CSA Group

# O86-14

# Engineering design in wood

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# Δ Preface

This is the tenth edition of CSA O86, *Engineering design in wood*. It is presented in limit states design (LSD) format, and supersedes the previous editions published in 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first (1984) and second (1989) LSD versions, *Engineering Design in Wood (Limit States Design).* The WSD version was withdrawn on publication of the 1994 LSD edition.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.

Changes in this edition include the following:

- Clause 8 been added for CLT in compression and out-of-plane bending applications;
- Clause 11.9 has been added for the design of CLT shearwalls and diaphragms for platform-type construction;
- Clause 12 has been modified to cover connections in CLT;
- Clause 12 has been modified to cover embedment resistance of nails, screws, lags, bolts, and dowels in mild and cold-formed steel as well as head pull-through resistance of screws in steel;
- Annex B, on fire resistance of large cross-section wood elements, has been modified to cover CLT;
- reduction in the concentrated loaded area on roof deck has been incorporated;
- requirements for anticipated building movement due to shrinkage and swelling have been added;
- requirements for lateral brace forces for wood truss compression webs have been included;
- revisions have been made to finger-jointed lumber grades;
- revisions have been made to the shear and bending moment resistance of glued-laminated timber;
- lateral-load-resisting provisions have been modified;
- design of diaphragms and shearwalls have been revised;
- lag screw requirements have been adjusted;
- reaction requirements for proprietary wood products have been added; and
- strength resistance of truss plates and ultimate load of joist hangers have changed;

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

**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 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.*

# O86-14
# Engineering design in wood

## 1 Scope

### Δ 1.1
This Standard provides criteria for the structural design and appraisal of structures or structural elements made from wood or wood products, including graded lumber, glued-laminated timber, cross-laminated timber (CLT), unsanded plywood, oriented strandboard (OSB), composite building components, shearwalls and diaphragms, timber piling, pole-type construction, prefabricated wood I-joists, structural composite lumber products, preserved wood foundations, and their structural connections. This Standard employs the limit states design method.

### 1.2
In CSA Standards, "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.

## Δ 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 (withdrawn)
*Wire nails, spikes and staples*

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

CAN/CSA-O15-05 (R2009)
*Wood utility poles and reinforcing stubs*

CAN/CSA-O56-10
*Round wood piles*

CAN/CSA-O80 Series-08 (R2012)
*Wood preservation*

O112 Series-M1977 (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
*Evaluation of adhesives for structural wood products (exterior exposure)*

O112.10-08 (R2013)
*Evaluation of adhesives for structural wood products (limited moisture exposure)*

O121-08 (R2013)
*Douglas fir plywood*

CAN/CSA-O122-06 (R2011)
*Structural glued-laminated timber*

O141-05 (R2009)
*Softwood lumber*

O151-09
*Canadian softwood plywood*

O153-13
*Poplar plywood*

O177-06 (R2011)
*Qualification code for manufacturers of structural glued-laminated timber*

O322-02 (R2012)
*Procedure for certification of pressure-treated wood materials for use in preserved wood foundations*

O325-07 (R2012)
*Construction sheathing*

S16-14
*Design of steel structures*

S136-12
*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*

S406-14
*Specification of permanent wood foundations for housing and small buildings*

**American Wood Council**
Technical Report 10-2003
*Calculating the Fire Resistance of Exposed Wood Members*

**ANSI (American National Standards Institute)**
ANSI/APA PRG 320-2012
*Standard for Performance-Rated Cross-Laminated Timber*

**APEGBC (Association of Professional Engineers and Geoscientists of British Columbia)**
Technical and Practice Bulletin 2013
*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)**
B18.2.1-1996
*Square and Hex Bolts and Screws (Inch Series)*

B18.6.1-1981 (R2008)
*Wood Screws (Inch Series)*

**ASTM International (American Society for Testing and Materials)**
A36/A36M-08
*Standard Specification for Carbon Structural Steel*

A47/A47M-99 (2004)
*Standard Specification for Ferritic Malleable Iron Castings*

A307-07b
*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/A 653M replaces ASTM A446/A446M. Their chemical and mechanical requirements are the same.*

C1002-07
*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
*Standard Specification for Gypsum Board*

D5055-13e1
*Standard Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists*

D5456-14b
*Standard Specification for Evaluation of Structural Composite Lumber Products*

D5457-12
*Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design*

D7147-11
*Standard Specification for Testing and Establishing Allowable Loads of Joist Hangers*

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

**CCMC (Canadian Construction Materials Centre)**
*Registry of Product Evaluations*
**Note:** *See CCMC website.*

**CWC (Canadian Wood Council)**
*Standard Practice Relating Specified Strengths of Structural Members to Characteristic Structural Properties* (2001)

*Wood Design Manual including CSA O86* (2015)

*Commentary on CSA O86*

**European Committee for Standardisation**
EN 1995-1-2:2004
*Eurocode 5 — Design of Timber Structures. Part 1-2 — Structural fire design*

**NLGA (National Lumber Grades Authority)**
*Standard Grading Rules for Canadian Lumber* (2014), including *National Grading Rules for Dimension Lumber*

SPS 1-2013
*Special Products Standard for Fingerjoined Structural Lumber*

SPS 2-2013
*Special Products Standard for Machine Graded Lumber*

SPS 3-2013
*Special Products Standard for Fingerjoined "Vertical Stud Use Only" Lumber*

SPS 4-2013
*Special Products Standard for Fingerjoined Machine Graded Lumber (FJ-MGL)*

**NRCC (National Research Council Canada)**
*National Building Code of Canada,* 2010

*Structural Commentary A of the National Building Code of Canada*

*User's Guide — NBC 2010* (2015 edition in preparation)
*Structural Commentaries (Part 4 of Division B)*

**SAE International (Society of Automotive Engineers)**
*SAE Handbook (2008)*

J429-1999
*Mechanical and Material Requirements for Externally Threaded Fasteners*

**TPIC (Truss Plate Institute of Canada)**
TPIC-2014
*Truss design procedures and specification for light metal plate connected Wood Trusses*

**Underwriters Laboratories of Canada**
CAN/ULC-S101-07
*Standard Methods of Fire Endurance Tests of Building Construction and Materials*

# ∆ 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.

Δ **Axis** —

**Major axis (**or **major strength axis)** —
(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 axis)** —
(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 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 ASTM D5456, respectively, carried out to characterize strength properties of I-joists or structural composite lumber or their component materials. The term is used in combination with a specific property, such as tensile capacity or shear capacity.
**Note:** *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.*

Δ **CLT stress 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.

**Concentrically braced heavy timber space frame** — a structural system with an essentially complete timber space frame providing support for gravity loads and diagonal bracing with ductile connections resisting lateral loads.

Δ **Cross-laminated timber (CLT)** — an engineered wood product made of at least three orthogonal layers of stress graded structural sawn lumber that are laminated by gluing with structural adhesives to form a solid panel that meets the requirements of ANSI/APA PRG 320 standard.

**Layer** — in CLT, an arrangement of lumber 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 axis of the panel.

**Transverse** — a layer with laminations oriented perpendicular to the major axis of the panel.

**Layup** — in CLT, a cross-sectional 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. The term "diaphragm" 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 particular material, design, practice, or construction method satisfies the intent of this Standard.

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

**Drift pin** — a round mild steel bar, without head or threads, used to provide a laterally loaded connection between overlapping timbers in heavy engineering structures such as cribwork.

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

**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. 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 12.3.3A and 12.3.3B).

**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 connection** — a connection at the corner of a shearwall or shearwall segment that is designed to provide a structural load path between the boundary chords of the segment and the
(a) foundation or beam supporting the shearwall; or
(b) corresponding chord member of the shearwall segment above or below.

**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.

**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.

**Moment-resisting wood space frame** — a structural system with an essentially complete wood space frame providing support for gravity loads, with resistance to lateral loads provided primarily by flexural action of members.

**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 6.5.13), the distance between two adjacent panel points.

**Panel point** — with respect to the design of lumber members in metal-plated trusses (see Clause 6.5.13), 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 6.5.13), 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.

Δ **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:** *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 or not 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,** $\phi$ — 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.

**Service condition** —

**Dry service condition** — a climatic condition in which the average equilibrium moisture content of solid wood over a year is 15% or less and does not exceed 19%.

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

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

**Shearwall** — a stud wall system designed to resist lateral forces parallel to the plane of the wall. A shearwall can consist of one or more shearwall segments in the plane of the wall.
**Note:** *Also referred to as a "vertical diaphragm" or a "structural 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 beams** — 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 so as 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 unit as a whole.

**Structural composite lumber** — 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**.

**Structural panel** — in this Standard, plywood and 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.

**Timber connector** — a metal ring or plate that, by being embedded in adjacent wood faces or in one wood face, acts in shear to transmit loads from one timber to another or from a timber to a bolt and, in turn, to a steel plate or another connector.

**Truss plate** — a light steel plate fastener, 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 structural 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.

## 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$^2$

Δ $A_{eff}$ = 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

$d$ = 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 |
| Δ | $E_{\perp}$ | = | transverse modulus of elasticity, MPa |
| | $E_{05}$ | = | modulus of elasticity for design of compression members, MPa |
| | $E_S$ | = | modulus of elasticity for stiffness calculations, MPa |
| | $(EI)_e$ | = | effective stiffness of structural panel assemblies, N•mm$^2$ |
| Δ | $(EI)_{eff}$ | = | effective 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 |
| Δ | $f_f$ | = | specified fracture shear strength at notch, MPa |
| Δ | $f_s$ | = | specified strength in rolling shear, MPa |
| Δ | $f_t$ | = | 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 |
| Δ | $f_{tn}$ | = | 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 |
| Δ | $G_{\perp}$ | = | rolling shear modulus, MPa |
| Δ | $(GA)_{eff}$ | = | effective in-plane (planar) shear rigidity of CLT panels, N |
| | $I$ | = | moment of inertia, mm$^4$ |
| Δ | $I_{eff}$ | = | effective out-of-plane moment of inertia of CLT panels, mm$^4$ |
| Δ | $J$ | = | 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 |
| | $K_F$ | = | foundation factor for plywood |
| Δ | $K_{fi}$ | = | 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 |
| | $K_R$ | = | radial stress factor |
| Δ | $K_{rb}$ | = | adjustment factor for bending moment resistance of CLT panels |
| Δ | $K_S$ | = | service condition factor for sawn lumber, glued-laminated timber, CLT, structural panels, poles, and piling |
| | $K_{Sb}$ | = | 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 |
| Δ | $K_{SF}$ | = | service condition factor for connections |
| Δ | $K_{Sf}$ | = | 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 |
| Δ | $K_{Zc}$ | = | 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 |
| | $\ell$ | = | 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 |
| Δ | | = | factored lateral strength resistance of a connection, N or kN |
| Δ | $N_{rs}$ | = | factored lateral slip resistance of a connection, N |
| Δ | $N_s$ | = | lateral slip resistance of a connection, N |
| Δ | $N_u$ | = | 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 |
| | | = | 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 |
| Δ | $P_u$ | = | 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 |
| Δ | $r_{eff}$ | = | effective radius of gyration, mm |
| | $S$ | = | section modulus, $mm^3$ |
| Δ | $S_{eff}$ | = | effective out-of-plane section modulus of CLT panels, $mm^3$ |
| | $T_f$ | = | 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
$v_{psb}$ = specified strength capacity of structural panels in planar shear (due to bending), N/mm
$v_{pf}$ = specified strength capacity of structural panels in planar shear (due to in-plane forces), MPa
$W_f$ = 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$^3$
$\phi$ = resistance factor

## Δ 3.3 Spacing and dimensions

For the purpose of this Standard, the following apply:

(a) 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) Panel dimensions may be used interchangeably:
  (i) 1200 mm and 1220 mm; and
  (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 acceptable assurances that the structure, when correctly designed and built, will be fit for the intended use.

## 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.

## 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 adequate 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, 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.

### 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

Where design drawings are required, they shall be drawn to a scale adequate to convey 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
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
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, supplemental material, or both.

#### 4.3.6.4
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 Ultimate and serviceability limit states

### 5.1.1 Method 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.

### 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.

### 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, $\phi$ are given in the appropriate sections of this Standard for all applicable limit states for wood members and connections.

## 5.2 Specified loads, load effects, and load combinations

### 5.2.1 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 NRCC* User's Guide — NBC Structural Commentaries (Part 4 of Division B)*, and the Canadian Geotechnical Society's* Canadian Foundation Engineering Manual*. In the case of seismic design, Clause 11 provides provisions additional to those in Division B of the* National Building Code of Canada*.*

### 5.2.2 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 include the following, where applicable, and the minimum specified values of these loads shall be increased to account for dynamic effects, where applicable:

(a) *D* — dead load due to weight of members; the weight of all materials of construction incorporated into the building to be supported permanently by the member, including permanent partitions and allowance for non-permanent partitions; the weight of permanent equipment;
(b) *E* — load due to earthquake, including the effect of the importance factors in Clause 5.2.3.2;
(c) *L* — live load due to intended use and occupancy, including loads due to cranes and the pressure of liquids in containers;
(d) *S* — load due to snow, including ice and associated rain, and also including the effect of the importance factors in Clause 5.2.3.2;
(e) *W* — load due to wind, including the effect of the importance factors in Clause 5.2.3.2;
(f) *H* — permanent load due to lateral earth pressure, including groundwater;
(g) *P* — permanent effects caused by prestress; and
(h) *T* — load due to contraction or expansion caused by temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.

#### 5.2.3.2 Importance factors

For the purpose of determining the specified *S*, *W*, or *E* loads of Clause 5.2.3.1, importance factors shall be applied in accordance with Table 5.2.3.2.

**Note:** *For further information on specified loads and importance factors, see the* National Building Code of Canada*.*

**Table 5.2.3.2**
**Importance factors for determining *S*, *W*, or *E* loads**

| Importance category | Importance factors for snow loads, $I_S$ | | Importance factors for wind loads, $I_W$ | | Importance factors for earthquake loads, $I_E$ | |
|---|---|---|---|---|---|---|
| | Ultimate limit state | Serviceability limit state | Ultimate limit state | Serviceability limit state | Ultimate limit state | Serviceability limit state |
| Low | 0.8 | 0.9 | 0.8 | 0.75 | 0.8 | — |
| Normal | 1.0 | 0.9 | 1.0 | 0.75 | 1.0 | — |
| High | 1.15 | 0.9 | 1.15 | 0.75 | 1.3 | — |
| Post-disaster | 1.25 | 0.9 | 1.25 | 0.75 | 1.5 | — |

## 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 Table 5.2.4.1. The applicable combination shall be that which results in the most unfavourable effect.

**Table 5.2.4.1**
**Load combinations for ultimate limit states**

| Case | Principal loads* | Companion loads |
|---|---|---|
| 1 | 1.4*D* | — |
| 2 | (1.25*D*† or 0.9*D*) + 1.5*L*‡ | 1.0*S*§ or 0.4*W* |
| 3 | (1.25*D*† or 0.9*D*) + 1.5*S* | 1.0*L*§** or 0.4*W* |
| 4 | (1.25*D*† or 0.9*D*) + 1.4*W* | 0.5*L*** or 0.5*S* |
| 5 | 1.0*D* + 1.0*E* | 0.5*L*§** + 0.25*S*§ |

**Refer to the* National Building Code of Canada *for loads due to lateral earth pressure (H), prestress (P), and imposed deformation (T).*
†*Refer to the* National Building Code of Canada *for a dead load (D) for soil.*
‡*The principal load factor of 1.5 for a live load (L) may be reduced to 1.25 for liquids in tanks.*
§*Refer to the* National Building Code of Canada *for loads on exterior areas.*
***The companion load factor for a live load (L) shall be increased by 0.5 for storage occupancies, equipment areas, and service rooms.*

### 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 Table 5.2.4.2. The applicable combination shall be that which results in the most unfavourable effect.

**Table 5.2.4.2**
**Load combinations for serviceability limit states**

| Case | Principal loads | Companion loads |
|---|---|---|
| 1 | 1.0*D** | — |
| 2 | 1.0*D** + 1.0*L* | 0.5*S*† or 0.4*W* |
| 3 | 1.0*D** + 1.0*S* | 0.5*L*†‡ or 0.4*W* |
| 4 | 1.0*D** + 1.0*W* | 0.5*L*‡ or 0.5*S* |

**Dead loads include permanent loads due to lateral earth pressure (H) and prestress (P).*
†*Refer to the* National Building Code of Canada *for loads on exterior areas.*
‡*The companion load factor of 0.5 for a live load (L) shall be increased to 1.0 for storage occupancies, equipment areas, and service rooms.*

## 5.3 Conditions and factors affecting resistance

### 5.3.1 General

Specified strengths and capacities for materials and connections shall be multiplied by the modification factors in this Clause and the appropriate materials or connection 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 Specified strengths and capacities

The specified strengths and capacities in this Standard are based on the standard-term duration of the specified loads.

#### 5.3.2.2 Load duration factor

Except as specified in Clause 5.3.2.3, the specified strengths and capacities shall be multiplied by a load duration factor, $K_D$, in accordance with Table 5.3.2.2, but not exceeding 1.15.

**Table 5.3.2.2**
**Load duration factor, $K_D$**

| Load duration | $K_D$ | Explanatory notes |
|---|---|---|
| Short term | 1.15 | Short-term loading means the condition of loading where the duration of the specified loads is not expected to last more than 7 days continuously or cumulatively throughout the life of the structure.<br>Examples include wind loads, earthquake loads, falsework, and formwork, as well as impact loads. |
| 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.<br>Examples include snow loads, live loads due to occupancy, wheel loads on bridges, and dead loads in combination with all of the above. |
| Long term | 0.65 | Long-term duration means the condition of loading under which a member is subjected to more or less continuous specified load.<br>Examples include dead loads or dead loads plus live loads of such character that they are imposed on the member for as long a period of time 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 should be considered to be 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.*

#### 5.3.2.3 Long-term load factor

For standard-term loads where the specified long-term load, $P_L$, is greater than the specified standard-term load, $P_s$, the long-term load factor may be used or the factor can 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 $S$ and $L$ loads acting alone or in combination
= $S$, $L$, $S + 0.5L$, or $0.5S + L$, determined using importance factors equal to 1.0

#### 5.3.2.4 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.

### 5.3.3 Service condition factor, $K_S$
Where materials or connections are used in service conditions other than dry, specified strengths and capacities shall be multiplied by the service condition factor, $K_S$, in the appropriate materials or connection clause 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, $K_T$, in the appropriate materials or connection clause 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.

#### 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 $K_T$ tabulated in the appropriate clause of this Standard.

### Δ 5.3.5 System factor, $K_H$
Specified strengths may be multiplied by a system factor, $K_H$, as specified in Clauses 6.4.4, 7.4.3, 8.3.4, 15.2.2.4, and 15.3.2.4.

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

### Δ 5.3.6 Size factor, $K_Z$
Where size influences the specified strengths of members, the specified strengths shall be multiplied by the size factor, $K_Z$, in accordance with Clauses 6.4.5, 7.5.6, 7.5.8, 7.5.9, 8.4.5, 15.3.2.5, 15.3.2.6, and 15.3.3.

**Note:** *See the CWC* Commentary on CSA O86.

### 5.3.7 Lateral stability factor, $K_L$
The effect of width-to-depth ratios and of the degree of lateral support on the factored bending moment resistance is specified in Clauses 6.5.4.2 and 7.5.6.4.

### 5.3.8 Reduction in cross-section

#### Δ 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 taken 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.*

### 5.4.3 Permanent deformation

Structural members that support long term loads in excess of 50% of the load combinations for serviceability limit states shall be designed to limit permanent deformation. In lieu 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 loads.

### 5.4.4 Ponding

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

(a) the roof surface is provided with sufficient slope toward points of free drainage to prevent 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 of the components of the system (decking, secondary beams, primary beams, etc.)

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

### 5.4.5 Vibration

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

**Note:** *See Clause A.5.4.5 for information on floor vibration. Additional information can be found in the commentary on serviceability criteria for deflections and vibrations in the User's* Guide — NBC Structural Commentaries (Part 4 of Division B)*.*

### 5.4.6 Building movements due to moisture content change

Anticipated building movements due to shrinkage and swelling shall be specified by the designer.

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

### Δ 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 metal plate connected wood trusses shall be designed for a force equal to 1.25% of the axial 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.

Lateral bracing shall be connected to truss webs in accordance with Clause 12.

### Δ 5.6 Fire resistance

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

**Note:** *See Annex B for a methodology that provides useful information in the development of a proposal for an alternative solution to meet the objectives of the NBC.*

## 6 Sawn lumber

### 6.1 Scope

The design tables, 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 6.2.1.2, 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.*

**Table 6.2.1.2**
**Species combinations**

| 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), 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 many species designations 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 6.2.1.3.

**Table 6.2.1.3**
**Lumber species equivalents**

| 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

Table 6.2.2.1 lists categories, limiting dimensions, and structural grades for which design data are assigned in this Standard. These grades are specified in the NLGA *Standard Grading Rules for Canadian Lumber.*

**Table 6.2.2.1**
**Visual grades and their dimensions**

| Grade category | Smaller dimension, mm | Larger dimension, mm | Grades |
|---|---|---|---|
| Light framing | 38 to 89 | 38 to 89 | Construction, Standard |
| Stud | 38 to 89 | 38 or more | Stud |
| Structural light framing | 38 to 89 | 38 to 89 | Select Structural<br>No. 1, No. 2, No. 3 |
| Structural joists and planks | 38 to 89 | 114 or more | Select Structural<br>No. 1, No. 2, No. 3 |
| Beam and stringer | 114 or more | Exceeds smaller dimension by more than 51 | Select Structural<br>No. 1, No. 2 |
| Post and timber | 114 or more | Exceeds smaller dimension by 51 or less | Select Structural<br>No. 1, No. 2 |
| Plank decking | 38 to 89 | 140 or more | Select, Commercial |

#### 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 and grade-stamped 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 lumber

#### 6.2.3.1

Except as limited in Clause 6.2.3.2 or 6.2.3.3, the design data specified in this Standard apply to finger-joined lumber that has been produced and grade-stamped in accordance with NLGA SPS 1, SPS 3, or SPS 4.
**Note:** *Finger-joined lumber is produced to specifications that permit the same specified strength and stiffness to be assigned as non-finger-joined lumber of the same grade, species and size. See the CWC* Commentary on CSA O86 *for additional information.*

#### 6.2.3.2 NLGA SPS 3 "Vertical Stud Use Only" lumber

#### 6.2.3.2.1

Finger-joined lumber that has been produced and grade-stamped in accordance with NLGA SPS 3 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 at all times and not in an environment where the equilibrium moisture content can be expected to exceed 19%, or the temperature can be expected to exceed 50 °C for an extended period of time.

