08	1.08	1.06	1.04	1.03
	32.0	2.03	1.54	1.34	1.22	1.14	1.10	1.09	1.07	1.06	1.05
	40.0	2.37	1.72	1.46	1.29	1.18	1.14	1.12	1.10	1.08	1.06
32	12.5	0.94	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
	15.0	1.05	0.99	0.97	0.95	0.95	0.94	0.94	0.94	0.94	0.94
	25.0	1.58	1.27	1.15	1.07	1.02	1.00	1.00	0.98	0.98	0.97
	32.0	1.90	1.44	1.26	1.14	1.06	1.02	1.01	0.99	0.98	0.98
	40.0	2.23	1.61	1.36	1.21	1.11	1.06	1.04	1.02	1.00	1.00
40	12.5	0.87	0.87	0.87	0.87	0.87	0.87	0.88	0.87	0.87	0.87
	15.0	0.97	0.92	0.90	0.89	0.88	0.88	0.88	0.88	0.88	0.88
	25.0	1.48	1.18	1.07	1.00	0.96	0.93	0.92	0.92	0.91	0.90
	32.0	1.78	1.34	1.17	1.06	0.99	0.96	0.94	0.93	0.93	0.92
	40.0	2.08	1.50	1.27	1.13	1.04	0.99	0.97	0.95	0.94	0.93
50	12.5	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
	15.0	0.84	0.79	0.77	0.76	0.76	0.76	0.76	0.75	0.75	0.75
	25.0	1.27	1.00	0.91	0.85	0.81	0.79	0.79	0.78	0.78	0.77
	32.0	1.54	1.14	1.00	0.91	0.85	0.82	0.82	0.80	0.80	0.79
	40.0	1.80	1.27	1.09	0.97	0.89	0.86	0.85	0.83	0.82	0.81

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Δ

Table A.15
Values of E_t*
(See Clause A.12.7.3.3.1.)

Rivet penetration, L_p, mm	Width of glulam member, b (mm)						
	80	130	175	215	265	315	365
30	24.0	18.6	16.8	16.8	16.8	16.8	16.8
55	25.3	21.2	18.4	16.4	14.8	14.1	14.0
80	25.6	22.7	20.3	18.5	16.7	15.2	14.1

* *For intermediate sawn timber widths, linear interpolation or the following equation may be used to calculate E_t:*

$$E_t = \frac{2\alpha_t Y}{L_p}$$

α_t = 1.0 for $b \geq 6L_p$

= $1.0 + 0.155\left(3 - \frac{b}{2L_p}\right)^2$ for $b < 6L_p$

γ = $90.5 + 5.4L_p$

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Table A.16
Values of K_v
(See Clause A.12.7.3.3.1.)

Rivets per row, n_C	No. of rows, n_R									
	2	**4**	**6**	**8**	**10**	**12**	**14**	**16**	**18**	**20**
2	2.23	1.61	1.15	0.81	0.60	0.48	0.40	0.32	0.25	0.19
4	2.31	1.69	1.22	0.88	0.66	0.53	0.45	0.37	0.30	0.24
6	2.35	1.73	1.27	0.93	0.70	0.57	0.48	0.40	0.33	0.27
8	2.36	1.76	1.30	0.96	0.73	0.60	0.51	0.43	0.36	0.30
10	2.37	1.78	1.32	0.98	0.75	0.62	0.53	0.45	0.38	0.31
12	2.36	1.78	1.33	1.00	0.77	0.63	0.55	0.46	0.39	0.32
14	2.35	1.78	1.34	1.01	0.78	0.64	0.56	0.47	0.40	0.33
16	2.34	1.78	1.34	1.02	0.79	0.65	0.57	0.48	0.40	0.33
18	2.33	1.78	1.35	1.02	0.80	0.66	0.57	0.48	0.40	0.34
20	2.32	1.78	1.35	1.03	0.80	0.66	0.57	0.48	0.40	0.34
22	2.31	1.78	1.35	1.03	0.80	0.66	0.57	0.48	0.40	0.34
24	2.30	1.78	1.35	1.03	0.80	0.66	0.57	0.48	0.40	0.34
26	2.30	1.78	1.35	1.03	0.80	0.66	0.57	0.48	0.40	0.35

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Table A.17
Values of β_v
(See Clause A.12.7.3.3.1.)