#### 6.2.3.2.2

For SPS-3 "vertical use only" stud 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 and grade-stamped in accordance with NLGA SPS 4 and designated “Dry Use Only” shall be used only where protected from wet service conditions at all times (i.e., shall not be used in environments where the equilibrium moisture content can be expected to exceed 19%).

### 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

### 6.3.1 Visually stress-graded lumber

#### 6.3.1.1
The specified strengths (MPa) for visually stress-graded lumber are tabulated as follows:
(a) structural joist and plank, structural light framing, and stud grade categories of lumber in Table 6.3.1A;
(b) light framing grades in Table 6.3.1B;
(c) beam and stringer grade categories of lumber in Table 6.3.1C; and
(d) post and timber grade categories of lumber in Table 6.3.1D.

#### 6.3.1.2
The specified strengths (MPa) for plank decking shall be derived from Table 6.3.1A using the following grade equivalents:

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

### 6.3.2 Machine stress-rated and machine evaluated lumber
The specified strengths (MPa) for machine stress-rated lumber are given in Table 6.3.2. The specified strengths (MPa) for machine evaluated lumber are given in Table 6.3.3. Specified strengths in shear are not grade dependent and shall be taken from Table 6.3.1A for the appropriate species.

## Table 6.3.1A
## Specified strengths and modulus of elasticity for structural joist and plank, structural light framing, and stud grade categories of lumber, MPa

| Species identification | Grade | Bending at extreme fibre, $f_b$ | Longi-tudinal shear, $f_v$ | Compression: Parallel to grain, $f_c$ | Compression: Perpen-dicular to grain, $f_{cp}$ | Tension parallel to grain, $f_t$ | Modulus of elasticity: $E$ | Modulus of elasticity: $E_{05}$ |
|---|---|---|---|---|---|---|---|---|
| D Fir-L | SS | 16.5 | | 19.0 | | 10.6 | 12 500 | 8 500 |
| | No. 1/No. 2 | 10.0 | 1.9 | 14.0 | 7.0 | 5.8 | 11 000 | 7 000 |
| | No. 3/Stud | 4.6 | | 7.3 | | 2.1 | 10 000 | 5 500 |
| Hem-Fir | SS | 16.0 | | 17.6 | | 9.7 | 12 000 | 8 500 |
| | No. 1/No. 2 | 11.0 | 1.6 | 14.8 | 4.6 | 6.2 | 11 000 | 7 500 |
| | No. 3/Stud | 7.0 | | 9.2 | | 3.2 | 10 000 | 6 000 |
| Spruce-Pine-Fir | SS | 16.5 | | 14.5 | | 8.6 | 10 500 | 7 500 |
| | No. 1/No. 2 | 11.8 | 1.5 | 11.5 | 5.3 | 5.5 | 9 500 | 6 500 |
| | No. 3/Stud | 7.0 | | 9.0 | | 3.2 | 9 000 | 5 500 |
| Northern | SS | 10.6 | | 13.0 | | 6.2 | 7 500 | 5 500 |
| | No. 1/No. 2 | 7.6 | 1.3 | 10.4 | 3.5 | 4.0 | 7 000 | 5 000 |
| | No. 3/Stud | 4.5 | | 5.2 | | 2.0 | 6 500 | 4 000 |

**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.*

Δ

## Table 6.3.1B
## Specified strengths and modulus of elasticity for light framing grades, MPa, applicable to sizes 38 by 38 mm to 89 by 89 mm

| Species identification | Grade | Bending at extreme fibre, $f_b$ | Longi-tudinal shear, $f_v$ | Compression: Parallel to grain, $f_c$ | Compression: Perpen-dicular to grain, $f_{cp}$ | Tension parallel to grain, $f_t$ | Modulus of elasticity: $E$ | Modulus of elasticity: $E_{05}$ |
|---|---|---|---|---|---|---|---|---|
| D Fir-L | Const. | 13.0 | 3.2 | 16.0 | 7.0 | 6.6 | 10 000 | 5 500 |
| | Stand. | 7.3 | | 13.1 | | 3.7 | 9 000 | 5 000 |
| Hem-Fir | Const. | 14.3 | 2.7 | 16.9 | 4.6 | 7.0 | 10 000 | 6 000 |
| | Stand. | 8.0 | | 13.9 | | 3.9 | 9 000 | 5 500 |
| Spruce-Pine-Fir | Const. | 15.3 | 2.6 | 13.1 | 5.3 | 6.2 | 9 000 | 5 500 |
| | Stand. | 8.6 | | 10.8 | | 3.5 | 8 000 | 5 000 |
| Northern | Const. | 9.9 | 2.2 | 11.9 | 3.5 | 4.5 | 6 500 | 4 000 |
| | Stand. | 5.5 | | 9.8 | | 2.5 | 6 000 | 3 500 |

**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 6.5.6.2.3 and $K_{Zcp}$ shall be determined in accordance with Clause 6.5.7.4.*
**(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.*

Δ

## Table 6.3.1C Specified strengths and modulus of elasticity for beam and stringer grades, MPa

| Species identification | Grade | Bending at extreme fibre, $f_b$* | Longi-tudinal shear, $f_v$ | Compression: Parallel to grain, $f_c$ | Compression: Perpen-dicular to grain, $f_{cp}$ | Tension parallel to grain, $f_t$ | Modulus of elasticity: $E$* | Modulus of elasticity: $E_{05}$* |
|---|---|---|---|---|---|---|---|---|
| D Fir-L | SS | 19.5 | 1.5 | 13.2 | 7.0 | 10.0 | 12 000 | 8 000 |
| | No. 1 | 15.8 | | 11.0 | | 7.0 | 12 000 | 8 000 |
| | No. 2 | 9.0 | | 7.2 | | 3.3 | 9 500 | 6 000 |
| Hem-Fir | SS | 14.5 | 1.2 | 10.8 | 4.6 | 7.4 | 10 000 | 7 000 |
| | No. 1 | 11.7 | | 9.0 | | 5.2 | 10 000 | 7 000 |
| | No. 2 | 6.7 | | 5.9 | | 2.4 | 8 000 | 5 500 |
| Spruce-Pine-Fir | SS | 13.6 | 1.2 | 9.5 | 5.3 | 7.0 | 8 500 | 6 000 |
| | No. 1 | 11.0 | | 7.9 | | 4.9 | 8 500 | 6 000 |
| | No. 2 | 6.3 | | 5.2 | | 2.3 | 6 500 | 4 500 |
| Northern | SS | 12.8 | 1.0 | 7.2 | 3.5 | 6.5 | 8 000 | 5 500 |
| | No. 1 | 10.8 | | 6.0 | | 4.6 | 8 000 | 5 500 |
| | No. 2 | 5.9 | | 3.9 | | 2.2 | 6 000 | 4 000 |

**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.*

**(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.*

# Table 6.3.1D
# Specified strengths and modulus of elasticity for post and timber grades, MPa

| Species identification | Grade | Bending at extreme fibre, $f_b$ | 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: $E_{05}$ |
|---|---|---|---|---|---|---|---|---|
| D Fir-L | SS | 18.3 | 1.5 | 13.8 | 7.0 | 10.7 | 12 000 | 8 000 |
| | No. 1 | 13.8 | | 12.2 | | 8.1 | 10 500 | 6 500 |
| | No. 2 | 6.0 | | 7.5 | | 3.8 | 9 500 | 6 000 |
| Hem-Fir | SS | 13.6 | 1.2 | 11.3 | 4.6 | 7.9 | 10 000 | 7 000 |
| | No. 1 | 10.2 | | 10.0 | | 6.0 | 9 000 | 6 000 |
| | No. 2 | 4.5 | | 6.1 | | 2.8 | 8 000 | 5 500 |
| Spruce-Pine-Fir | SS | 12.7 | 1.2 | 9.9 | 5.3 | 7.4 | 8 500 | 6 000 |
| | No. 1 | 9.6 | | 8.7 | | 5.6 | 7 500 | 5 000 |
| | No. 2 | 4.2 | | 5.4 | | 2.6 | 6 500 | 4 500 |
| Northern | SS | 12.0 | 1.0 | 7.5 | 3.5 | 7.0 | 8 000 | 5 500 |
| | No. 1 | 9.0 | | 6.7 | | 5.3 | 7 000 | 5 000 |
| | No. 2 | 3.9 | | 4.1 | | 2.5 | 6 000 | 4 000 |

**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.*

## Table 6.3.2
## Specified strengths and modulus of elasticity for machine stress-rated grades 38 mm wide by all depths, MPa

| Grade | Bending at extreme fibre, $f_b$ | Modulus of elasticity, $E$ | Tension parallel to grain, $f_t$: 89 to 184 mm | Tension parallel to grain, $f_t$: >184 mm* | Compression: Parallel to grain, $f_c$ | Compression: Perpendicular to grain, $f_{cp}$† |
|---|---|---|---|---|---|---|
| 1200$F_b$ -1.2E | 17.4 | 8 300 | 6.7 | — | 15.1 | 5.3 |
| 1350$F_b$ -1.3E | 19.5 | 9 000 | 8.4 | — | 16.9 | 5.3 |
| 1450$F_b$ -1.3E | 21.0 | 9 000 | 9.0 | — | 17.3 | 5.3 |
| 1500$F_b$ -1.4E | 21.7 | 9 700 | 10.1 | — | 17.5 | 5.3 |
| 1650$F_b$ -1.5E | 23.9 | 10 300 | 11.4 | — | 18.1 | 5.3 |
| 1800$F_b$ -1.6E | 26.1 | 11 000 | 13.2 | — | 18.7 | 5.3 |
| 1950$F_b$ -1.7E | 28.2 | 11 700 | 15.4 | — | 19.3 | 5.3 |
| 2100$F_b$ -1.8E | 30.4 | 12 400 | 17.7 | — | 19.9 | 6.5 |
| 2250$F_b$ -1.9E | 32.6 | 13 100 | 19.6 | — | 20.5 | 6.5 |
| 2400$F_b$ -2.0E | 34.7 | 13 800 | 21.6 | — | 21.1 | 6.5 |
| 2550$F_b$ -2.1E | 36.9 | 14 500 | 23.0 | — | 21.7 | 6.5 |
| 2700$F_b$ -2.2E | 39.1 | 15 200 | 24.1 | — | 22.3 | 6.5 |
| 2850$F_b$ -2.3E | 41.3 | 15 900 | 25.8 | — | 22.9 | 6.5 |
| 3000$F_b$ -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.* | | | | | | |
| 1400$F_b$ -1.2E | 20.3 | 8 300 | 9.0 | 9.0 | 17.1 | 5.3 |
| 1600$F_b$ -1.4E | 23.2 | 9 700 | 10.7 | 10.7 | 17.9 | 5.3 |
| 1650$F_b$ -1.3E | 23.9 | 9 000 | 11.4 | 11.4 | 18.1 | 5.3 |
| 1800$F_b$ -1.5E | 26.1 | 10 300 | 14.6 | 14.6 | 18.7 | 5.3 |
| 2000$F_b$ -1.6E | 29.0 | 11 000 | 14.6 | 14.6 | 19.5 | 5.3 |
| 2250$F_b$ -1.7E | 32.6 | 11 700 | 19.6 | 19.6 | 20.5 | 5.3 |
| 2250$F_b$ -1.8E | 32.6 | 12 400 | 19.6 | 19.6 | 20.5 | 6.5 |
| 2400$F_b$ -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 6.3.1A 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 $K_{Zv}$ is given in Table 6.4.5, $K_{Zcp}$ is determined in accordance with Clause 6.5.7.5, and $K_{Zc}$ is calculated in accordance with Clause 6.5.6.2.3.*

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

**Table 6.3.3**
**Specified strengths and modulus of elasticity for machine evaluated lumber grades 38 mm wide by all depths, MPa**

| Grade | Bending at extreme fibre, $f_b$ | 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 | 8 300 | 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 | 9 700 | 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 6.3.1A 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 $K_{Zv}$ is given in Table 6.4.5, $K_{Zcp}$ is determined in accordance with Clause 6.5.7.5, and $K_{Zc}$ is calculated in accordance with Clause 6.5.6.2.3.*

**(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, $K_D$, as given in Clause 5.3.2.2.

### 6.4.2 Service condition factor, $K_S$

The specified strength of lumber shall be multiplied by a service condition factor, $K_S$, as given in Table 6.4.2.

## 6.4.3 Treatment factor, $K_T$

### 6.4.3.1

The specified strength of lumber shall be multiplied by a treatment factor, $K_T$, as given in Table 6.4.3.

### 6.4.3.2

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. See the CWC Commentary on CSA O86.*

## 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 6.4.4.

**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 6.4.4, 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 6.4.4.

## 6.4.5 Size factor, $K_Z$

### 6.4.5.1

Some specified strengths of visually stress-graded lumber vary with member size and shall be multiplied by a size factor, $K_Z$, in accordance with Table 6.4.5.

**Note:** *See Clauses 6.4.5.2 and 6.4.5.3 for exceptions.*

### 6.4.5.2
The size factor, $K_Z$, for light framing grades shall be 1.00, except that $K_{Zc}$ shall be calculated in accordance with Clause 6.5.6.2.3, and $K_{Zcp}$ may be determined in accordance with Clause 6.5.7.5.

### 6.4.5.3
The size factor, $K_Z$, for machine stress-rated lumber and machine evaluated lumber shall be 1.00, except that $K_{Zv}$ shall be as given in Table 6.4.5, $K_{Zc}$ shall be calculated in accordance with Clause 6.5.6.2.3, and $K_{Zcp}$ may be determined in accordance with Clause 6.5.7.5.

Δ

**Table 6.4.2**
**Service condition factors, $K_S$**

| $K_S$ | 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 at extreme fibre | 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 |

**Table 6.4.3**
**Treatment factor, $K_T$**

| Product | Dry service conditions | Wet service conditions |
|---|---|---|
| Untreated lumber | 1.00 | 1.00 |
| Preservative-treated unincised 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 6.4.4
## System factor, $K_H$

| For specified strength in | Case 1* | Case 2† 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.

## Table 6.4.5
## Size factor, $K_Z$, for visually stress-graded lumber

| Larger dimension, mm | Bending and shear $K_{Zb}$, $K_{Zv}$ — Smaller dimension, mm: 38 to 64 | 89 to 102 | 114 or more | Tension parallel to grain, $K_{Zt}$ — All | Compression perpendicular to grain, $K_{Zcp}$ — All | Compression parallel to grain, $K_{Zc}$ — All | All other properties — All |
|---|---|---|---|---|---|---|---|
| 38 | 1.7 | — | — | 1.5 | See Clause 6.5.7.5 | Value computed using formula in Clause 6.5.6.2.3 | 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 to 191 | 1.2 | 1.3 | 1.3 | 1.2 | | | 1.0 |
| 235 to 241 | 1.1 | 1.2 | 1.2 | 1.1 | | | 1.0 |
| 286 to 292 | 1.0 | 1.1 | 1.1 | 1.0 | | | 1.0 |
| 337 to 343 | 0.9 | 1.0 | 1.0 | 0.9 | | | 1.0 |
| 387 or larger | 0.8 | 0.9 | 0.9 | 0.8 | | | 1.0 |

# 6.5 Strength and resistance

## 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
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
In conjunction with Tables 6.3.1C and 6.3.1D, 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.6.5.2.*

## Δ 6.5.3 — *Deleted*

## 6.5.4 Bending moment resistance

### Δ 6.5.4.1 General
The factored bending moment resistance, $M_r$, of sawn lumber members shall be taken as follows:

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

where

$\phi$ = 0.9

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

where

$f_b$ = specified strength in bending, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3)

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

$K_L$ = lateral stability factor (Clause 6.5.4.2)

### 6.5.4.2 Lateral stability factor, $K_L$

#### 6.5.4.2.1
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 adequate 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.

Alternatively, $K_L$ may be calculated in accordance with Clause 7.5.6.4.

#### 6.5.4.2.2
For built-up beams consisting of two or more individual members of the same depth, the ratio in Clause 7.5.6.3.1 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.

## 6.5.5 Shear resistance

### 6.5.5.1 Loads near supports

#### 6.5.5.1.1
In Clause 6.5.5.2, the effect of all loads acting within a distance from a support equal to the depth of the member need not be taken into account.

#### 6.5.5.1.2
In Clause 6.5.5.3.1, 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.

### 6.5.5.2 Shear resistance
The factored shear resistance, $V_r$, shall not be less than the maximum factored shear force, $V_f$, and shall be taken as follows:

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

where
$\phi$ = 0.9
$F_v$ = $f_v (K_D K_H K_{Sv} K_T)$
where
$f_v$ = specified strength in shear, MPa (Clause 6.3)
$A_n$ = net area of cross-section, mm$^2$ (Clause 5.3.8)
$K_{Zv}$ = size factor in shear (Clause 6.4.5)

### Δ 6.5.5.3 Fracture shear resistance at notch

#### 6.5.5.3.1 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, $V_f$ (Clause 6.5.5.3.3), and shall be taken as follows:

$$F_r = \phi F_f A_g K_N$$

where
$\phi$ = 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
$K_{Sf}$ = service condition factor for fracture shear
$A_g$ = $b \times d$ = gross cross-section area, mm$^2$
$K_N$ = notch factor (Clause 6.5.5.3.2)

### 6.5.5.3.2 Notch factor, $K_N$

The notch factor for members of rectangular cross-sections shall be taken 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

$\alpha$ = 1 – ($d_n$/$d$)

where

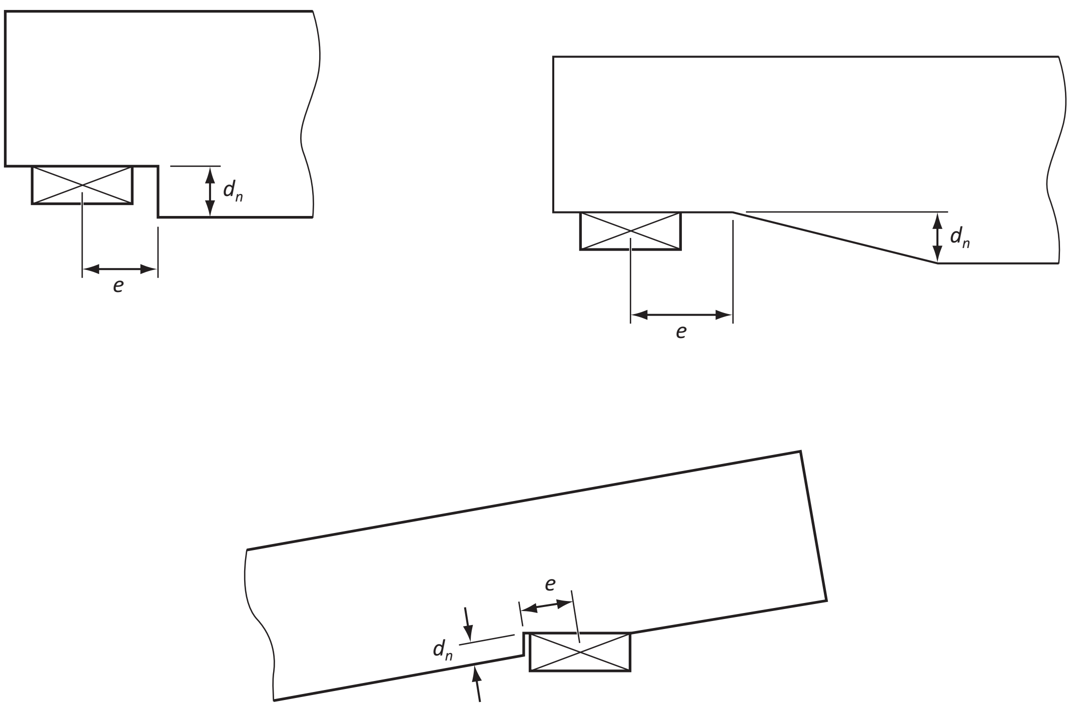
$d_n$ = depth of notch measured normal to the member axis in accordance with Figure 6.5.5.3.2, mm, which shall not exceed 0.25$d$

$\eta$ = $e$/$d$

where

$e$ = length of notch measured parallel to the member axis, mm, from the centre of the nearest support to re-entrant corner of notch (Figure 6.5.5.3.2). For a member notched over an end support, the length of support may be taken as the lesser of minimum required bearing length (Clause 6.5.7) 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 $\alpha$ and $\eta$ are given in Table 6.5.5.3.2.*




**Legend:**
$d_n$ = depth of notch
$e$ = length of notch

**Figure 6.5.5.3.2**
**Determination of length and depth of notch**

**Table 6.5.5.3.2**
**Values of $K_N\sqrt{d}$**

| $\eta$ | $\alpha$ 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 |
| 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 $\alpha$ and $\eta$*

### Δ 6.5.5.3.3 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 notch shear force resistance 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.*

## 6.5.6 Compressive resistance parallel to grain

### 6.5.6.1 Effective length

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. Recommended effective length factors, $K_e$, for compression members are given in Table A.6.5.6.1.

## 6.5.6.2 Simple compression members

### 6.5.6.2.1 General

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

### 6.5.6.2.2 Constant rectangular cross-section

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:

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

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

### 6.5.6.2.3 Factored compressive resistance parallel to grain

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

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

where

$\phi$ = 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 (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3)

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

where

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

$L$ = length associated with member dimension, mm

### 6.5.6.2.4 Slenderness factor, $K_C$

For both axes, the slenderness factor, $K_C$, shall be taken 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

= $0.75E$ for MEL lumber

= as specified in Tables 6.3.1A to 6.3.1D for visually graded lumber

## 6.5.6.3 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.6.5.6.3.*

## 6.5.6.4 Built-up compression members

### 6.5.6.4.1 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 6.5.6.4.2 to 6.5.6.4.4, 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 6.5.6.4.2, 6.5.6.4.3, or 6.5.6.4.4, or the combined factored resistance of the individual pieces taken as independent members.

**Note:** *Slenderness ratios are calculated according to Clause 6.5.6.2.2 using the overall dimensions of the composite member or the dimensions of the individual pieces, as appropriate.*

### 6.5.6.4.2 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 6.5.6.2, 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.

### 6.5.6.4.3 Bolted built-up compression members

The factored compressive resistance of a built-up compression member fastened together by minimum 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 6.5.6.2, 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.

### 6.5.6.4.4 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 1/2 in diameter bolts and 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 6.5.6.2.

### 6.5.6.4.5 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 6.5.6.4.1 to 6.5.6.4.4 shall be taken as the combined factored compressive strength of the individual pieces considered as independent members.

### 6.5.6.4.6 Strong axis buckling

The strength reduction factors given in Clauses 6.5.6.4.2 to 6.5.6.4.4 may be omitted for buckling in the strong axis of the laminations.

### 6.5.6.5 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.

## 6.5.7 Compressive resistance perpendicular to grain

**Note:** *See Clause A.6.5.7.*

### 6.5.7.1 General

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 6.5.7.2 and 6.5.7.3.

### Δ 6.5.7.2 Effect of all applied loads

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

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

where

$\phi$ = 0.8

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

where

$f_{cp}$ = specified strength in compression perpendicular to grain, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3)

$A_b$ = bearing area, $mm^2$

$K_B$ = length of bearing factor (Clause 6.5.7.5)

$K_{Zcp}$ = size factor for bearing (Clause 6.5.7.4)

### Δ 6.5.7.3 Effect of loads applied near a support

#### 6.5.7.3.1 Factored compressive resistance perpendicular to grain

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

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

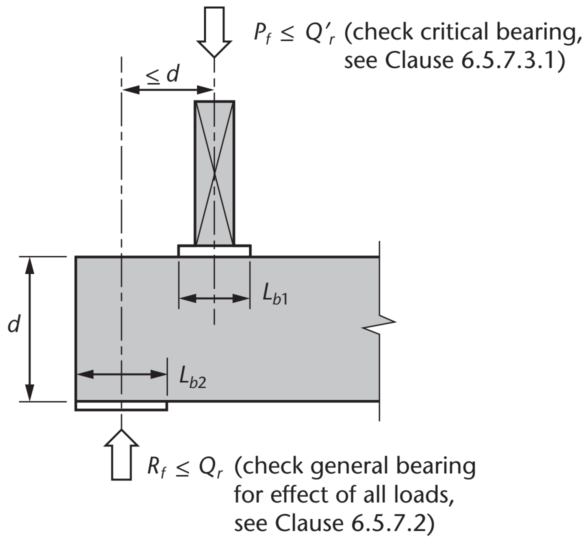
where

$\phi$ = 0.8

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

$A_b'$ = average bearing area, $mm^2$ (see Clause 6.5.7.3.2)

**Note:** *See Figure 6.5.7.3 and CWC* Commentary on CSA O86.




Δ **Figure 6.5.7.3**
**Load applied near a support**

### Δ 6.5.7.3.2 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:

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

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 taken as $1.5b(L_{b1})$.*

### Δ 6.5.7.4 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 6.5.7.4.

Δ **Table 6.5.7.4**
**Size factor for bearing, $K_{Zcp}$**

| 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.*

### Δ 6.5.7.5 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 6.5.7.5, 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.

Δ **Table 6.5.7.5**
**Length of bearing factor, $K_B$**

| Bearing length (parallel to grain) or washer diameter, mm | Modification factor, $K_B$ |
|---|---|
| 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 |

## 6.5.8 Compressive resistance at an angle to grain

The factored compressive resistance at an angle to grain shall be taken 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 (Clause 6.5.6.2.3, assuming $K_C = 1.00$)
$Q_r$ = factored compressive resistance perpendicular to grain, N (Clause 6.5.7.2)
$\theta$ = angle between direction of grain and direction of load, degrees

## 6.5.9 Tensile resistance parallel to grain

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

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

where
$\phi$ = 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 (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3)
$A_n$ = net area of cross-section, mm$^2$ (Clause 5.3.8)
$K_{Zt}$ = size factor in tension (Clause 6.4.5)

## 6.5.10 Resistance to combined bending and axial load

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

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

or

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

where

$P_f$ = factored compressive axial load

$P_r$ = factored compressive load resistance parallel to grain calculated in accordance with Clause 6.5.6

$M_f$ = factored bending moment

$M_r$ = factored bending moment resistance calculated in accordance with Clause 6.5.4

$P_E$ = 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

$K_e$ = the effective length factor given in Clause A.6.5.6.1

$T_f$ = factored tensile axial load

Δ $T_r$ = factored tensile load resistance parallel to grain calculated in accordance with Clause 6.5.9

See *Clause A.6.5.10* for a more detailed interaction formula for combined bending and compressive loads.

## 6.5.11 Decking

### 6.5.11.1 General

To utilize continuity in the design of decking, the conditions specified in Clauses 6.5.11.2 to 6.5.11.5 shall apply.

### 6.5.11.2 Plank decking

**Note:** *Information on point loads supported by plank decking can be found in the CWC* Commentary on CSA O86*.*

#### 6.5.11.2.1 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 2-1/2 in. Planks thicker than 38 mm shall be toe-nailed to the supporting members with one 5 in toe-nail and face-nailed with one or more nails not less than 6 inches 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.

#### 6.5.11.2.2 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, provided that 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.

### 6.5.11.3 Nail laminated decking

#### 6.5.11.3.1 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.

#### 6.5.11.3.2 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.

### 6.5.11.4 Deflection calculations

For uniform design loads, decking deflections for the laying patterns described in Table 6.5.11.4 shall be calculated using the formulas given in Table 6.5.11.4. For other loading conditions or laying patterns, deflections shall be calculated using recognized engineering formulas.

### 6.5.11.5 Bending

Bending moments for plank decking laid in a controlled, random pattern, as described in Table 6.5.11.4, shall be calculated on the basis of simple span moments. For other deck patterns, bending moments shall be calculated on the basis of recognized engineering formulas.

**Table 6.5.11.4**
**Laying patterns and deflection formulas for decking**

| Pattern | Description | Deflection formula* |
|---|---|---|
| Simple span | All pieces bear on two supports only | $\Delta_1 = \dfrac{5w\ell^4}{384E_S I}$ |
| Controlled random | Decking continuous for three or more spans<br>End joints staggered in adjacent planks not less than 610 mm<br>Joints in same general line separated by at least two intervening courses<br>End joints in first half of end spans avoided<br>Requirements of Clause 6.5.11.2.2 or 6.5.11.3.2 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
$\Delta$ = *deflection,* mm
$w$ = *uniformly distributed specified load,* kN/m$^2$
$\ell$ = *span,* mm
$E_S$ = *modulus of elasticity,* MPa
$I$ = *moment of inertia of the decking,* mm$^4$ *per* m *of width*

## 6.5.12 Preserved wood foundations

### 6.5.12.1 General

All lumber and plywood in preserved 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 preserved 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 preserved wood foundations. See Clause A.6.5.12 for more details about wood foundations.*

### 6.5.12.2 Wall footings

In the design of wall footings for preserved 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.

## 6.5.13 Sawn lumber design for specific truss applications

**Note:** *See Clause A.6.5.13.*

### 6.5.13.1 Scope

The design methods specified in Clause 6.5.13 apply only to fully triangulated 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 6.5.13 does not apply to girder, bowstring, semi-circular, or attic trusses (which have non-triangulated sections), or to flat or floor trusses.

Δ **Notes:**

**(1)** *The provisions of Clause 6.5.13 are predicated on the determination of load effects on members and connections in accordance with recognized methods of analysis such as those in TPIC.*

**(2)** *The provisions of Clause 6.5.13 are predicated on truss fabrication and erection in accordance with recognized practices such as those referenced by TPIC.*

### 6.5.13.2 General

Except as modified in Clauses 6.5.13.3 to 6.5.13.5, truss member design shall be in accordance with the sawn lumber provisions of Clauses 6.5.1 to 6.5.10. Truss plate design shall be in accordance with Clause 12.8.

### 6.5.13.3 Compressive resistance parallel to grain

#### Δ 6.5.13.3.1

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.*

#### 6.5.13.3.2

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.

#### 6.5.13.3.3

The member length, $L$, used in Clause 6.5.6.2.3 to compute the factor $K_{Zc}$ shall be the greater of the panel length or one-half the chord length between pitch breaks.

### Δ 6.5.13.4 Compressive resistance perpendicular to grain

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 6.5.7.2 and 6.5.7.3.1.

The requirements of Clause 6.5.7.3.1 may be met by providing adequate 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.*

### Δ 6.5.13.5 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:

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

or

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

where

$P_f$ = factored compressive axial load

$P_r$ = factored compressive load resistance parallel to grain calculated in accordance with Clauses 6.5.6 and 6.5.13.3

$M_f$ = 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 6.5.10

$K_M$ = bending capacity modification factor as defined in Table 6.5.13.5

$M_r$ = factored bending moment resistance calculated in accordance with Clause 6.5.4

$T_f$ = factored tensile axial load

$T_r$ = factored tensile load resistance parallel to grain calculated in accordance with Clause 6.5.9

## Table 6.5.13.5
## Bending capacity modification factor, $K_M$, for specific truss applications

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

*where

$M_1$ = *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 6.5.13.5 is the larger of the absolute values of $M_1$ and $M_2$.*

# 7 Glued-laminated timber (glulam)

## 7.1 Scope
The characteristic strengths, 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 7.2.1.

**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 7.2.1**
**Glued-laminated timber stress grades**

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

### 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, it is possible that the designer will need to compensate for any resulting reduction of cross-section and/or specified strength of the member.

## 7.3 Specified strengths
The specified strengths for glued-laminated timber are given in Table 7.3.

# Table 7.3
# Specified strengths and modulus of elasticity for glued-laminated timber, MPa

(See Clauses 7.5.9.3, 10.5.3, 10.5.4, 10.5.5, 10.6.3.1, 10.6.3.6, 10.6.3.7, A.6.5.6.3.6.)

| | 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, $f_{cp}$ Compression face bearing | 7.0 | 7.0 | 7.0 | 7.0 | 7.0 | 7.0 |
| 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 |
| | 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, $f_{cp}$ Compression face bearing | 5.8 | 5.8 | 5.8 | 5.8 | 4.6 | 7.0 |
| 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 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

The specified strengths for glued-laminated timber are tabulated for dry service conditions. For wet service conditions, tabulated values shall be multiplied by a service condition factor, $K_S$, in accordance with Table 7.4.2.

#### 7.4.2.2

Where glued-laminated 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 7.4.2 may be used.

### 7.4.3 System factor, $K_H$

The specified strengths 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 and 1.10 for all other strength properties.

### 7.4.4 Treatment factor, $K_T$

For preservative treatment, the treatment factor for unincised glued-laminated timber may be taken as unity. 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.

Δ **Table 7.4.2**
**Service condition factors, $K_S$**

| | | Glued-laminated timber | |
|---|---|---|---|
| $K_S$ | Property | Dry service conditions | Wet service conditions |
| $K_{Sb}$ | Bending at extreme fibre | 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 Strength and resistance

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

### 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 beams, the narrow faces of whose 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 Sizes
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
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.

##### 7.5.6.3.2
In the case of glued-laminated members of rectangular section subjected to combined bending and axial loads, the provisions of Clause 6.5.4.2 may be applied.

##### 7.5.6.3.3
For beams composed of two or more individual members of the same depth, the ratio in Clause 7.5.6.3.1 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.

### 7.5.6.4 Calculation of lateral stability factor, $K_L$

#### 7.5.6.4.1 Unsupported length, $\ell_u$

When no additional intermediate support is provided, the unsupported length, $\ell_u$, 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 7.5.6.4.3).

#### 7.5.6.4.2 Prevention of lateral displacement

When the compressive edge of the bending member is supported throughout its length so as 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 taken as follows:

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

where

$L_e$ = effective length, mm, from Table 7.5.6.4.3

**Table 7.5.6.4.3**
**Effective length, $L_e$, for bending members**

| | Intermediate support | |
|---|---|---|
| | 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 | | $1.92\ell_u$ |
| Uniformly distributed load | | $1.23\ell_u$ |
| Concentrated load at free end | | $1.69\ell_u$ |

**Note:** *$\ell_u$ and $a$ are as defined in Clause 7.5.6.4.1.*

### 7.5.6.4.4 Calculation of lateral stability factor, $K_L$

The lateral stability factor shall be taken 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

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

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

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

where

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

where

$f_b$ = specified strength in bending, MPa (Table 7.3)

$K_X$ = curvature factor (Clause 7.5.6.5.2)

## 7.5.6.5 Moment resistance

### 7.5.6.5.1

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:

$M_{r1} = \phi F_b S K_x K_{Zbg}$

$M_{r2} = \phi F_b S K_x K_L$

where

$\phi$ = 0.9

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

where

$f_b$ = specified strength in bending, MPa (Table 7.3)

$K_X$ = curvature factor (Clause 7.5.6.5.2)

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

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 from point of zero moment to point of zero moment, mm

$K_L$ = lateral stability factor (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

For the curved portion only of glued-laminated timber members, the specified strength in bending shall be multiplied by the curvature factor, taken 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.7.5.5).

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

### 7.5.6.5.3

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 tan$\alpha$], where $\alpha$ = slope of upper surface of member (roof slope) in degrees.

### 7.5.6.6 Radial resistance

**Notes:**

**(1)** *The provisions of this Clause apply to members of rectangular cross-section.*

**(2)** *Radial stresses occur in curved and/or double-tapered glued-laminated members and can limit their factored bending moment resistance.*

**(3)** *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

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.

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

where

$\phi$ = 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 (Table 7.3)

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

$K_{Ztp}$ = size factor in tension perpendicular to grain (Table 7.5.6.6.1)

## Table 7.5.6.6.1
## Size factor, $K_{Ztp}$, for tension perpendicular to grain*

| | 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,* mm$^2$
*R = radius of curvature at centreline of member,* mm
*$\beta$ = enclosed angle in radians (1 radian = 57.3°)*

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

### 7.5.6.6.2
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
$\phi$ = 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 (Table 7.3)
$S$ = section modulus at apex, mm$^3$
$K_{Ztp}$ = size factor in tension perpendicular to grain (Table 7.5.6.6.1)
$K_R$ = radial stress factor (Clause 7.5.6.6.3)

### 7.5.6.6.3
The radial stress factor, $K_R$, shall be taken as follows:

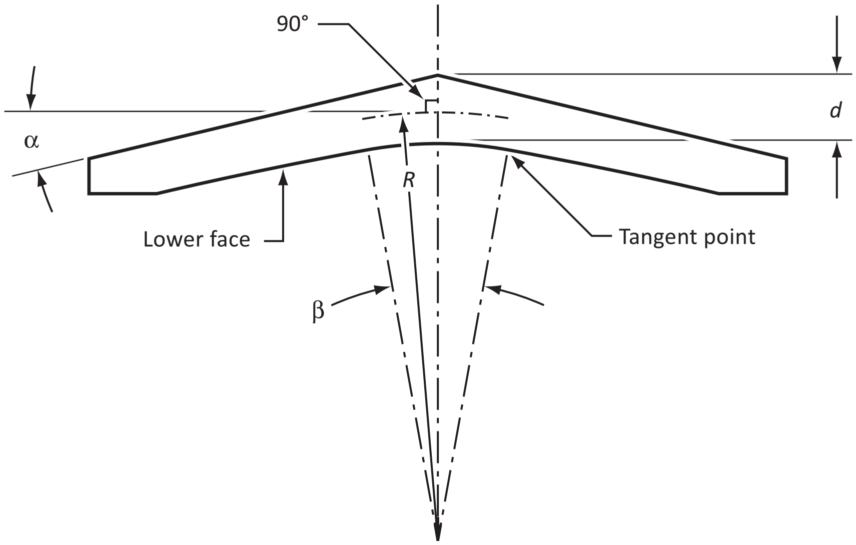
$$K_R = \left[A + B\left(\frac{d}{R}\right) + C\left(\frac{d}{R}\right)^2\right]^{-1}$$

where
$A, B, C$ = constants specified in Table 7.5.6.6.3
$d$ = maximum depth at apex, mm
$R$ = radius of curvature at centreline of member, mm

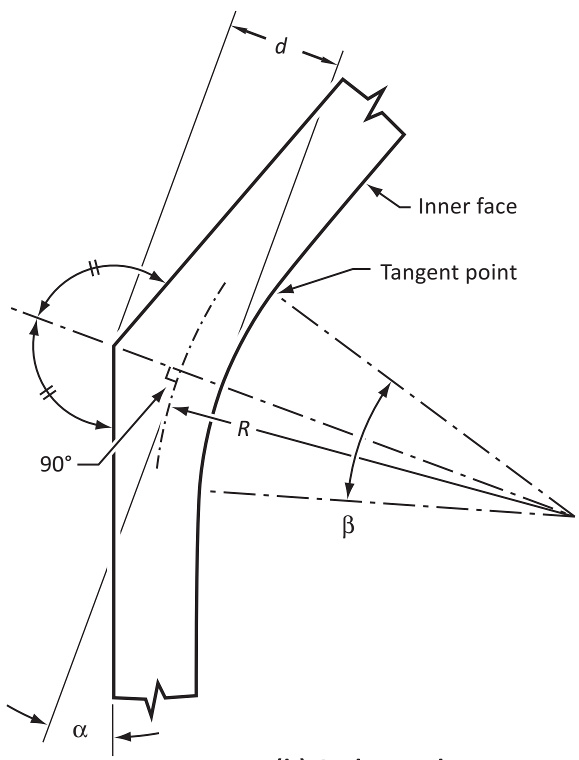
**Table 7.5.6.6.3**
**Values of constants for determination of radial stress in double-tapered curved members**

| 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 7.5.6.6.3.*




**(a) Pitched cambered beam**




**(b) Arch member**

**Legend:**

$\alpha$ = slope of upper surface of member
β = enclosed angle in radians
*d* = maximum depth at apex
*R* = radius of curvature at centreline of member

**Figure 7.5.6.6.3**
**Double-tapered member**

## 7.5.7 Shear resistance

### 7.5.7.1 Loads near supports

#### 7.5.7.1.1

In Clauses 7.5.7.2(b), 7.5.7.3, and 7.5.7.4.1, the effect of all loads acting within a distance from a support equal to the depth of the member need not be taken into account.

#### 7.5.7.1.2

In Clauses 7.5.7.2(a) and 7.5.7.4.2, the effect of all loads acting on the member shall be taken into account, including the loads acting within a distance from the support equal to the depth of the member.

### 7.5.7.2 Shear resistance at locations other than end notches

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

(a) For beams of any volume, the total factored loading, $W_f$, 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 m$^3$ *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 taken as follows:

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

where

$\phi$ = 0.9

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

where

$f_v$ = specified strength in shear, MPa (Table 7.3)

$A_g$ = $b \times d$ = gross cross-sectional area of member, mm$^2$ (Clause 5.3.8)

$C_V$ = shear load coefficient (Clause 7.5.7.5)

$Z$ = beam volume, m$^3$

**Note:** *The shear resistance requirements of this clause are additional to those applicable to notched members (Clauses 7.5.7.3 and 7.5.7.4).*

### 7.5.7.3 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 taken as follows:

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

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

where

$\phi$ = 0.9

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

where

$f_v$ = specified strength in shear, MPa (Table 7.3)

Δ $A_n$ = $b(d - d_n)$ = net cross-sectional area of member, mm$^2$ (Clause 7.5.4)
$A_g$ = $b \times d$ = gross cross-sectional area of member, mm$^2$
$b$ = member width, mm
$d$ = member depth, mm
$d_n$ = notch depth, mm (which shall not exceed $0.25d$)
$e_c$ = length of notch, mm, from inner edge of closest support to farthest edge of notch

### 7.5.7.4 Shear resistance at locations with tension side end notches

#### 7.5.7.4.1 Longitudinal shear resistance of residual member above notch

Tension side notches not exceeding $0.25d$ may be permitted 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 7.5.7.2.

#### Δ 7.5.7.4.2 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 taken as follows:

$$F_r = \phi F_f A_g K_N$$

where
$\phi$ = 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)

$K_{Sf}$ = service condition factor for fracture shear
$K_T$ = treatment factor

$A_g$ = $b \times d$ = gross cross-sectional area, mm$^2$
$K_N$ = notch factor

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

where
$d$ = member depth (unreduced), mm
$\alpha$ = $1 - (d_n/d)$

where
$d_n$ = notch depth, mm (Figure 6.5.5.3.2, which shall not exceed $0.25\,d$)

$\eta$ = $e/d$

where
$e$ = notch length, mm (Figure 6.5.5.3.2)

### 7.5.7.5 Shear load coefficient, $C_V$

For any load condition not specified in Tables 7.5.7.5A to 7.5.7.5F, the coefficient 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, $\ell_a$, such that within each segment there are neither abrupt changes nor changes from negative to positive in the shear force 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:

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

(d) Determine the coefficient, $C_V$, as follows:
   (i) for stationary loads:

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

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

   (ii) for moving loads:

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

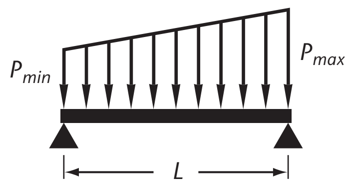
   where
   $W_f$ = the total of all factored moving loads and all factored distributed loads applied to the beam, N

**Table 7.5.7.5A**
**Shear load coefficient, $C_V$, for simple span beams**

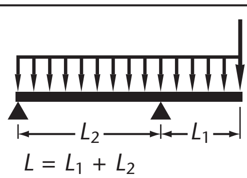
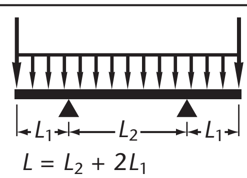
| Number of equal loads equally and symmetrically spaced | r* | | | |
|---|---|---|---|---|
| | 0.0 | 0.5 | 2.0 | 10.0 and over |
| 1 | 3.69 | 3.34 | 2.92 | 2.46 |
| 2 | 3.69 | 3.37 | 3.01 | 2.67 |
| 3 | 3.69 | 3.41 | 3.12 | 2.84 |
| 4 | 3.69 | 3.45 | 3.21 | 2.97 |
| 5 | 3.69 | 3.48 | 3.28 | 3.08 |
|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

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

## Table 7.5.7.5B
## Shear load coefficient, $C_V$, for distributed loads

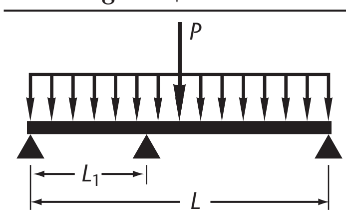
| Type of loading | $P_{min}/P_{max}$ 0.0 | 0.2 | 0.4 | 0.6 | 0.8 | 1.0 |
|---|---|---|---|---|---|---|
|  | 3.40 | 3.55 | 3.63 | 3.67 | 3.69 | 3.69 |

## Table 7.5.7.5C
## Shear load coefficient, $C_V$, for cantilevered beams

| Beam type and loading | $L_1/L_2$ | $r^*$ 0.0 | 0.5 | 2.0 | 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 |
|  | 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 |

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

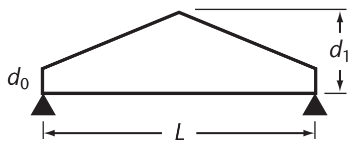
## Table 7.5.7.5D
## Shear load coefficient, $C_V$, for 2-span continuous beams

| Loading case† | $L_1/L$ | $r^*$ 0.0 | 0.5 | 2.0 | 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 |

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

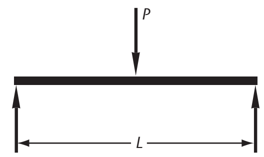
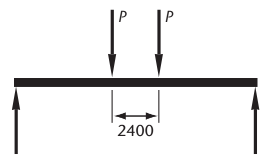
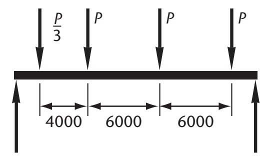
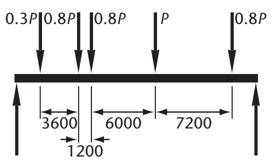
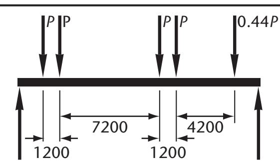
†*The specified values correspond to the worst position for the concentrated loads.*

## Table 7.5.7.5E
## Shear load coefficient, $C_V$, for tapered beams — Uniformly distributed loads

| | | m | | | | | |
|---|---|---|---|---|---|---|---|
| **Beam case** | **$L/d_0$*** | **0.20** | **0.10** | **0.05** | **0.00** | **–0.04** | **–0.08** |
|   $m = \frac{(d_1 - d_0)}{L}$ | 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 | — | — |
|   $m = \frac{2(d_1 - d_0)}{L}$ | 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 | — |

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

## Table 7.5.7.5F
## Shear load coefficient, $C_V$, for moving loads

| 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 |

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

## 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, $L_e = K_e L$, shall be used in determining the slenderness ratio of compression members.