		No. of rows, n_R									
S_p, mm	S_Q, mm	2	4	6	8	10	12	14	16	18	20
25	12.5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	15.0	1.00	0.97	0.95	0.95	0.94	0.94	0.94	0.93	0.93	0.93
	25.0	0.97	0.81	0.71	0.67	0.63	0.62	0.61	0.60	0.58	0.57
	32.0	0.95	0.72	0.57	0.51	0.44	0.43	0.42	0.40	0.38	0.35
	40.0	0.94	0.63	0.43	0.34	0.26	0.24	0.23	0.20	0.17	0.13
32	12.5	1.06	1.06	1.06	1.06	1.05	1.05	1.04	1.04	1.03	1.02
	15.0	1.05	1.02	1.00	0.99	0.97	0.97	0.96	0.95	0.94	0.93
	25.0	1.02	0.84	0.71	0.66	0.60	0.58	0.56	0.54	0.51	0.49
	32.0	1.02	0.75	0.58	0.50	0.42	0.40	0.38	0.36	0.33	0.30
	40.0	1.02	0.68	0.46	0.36	0.27	0.25	0.24	0.21	0.18	0.14
40	12.5	1.11	1.12	1.12	1.11	1.10	1.09	1.08	1.07	1.06	1.05
	15.0	1.11	1.08	1.05	1.03	1.01	1.00	0.98	0.97	0.96	0.94
	25.0	1.07	0.84	0.68	0.61	0.53	0.49	0.45	0.42	0.38	0.35
	32.0	1.10	0.79	0.60	0.51	0.42	0.40	0.37	0.34	0.31	0.27
	40.0	1.11	0.73	0.48	0.38	0.27	0.26	0.24	0.22	0.18	0.14
50	12.5	1.22	1.24	1.24	1.22	1.20	1.18	1.16	1.14	1.12	1.10
	15.0	1.23	1.21	1.18	1.15	1.12	1.10	1.08	1.06	1.04	1.02
	25.0	1.26	1.04	0.89	0.82	0.74	0.72	0.71	0.68	0.65	0.62
	32.0	1.27	0.93	0.71	0.61	0.52	0.50	0.48	0.46	0.42	0.39
	40.0	1.29	0.83	0.53	0.41	0.29	0.27	0.25	0.23	0.19	0.15

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Table A.18
Values of K_{tp}
(See Clause A.12.7.3.3.2.)

Rivets per row, n_C	No. of rows, n_R				
	2	4	6	8	10
2	0.29	0.67	0.88	0.98	1.04
4	0.22	0.50	0.68	0.78	0.85
6	0.17	0.39	0.54	0.63	0.69
8	0.13	0.31	0.44	0.52	0.58
10	0.11	0.25	0.36	0.44	0.48
12	0.09	0.21	0.31	0.37	0.41
14	0.07	0.18	0.26	0.32	0.36
16	0.06	0.15	0.23	0.28	0.31
18	0.05	0.13	0.20	0.25	0.28
20	0.05	0.12	0.18	0.22	0.25

Table A.19
Values of β_{tp}
(See Clause A.12.7.3.3.2.)

S_Q, mm	S_p, mm	No. of rows, n_R				
		2	4	6	8	10
15	25	1.29	1.25	1.24	1.23	1.23
	40	1.76	1.61	1.49	1.45	1.40
25	25	1.00	1.00	1.00	1.00	1.00
	32	1.18	1.13	1.10	1.09	1.07
	40	1.36	1.27	1.20	1.17	1.14
	50	1.72	1.53	1.40	1.34	1.28
32	25	0.82	0.84	0.85	0.86	0.86
	32	0.97	0.96	0.94	0.93	0.93
	40	1.12	1.07	1.03	1.00	0.98
	50	1.42	1.29	1.20	1.15	1.10
40	25	0.63	0.68	0.70	0.71	0.71
	32	0.75	0.77	0.78	0.77	0.76
	40	0.87	0.86	0.84	0.83	0.82
	50	1.11	1.04	0.98	0.95	0.92

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Table A.20
Values of β_D
(See Clause A.12.7.3.3.2.)