Recommended effective length factors, $K_e$, for compression members are given in Table A.6.5.6.1.

### 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 taken 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 Factored compressive resistance parallel to grain

### 7.5.8.4.1

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

### 7.5.8.4.2

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

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

where

$\phi$ = 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 (Table 7.3)

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

where

$Z$ = member volume, m$^3$

$K_C$ = slenderness factor (Clause 7.5.8.5)

### 7.5.8.5 Slenderness factor, $K_C$

The slenderness factor, $K_C$, shall be taken as follows:

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

where

$E_{05}$ = $0.87E$

### 7.5.8.6 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.6.5.6.3.*

### 7.5.8.7 Built-up compression members

Built-up compression members shall be designed in accordance with Clause 6.5.6.4 using the specified strengths and adjustment 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.

### Δ 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 taken as follows:

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

where

$\phi$ = 0.8

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

where

$f_{cp}$ = specified strength in compression perpendicular to grain, MPa (Table 7.3)

$A_b$ = bearing area, mm$^2$

$K_B$ = length of bearing factor (Clause 6.5.7.5)

$K_{Zcp}$ = size factor for bearing, where the depth is the thickness of lamination (Clause 6.5.7.4)

### 7.5.9.3 Effect of loads applied near a support

### Δ 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 taken as follows:

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

where

$\phi$ = 0.8

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

where

$f_{cp}$ = f-E grade value from Table 7.3 for compression face bearing strength, MPa

$A'_b$ = average bearing area, mm$^2$ (see Clause 7.5.9.3.2)

**Note:** *See Figure 6.5.7.3 and CWC* Commentary on CSA O86.

### Δ 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:

$$A'_b = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \le 1.5b(L_{b1})$$

where

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

$L_{b1}$ = lesser bearing length, mm

$L_{b2}$ = larger bearing length, mm

### Δ 7.5.9.4 — *Deleted*

### 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 6.5.8 using the appropriate specified strengths and resistances for glued-laminated timber.

## Δ 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

$\phi$ = 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 (Table 7.3)

$A_n$ = net area of cross-section, mm$^2$

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

where

$f_{tg}$ = specified strength in tension parallel to grain at gross section, MPa (Table 7.3)

$A_g$ = gross area of cross-section, mm$^2$

## 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:

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

or

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

where

$P_f$ = factored compressive axial load

$P_r$ = factored compressive load resistance parallel to grain calculated in accordance with Clause 7.5.8.4 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, 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.6.5.6.1

**Note:** *See Clause A.6.5.10 for a more detailed interaction formula for combined bending and compressive loads.*

# Δ 8 Cross-laminated timber (CLT)

## 8.1 Scope

The design values and methods given in Clause 8 apply only to panels of primary and custom CLT stress grades manufactured and certified in accordance with ANSI/APA PRG 320 and layups as defined in Clause 8.2. 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 out-of-plane bending applications.*

## 8.2 Materials

### 8.2.1 General

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

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

### 8.2.2 CLT layups

For the purpose of this Standard, the CLT layup shall be a balanced combination of orthogonal layers where all laminations oriented in the same direction are made of structural 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 CLT stress grades

The primary CLT stress grades shall be as specified in Table 8.2.3. Custom CLT stress grades shall be specified by the product manufacturer.

**Table 8.2.3**
**Primary CLT stress grades**

| 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 of laminations

Specified strength values and moduli of elasticity for the laminations in longitudinal and transverse layers of the primary CLT stress grades shall be as specified in Table 8.2.4. Those properties for custom CLT stress grades shall be as published by the product manufacturer.

## Table 8.2.4
## Specified strengths and moduli of elasticity of laminations in primary CLT stress grades, MPa

| 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 | 11700 | 15.4 | 19.3 | 0.50 | 5.3 | 7.0 | 9000 | 3.2 | 9.0 | 0.50 | 5.3 |
| E2 | 23.9 | 10300 | 11.4 | 18.1 | 0.63 | 7.0 | 4.6 | 10000 | 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 | 11000 | 5.8 | 14.0 | 0.63 | 7.0 | 4.6 | 10000 | 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; and*
*(b) standard-term duration of load.*
**(2)** *The specified values are taken from Table 6.3.2 for MSR lumber and Table 6.3.1A 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 8.2.4 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 8.2.4 for clarification of rolling shear.*
**(6)** *The modulus of elasticity for design of compression members, $E_{05}$, shall be taken form Table 6.3.1A for visually stress-graded lumber and 0.82E for MSR lumber.*

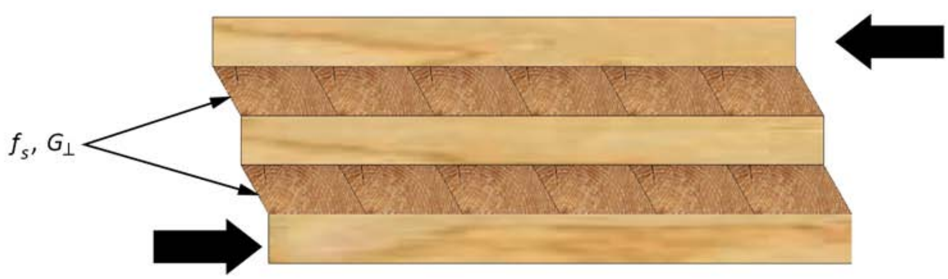



**Figure 8.2.4**
**Rolling shear in CLT**

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

## 8.3 Modification factors

### 8.3.1 Load duration factor, $K_D$
The specified strength 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 which $K_{Sb} = K_{Sc} = K_{Scp} = K_{St} = K_{Sv} = K_{SE} = 1.0$.
**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 is 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 Strength and resistance

### 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 values of laminations (Clause 8.2.4), modification factors (Clause 8.3), and section properties determined in accordance 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 Bending moment resistance

#### 8.4.3.1 General

The out-of-plane factored bending moment resistance, $M_r$, of CLT panels shall be calculated as follows:

(a) for the major strength axis (Figure 8.4.3.2a):

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

where

$\phi$ = 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 (Clause 8.2.4)

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

where

$(EI)_{eff,y}$ = effective bending stiffness of the panel for the major strength axis, N•mm$^2$ (Clause 8.4.3.2)

$E$ = specified modulus of elasticity of laminations in the longitudinal layers, MPa (Clause 8.2.4, Figure 8.4.3.2a)

$h$ = thickness of the panel, mm (Figure 8.4.3.2a)

$K_{rb,y}$ = 0.85

(b) for the minor strength axis (Figure 8.4.3.2b):

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

where

$\phi$ = 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 (Clause 8.2.4)

$$S_{eff,x} = \frac{(EI)_{eff,x}}{E}\frac{2}{h_x}$$

where

$(EI)_{eff,x}$ = effective bending stiffness of the panel for the minor strength axis, N•mm$^2$ (Clause 8.4.3.2)

$E$ = specified modulus of elasticity of laminations in the transverse layers, MPa (Clause 8.2.4, Figure 8.4.3.2b)

$h_x$ = thickness of the panel without the outer longitudinal layers, mm (Figure 8.4.3.2b)

$K_{rb,x}$ = 1.0

### 8.4.3.2 Effective bending stiffness and in-plane (planar) shear rigidity

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 bending stiffness, $(EI)_{eff}$, and effective in-plane (planar) shear rigidity, $(GA)_{eff}$, of CLT panels with alternating orthogonal layers shall be calculated as follows:

(a) For the major strength axis (Figure 8.4.3.2a):

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

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

where

$b_y$ = width of the panel for the major strength axis (Figure 8.4.3.2a), mm

$E_i$ = modulus of elasticity of laminations in the $i$-th layer, MPa

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

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

$G_i$ = shear modulus of laminations in the $i$-th layer, MPa

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

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

$h$ = thickness of the panel (Figure 8.4.3.2a)

$n$ = number of layers in the panel

$t_i$ = thickness of laminations in the $i$-th layer, mm

$z_i$ = distance between the center point of the $i$-th layer and the neutral axis, mm (Figure 8.4.3.2a)

(b) For the minor strength axis (Figure 8.4.3.2b):

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

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

where

$b_x$ = width of the panel for the minor strength axis, mm (Figure 8.4.3.2b)

$E_i$ = modulus of elasticity of laminations in the *i*-th layer, MPa
= $E$, for laminations in the transverse layers, MPa
= $E_\perp$, for laminations in the longitudinal layers, MPa

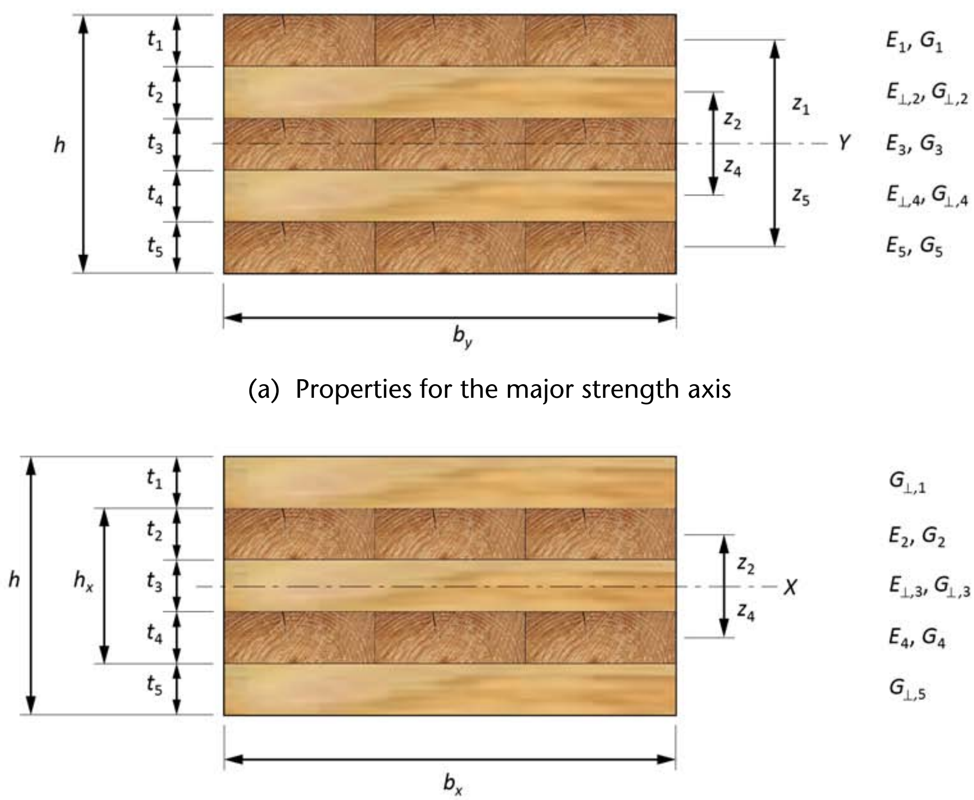
$G_i$ = shear modulus of laminations in the *i*-th layer, MPa
= $G$, for laminations in the transverse layers, MPa
= $G_\perp$, for laminations in the longitudinal layers, MPa

$n$ = number of layers in the panel

$h$ = thickness of the panel, mm (Figure 8.4.3.2b)

$t_i$ = thickness of laminations in the *i*-th layer, mm

$z_i$ = distance between the center point of the i-th layer and the neutral axis, mm (Figure 8.4.3.2b)




(a) Properties for the major strength axis

(b) Properties for the minor strength axis

**Figure 8.4.3.2**
**Properties of a typical five-layer CLT panel**

## 8.4.4 Shear resistance

### 8.4.4.1 General

In the calculation of the shear resistance in Clause 8.4.4.2, the effect of loads acting out of plane within a distance from a support equal to the thickness of the panel need not be taken into account.

### 8.4.4.2 Factored shear resistance

The factored shear resistance, $V_r$, of CLT panels shall be calculated as follows:

(a) for the major strength axis:

$$V_{r,zy} = \phi F_s \frac{2A_{g,zy}}{3}$$

where

$\phi$ = 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 (Clause 8.2.4)

$A_{g,zy}$ = gross cross-sectional area of the panel for the major strength axis, $mm^2$

(b) for the minor strength axis:

$$V_{r,zx} = \phi F_s \frac{2A_{g,zx}}{3}$$

where

$\phi$ = 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 (Clause 8.2.4)

$A_{g,zx}$ = gross cross-sectional area of the panel for the minor strength axis, $mm^2$

## 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.6.5.6.1.

### 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 axial load, $mm^4$

$A_{eff}$ = effective cross-sectional area of the panel accounting only for the layers with laminations oriented parallel to the axial load, $mm^2$

### 8.4.5.4 Factored compressive resistance

#### 8.4.5.4.1
Bending moments due to eccentrically applied axial loads shall be taken into account in accordance with Clause 8.4.6.

#### 8.4.5.4.2
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

$\phi$ = 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 (Clause 8.2.4)

$$K_{Zc} = 6.3\left(\sqrt{12}\ r_{eff}\ L\right)^{-0.13} \le 1.3$$

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

where

$E_{05}$ = modulus of elasticity for design of compression members, only for the laminations oriented parallel to the axial load, MPa (Clause 8.2.4)

$L$ = height of the panel, mm

### 8.4.6 Resistance to combined bending and axial compressive load
CLT panels subject to combined out-of-plane bending and compressive axial load shall be designed to satisfy the interaction equation:

$$\frac{P_f}{P_r} + \frac{M_f}{M_r}\left[\frac{1}{1-\dfrac{P_f}{P_{E,v}}}\right] \le 1$$

where

$P_f$ = factored compressive axial load

$P_r$ = factored compressive resistance under axial load, calculated in accordance with Clause 8.4.5.4

$M_f$ = factored bending moment

$M_r$ = factored 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

$$= \frac{P_E}{1+\dfrac{\kappa\ P_E}{(GA)_{eff}}}$$

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 (Clauses 8.2.4 and 8.4.5.3)

$\kappa$ = form factor

= 1.2 for rectangular cross-sections

$(GA)_{eff}$ = effective in-plane (planar) shear rigidity of CLT panel accounting for all layers, N (Clause 8.4.3.2)

## 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

$\phi$ = 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 (Clause 8.2.4), MPa

$A_b$ = bearing area, mm$^2$

$K_B$ = length of bearing factor (Clause 6.5.7.5)

$K_{Zcp}$ = size factor for bearing

= 1.0

### 8.4.7.3 Effect of loads applied near a support

#### 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

$\phi$ = 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 (Clause 8.2.4), MPa

$A'_b$ = average bearing area, mm$^2$ (Clause 8.4.7.3.2)

**Note:** *See Figure 6.5.7.3 and CWC* Commentary on CSA O86.

#### 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:

$$A'_b = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \le 1.5b(L_{b1})$$

where

$b$ = average bearing width, mm

$L_{b1}$ = lesser bearing length, mm

$L_{b2}$ = larger bearing length, mm

#### 8.4.7.4 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 6.5.8 using the factored resistance values calculated in Clause 8.4.5 with $K_{Zc} = K_C = 1.0$ and Clause 8.4.7.1.

## 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 bending stiffness $(EI)_{eff}$, and the effective in-plane (planar) shear rigidity, $(GA)_{eff}$, 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 equations, data, and construction requirements specified in Clause 9 apply to the materials specified in Clause 9.2.

Design equations 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. Other tests and criteria might be required to characterize panels for special construction uses or other applications.

**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.*

### 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 9.3A and for Canadian softwood plywood in Table 9.3B.

### 9.3.2 OSB

The specified capacities for construction sheathing OSB conforming to CSA O325 are given in Table 9.3C.

Δ

# Table 9.3A
## Specified strength, stiffness, and rigidity capacities for standard constructions of regular grades of unsanded Douglas fir plywood (DFP)

| 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 |
| | 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 |

(Continued)

## Table 9.3A (Concluded)

| 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 | 4 600 |
| 9.5 | 3* | 840 000 | 27 000 | 70 000 | 28 000 | 5 500 |
| 12.5 | 3 | 2 100 000 | 79 000 | 120 000 | 39 000 | 7 800 |
| | 4* | 1 700 000 | 190 000 | 70 000 | 57 000 | 6 900 |
| | 5 | 1 700 000 | 350 000 | 94 000 | 47 000 | 6 900 |
| 15.5 | 4 | 3 100 000 | 430 000 | 77 000 | 75 000 | 8 500 |
| | 5* | 3 000 000 | 630 000 | 94 000 | 47 000 | 8 400 |
| | 6 | 3 000 000 | 760 000 | 94 000 | 47 000 | 8 400 |
| 18.5 | 5 | 4 600 000 | 1 300 000 | 110 000 | 69 000 | 9 800 |
| | 6* | 4 600 000 | 1 300 000 | 94 000 | 47 000 | 9 800 |
| | 7 | 4 900 000 | 1 400 000 | 120 000 | 71 000 | 9 800 |
| 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 |
| | 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 bending on edge, 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), $q_p$, is 4.5 MPa.*

**(4)** *See Figure 9.3 for clarification of shear orientations.*

Δ

# Table 9.3B
## Specified strength, stiffness, and rigidity capacities for standard constructions of regular grades of unsanded Canadian softwood plywood (CSP)

| 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 |
| 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 |

*(Continued)*

## Table 9.3B (Concluded)

| 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 | 3 400 |
| 9.5 | 3* | 610 000 | 27 000 | 55 000 | 28 000 | 4 300 |
| 12.5 | 3 | 1 400 000 | 79 000 | 81 000 | 39 000 | 5 700 |
| | 4* | 1 300 000 | 190 000 | 55 000 | 57 000 | 5 700 |
| | 5 | 1 400 000 | 350 000 | 79 000 | 47 000 | 5 700 |
| 15.5 | 4 | 2 300 000 | 430 000 | 59 000 | 75 000 | 7 100 |
| | 5* | 2 300 000 | 630 000 | 79 000 | 47 000 | 7 100 |
| | 6 | 2 400 000 | 760 000 | 79 000 | 47 000 | 7 100 |
| 18.5 | 5 | 3 600 000 | 1 300 000 | 83 000 | 69 000 | 8 600 |
| | 6* | 3 600 000 | 1 300 000 | 79 000 | 47 000 | 8 600 |
| | 7 | 3 900 000 | 1 400 000 | 100 000 | 71 000 | 8 600 |
| 20.5 | 5 | 4 600 000 | 2 600 000 | 100 000 | 89 000 | 9 500 |
| | 6 | 4 600 000 | 1 900 000 | 79 000 | 47 000 | 9 500 |
| | 7 | 4 900 000 | 2 000 000 | 100 000 | 71 000 | 9 500 |
| | 8 | 4 800 000 | 2 100 000 | 100 000 | 71 000 | 9 500 |
| 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 |
| | 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 bending on edge, 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 9.3 for clarification of shear orientations.*

Δ

## Table 9.3C
## Specified strength, stiffness, and rigidity capacities for construction sheathing OSB

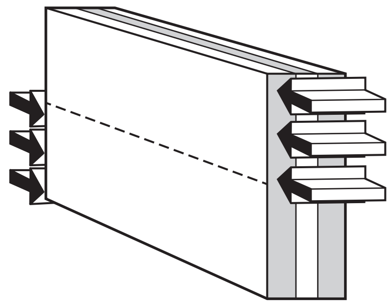
| Panel mark (CSA O325) | Minimum nominal thickness, mm | Bending, $m_p$, N•mm/mm | | Axial tension, $t_p$, N/mm | | Axial compression, $p_p$, N/mm | | Shear-through-thickness, $v_p$, N/mm | Planar shear: Bending, $v_{psb}$, N/mm | | Planar shear: Shear in-plane, $v_{pf}$, MPa | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Capacities relative to major axis* | | | | | | | | | | |
| | | 0° | 90° | 0° | 90° | 0° | 90° | 0° and 90° | 0° | 90° | 0° | 90° |
| 2R24 | 9.5 | 180 | 57 | 53 | 18 | 62 | 54 | 42 | 3.8 | 2.4 | 0.60 | 0.38 |
| 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 |

| Panel mark (CSA O325) | Minimum nominal thickness, mm | Bending stiffness, $B_b = EI$, N•mm$^2$/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° |
| 2R24 | 9.5 | 560 000 | 100 000 | 44 000 | 33 000 | 10 000 |
| 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 | 11 000 000 | 4 400 000 | 108 000 | 61 000 | 20 000 |

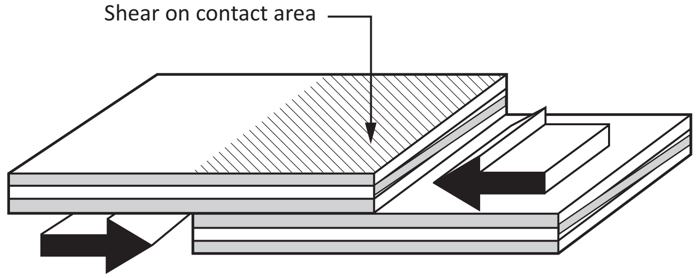
*Orientation of applied force relative to panel's long direction.*

**Notes:**

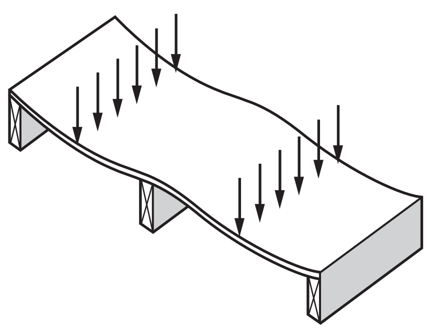
**(1)** *For specified stiffness in bending on edge, use axial stiffness values.*
**(2)** *The tabulated values are based on dry service conditions and standard-term duration of load.*
**(3)** *The specified strength in bearing (normal to plane of panel), $q_p$, is* 4.2 MPa.
**(4)** *The design values do not apply to panels marked W only.*



Shear through thickness $v_p$, N/mm




Planer shear due to in-plane forces in a gusset or splice plate, $v_{pf}$, MPa



Planer shear due to bending, $v_{psb}$, N/mm

Δ

**Figure 9.3**
**Shear orientation in panel products**

## 9.4 Modification factors

### 9.4.1 Load duration factor, $K_D$

#### 9.4.1.1

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 5.3.2.2.

#### 9.4.1.2

For OSB used in structures subject to long term loads in excess of 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 9.4.2.

**Note:** *OSB is specified for use under dry service conditions only.*

**Table 9.4.2**
**Service condition factor, $K_S$**

| | 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, $K_T$ shall be 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 that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 4.3.2.

**Note:** *See the CWC* Commentary on CSA O86 *for additional information.*

### 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 9.4.4.1. Construction requirements for scarf joints are given in Clause 10.4.3.

**Table 9.4.4.1**
**Stress joint factor, $X_J$, for scarf joints**

| 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 9.4.4.2. Construction requirements for butt joints are given in Clause 10.4.4.

**Table 9.4.4.2**
**Stress joint factor, $X_J$, for butt joints**

| Main/side member | Nominal thickness of panel splice plate, mm | One side: Minimum length of splice plate, mm | One side: Tension | One side: Compression, shear | Both sides: Minimum length of splice plate, mm | Both sides: Tension | Both sides: 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.*

### 9.4.5 Factor $K_F$ for preserved wood foundations

For plywood in preserved 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 Resistance of structural panels

**Note:** *Information on point loads supported by structural sheathing can be found in the CWC* Commentary on CSA O86.

### 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 face grain (plywood) or the orientation of the major axis (OSB) intended in the design.

### Δ 9.5.2 Bending as a panel

The factored bending resistance, $M_r$, of a structural panel in the plane perpendicular to the plane of the panel shall be taken as follows:

$$M_r = \phi M_p b_p$$

where

$\phi$ = 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 (Tables 9.3A and 9.3B for plywood and Table 9.3C for OSB)

$b_p$ = width of panel, mm

### 9.5.3 Bending on edge

The factored bending resistance, $M_r$, of structural panels loaded on edge in the plane of a panel that is adequately braced to prevent lateral buckling shall be taken as follows:

$$M_r = \phi T_p \frac{d_p^2}{6}$$

where

$\phi$ = 0.95

$T_p = t_p(K_D K_S K_T)$

where

$t_p$ = specified strength capacity in tension, N/mm (Tables 9.3A and 9.3B for plywood and Table 9.3C for OSB)

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

### 9.5.4 Planar shear

#### Δ 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 taken as follows:

$$V_{rp} = \phi V_{psb} b_p$$

where

$\phi$ = 0.95

$V_{psb} = v_{psb}(K_D K_S K_T K_F)$ 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 (Tables 9.3A and 9.3B for plywood and Table 9.3C 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 taken as follows:

$$V_{rp} = \phi V_{pf} A_c$$

where

$\phi$ = 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 (Tables 9.3A and 9.3B for plywood and Table 9.3C 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 bending of structural panel on edge

The factored resistance in shear, $V_r$, through the thickness of a structural panel due to bending in the plane of the panel shall be taken as follows:

$$V_r = \phi V_p \frac{2d_p}{3}$$

where

$\phi$ = 0.95

$V_p$ = $v_p(K_D K_S K_T)$

where

$v_p$ = specified strength capacity in shear-through-thickness, N/mm (Tables 9.3A and 9.3B for plywood and Table 9.3C 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 taken as follows:

$$V_r = \phi V_p L_G$$

where

$\phi$ = 0.95

$V_p$ = $v_p(K_D K_S K_T)$

where

$v_p$ = specified strength capacity in shear-through-thickness, N/mm (Tables 9.3A and 9.3B for plywood and Table 9.3C for OSB)

$L_G$ = length of splice or gusset plate subjected to shear, mm

## 9.5.6 Compression parallel to panel edge

The factored compressive resistance, $P_r$, parallel to a laterally supported panel edge shall be taken as follows:

$$P_r = \phi P_p b_p$$

where

$\phi$ = 0.95

$P_p = p_p(K_D K_S K_T)$

where

$p_p$ = specified strength capacity in axial compression, N/mm (Tables 9.3A and 9.3B for plywood and Table 9.3C for OSB)

### 9.5.7 Tension parallel to panel edge

The factored tensile resistance, $T_r$, parallel to a panel edge shall be taken as follows:

$T_r = \phi T_p b_n$

where

$\phi$ = 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 (Tables 9.3A and 9.3B for plywood and Table 9.3C 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 taken as follows:

$Q_r = \phi Q_p A_b$

where

$\phi$ = 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 (Tables 9.3A and 9.3B for plywood and Table 9.3C for OSB)

$A_b$ = bearing area, mm$^2$

# 10 Composite building components

## 10.1 Scope

Clause 10 specifies design equations and data for glued composite building components using plywood, OSB, and lumber or glued-laminated timber. The design requirements of Clause 10 apply if the glue joints are made in strict compliance 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 covered by Clause 15.*

## 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, O112.9 or O112.10.

**Note:** *For additional information on equivalent adhesive systems, see the CWC* Commentary on CSA O86*.*

### 10.2.3 Lumber
Clauses 10.3 to 10.6 apply to 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 CSA O141.

### 10.2.4 Glulam
Clauses 10.3 to 10.6 apply to glued-laminated timber that is manufactured in accordance with CAN/CSA-O122.

## 10.3 Stress joint factor, $X_J$

### 10.3.1 Joint requirements
The stress joint factors, $X_J$, specified in Clauses 10.3.2 and 10.3.3 apply to glued plywood and OSB stress joints fabricated in accordance with Clause 10.4.

### 10.3.2 Scarf joints
The stress joint factor for plywood scarf joints across the face grain or for OSB scarf joints across the major axis stressed in tension, compression, or shear-through-thickness shall be as specified in Table 9.4.4.1.

### 10.3.3 Butt joints

#### 10.3.3.1 General
For the length of splice plates perpendicular to the joint, the stress joint factors for butt joints across the face grain of plywood or the major axis of OSB stressed in tension, compression, or shear-through-thickness shall be as specified in Table 9.4.4.2.

#### 10.3.3.2 Butt joints with short plywood plates in compression
For plywood butt joints with splice plates shorter than the minimum length specified in Table 9.4.4.2, the stress joint factor for compression shall be reduced in direct proportion to such reduction in length.

## 10.4 Construction requirements for stress joints

### 10.4.1 Types of stress joints
Joints transmitting forces from one panel to another may be either scarf joints or butt joints.

### 10.4.2 Adhesives for stress joints
Adhesives for the assembly of stress joints shall meet the requirements of Clause 10.2.2.

### 10.4.3 Scarf joints

#### 10.4.3.1 Scarf joints in shear
The slope of plywood scarf joints shall be not steeper than 1:8. The slope of OSB scarf joints shall not be steeper than 1:4.

#### 10.4.3.2 Scarf joints in tension, compression, or bending
The slope of plywood scarf joints shall be not steeper than 1:5. The slope of OSB scarf joints shall not be steeper than 1:4.

### 10.4.4 Butt joints

#### 10.4.4.1 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.

#### 10.4.4.2 Splice plate thickness
Splice plates shall have a minimum thickness equal to that of the panel being spliced.

#### 10.4.4.3 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.

#### 10.4.4.4 Butt joints in tension
Splice plates stressed in tension shall have a minimum length as specified in Table 9.4.4.2.

#### 10.4.4.5 Butt joints in compression
Splice plates stressed in compression may have a length as specified in Table 9.4.4.2. For plywood joints, if shorter lengths are used, the strength of the joint shall be reduced as specified in Clause 10.3.3.2.

## 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/glued 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 10.5.1).

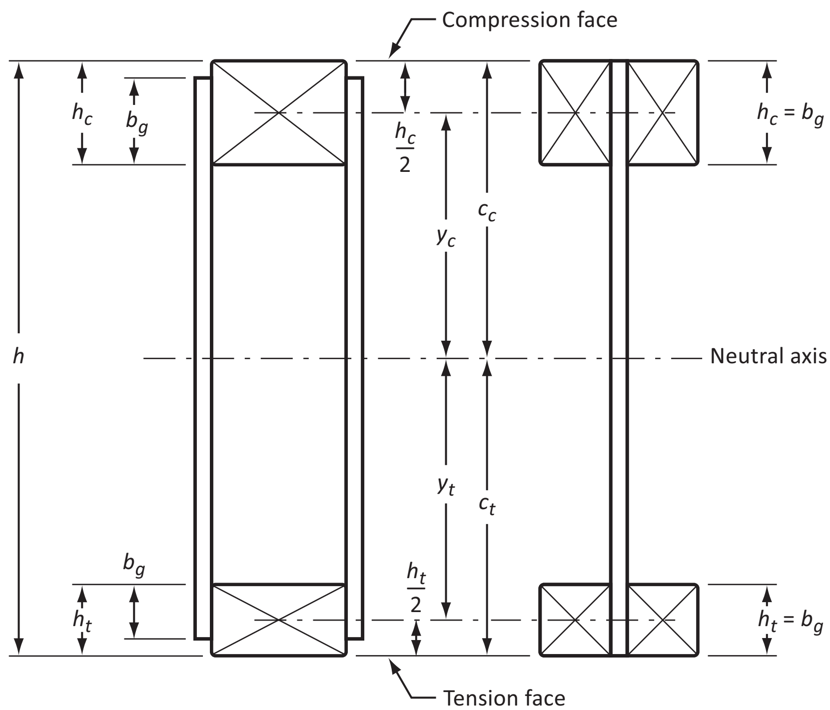



**Figure 10.5.1**
**Panel web beam dimensions, mm**

## 10.5.2 Effective stiffness

The effective stiffness, $(EI)_e$, of a panel web beam shall be taken as follows:

$$(EI)_e = (\Sigma B_a) K_S \frac{\left(c_t^3 + c_c^3\right)}{3} + (EI)_f K_{SE}$$

where

$(\Sigma B_a)$ = sum of axial stiffness of panel webs, N/mm (Tables 9.3A to 9.3C)

$K_S$ = service condition factor for web material (Table 9.4.2)

$(EI)_f$ = 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 (Table 6.4.2 for sawn lumber and Table 7.4.2 for glulam)

## 10.5.3 Bending 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, taken as follows:

(a) compression flange:

$$M_r = \phi F_c K_{Zc} \frac{(EI)_e}{E K_{SE} c_c}$$

where

$\phi$ = 0.8 for sawn lumber

= 0.9 for glulam

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

where
$f_c$ = specified strength of flange in compression, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam)
$K_H$ = system factor (Clause 6.4.4 for sawn lumber and Clause 7.4.3 for glulam)

$K_{Zc}$ = size factor for compression for sawn lumber (Clause 6.4.5)
$E$ = modulus of elasticity of flange (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam), MPa
$K_{SE}$ = service condition factor for modulus of elasticity of flange (Table 6.4.2 for sawn lumber and Table 7.4.2 for glulam)
$c_c$ = distance from neutral axis to compression face (Figure 10.5.1)

(b) tension flange:

$$M_r = \phi F_t K_{Zt} \frac{(EI)_e}{EK_{SE}c_t}$$

where
$\phi$ = 0.9
$F_t = f_t(K_D K_{St} K_T K_H)$

where
$f_t$ = specified strength of flange in tension (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam), MPa

$K_{Zt}$ = size factor for tension for sawn lumber (Table 6.4.5)
$E$ = modulus of elasticity of flange (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam), MPa
$c_t$ = distance from neutral axis to tension face (Figure 10.5.1)

### 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 taken as follows:

$$V_r = \phi V_p X_J \frac{(EI)_e}{EK_{SE}Q_f + 0.5(\Sigma B_a)K_S c_w^2}$$

where
$\phi$ = 0.95
$V_p = (\Sigma v_p)(K_D K_S K_T)$

where
$\Sigma v_p$ = sum of specified strengths of all panel webs in shear-through-thickness, N/mm (Tables 9.3A to 9.3C)

$X_J$ = stress joint factor (Clause 10.3)
$E$ = modulus of elasticity of flange, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam)
$Q_f$ = moment of area of flange about neutral axis, $mm^3$
$\Sigma B_a$ = sum of specified axial stiffness for all panel webs, N/mm (Tables 9.3A to 9.3C)
$c_w$ = greatest distance from neutral axis to outer edge of web, mm

### 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, taken as follows:

$$V_{rp} = \phi V_g \left(\Sigma b_g X_v\right)\frac{(EI)_e}{EK_{SE}Q_f}$$

where

$\Sigma b_g$ = sum of contact widths between flange and web

$E$ = modulus of elasticity of flange, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam)

$Q_f$ = moment of area of flange about neutral axis, mm3

(a) for web:

$\phi$ = 0.95

$V_g = v_{pf}\,(K_D K_S K_T)$

where

$v_{pf}$ = specified strength in planar shear, MPa (Tables 9.3A to 9.3C)

$X_v$ = shear modification factor (Figure 10.5.5)

(b) for flange:

$\phi$ = 0.9

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

where

$f_v$ = specified strength in shear, MPa (Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.3 for glulam)

$X_v$ = 2.00

### 10.5.6 Deflection

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, $\Delta_s$, taken as follows:

$$\Delta_s = \frac{B_a M h^2 X_s}{B_v\,(EI)_e}$$

where

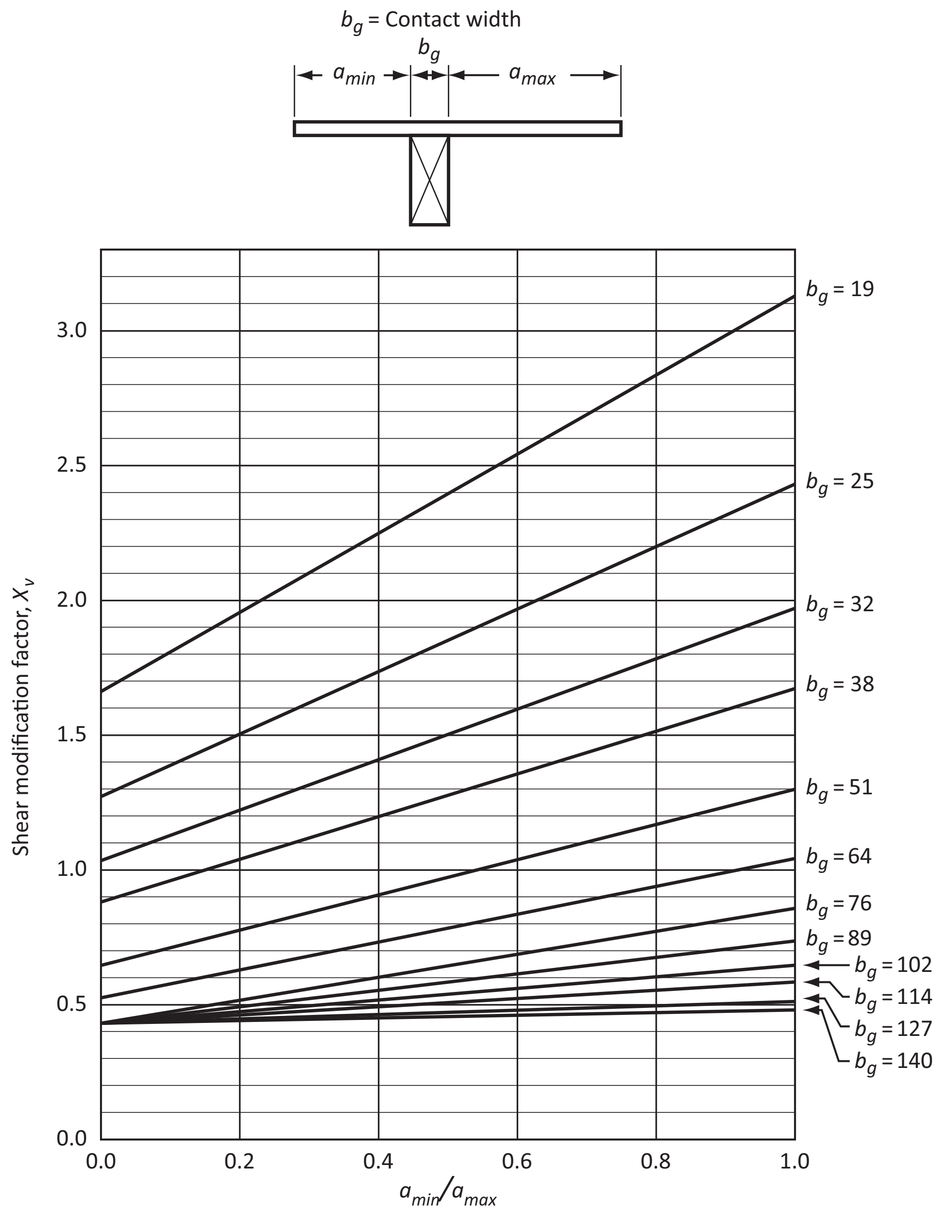
$B_a$ = specified axial stiffness, N/mm (Tables 9.3A to 9.3C)

$M$ = maximum bending moment due to specified loads, N•mm

$h$ = height of web beam, mm (Figure 10.5.1)

$X_s$ = section shear coefficient (Figure 10.5.6)

$B_v$ = specified shear rigidity, N/mm (Tables 9.3A to 9.3C)




**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:*

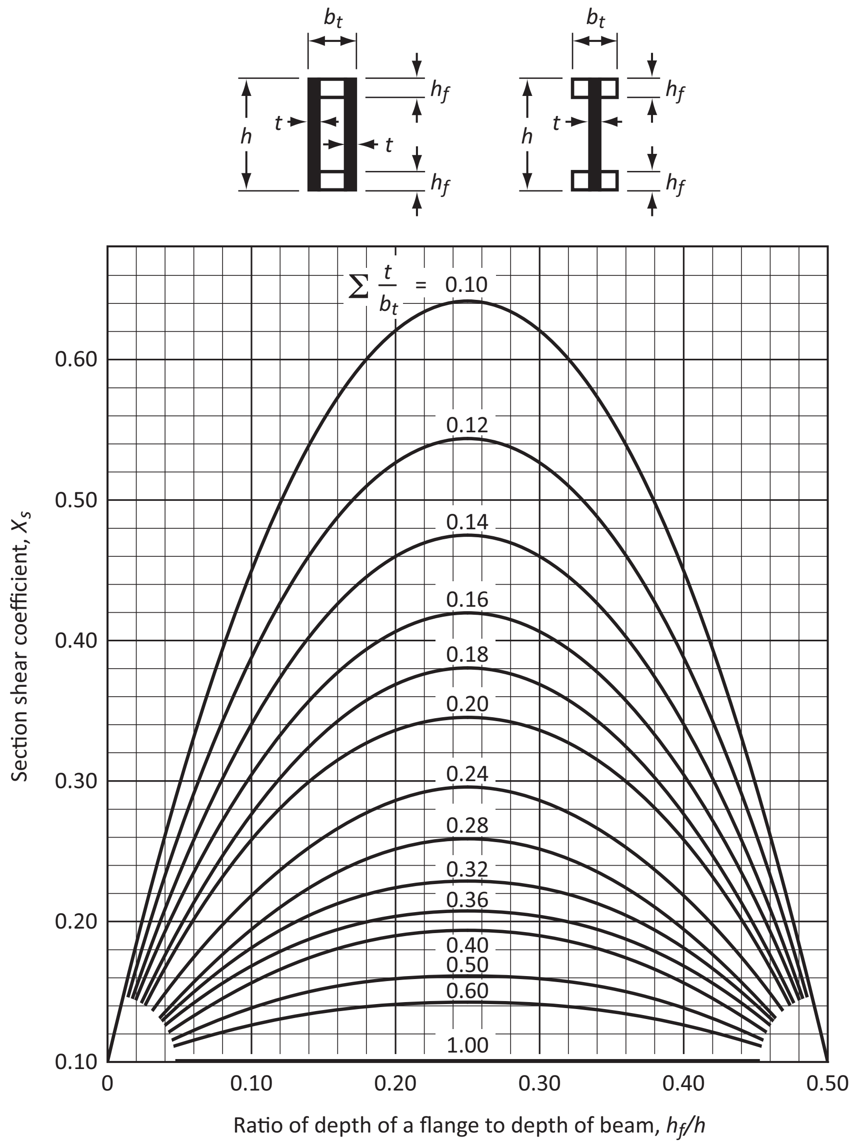
*(1) $a_{min}$ = the overhang at the edge; and*

*(2) $a_{max}$ = 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.*

**Figure 10.5.5**
**Shear modification factor, $X_v$**




**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

**(2)** *This Figure is valid only for box and I-beams symmetrical about two axes.*

**Figure 10.5.6**
**Section shear coefficient, $X_s$**

### 10.5.7 Lateral stability of panel web beams

Lateral stability of a 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 adequately 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 10.6.1).

### 10.6.2 Effective stiffness

The effective stiffness, $(EI)_e$, of a stressed skin panel shall be taken as follows:

$$(EI)_e = (EI)_w K_{SE} + b_f \left(B_{at} y_t^2 + B_{ac} y_c^2\right) K_S$$

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 9.3A to 9.3C)

$B_{ac}$ = specified axial stiffness of compression flange, N/mm (Tables 9.3A to 9.3C)

$b_f$, $y_t$, $y_c$ = panel dimensions, mm, in accordance with Figure 10.6.1

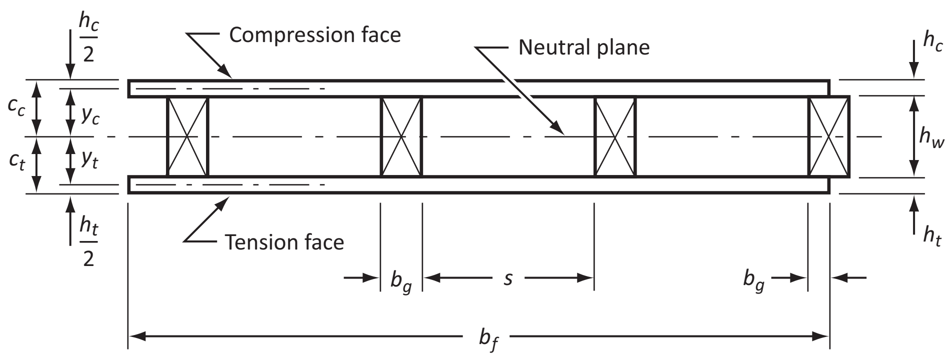



**Figure 10.6.1**
**Stressed skin panel dimensions**

## 10.6.3 Bending resistance

### 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, taken as follows:

(a) tension flange:

$$M_r = \phi T_p X_J X_G \frac{(EI)_e}{B_a K_S c_t}$$

where

$\phi$ = 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 (Tables 9.3A to 9.3C)

$X_J$ = stress joint factor (Clause 10.3)

$X_G$ = panel geometry reduction factor (Clause 10.6.3.2)

$B_a$ = specified axial stiffness, N/mm (Tables 9.3A to 9.3C)

(b) compression flange:

$$M_r = \phi P_p X_J X_G \frac{(EI)_e}{B_a K_S c_c}$$

where

$\phi$ = 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 (Tables 9.3A to 9.3C)

(c) web:

$$M_r = \phi F_b K_{Zb} K_L X_G \frac{(EI)_e}{E K_{SE} c_w}$$

where

$\phi$ = 0.9

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

where

$f_b$ = specified strength in bending of webs, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam)

$E$ = modulus of elasticity of web, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 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 taken as follows:

$$X_G = 1 - 4.8\left(\frac{s}{\ell_p}\right)^2$$

where

$s$ = clear spacing between longitudinals, mm

$\ell_p$ = span of stressed skin panel, mm

**Note:** *This formula accounts for shear lag and is valid for values of $s/\ell_p$ 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 ($\ell_p$); or

(b) $s \leq 40h_c$ for panels with their major axis perpendicular to the span ($\ell_p$)

where s and $h_c$ are as shown in Figure 10.6.1.