$(n_c - 1)S_Q$	β_D	$(n_c - 1)S_Q$	β_D
0.1	0.275	1.0	0.83
0.2	0.433	1.2	0.88
0.3	0.538	1.4	0.92
0.4	0.60	1.6	0.95
0.5	0.65	1.8	0.97
0.6	0.69	2.0	0.98
0.7	0.73	2.4	0.99
0.8	0.77	2.8 and more	1.00
0.9	0.80		

Note: *For $e_P/[(n_C - 1)S_Q]$ between 0.1 and 0.3, β_D is given to three significant digits due to the sensitivity in this range.*

Table A.21
CSA B111 Nail and spike sizes
(See Clause 12.9.1.)

	Gauge length			Diameter*	
Type	**No.**	**mm**	**in**	**mm**	**in**
Common wire nails	15	25.40	1	1.83	0.072
	15	28.58	1-1/8	1.83	0.072
	14	31.75	1-1/4	2.03	0.083
	13	38.10	1-1/2	2.34	0.092
	12	44.45	1-3/4	2.64	0.109
	11-3/4	50.80	2	2.84	0.112
	11	57.15	2-1/4	2.95	0.116
	10	63.50	2-1/2	3.25	0.128
	10	69.85	2-3/4	3.25	0.128
	9	76.20	3	3.66	0.144
	9	82.55	3-1/4	3.66	0.144
	8	88.90	3-1/2	4.06	0.160
	6	101.6	4	4.88	0.192
	5	114.3	4-1/2	5.38	0.220

(Continued)

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Table A.21 (Concluded)

	Gauge length			Diameter*	
Type	**No.**	**mm**	**in**	**mm**	**in**
	4	127.0	5	5.89	0.238
	3	139.7	5-1/2	6.40	0.250
	2	152.4	6	7.01	0.275
Common spikes	3	101.6	4	6.40	0.250
	1	152.4	6	7.62	0.300
	1	177.8	7	7.62	0.300
	0	203.2	8	8.23	0.324
	0	228.6	9	8.23	0.325
	0	254.0	10	8.84	0.348
	0	304.8	12	8.84	0.348
Common spiral nails	10-1/2	63.50	2-1/2	2.77	0.109
	9-3/4	76.20	3	3.10	0.122
	9-3/4	82.55	3-1/4	3.10	0.122
	8	88.90	3-1/2	3.86	0.152
	7	101.6	4	4.33	0.170
	5	127.0	5	4.88	0.192

* *In this Table, "diameter" for spiral nails is the effective diameter based on the projected lateral width.*

Table A.22
ASTM F1667 Nail sizes
(See Clause 12.9.1.)

Type	**Length, in**	**Pennyweight**	**Diameter, mm**
Common	2	6d	2.87
	2-1/4	7d	2.87
	2-1/2	8d	3.33
	3	10d	3.76
	3-1/4	12d	3.76
	3-1/2	16d	4.11
	4	20d	4.88
	4-1/2	30d	5.26

(Continued)

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Table A.22 (Concluded)

Type	Length, in	Pennyweight	Diameter, mm
	5	40d	5.72
	5-1/2	50d	6.20
	6	60d	6.68

A.12.9.3.3 Lateral deformation of wood-to-wood connections with nails, spikes, or wood screws

For specified loads, P, not greater than $n_u/3$, the lateral deformation of wood-to-wood connections with nails, spikes, or wood screws, including structural panel-to-lumber connections, may be estimated as follows:

$$\Delta = 0.5 d_F K_m \left(\frac{P}{n_u}\right)^{1.7}$$

where

Δ = lateral deformation, mm

d_F = nominal diameter of fastener, mm

K_m = service-creep factor (see Table A.23)

P = specified load per fastener per shear plane, N

n_u = unit lateral strength resistance per shear plane, N (see Clause 12.9.3.2)

Table A.23
Service-creep factor, K_m
(See Clauses 11.7.1.2 and A.12.9.3.3.)

Load duration class	Moisture condition		
	Nailed dry, loaded dry	Nailed wet, loaded dry	Nailed wet, loaded wet
Long term	1.5	2.0	3.0
Standard term	1.2	1.5	2.0
Short term	1.0	1.2	1.5

A.16 Proprietary structural products — Materials and evaluation

Δ

A.16.2.3.1 Length-adjustment factor, K_{LN}

The length-adjustment factor, K_{LN}, is used by manufacturers to develop proprietary I-joist moment resistance values and is not intended as an adjustment factor for specific applications. The value of K_{LN} is taken from Section 6.4.1.5 of ASTM D5055 and is the lesser of 1.0 or the value calculated as follows:

$$K_{LN} = K_{SD}\left(\frac{L_1}{L}\right)^z \leq 1.0$$

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where

K_{SD} = stress distribution adjustment factor

= the adjustment of specified axial strength, f_a, from full-length constant stress (such as a tension test) to the reference stress condition (simple span and uniform load)

= 1.15

L_1 = gauge length (distance between grips) used in tension tests, parallel to grain, for I-joist flange material

L = I-joist span

= 18 times the joist depth

Z = coefficient that accounts for the variation in tensile strength of the flange material (see Table A.24)

Table A.24
***Z* for calculating the length-adjustment factor for prefabricated wood I-joists**
(See Clause A.16.2.3.1.)