**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 10.6.3.6.

### 10.6.3.6 Shear resistance

The factored shear resistance at the neutral plane of a stressed skin panel shall be taken as follows:

$$V_r = \phi F_v K_{Zv} \frac{(EI)_e \Sigma b_g}{EK_{SE}\Sigma Q_w + B_a K_S b_f y}$$

where

$\phi$ = 0.9

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

where

$f_v$ = specified strength in shear of webs, MPa (Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.3 for glulam)

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

$b_g$ = contact width between flange and web (Figure 10.6.1)

$E$ = modulus of elasticity of web, MPa (Tables 6.3.1A to 6.3.1D, 6.3.2, and 6.3.3 for sawn lumber and Table 7.3 for glulam)

$\Sigma Q_w$ = sum of moments of area of all webs about neutral plane, $mm^3$

$B_a$ = specified axial stiffness, N/mm (Tables 9.3A to 9.3C)

$b_f$ = 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, taken 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, N/mm (Tables 9.3A to 9.3C)

$y$ = the greater value of $y_t$ or $y_c$, mm

(a) for flange:

$\phi$ = 0.95

$V_g = v_{pf}\,(K_D K_S K_T)$

where

$v_{pf}$ = specified strength capacity in planar shear, MPa (Tables 9.3A to 9.3C)

$X_v$ = shear modification factor (Figure 10.5.5)

(b) for web:

$\phi$ = 0.9

$V_g = f_v(K_D K_S K_T)$

where

$f_v$ = specified strength in shear, MPa (Clauses 6.3.1 and 6.3.2 for sawn lumber and Table 7.3 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.

# 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, or earth pressure loads.

## Δ 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 structural composite lumber 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.

## Δ 11.3 Design of light-frame shearwalls and diaphragms

### 11.3.1 General

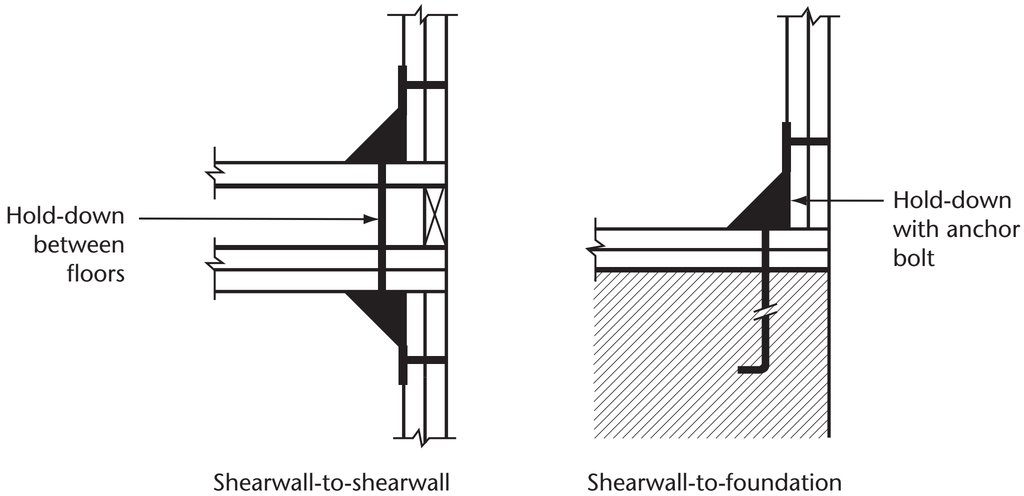
#### 11.3.1.1 Standard methods
Provisions in Clause 11.3 to 11.8 apply to shearwalls and diaphragms constructed with shear panels consisting of structural wood panels, gypsum wallboard, and diagonal lumber sheathing fastened to framing members using nails of diameter 3.66 mm or smaller, unless otherwise stated in these Clauses.

#### 11.3.1.2 Alternative methods
Factored shear resistance of shearwalls and diaphragms may be derived from full-scale testing, provided that the tests are conducted in accordance with recognized test standards.

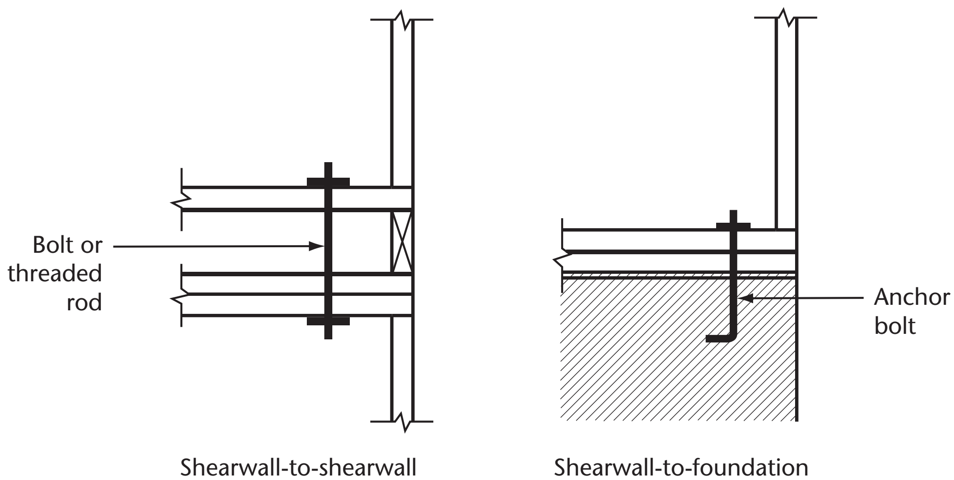
### 11.3.2 Resistance to overturning

#### 11.3.2.1 Shearwall segments with hold-downs
Where the factored dead loads are not sufficient to prevent overturning, hold-down connections (see Figure 11.3.2(a)) shall be designed to resist the factored uplift forces and transfer the forces through a continuous load path to the foundation.

#### 11.3.2.2 Shearwall segments without hold-downs
Where the factored dead loads are not sufficient to prevent overturning and hold-down connections are not used, anchorage (see Figure 11.3.2 b) on the bottom plate within 300 mm from both ends of the shearwall segment shall transfer the uplift force, $R_{ij}$ (see Clause 11.4.5.2), to the supporting structure (i.e., to the top plate of the shearwall below or to the foundation).




**(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 11.3.2.1 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 11.3.2.2 typically transfer loads from the sill plate of the upper storey shearwall segment to the lower storey top plates, beams, or foundations.*

**Figure 11.3.2**
**Examples of hold-downs and anchorages**

## 11.3.3 Shearwalls with segments

### 11.3.3.1 General

The factored shear resistance of a shearwall shall equal the sum of the factored shear resistance of the wall segments determined in accordance with Clause 11.5.1. The factored shear resistance of the shearwall shall be determined for lateral loads acting in opposite directions.

#### 11.3.3.2 Shearwall segment aspect ratio
The maximum aspect ratio (height-to-length ratio) of a shearwall segment shall be 3.5:1. 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.

#### 11.3.3.3 Shearwalls with openings
Shearwalls with openings shall be analyzed as the sum of the separate shearwall segments. The contribution of sheathing above and below openings shall not be included in the calculation of shearwall resistance.

#### 11.3.3.4 Shearwalls with dissimilar materials
Except as permitted by Clause 11.3.3.5, shearwalls constructed with dissimilar materials, thicknesses, or fastener spacings along the length of the shearwall shall be analyzed as the sum of the separate shearwall segments.

#### 11.3.3.5 Alternative method for shearwalls with dissimilar materials
Shearwalls constructed with dissimilar materials, thicknesses, or fastener spacings along the length of the shearwall may be analyzed, in accordance with Clause 11.5.1, as a shearwall of uniform construction, provided that the least value of $v_d\, J_{us}\, J_s$ is assumed to apply over the entire shearwall.

### 11.3.4 Shearwalls with multiple layers

#### Δ 11.3.4.1 Shearwalls with two layers of panels on one side
The factored shear resistance for a shearwall with two layers of the same or different 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 framing member, as long as minimum fastener penetration in the framing is satisfied.

#### Δ 11.3.4.2 Two-sided shearwalls
The factored shear resistance from each side of the same shearwall shall be cumulative when panels of the same or different materials are applied on both sides.

**Notes:**

**(1)** *See Table 11.8.1 for the appropriate seismic design factors for shearwalls with different sheathing materials.*

**(2)** *See seismic design requirements for shearwalls using gypsum wallboard in Clause 11.8.8.*

### 11.3.5 Concrete or masonry wall anchorage

#### 11.3.5.1 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 capable of resisting the lateral force induced by the wall, but not less than 3 kN per lineal metre of the wall.

#### 11.3.5.2 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.

### 11.3.6 Shearwall anchorage
The anchor bolt connections that resist the lateral forces shall be designed in accordance with Clause 12.

## 11.4 Modification factors

### 11.4.1 Fastener spacing factor, $J_s$

The shear strength of shearwalls and diaphragms built with structural wood-based panels 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

$$= 1 - \left(\frac{150 - s}{150}\right)^{4.2} \text{ for } 50 \text{ mm} \leq s < 150 \text{ mm}$$

where

$s$ = fastener spacing along panel edges, mm

**Note:** *For proprietary framing material, appropriate adjustment factors can be obtained from product manufacturer (i.e., proprietary design literature and Product Evaluation Reports in the CCMC Registry of Product Evaluations).*

### 11.4.2 Fastener row factor for blocked diaphragms, $J_f$

The shear strength of blocked diaphragms sheathed with wood-based structural panels shall be multiplied by the fastener row factor, $J_f$, given in Table 11.4.2, to account for the number of rows of fasteners attaching the sheathing to framing:

**Table 11.4.2**
**Fastener row factor, $J_f$, for blocked diaphragms**

| Number 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.*

### 11.4.3 Strength adjustment factor for unblocked diaphragms, $J_{ud}$

The shear strength of unblocked diaphragms sheathed with wood-based structural panels shall be multiplied by the strength adjustment factor, $J_{ud}$, given in Table 11.4.3.

**Table 11.4.3**
**Strength adjustment factor, $J_{ud}$, for unblocked diaphragms***

| Configuration case† | $J_{ud}$ |
|---|---|
| 1 | 0.89 |
| 2, 3 and 4 | 0.67 |

**The shear strength of an unblocked diaphragm shall be calculated by multiplying the strength adjustment factor by the specified shear strength of a blocked diaphragm with fasteners spaced at* 150 mm *on centre along panel edges and* 300 mm *on centre along intermediate framing members.*
*†See Figure 11.4.3.*

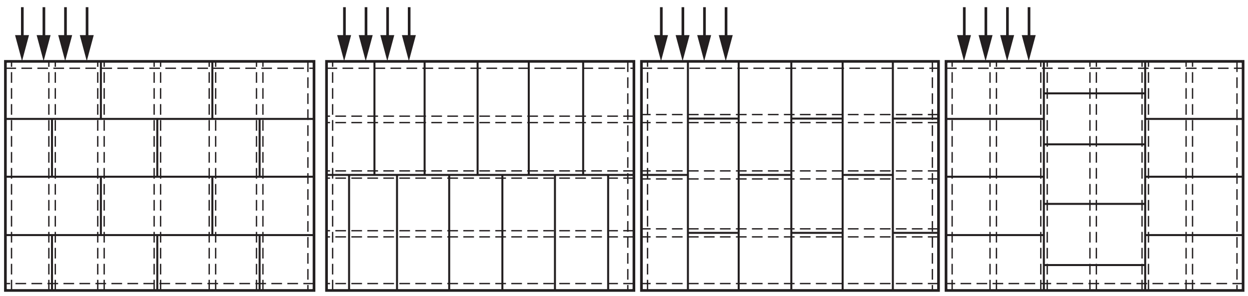



**Figure 11.4.3**
**Configurations of unblocked diaphragms**

### 11.4.4 Strength adjustment factor for unblocked shearwalls, $J_{us}$

The specified shear strengths for unblocked shearwalls sheathed with wood-based structural panels shall be multiplied by the strength adjustment factor, $J_{us}$, given in Table 11.4.4.

The maximum height, $H$, of unblocked shearwalls shall be 4.88 m. The maximum aspect ratio for unblocked shearwalls shall be 2:1.

**Table 11.4.4**
**Strength adjustment factor, $J_{us}$, for unblocked shearwalls***

| 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 |

**The shear strength of an unblocked shearwall shall be calculated by multiplying the strength adjustment factor by the specified shear strength of a blocked shearwall with fasteners spaced at* 150 mm *on centre along panel edges and* 300 mm *on centre along intermediate framing members.*

**Notes:**

**(1)** *The strength adjustment factor shall be applicable only to structural wood-based panels and the stated fastener spacings.*

**(2)** *See Figure 11.4.4 for configurations of unblocked shearwalls.*

**(3)** *See the CWC* Commentary on CSA O86 *for additional information on the diagrams.*

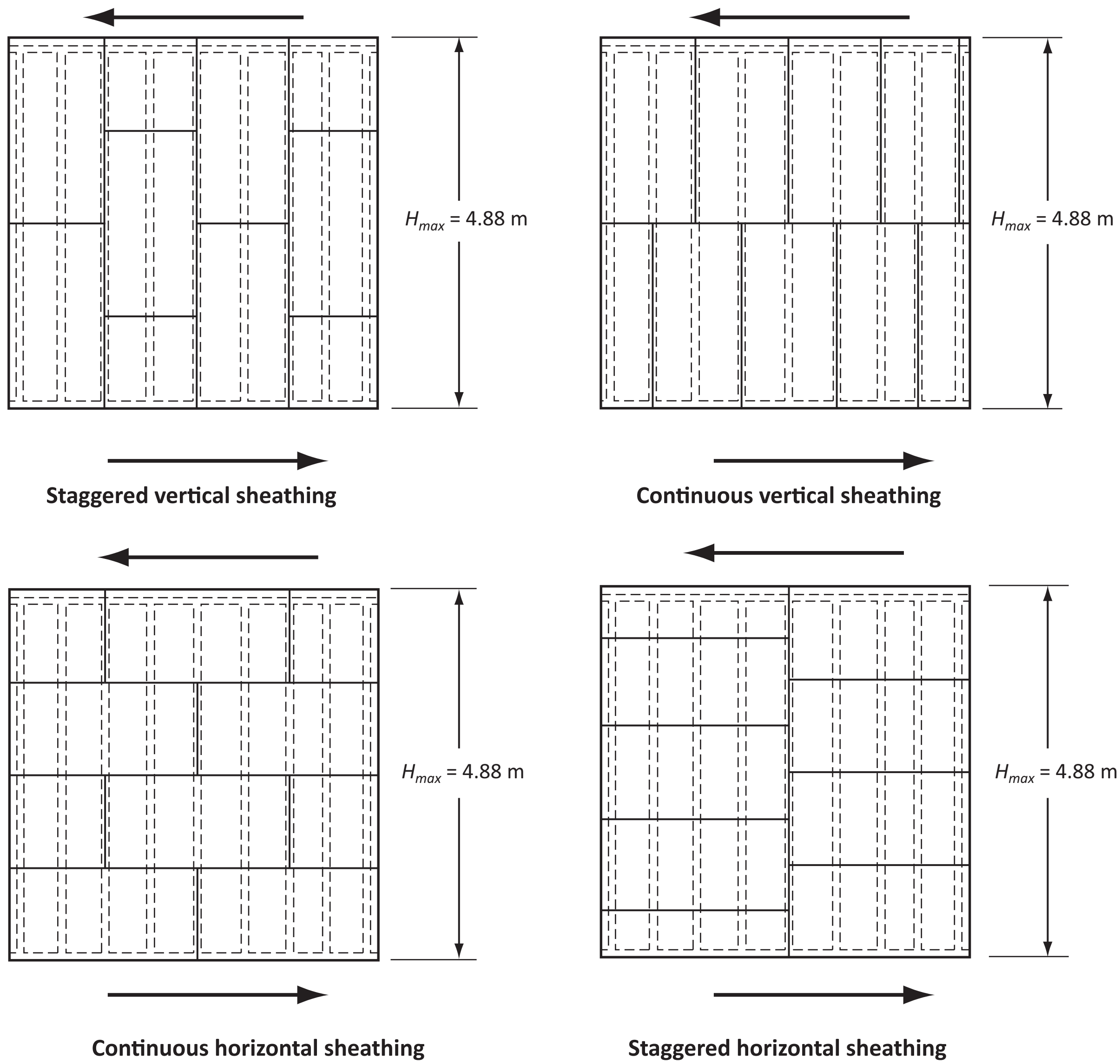



**Figure 11.4.4**
**Configurations for unblocked shearwalls**

## 11.4.5 Hold-down effect factor for shearwall segments, $J_{hd}$

### 11.4.5.1 Shearwall segments with hold-downs to resist all overturning tension forces

For shearwall segments with hold-down connections that are designed to resist all of the factored tension forces due to overturning, $J_{hd}$ shall equal 1.0.

### 11.4.5.2 Shearwall segments without hold-downs

For shearwall segments
(a) without hold-down connections at either end;
(b) meeting the requirements of Clauses 11.3.2.2 and 11.4.5.5; and

(c) having $P_{ij}$ equal to or greater than zero, $J_{hd}$ shall be taken as follows:

$$J_{hd} = \sqrt{1+2\frac{P_{ij}}{V_{hd}}+\left(\frac{H_s}{L_s}\right)^2} - \frac{H_s}{L_s} \le 1.0$$

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 basic shear resistance, kN

= factored shear resistance of the shearwall segment calculated with $J_{hd}$ equal to 1.0

$H_s$ = height of shearwall segment, m

$L_s$ = length of shearwall segment, m

**Note:** *See Figure 11.4.5.2.*

### 11.4.5.3 Shearwall segments with hold-downs only at the bottom of the segment

For a lower-storey shearwall segment with hold-down connections at the bottom of the shearwall segment and without hold-down connections at the top of the shearwall segment, and having $P$ less than zero,

$$J_{hd} = \frac{V_{hd} + P}{V_{hd}} \le 1.0$$

where

$V_{hd}$ = factored basic shear resistance, kN

= factored shear resistance of the shearwall segment calculated with $J_{hd}$ equal to 1.0

$P$ = uplift restraint force at the top of the end stud of a shearwall segment, kN

### 11.4.5.4 Shearwall segments with hold-downs only on one side

For a shearwall segment with hold-down connections only on one side of the segment, the following shall apply:

(a) $J_{hd}$ shall be determined in accordance with Clause 11.4.5.1 if designed to resist all tension forces due to overturning;

(b) $J_{hd}$ shall be determined in accordance with Clause 11.4.5.2 if there is no hold-down on the tension side of the segment; and

(c) $J_{hd}$ shall be determined in accordance with Clause 11.4.5.3 if there is a hold-down only at the bottom of the tension side of a lower-storey segment.

### 11.4.5.5 Conditions for shearwall segments with $J_{hd}$ < 1.0

The conditions for calculating $J_{hd}$ in Clauses 11.4.5.2 to 11.4.5.4 shall be as follows:

(a) the maximum fastener diameter shall be 3.25 mm and the minimum fastener spacing shall be 100 mm;

(b) the maximum specified shear strength, including both sides of a shearwall where applicable, shall be 10.3 kN/m; and

(c) the height of the shearwall segment shall be less than 3.6 m.

## 11.5 Strength and resistance

### Δ 11.5.1 Shear resistance of shearwalls

### Δ 11.5.1.1 General

The factored shear resistance, $V_r$, of shearwalls constructed with wood-based structural panels, gypsum wallboard, or diagonal lumber sheathing in accordance with Clauses 11.5.3, 11.5.4, or 11.5.5, respectively, shall be taken as follows:

$V_r = \Sigma V_{rs}$

where

$V_{rs}$ = factored shear resistance for each shearwall segment along the length of the shearwall, kN, in accordance with Clauses 11.5.1.2, 11.5.1.3, or 11.5.1.4, respectively.

### Δ 11.5.1.2 Shearwall segments with wood-based structural panels

The factored shear resistance, $V_{rs}$, for a shearwall segment with wood-based 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

$\phi$ = 0.8

$v_d$ = $N_u/s$, kN/m

where

$N_u$ = lateral strength resistance of sheathing-to-framing connections along panel edges, per fastener (see Clause 12.9.4 for nails), N

$s$ = fastener spacing along panel edges, mm

$J_D$ = adjustment factor for diaphragm and shearwall construction (Clause 12.9.4.1)

$n_S$ = number of shear planes in sheathing-to-framing connections (see Clause 11.5.3.4)

$J_s$ = fastener spacing adjustment factor (Clause 11.4.1)

$J_{us}$ = strength adjustment factor for unblocked shearwalls (Clause 11.4.4)

$J_{hd}$ = hold-down effect factor for shearwall segment (Clause 11.4.5)

$L_s$ = length of shearwall segment parallel to direction of factored load, m

(b) Shear resistance governed by sheathing panel buckling:

$$V_{rs} = \phi\, v_{pb} K_D K_S K_T\, L_s$$

where

$\phi$ = 0.8

$K_S$ = service condition factor (Table 9.4.2)

$v_{pb}$ = 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

$$= 1.7(\eta+1)\exp\left(\frac{-\alpha}{0.05\eta+0.75}\right)+(0.5\eta+0.8)$$

where

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

$$\eta = \frac{2\,B_v}{\sqrt{B_{a,0}B_{a,90}}}$$

where

$a$ = larger dimension of panel, mm

$b$ = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

$B_{a,90}$ = axial stiffness of panel 90° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

$B_v$ = shear-through-thickness rigidity, see Tables 9.3A, 9.3B and 9.3C, N/mm

$t$ = panel thickness, mm

### Δ 11.5.1.3 Shearwall segments with gypsum wallboard sheathing

The factored shear resistance, $V_{rs}$, for a shearwall segment with gypsum wallboard sheathing shall be taken as follows:

$$V_{rs} = \phi\, v_d\, J_{hd} L_s$$

where

$\phi$ = 0.7

$v_d$ = specified shear strength for shearwall segment sheathed with gypsum wallboard, kN/m (Table 11.5.4)

$J_{hd}$ = hold-down effect factor for shearwall segment (Clause 11.4.5)

$L_s$ = length of shearwall segment parallel to direction of factored load, m

### Δ 11.5.1.4 Shearwall segments with diagonal lumber sheathing

The factored shear resistance, $V_{rs}$, for a shearwall segment with diagonal lumber sheathing shall be calculated as follows:

$$V_{rs} = \phi\, v_d\, J_D\, n_L\, L_s$$

where

$\phi$ = 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 (see Clause 12.9.4, 11.5.5.2, and 11.5.5.3), N

$s$ = fastener spacing evenly distributed along boundary frame members, but not less than 65 mm (see Clause 11.5.5.2)

$J_D$ = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.4.1)

$n_L$ = number of sheathing layers

= 1 for single-layer lumber sheathing

= 2 for double-layer lumber sheathing

$L_s$ = length of shearwall segment parallel to direction of factored load, m

**Note:** *See CWC* Commentary on CSA O86 *on nailed shearwalls and diaphragms using diagonal lumber sheathing.*

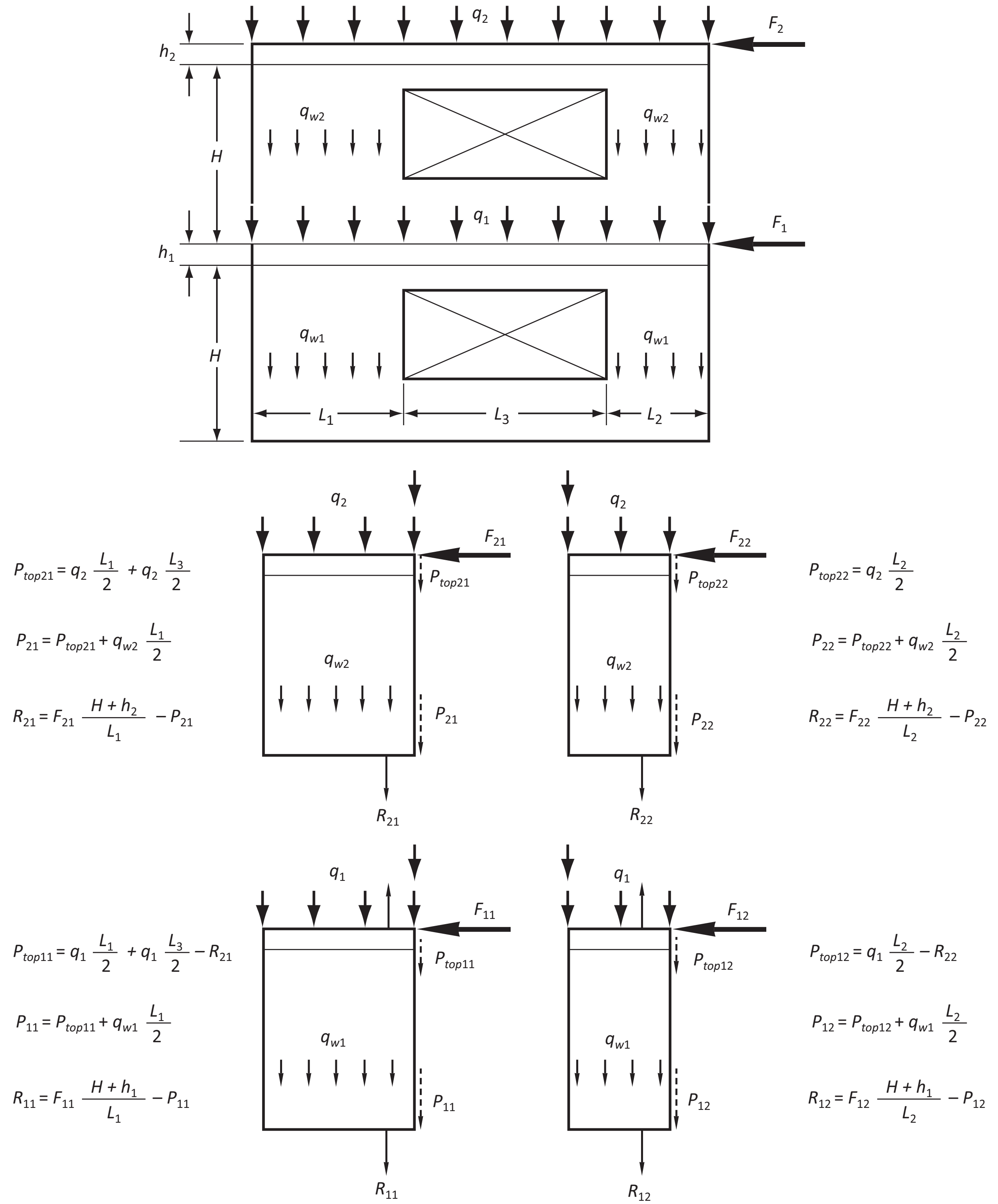



**Figure 11.4.5.2**
**Multi-storey shearwall force diagrams**

*(Continued)*

**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

$R_{ij}$ = resultant overturning 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 11.5.1, kN

$\Sigma V_{rs}$ = sum of factored shear resistances, kN, for each segment in a shearwall

**Figure 11.4.5.2 (Concluded)**

## Δ 11.5.2 Shear resistance of diaphragms

### Δ 11.5.2.1 General

The factored shear resistance of diaphragms constructed with diagonal lumber sheathing in accordance with Clause 11.5.5 shall be calculated in accordance with Clause 11.5.2.2. The factored shear resistance of diaphragms constructed with wood-based structural panels in accordance with Clause 11.5.3 shall be calculated in accordance with Clause 11.5.2.2.

### Δ 11.5.2.2 Diaphragms with diagonal lumber sheathing

The factored shear resistance, $V_{rs}$, of a diaphragm with diagonal lumber sheathing shall be taken as follows:

$$V_{rs} = \phi\, v_d\, J_D\, n_L L_D$$

where

$\phi$ = 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 (see Clauses 12.9.4, 11.5.5.2, and 11.5.5.3), N

$s$ = fastener spacing evenly distributed along boundary frame members, but not less than 65 mm (see Clause 11.5.5.2)

$J_D$ = adjustment factor for diaphragm and shearwall construction (see Clause 12.9.4.1)

$n_L$ = number of sheathing layers

= 1 for single-layer lumber sheathing

= 2 for double-layer lumber sheathing

$L_D$ = dimension of diaphragm parallel to direction of factored load, m

**Note:** *See CWC* Commentary on CSA O86 *on nailed shearwalls and diaphragms using diagonal lumber sheathing.*

### Δ 11.5.2.3 Diaphragms with wood-based structural panels

The factored shear resistance, $V_{rs}$, of a diaphragm with wood-based 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

$\phi$ = 0.8

$v_d$ = $N_u/s$, kN/m

where

$N_u$ = lateral strength resistance of sheathing-to-framing connections along panel edges, per fastener (see Clause 12.9.4 for nails), N

$s$ = fastener spacing along panel edges, mm

$J_D$ = adjustment factor for diaphragm and shearwall construction (Clause 12.9.4.1)

$J_s$ = fastener spacing factor (Clause 11.4.1)

$J_f$ = fastener row factor for blocked diaphragms (Clause 11.4.2)

$J_{ud}$ = strength adjustment factor for unblocked diaphragms (Clause 11.4.3)

$L_D$ = dimension of diaphragm parallel to direction of factored load, m

(b) shear resistance governed by sheathing panel buckling:

$$V_{rs} = \phi\, v_{pb} K_D K_s K_T\, L_D$$

where

$\phi$ = 0.8

$K_S$ = service condition factor (Table 9.4.2)

$v_{pb}$ = 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

$$= 1.7(\eta+1)\exp\left(\frac{-\alpha}{0.05\eta+0.75}\right)+(0.5\eta+0.8)$$

where

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

$$\eta = \frac{2\,B_v}{\sqrt{B_{a,0}B_{a,90}}}$$

where

$a$ = larger dimension of panel, mm

b = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

$B_{a,90}$ = axial stiffness of panel 90° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

$B_v$ = shear-through-thickness rigidity, see Tables 9.3A, 9.3B and 9.3C, N/mm

$t$ = panel thickness, mm

### 11.5.3 Shearwalls and diaphragms using plywood or OSB

#### 11.5.3.1 General

Shearwalls and diaphragms sheathed with plywood or OSB may be used to resist lateral forces based on the factored shear strength determined in accordance with Clause 11.5.1 for shearwalls and Clause 11.5.2 for diaphragms. Shearwalls and diaphragms shall be designed to resist lateral load only and perimeter members shall be provided to resist axial forces resulting from the application of lateral design forces.

#### 11.5.3.2 Framing members

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 or adjoining panel edges.

#### 11.5.3.3 Framing and panels

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, where up to two short or narrow panels may be used. Except as provided in Clauses 11.4.3 and 11.4.4, adjoining panel edges shall bear and be attached to the framing members and a gap of not less than 2 mm shall be left between wood-based panel sheets.Panels for shearwalls and diaphragms shall be arranged as indicated in Figures 11.5.3.3A and 11.5.3.3B respectively. Framing members shall be provided at the edge of all panels in shearwalls and diaphragms except for unblocked assemblies described in Clauses 11.4.3 and 11.4.4. For construction sheathing OSB, the product specification shall also include a panel mark identifying an end-use rating. Table A.9.2.2A 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. Panels less than 300 mm wide shall be blocked.

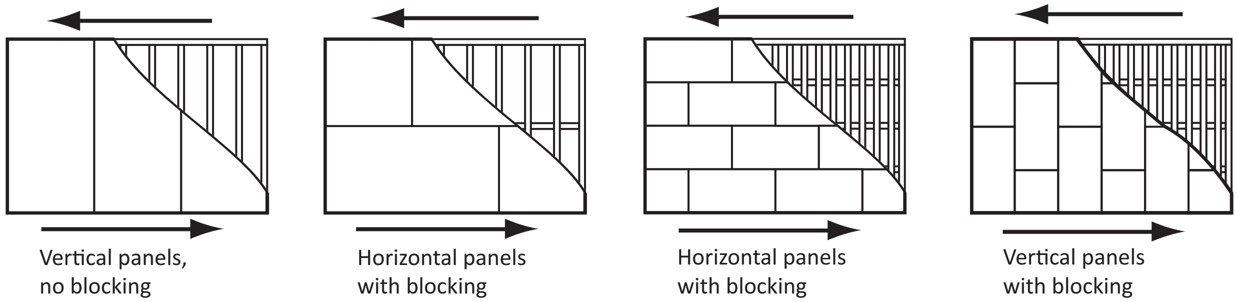



**Figure 11.5.3.3A**
**Shearwall configurations**

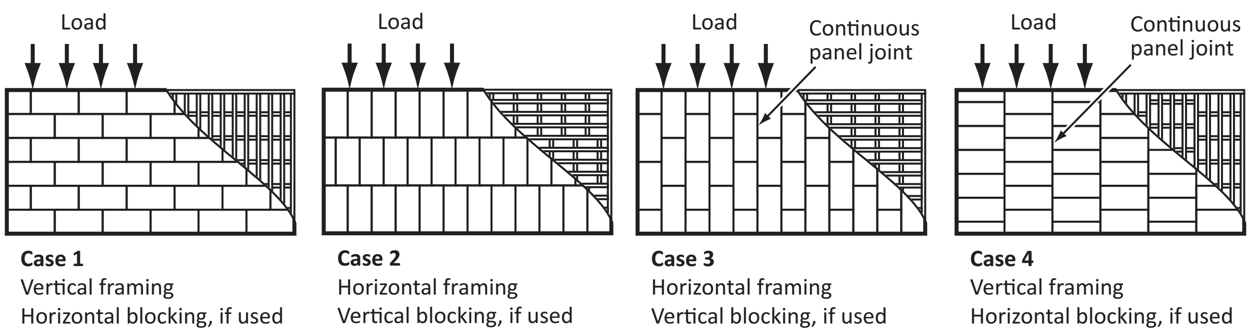



**Figure 11.5.3.3B**
**Diaphragm configurations**

### 11.5.3.4 Sheathing fastening

Perimeter members shall be adequately interconnected at corner intersections and member joints shall be spliced adequately. Fasteners shall be placed not less than 9 mm from the framing edge and panel edge and shall be placed along all the edges of each panel not less than 50 mm and not more than 150 mm on centre. Fasteners along all intermediate framing members shall not be greater than 300 mm on centre. Fasteners shall be firmly driven into framing members but shall not be over-driven into sheathing. For structural wood-based sheathing, fasteners shall not be over-driven more than 15% of the panel thickness.

Where the fastening in a shearwall is a double-shear joint with a panel in the middle and framing members as outside joint members, the framing members and mid-panel shall be connected using fasteners with sufficient penetration 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 11.5.3.4).

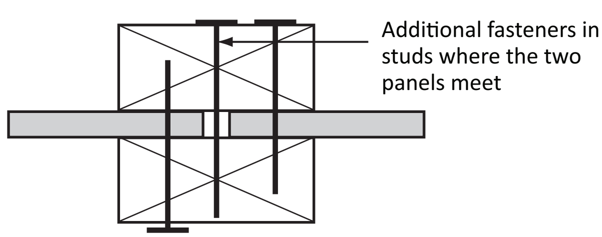



**Figure 11.5.3.4**
**Fastening for mid-panel shearwalls**

For diaphragms with multiple rows of fasteners, the fasteners shall be placed along all the edges of each panel not less than 64 mm on centre and the minimum distance between rows is 9 mm. The fasteners between rows shall be staggered.

**Note:** *See the CWC* Commentary on CSA O86 *for additional detailing for shearwalls with double-shear fastening and diaphragms with multiple rows of fasteners described in this Clause.*

### 11.5.3.5 Additional construction requirements

(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 diameter penetrate more than 41 mm into framing and are spaced 75 mm or less on centre, 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.

For shearwalls, where panels are applied on both faces of a wall and fastener spacing is less than 150 mm on centre on either side, panel joints shall be offset to fall on different framing members or framing shall be 64 mm or thicker and fasteners on each side shall be staggered.

### 11.5.4 Shearwalls using gypsum wallboard

Shearwalls sheathed with gypsum wallboard may be used to resist shear due to lateral forces based on the specified shear strength in Table 11.5.4.

The application of gypsum wallboard shall be restricted to platform frame construction where the height of a storey does not exceed 3.6 m.

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.

Gypsum wallboard application nails or screws shall be placed not less than 9 mm from the panel edge.

**Notes:**

**(1)** *There should exist a balanced spatial distribution of the gypsum wallboard and wood-based panels resisting shear in a given direction in a particular storey.*

**(2)** *See the CWC* Commentary on CSA O86.

**(3)** *See Clause 11.8.8 for seismic design requirements for shearwalls using gypsum wallboard.*

**(4)** *See Clause 11.8.9 for seismic design requirements for load bearing walls constructed with gypsum wallboard only.*

**Table 11.5.4**
**Specified shear strength, $v_d$, for gypsum wallboard shearwalls, kN/m**

| 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 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.*

**(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 11.5.5 Shearwalls and diaphragms using diagonal lumber sheathing

### Δ 11.5.5.1 General

The shearwalls and diaphragms described in Clauses 11.5.5.2 to 11.5.5.4 may be used to resist lateral forces using a single layer of diagonally sheathed lumber (see Clause 11.5.5.2) or a double layer of diagonally sheathed lumber (see Clause 11.5.5.3). There shall be not more than two layers of diagonal lumber sheathing.

### Δ 11.5.5.2 Single-layer diagonal sheathing

Single-layer diagonal sheathing shall be made up of 19 mm boards laid at an angle of approximately 45° to supports. Boards shall be fastened to each intermediate 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 shear panel boundaries.

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 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.

### Δ 11.5.5.3 Double-layer diagonal sheathing

Double-layer diagonal sheathing shall meet the requirements of Clause 11.5.5.2 and consist of two layers of diagonal boards no more than 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.

### 11.5.5.4 Boundary members

**Note:** *See the CWC* Commentary on CSA O86.

#### 11.5.5.4.1

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.

#### 11.5.5.4.2

$M_f$ 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

#### 11.5.5.4.3

$M_f$ 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

## 11.5.6 Moment resistance of shearwalls and diaphragms

### 11.5.6.1 General

Except as provided in Clause 11.5.6.2, the factored moment resistance of shearwalls and diaphragms shall be taken 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-down connections at both ends of the shearwall segment, mm

= length of shearwall segment minus 300 mm for shearwall segments designed without hold-down connections at both ends of the segment, mm

#### 11.5.6.2 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 Detailing

### 11.6.1 General

All boundary members, chords, and struts of shearwalls and diaphragms shall be designed and detailed to transmit the induced axial forces. The boundary members shall be fastened together at all corners.

### 11.6.2 Connections to shearwalls and diaphragms

Connections and anchorages capable of resisting the forces to be transferred shall be provided between the shearwall or diaphragm and the attached components.

## 11.7 Deflection of 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 Clause 11.7.1.2 and 11.7.1.3 respectively.

In the calculation of deflection for multi-storey shearwalls, multi-storey effects shall be considered.

**Note:** *See Clause A.11.7.1 for additional information on multi-storey effects.*

#### 11.7.1.2 Blocked shearwall segments

The static deflection at the top of the wall, $\Delta_{sw}$, mm, of a blocked shearwall segment with wood-based panels constructed in accordance with Clauses 11.5.3 to 11.5.5 may be taken as follows:

$$\Delta_{sw} = \frac{2\upsilon H_s^3}{3EAL_s} + \frac{\upsilon H_s}{B_v} + 0.0025H_s e_n + \frac{H_s}{L_s} d_a$$

where

$\upsilon$ = maximum shear due to specified loads at the top of the wall, N/mm

$H_s$ = height of shearwall segment, mm

$E$ = elastic modulus of boundary element (vertical member at shearwall segment boundary), 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 9.3A, 9.3B, and 9.3C for wood-based panel sheathing. For gypsum wallboard, $B_v$ may be taken as 7000 N/mm)

$e_n$ = nail deformation, mm (Clause A.11.7)

$d_a$ = total vertical elongation of the wall anchorage system (including fastener slip, device elongation, anchor or rod elongation, etc.) at the induced shear load

For shearwall segments without hold-downs, the total vertical elongation, $d_a$, may be taken as follows:

$$d_a = 2.5 d_F K_m \left[ \frac{(\upsilon H_s - P_{ij}) \frac{s_n}{L_s}}{n_u} \right]^{1.7}$$

**Note:** *This equation is an approximation.*

where
$d_F$ = fastener diameter, mm
$K_m$ = service creep factor (Table A.12.9.3.2)
$\upsilon$ = 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
$s_n$ = fastener spacing around panel edge, mm
$L_s$ = length of shearwall segment, mm
$n_u$ = unit lateral strength resistance of sheathing-to-framing connection, N (Clause 12.9.4)

### 11.7.1.3 Unblocked shearwall segments

The deflection of an unblocked shearwall segments with wood-based panels, $\Delta_{ub}$, may be taken as follows:

$$\Delta_{ub} = \frac{\Delta_{sw}}{J_{us}}$$

where
$\Delta_{sw}$ = 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 shearwall segment (Table 11.4.4)

**Note:** *See the CWC* Commentary on CSA O86*.*

### 11.7.2 Deflection of wood diaphragms

The lateral static deflection at mid-span, $\Delta_d$, mm, of a simply supported blocked diaphragm with wood-based panels constructed in accordance with Clauses 11.5.3 to 11.5.5 may be taken as follows:

$$\Delta_d = \frac{5\upsilon L^3}{96EAL_D} + \frac{\upsilon L}{4B_v} + 0.000614Le_n + \frac{\sum(\Delta_c x)}{2L_D}$$

where
$\upsilon$ = maximum shear due to specified loads in the direction under consideration, N/mm
$E$ = elastic modulus of the flange, N/mm$^2$
$A$ = area of the chord cross-section, mm$^2$
$L_D$ = dimension of diaphragm parallel to direction of load, mm
$L$ = dimension of diaphragm perpendicular to direction of load, mm
$B_v$ = shear-through-thickness rigidity of the sheathing, N/mm (Tables 9.3A, 9.3B, and 9.3C)
$e_n$ = nail deformation, mm (Clause A.11.7)
$\Sigma(\Delta_c x)$ = sum of the individual chord-splice slip values, $\Delta_c$, on both sides of the diaphragm, each multiplied by its distance $x$ to the nearest support

**Note:** *See the CWC* Commentary on CSA O86*.*

## Δ 11.8 Seismic design considerations for light-frame shearwalls and diaphragms

### 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 11.8.1, provided failure is governed by sheathing-to-framing connection. The sheathing-to-framing connection shall be designed to fail in mode (d), (e) or (g) shown in Clause 12.9.4.2 for nails to ensure sufficient ductility in the shearwall and diaphragm.

**Table 11.8.1**
**Seismic design factors for shearwalls**

| Construction | $R_d$ | $R_0$ |
|---|---|---|
| Shearwall with nailed structural wood-based panels | 3 | 1.7 |
| Shearwalls: wood-based and gypsum panels in combination | 2 | 1.7 |

Clauses 11.8.2 to 11.8.6 apply to structures that are located in seismic zones where the value of the product $I_E F_a S_a(0.2)$ determined in accordance with the *National Building Code of Canada* is equal to or greater than 0.35. Clause 11.8.7 applies to structures where the value of the product $I_E F_a S_a(0.2)$ is lower than 0.35.

## 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 seismic loads that are at least 20% greater than the force that is being transferred.
**Note:** *See the CWC* Commentary on CSA O86*.*

## 11.8.3 Over-capacity of wood-based seismic force resisting system (SFRS)

### 11.8.3.1 Storey over-capacity coefficient, $C_i$

The over-capacity coefficient, $C_i$, for a given storey, *i*, for each horizontal direction, shall be greater than or equal to 1.0 and shall be taken as follows:

$$C_i = \frac{V_{ri}}{V_{fi}}$$

where

$V_{ri}$ = factored resistance of the wood-based SFRS in a given storey *i*

$V_{fi}$ = factored seismic shear load for storey *i* obtained using $R_d R_o$ coefficients for the wood-based 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:

$$0.9 < \frac{C_2}{C_1} \le 1.2$$

**Note:** *See the CWC* Commentary on CSA O86*.*

## 11.8.4 Wood diaphragms supported on wood shearwalls

### 11.8.4.1 General

Wood diaphragms that are designed and detailed in accordance with Clause 11.5 shall be designed for a force 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 taken 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 storey *i* calculated using $R_d R_o$ for the wood shearwalls

**Note:** *See the CWC* Commentary on CSA O86.

## 11.8.5 Wood diaphragms in buildings with SFRSs other than wood 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 wood shearwalls for supporting vertical SFRSs; and

(b) have wood diaphragms.