Coefficient of variation, %*†	*Z*‡
≤ 10	0.06
15	0.09
20	0.12
25	0.15
≥ 30	0.19

** The coefficient of variation is determined from the full test data set using the higher coefficient of variation attained from the tensile strength of flange material or the tensile strength of end joints.*
† The coefficient of variation for the tensile strength of flange material is taken as 20% for machine-graded lumber (including SPS-4 material) and 25% for visually stress-graded lumber.
‡ Interpolation between tabular values may be used.

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Annex B (informative)
Fire resistance of large-cross-section wood elements

Notes:

1) *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*
2) *When this informational (non-mandatory) Annex is not otherwise adopted formally by building regulatory authorities as additional requirements to this Standard, the methodology presented provides information that may be useful to users of the Standard in the development of a proposal for an alternative solution to meet the objectives of the* National Building Code of Canada *(NBC).*

B.1 Scope

B.1.1

The design tables, data, and methods specified in Annex B provide a design methodology to develop fire-resistance ratings of large-cross-section wood elements based on structural criteria.

B.1.2

The design methodology is intended to be used as an alternative approach for determining fire-resistance ratings for establishing compliance to the *National Building Code of Canada,* as determined by testing in conformance with CAN/ULC-S101.

Note: *The fire performance criteria for evaluating the separating function of building elements related to the passage of flames or hot gases and transmission of heat through the assembly, as defined in CAN/ULC-S101, are outside the scope of Annex B, except as otherwise noted.*

B.1.3

The structural resistance of a wood element reduces as a function of time when exposed to fire. A structural element is deemed to possess a fire-resistance rating for a duration of fire exposure provided the reduced structural resistance of the element, after the specified exposure time, is greater than the specified load effects.

B.1.4

The methodology in Annex B is an engineering approach, intended to predict the structural fire resistance of large-cross-section wood elements exposed to the standard fire-resistance test, CAN/ULC-S101. The standard test method requires load-bearing elements to be tested with a superimposed load that represents a full specified load condition or a restricted load use condition. When calculating the fire-resistance rating using the methodology in Annex B, the specified gravity loads are used (i.e., D + L).

Note: *When a performance-based fire safety design approach is used in which the specific fire scenario(s) has a design fire(s) having time-temperature relationships other than that specified in the CAN/ULC-S101 fire-resistance test, additional analysis may be required. For example, a heat transfer analysis may be needed to determine an appropriate charring rate and zero-strength layer depth. In this case, it may also be appropriate to use the load factors suggested in Paragraph 25 of the most recent edition of Structural Commentary A of the* National Building Code of Canada. *Such fire scenarios can be evaluated as an alternative solution to meet the objectives of the* National Building Code of Canada.

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B.1.5

It is possible that built-up beams as described in Clause 6.4.4.3 and built-up compression members as described in Clause 6.5.5.4 will not exhibit similar fire performance as similar-sized solid members and therefore are beyond the scope of Annex B.

Note: *Connections between built-up elements are critical for ensuring performance as a single solid member and, in some cases, have been shown to be unable to keep the individual elements from separating during fire exposure thus significantly increasing the charring rate.*

B.2 Materials

B.2.1 General

Annex B applies to the materials specified in Clause 6 for timbers and decking, Clause 7 for glued-laminated timber, Clause 8 for cross-laminated timber, and Clause 15.3 for structural composite lumber.

B.2.2 Glued-laminated timber

To calculate the fire resistance of glulam beams using the specified strength values given in Table 7.2, the following manufacturing requirements shall be fulfilled:

a) Beams targeting a 1-hour fire-resistance rating when protected from fire exposure on the top shall be manufactured to the specified layups, except that one core lamination shall be removed and one 38-mm-thick outer tension lamination added on the bottom [see Figure B.1 b)].
b) Beams targeting a 1-hour fire-resistance rating when not protected from fire exposure on the top and subjected to positive and negative bending moments shall be manufactured to the specified layups, except that two core laminations shall be removed and one 38-mm-thick outer tension lamination added on the top and on the bottom [see Figure B.2 b)].
c) Beams targeting a 1.5- or 2-hour fire-resistance rating when protected from fire exposure on the top shall be manufactured to the specified layups, except that two core laminations shall be removed and two 38-mm-thick outer tension laminations added on the bottom [see Figure B.1 c)].
d) Beams targeting a 1.5- or 2-hour fire-resistance rating when not protected from fire exposure on the top and subjected to positive and negative bending moments shall be manufactured to the specified layups, except that four core laminations shall be removed and two 38-mm-thick outer tension laminations added on the top and on the bottom [see Figure B.2 c)].

If the additional tension laminations are not substituted for core laminations, the specified strength values of the individual laminations shall be used to calculate the fire resistance of glulam beams.

B.2.3 CLT

When determining the axial or flatwise-bending moment resistance of CLT panels exposed to fire, only the layers parallel to the direction of the applied stress shall be considered in determination of the fire resistance of the panels.

Note: *When high shear stress governs design, such as a point load from a column on a floor element, shear resistance should be determined. In the case of shear resistance, all layers contributing to the resistance should be considered.*

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Figure B.1
Typical glued-laminated timber beam layup subjected to a positive bending moment or subjected to both a positive and a negative bending moment and protected from fire exposure on the top surface
(See Clause B.2.2.)

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Figure B.2
Typical glued-laminated beam layup subjected to both a positive and negative bending moment and not protected from fire exposure on the top surface
(See Clause B.2.2.)

B.3 Factors affecting resistance

B.3.1 General

The specified strengths of materials and connections used in calculating the resistance shall be multiplied by the modification factors specified in Clause B.3 and in the applicable clauses of this Standard.

B.3.2 Resistance factor, ϕ

The resistance factor, ϕ, shall be taken as 1.0.

B.3.3 Load-duration factor, K_D

The load-duration factor, K_D, shall be taken in accordance with Clause 5.3.2 for short-term load duration.

B.3.4 System factor, K_H

The system factor, K_H, shall be taken as 1.0.

B.3.5 Size factor, K_Z

The size factor, K_Z, shall be determined in accordance with applicable Clauses in this Standard using the original cross-sectional dimensions.

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B.3.6 Lateral-stability factor, K_L

The lateral-stability factor, K_L, shall be determined in accordance with Clause 6.5.3.2 for sawn lumber or Clause 7.5.6.4 for glued-laminated timber using the effective cross-section of the beam (see Clause B.6.2). To be considered in the calculations, the members providing the intermediate lateral support shall have at least the same fire-resistance rating as that sought for the beam.

Δ **B.3.7 Slenderness ratio, C_C**

The slenderness ratio, C_C, shall be determined using the effective cross-section (see Clause B.6.2). To be considered in the calculations, the members providing the intermediate lateral support shall have at least the same fire-resistance rating as that sought for the column.

B.3.8 Slenderness factor, K_C

The slenderness factor, K_C, shall be determined using the effective cross-section (see Clause B.6.2) and the specified mean modulus of elasticity (see Clause B.6.4).

B.3.9 Adjustment factor for fire resistance, K_{fi}

The adjustment factor for fire resistance, K_{fi}, shall be determined as given in Table B.1.

Table B.1
Adjustment factor for fire resistance, K_{fi}
(See Clauses B.3.9 and B.6.3.)

Product	K_{fi}
Timbers and plank decking	1.5
Glued-laminated timber	1.35
Cross-laminated timber	
– V1-V2 stress grades	1.5
– E1-E2-E3 stress grades	1.25
SCL	1.25

Note: *Adjustment factor for fire resistance, K_{fi}, converts specified strength to mean strength.*

B.4 Char layer depth

B.4.1 General

The charring rates in Table B.2 apply when members are exposed to the standard fire exposure in accordance with CAN/ULC-S101. Clause B.4.3 shall be used to determine the char layer depth for one-dimensional heat transfer, such as in solid wood walls and floors or when corner rounding is explicitly considered for rectangular cross-sections following Clause B.4.5. For CLT, the char layer depth shall be determined in accordance with Clause B.4.6. For all other cases, the char layer depth shall be determined in accordance with Clause B.4.4.

Note: *Use of the notional charring rate in Clause B.4.4 is recommended when considering rectangular cross-section members because the method implicitly accounts for the effect of cracking, formation of fissures, and corner rounding.*

Δ **B.4.2 Charring rates**

The one-dimensional and notational charring rates, β_o and β_n, are given in Table B.2.

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Table B.2
Charring rates, mm/min
(See Clauses B.4.1, B.4.2, and B.4.6.)

	β_o	β_n
Timbers and plank decking	0.65	0.80
Glued-laminated timber	0.65	0.70
Cross-laminated timber	0.65	0.80
SCL	0.65	0.70

Note: *When the smallest dimension of the effective cross-section of a member subjected to heating on parallel sides is reduced to less than 70 mm, the charring rate will increase as the temperature rise beyond the char front meets in the middle of the cross-section. In this case, a heat transfer analysis may be required to determine the proper effective cross-section to use.*

Δ

B.4.3 One-dimensional char layer depth

The char layer depth for one-dimensional charring, $x_{c,o}$ (mm), shall be calculated as follows:

$x_{c,o} = \beta_o t$

where

β_o = one-dimensional charring rate, mm/min

t = fire exposure duration, min

B.4.4 Notional char layer depth

The char layer depth for notional charring, $x_{c,n}$ (mm), shall be calculated as follows:

$x_{c,n} = \beta_n t$

where

β_n = notional charring rate, mm/min

t = fire exposure duration, min

B.4.5 Corner rounding

The corners of rectangular cross-sections exposed to fire are subjected to two-dimensional heat transfer resulting in a corner-rounding effect. When explicitly considering the effect of corner rounding, the radius of the corner shall be taken as the depth of the charred layer using the one-dimensional charring rate, in accordance with Clause B.4.3, and the section properties shall be determined accordingly.

Note: *The effect of corner rounding is not required to be considered explicitly when the char layer depth is determined in accordance with Clause B.4.4.*

Δ

B.4.6 Char layer depth for CLT

The one-dimensional charring rate, β_o, specified in Table B.2 and Clause B.4.3 shall be used when the char layer depth will not reach the first bond line between the exposed layer and the subsequent layer for the duration of the fire resistance required. If the char layer depth surpasses the first bond line during the fire-resistance period sought, the notional charring rate, β_n, specified in Table B.2 and Clause B.4.4 shall be used.

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B.5 Zero-strength layer depth

To account for a reduction in strength of the heated wood beyond the char front, a zero-strength layer shall be subtracted to further reduce the cross-section beyond the char layer depth determined in Clause B.4.3 or B.4.4. The zero-strength layer depth, x_t (mm), shall be calculated as follows:

$$x_t = \left(\frac{t}{20}\right) \times 7 \text{ (for } t < 20)$$

$x_t = 7$ (for $t \geq 20$)

where

t = fire exposure duration, min

B.6 Resistance of effective cross-section

B.6.1 General

Charring of all surfaces of a wood element directly exposed to fire shall be taken into account including, where relevant, any surfaces initially protected from fire exposure where wood charring could occur at some point during the standard fire exposure for fire-resistance rating required. The effective cross-section shall be assumed to have properties as specified in Clauses B.6.3 and B.6.4, while the char layer and zero-strength layer shall be assumed to provide no strength or stiffness.

Note: *Only surfaces protected from fire exposure for the duration of the exposure, such as the top surface of a beam protected by a fire-resistance-rated floor or roof assembly, may be considered to be protected from the fire for the duration of the exposure.*

B.6.2 Effective cross-section

The effective cross-section shall be the initial cross-section reduced by the char layer depth (see Clause B.4) and the zero-strength layer depth (see Clause B.5) on each exposed surface (see Figure B.2). Section properties shall be calculated using standard equations for area, section modulus, and moment of inertia using the dimensions of the effective cross-section.

Note: *In some instances, such as the effective cross-section of CLT, the location of the neutral axis must be determined prior to calculating the moment of inertia and section modulus. In addition, for calculating the section modulus of CLT, the height divided by 2 (h/2) must be replaced with c, the distance from the extreme (bottom) tension fibre to the neutral axis, thus replacing 2/h with 1/c in the section modulus equation.*

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Figure B.3
Schematic showing effective cross-section for a member exposed to fire on all sides
(See Clause B.4.2.)

B.6.3 Resistance of the effective cross-section

The resistances of the members exposed to fire shall be calculated in accordance with the applicable equations in Clauses 6, 7, 8 and 15.3 using the strength properties modified in accordance with Clause B.3 and the effective cross-section in accordance to Clause B.6.2.

B.6.4 Modulus of elasticity

The modulus of elasticity used in calculating the resistance of the member shall be taken as the specified mean value.

Note: *The modulus of elasticity is used in calculations of the lateral-stability factor for bending members, K_L, and the slenderness factor for compression members, K_c.*

B.7 Fire-resistance rating

A structural element shall be assigned a fire-resistance rating of a duration of fire exposure if the effective structural resistance of the element for that fire exposure is greater than the specified gravity load effects.

B.8 Surfaces protected by gypsum board

B.8.1 Gypsum board

Provided that surfaces are protected from fire exposure by fire-rated Type X gypsum board, the assigned fire-resistance duration calculated in accordance with Clause B.7 can be increased by the following times:

a) 15 min when one layer of 12.7 mm Type X gypsum board is used;

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b) 30 min when one layer of 15.9 mm Type X gypsum board is used;
c) 60 min when two layers of 15.9 mm Type X gypsum boards are used; or
d) 60 min when two layers of 12.7 mm Type X gypsum boards are applied to CLT.

Δ B.8.2 Gypsum board fasteners

The values in Clause B.8.1 shall only apply where the fasteners used to attach the gypsum board penetrate the wood element a minimum of 25 mm and are spaced a maximum of 300 mm on centre and each length of gypsum board is attached by a minimum of two rows of fasteners that are offset by half the fastener spacing if row spacing is less than 300 mm.

Note: *Steel or wood furring providing a gap between the gypsum board and wood member will not reduce the additional fire resistance attributed to the gypsum board, provided the gypsum board fastener spacing requirements in Clause B.8.2 are met and the fasteners used to attach the furring penetrate the wood structural elements a minimum of 25 mm.*

B.8.3 Joints

The values in Clause B.8.1 shall only apply where the exposed joints of the gypsum board are taped and finished. When multiple layers of 15.9 mm Type X gypsum board described in Clause B.8.1 are used, the joints shall be offset between the base layer and face layer.

B.9 Connections

Connections that are critical to support the gravity loads acting on the structure shall be designed to have at least the same fire-resistance rating as the elements they support. Connections in which the steel is located within the effective cross-section of the wood element shall be considered properly protected.

Note: *Additional information can be found in the American Wood Council's Technical Report 10 and in EN 1995-1-2.*

B.10 Decking

B.10.1 Double tongue-and-groove

Double tongued-and-grooved decking shall be designed as an assembly of wood beams fully exposed on one face.

B.10.2 Single tongue-and-groove or internal spline

Single tongued-and-grooved or single internal splined decking shall be designed as an assembly of wood beams fully exposed on one face, provided the loss in depth of the effective cross-section has not yet reached the tongue-and-groove or internal spline.

B.10.3 Butt-jointed

Butt joints shall be designed as an assembly of wood beams partially exposed on the sides and fully exposed on one face. To compute the effects of partial exposure of the decking elements on their sides, the char rate for this partial exposure shall be reduced to 1/3 of the char rate of that used on the exposed face.

B.10.4 Unexposed surface protection

The methodology in Clauses B.10.1 to B.10.3 shall apply only to solid wood decks that have the unexposed (top) surface of the decking protected by:

a) tongued-and-grooved wood flooring not less than 19 mm thick laid crosswise or diagonally;

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b) tongued-and-grooved plywood or OSB not less than 12.5 mm thick;
c) concrete topping of minimum 38 mm thickness; or
d) gypsum-concrete topping of minimum 25 mm thickness.

Note: *These prescriptive requirements are intended to address the fire performance criteria for evaluating the separating function of building assemblies related to the passage of flames or hot gases and transmission of heat through the assembly as defined in CAN/ULC-S101. Additional information can be found in Section 3.1.4.7 and Appendix D-2.4 of the* National Building Code of Canada *for solid wood floors and roofs.*

B.11 Fire performance criteria for evaluating separating function of building assemblies

The fire performance criteria for evaluating the separating function of building assemblies related to the passage of flames or hot gases and transmission of heat through the assembly, as defined in CAN/ULC-S101, shall be considered when required by the applicable building code.

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