Vertical SFRSs shall be designed in accordance with appropriate CSA material standards.

### 11.8.5.2 Seismic design forces for diaphragms

#### 11.8.5.2.1 Diaphragms designed to yield

Diaphragms that are designed and detailed in accordance with Clause 11.5 to exhibit ductile behaviour and will yield before the supporting SFRS shall be designed for seismic loads 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

When diaphragms are designed not to yield before the supporting SFRS, they shall be designed to resist a seismic force, $V_{Di}$, taken as follows:

$V_{Di} = 1.2\, C_i F_i$ (for wood-based SFRS other than shearwalls sheathed with wood-based panels)

$V_{Di} = \gamma_i F_i$ (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

$\gamma_i$ = over-strength coefficient applied at level *i* for the vertical SFRS, determined on the basis of principles of capacity based design in accordance with the applicable CSA material standard

$V_{Di}$ need not exceed the value of load $F_i$ 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 load-transfer elements, shall be designed for seismic loads that are at least 20% greater than the seismic design load 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 loads 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 load of the offset SFRS.

Forces used for the design of force transfer elements need not exceed the forces determined using $R_d R_o = 1.3$.

**Note:** *See the CWC* Commentary on CSA O86.

## 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 that are located in seismic zones where the value of $I_E F_a S_a(0.2)$ is lower than 0.35.

### 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.

## 11.8.8 Seismic design requirements for shearwalls using gypsum wallboard

For seismic design, gypsum wallboard shall be used in combination with wood-based 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 11.8.8.

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 higher than 4 storeys the contribution of the gypsum wallboard shall not be accounted for in seismic resistance.

**Note:** *See the CWC* Commentary on CSA O86.

**Table 11.8.8**
**Maximum percentage of total seismic shear forces resisted by gypsum wallboard in a storey**

| | 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 inter-storey 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 capable of resisting the factored axial load combination for seismic design (see load combination No. 5 in Table 5.2.4.1) 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.

**Note:** *See the CWC* Commentary on CSA O86.

## Δ 11.9 Design of CLT shearwalls and diaphragms

**Note:** *The provisions in this Clause should be used in conjunction with the CWC* Commentary on CSA O86.

### 11.9.1 General

#### 11.9.1.1

Clause 11.9 shall apply to platform-type constructions not exceeding 30 m in height. For high seismic zones (i.e., $I_E F_a S_a(0.2) > 0.75$), the height shall be limited to 20 m. Alternative systems shall be designed in accordance with Clause 4.3.2 of this Standard and *NBC* subsection 4.1.8.

#### 11.9.1.2

The factored shear resistance of CLT shearwalls shall be governed by the resistance of connections between the shearwalls and the foundations or floors, and connections between the individual panels, calculated using methods of mechanics, assuming each individual panel acts as a rigid body.

#### 11.9.1.3

The factored shear resistance of the diaphragms shall be governed by the resistance of the connections between the diaphragms and the supporting structure and the connections between the individual panels, calculated using methods of mechanics, assuming each individual panel acts as a rigid body.

### 11.9.2 Seismic design considerations for CLT structures

#### 11.9.2.1 General

Factors $R_d \leq 2.0$ and $R_o = 1.5$ shall apply to platform-type CLT structures where the energy is dissipated through connections as specified in Clause 11.9.2.2 following the capacity design principles given in Clause 11.9.2.4, and wall panels act in rocking or in combination of rocking and sliding. Type 4 or 5 irregularities as defined in the *NBC* shall not be allowed. Other types of irregularities shall be dealt with in accordance with the *NBC*. CLT structures with wall panels with aspect ratios (height-to-length) less than 1:1 or acting in sliding only shall be designed with $R_d R_o = 1.3$.

**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

#### 11.9.2.2 Energy dissipative connections

Energy dissipative connections of CLT structures shall satisfy all of 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 of the assumed rigid body motions of CLT panels; and

(c) connections shall possess sufficient deformation capacity to allow for the CLT panels to develop their assumed deformation behaviour, such as rocking, sliding, or combination thereof.

**Note:** *For further information on moderately ductile connections, see the CWC* Commentary on CSA O86.

#### 11.9.2.3 Non-dissipative connections

Non-dissipative connections shall be designed to remain elastic under the force and displacement demands that are induced in them when the energy-dissipative connections reach the $95^{th}$ percentile of their ultimate resistance or target displacement, in accordance with engineering principles of equilibrium and displacement compatibility. The seismic design force need not exceed the force determined using $R_d R_o = 1.3$.

**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

#### 11.9.2.4 Capacity design principles

##### 11.9.2.4.1 General

The following capacity design principles shall be used for seismic design of CLT structures. All inelastic deformations and energy dissipation shall occur in

(a) vertical joints between the panels in shearwalls;

(b) shear connections between the shearwalls and the foundations or floors underneath; and
(c) hold-down connections, except for continuous steel rods (Clause 11.9.3).
Items (a), (b) and (c) shall satisfy displacement compatibility based on the assumed rigid body motion.
**Note:** *See the CWC* Commentary on CSA O86 *for further information*

#### 11.9.2.4.2 CLT panels
CLT panels that are part of the lateral-force-resisting system shall be designed for seismic forces that are developed when energy dissipative connections in shearwalls reach the 95th percentile of their ultimate resistance but need not exceed the force determined using $R_d R_o = 1.3$. The in-plane shear resistance of the CLT panels shall be provided by the product manufacturer. Net section effects and openings shall be accounted for in the design.
**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

### 11.9.2.5 CLT Shearwalls

#### 11.9.2.5.1 Aspect ratio
Wall segments shall have an aspect ratio (height-to-length) of not less than 1:1 and not greater than 4:1. Wall segments with an aspect ratio of less than 1:1 shall be divided into sub-segments satisfying this requirement and joined with energy dissipative connections as per Clause 11.9.2.2.
**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

#### 11.9.2.5.2 Resistance to overturning
Where the factored dead loads are not sufficient to prevent overturning, hold-down connections shall be designed to resist the factored uplift forces and transfer the forces through a continuous load path to the foundation. If continuous steel rods are used, they shall be designed to remain elastic at all times and shall not restrict the motion in the direction of the assumed rigid body.
**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

### 11.9.3 Shear-uplift interaction in the connections
If connections of the CLT shearwall panels to the foundation or the floors underneath are designed to resist forces in both shear and uplift direction, the shear-uplift interaction shall be taken into account when determining the resistance of the CLT shearwalls.
**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

### 11.9.4 Deflections
Deflections shall be determined using established methods of mechanics. Calculations shall account for the main sources of shearwall deformations, such as panel sliding, rocking, and deformation of supports. CLT panels may be assumed to act as rigid bodies.
**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

# 12 Connections

## Δ 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 and wood screws are based on the relative density of the wood material. Relative density values are given in Table A.12.1.*

## 12.2 General requirements

### Δ 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 direction 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 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.

#### 12.2.1.3 Conditions conducive to corrosion

Under severe conditions conducive to corrosion, connection design should provide adequate protection.

#### 12.2.1.4 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 D Fir-L species group.

#### 12.2.1.5 Shear resistance 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 6.5.5 and 7.5.7, shall be based on the dimension $d_e$ shown in Figure 12.2.1.5, instead of the dimension $d$. The dimension $d_e$ 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.

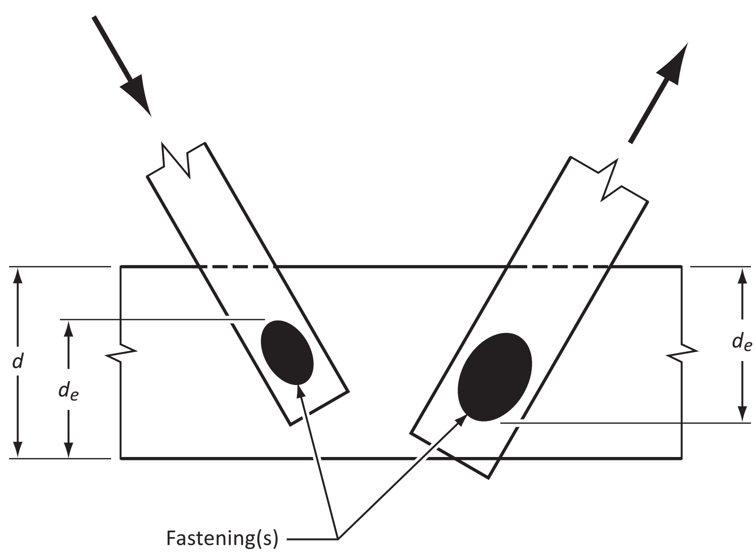



**Figure 12.2.1.5**
**Shear depth**

### 12.2.1.6 Service condition factors
The service condition factors for connections shall be as specified in Table 12.2.1.6.

### 12.2.1.7 Load duration factor, $K_D$
The load duration factor, $K_D$, for connections shall be as specified in Table 5.3.2.2.

### 12.2.1.8 Treatment factor, $K_T$
For connections containing wood-based members treated with fire-retardant or other strength-reducing chemicals, the strength capacities 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. See the CWC* Commentary on CSA O86.

### 12.2.1.9 Connection assembly
Connections shall be assembled so that the surfaces are brought into close contact.

Δ

**Table 12.2.1.6**
**Service condition factor, $K_{SF}$, for connections**

| | Moisture content of wood when connection is fabricated | | | | | |
|---|---|---|---|---|---|---|
| | Dry (≤ 19%) | | Green (> 19%) | | | |
| Service conditions | Dry | Wet | Dry | 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 rings, 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, 12.4.4.5 and 12.4.4.6).*

### 12.2.2 Split ring and shear plate connectors, bolts, and lag screws

**Note:** *See Clauses 12.3, 12.4, and 12.6 for further requirements on bolts, lag screws, split rings, and shear plate connectors.*

#### 12.2.2.1 Inspection and tightening

Structures that have been assembled with unseasoned or partially seasoned lumber or timbers shall be inspected regularly at intervals not exceeding six months until it becomes apparent that further shrinkage of the wood will not be appreciable. At each inspection the fasteners shall be tightened sufficiently to bring the faces of the connected members into close contact without deformation.

#### 12.2.2.2 Assembly

Grooves, daps, and holes shall be fabricated and oriented accurately in the contacting faces. 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 bolt or lag screw diameters.

#### 12.2.2.3 Group of fasteners

##### 12.2.2.3.1

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.

##### Δ 12.2.2.3.2

A row of fasteners consists of one or more bolts, lag screws, or timber connector units of the same type and size aligned with the direction of the load (see Figures 12.4.3.1, 12.4.3.2, and 12.4.3.3).

##### 12.2.2.3.3

When fasteners in adjacent rows are staggered and the distance between adjacent rows is less than one-quarter the distance between the closest fasteners in adjacent rows measured parallel to the rows, the adjacent rows shall be considered as one row for the purpose of determining 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

The modification factor, $J_G$, for groups of timber connectors and lag screws shall be as specified in Tables 12.2.2.3.4A and 12.2.2.3.4B.

#### 12.2.2.4 Washers

##### 12.2.2.4.1

A standard cut washer or its equivalent (see Table 12.2.2.4.1), or a metal strap of the same thickness as the washer, shall be placed between the wood and the head and between the wood and the nut.

##### 12.2.2.4.2

When a bolt head or nut bears directly on a steel plate, washers may be omitted.

##### 12.2.2.4.3

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 be not 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
The resistance of connections made using bolts, lag screws, and split ring and shear plate connectors shall be checked for net section in accordance with Clause 5.3.8.

#### 12.2.2.5.2
For a bolted or lag screw connection under parallel-to-grain loading, staggered adjacent bolts 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 eight bolt or lag screw shank diameters.

#### 12.2.2.5.3
For connections using timber 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 centre-to-centre parallel to grain is more than two connector diameters.

**Table 12.2.2.3.4A**
**Modification factor, $J_G$, for timber connector and lag screw connections with wood side plates**

| Area ratio* | The lesser of $A_m$† or $A_s$‡ | Number of fasteners in a row | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| 0.5 | < 8 000 | 1.00 | 0.92 | 0.84 | 0.76 | 0.68 | 0.61 | 0.55 | 0.49 | 0.43 | 0.38 | 0.34 |
| | 8 001–12 000 | 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 | < 8 000 | 1.00 | 0.97 | 0.92 | 0.85 | 0.78 | 0.71 | 0.65 | 0.59 | 0.54 | 0.49 | 0.44 |
| | 8 001–12 000 | 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,* mm$^2$
‡$A_s$ *= sum of gross cross-sectional areas of side members,* mm$^2$
**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.*

## Table 12.2.2.3.4B
## Modification factor, $J_G$, for timber connector and lag screw connections with steel side plates

| Area ratio* | $A_m$ | Number 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 |

(Continued)

## Table 12.2.2.3.4B (Concluded)

| Area ratio* | $A_m$ | Number of fasteners in a row: 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |
| | 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,* mm$^2$
$A_s$ = *sum of gross cross-sectional area of steel side plates,* mm$^2$

## Table 12.2.2.4.1
## Minimum washer sizes for bolted, lag screw, and timber connector connections

| | | Bolted or lag screw connections | | | Timber connector connections | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 2-1/2 in split ring with 1/2 in bolt | | 4 in split ring with3/4 in bolt | | 2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt* | |
| Washer type | Use | Rod or bolt diameter, $d_F$, in | Outside dimension, $d_o$, mm | Thickness, $t$, mm | $t$, mm | $d_o$, mm | $t$, mm | $d_o$, mm | $t$, mm | $d_o$, mm |
| Standard cut (steel) | For bolts and lag screws only; too thin to resist any tensile loads | 1/2<br>5/8<br>3/4<br>7/8<br>1 | 35<br>45<br>50<br>60<br>65 | 3<br>4<br>4<br>4<br>4 | Cut washers shall not be used with connectors | | | | | |
| Square plate (steel) | For connector or tensile load | 1/2<br>5/8<br>3/4<br>7/8<br>1 | 65<br>70<br>75<br>85<br>90 | 5<br>6<br>6<br>9<br>8 | 3.2 | 50 | 4.8 | 75 | 6.4 | 75 |
| Round plate (steel) | For any use, unless tensile loading develops enough stress to crush wood | 1/2<br>3/4<br>7/8 | 65<br>75<br>85 | 5<br>6<br>8 | 3.2 | 50 | 4.8 | 75 | 6.4 | 75 |
| Ogee (cast iron) | Thicker and wider than normal or malleable iron washers (for increased stiffness and bearing strength) | 1/2<br>5/8<br>3/4<br>7/8<br>1 | 65<br>75<br>90<br>100<br>100 | 13<br>16<br>19<br>22<br>25 | 3.2 | 55 | 4.8 | 75 | 6.4 | 75 |

*(Continued)*

## Table 12.2.2.4.1 (Concluded)

| | | Bolted or lag screw connections | | | Timber connector connections | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 2-1/2 in split ring with 1/2 in bolt | | 4 in split ring with 3/4 in bolt | | 2-5/8 in shear plate and 4 in shear plate with 3/4 in bolt* | |
| **Washer type** | **Use** | **Rod or bolt diameter, $d_F$, in** | **Outside dimension, $d_o$, mm** | **Thickness, $t$, mm** | **$t$, mm** | **$d_o$, mm** | **$t$, mm** | **$d_o$, mm** | **$t$, mm** | **$d_o$, mm** |
| Malleable iron | Wider than normal washers (for added bearing strength) | 1/2 | 65 | 6 | 3.2 | 55 | 4.8 | 75 | 6.4 | 75 |
| | | 5/8 | 70 | 8 | | | | | | |
| | | 3/4 | 75 | 11 | | | | | | |
| | | 7/8 | 90 | 11 | | | | | | |
| | | 1 | 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.*

## 12.3 Split ring and shear plate connectors

### 12.3.1 General

#### 12.3.1.1 Connector unit

For the purpose of specifying connector 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 connector with its bolt or lag screw;
(b) one shear plate connector with its bolt or lag screw, used with a steel strap or plate in a wood-to-metal connection; or
(c) two shear plate connectors used back-to-back in the contact faces of a wood-to-wood connection with their bolt or 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 connectors that have dimensions in accordance with Table 12.3.1A 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 that have dimensions in accordance with Table 12.3.1A 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.

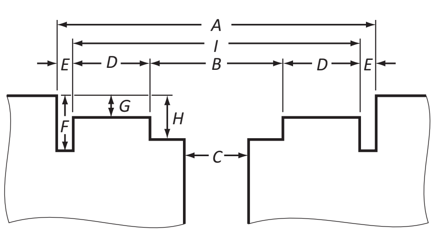
## Table 12.3.1A
## Timber connector dimensions, mm

| **Split ring** | **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 | |
| | **2-5/8 in** | **4 in** | |
| **Shear plate** | | **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 Table 12.3.1B.

## Table 12.3.1B
## Timber connector groove dimensions, mm

| **Split ring groove** | **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 | |
| | **2-5/8 in shear plate** | **4 in shear plate** | |
| **Shear plate groove** | | **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 |
| | *H* = not applicable | 12.7 | 12.7 |
| | *I* = 57.2 | 88.6 | 88.6 |



Δ

### 12.3.2 Service condition factors

The service condition factors specified in Table 12.2.1.6 are based on the moisture content within a depth of 20 mm from the connected surface.

### 12.3.3 Distance factors

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 12.3.3A to 12.3.3C shall not be considered to provide resistance. Factors for reduction of resistance 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 12.3.3A and 12.3.3B).

**Table 12.3.3A**
**Values of $J_C$ for timber connector edge distance**

| Edge distance, mm | 2-1/2 in split ring or 2-5/8 in shear plate | | | 4 in split ring or shear plate | | |
|---|---|---|---|---|---|---|
| | $\theta$ = 15° | $\theta$ = 30° | $\theta$ = 45° to 90° | $\theta$ = 15° | $\theta$ = 30° | $\theta$ = 45° to 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 |
| 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 $\theta$ = 0°, the minimum edge distance for the particular connector size gives a value of $J_C$ = 1.00. For intermediate values of $\theta$, 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 12.3.3B
# Values of $J_C$ for timber connector end distance

| 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 | $\theta = 0°$ to $90°$ | $\theta = 0°$ to $90°$ |
| 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 |
| 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 $\theta$ 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 12.3.3C**
**Timber connector spacing, mm, for values of $J_C$ between 0.75 and 1.0**

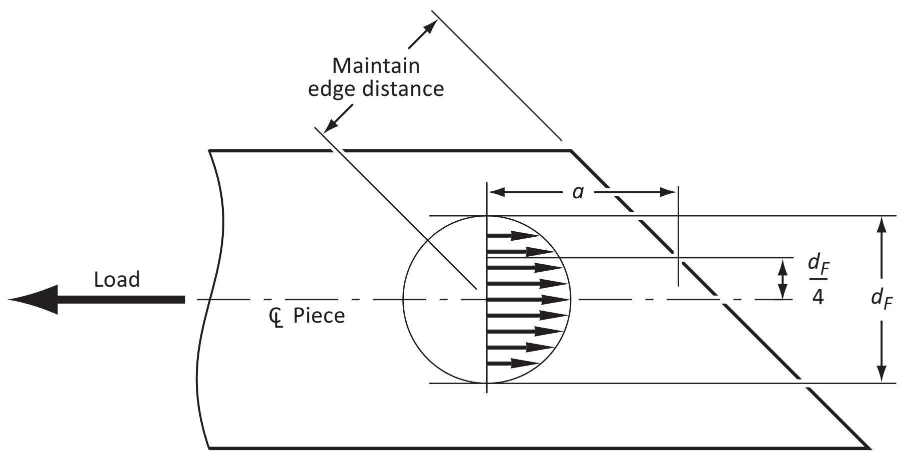
| Angle of load to grain, $\theta$° | Angle of connector row to grain, $\beta$° | Minimum spacing between connectors measured centre-to-centre, mm | | | |
|---|---|---|---|---|---|
| | | 2-1/2 in split rings and 2-5/8 in shear plates | | 4 in split rings and 4 in shear plates | |
| | | $J_C$ = 0.75 | $J_C$ = 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 |
| | 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 |

*(Continued)*

## Table 12.3.3C (Concluded)

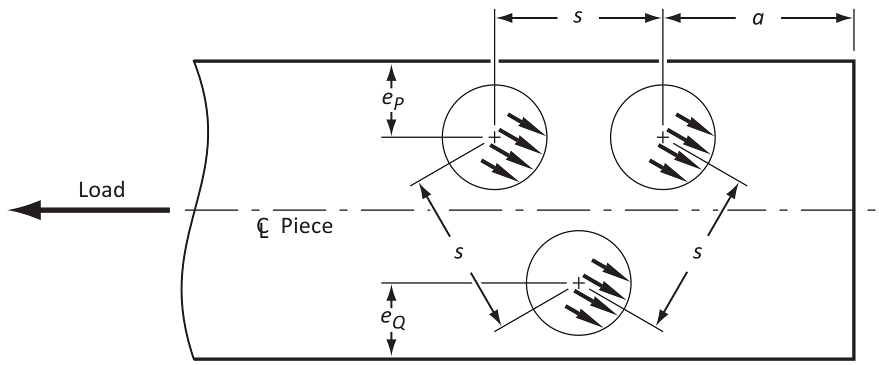
| Angle of load to grain, $\theta°$ | Angle of connector row to grain, $\beta°$ | Minimum spacing between connectors measured centre-to-centre, mm | | | |
|---|---|---|---|---|---|
| | | 2-1/2 in split rings and 2-5/8 in shear plates | | 4 in split rings and 4 in shear plates | |
| | | $J_C = 0.75$ | $J_C = 1.00$ | $J_C = 0.75$ | $J_C = 1.00$ |
| 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.*




**Legend:**
$a$ = end distance
$d_F$ = connector diameter

**Figure 12.3.3A**
**End distance for member with sloping end cut**




**Legend:**
$a$ = end distance
$e_p$ = unloaded edge distance
$e_Q$ = loaded edge distance
$s$ = spacing

**Figure 12.3.3B**
**End distance, edge distance, and spacing**

## 12.3.4 Lumber thickness

Connectors installed in lumber of a thickness less than the minimum specified in Table 12.3.4 for the connector type and use shall not be considered to provide resistance.

**Table 12.3.4**
**Thickness factor for timber connector, $J_T$**

| Connector type and size | Number 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 |

*(Continued)*

**Table 12.3.4 (Concluded)**

| Connector type and size | Number of faces of a piece containing connectors on a bolt | Thickness of piece, mm | $J_T$ |
|---|---|---|---|
| 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 Connections using lag screws with connectors

When lag screws instead of bolts are used with connectors, the resistance shall vary uniformly with penetration into the member receiving the point, 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 12.3.5 and shall be not less than the minimum value.

**Table 12.3.5**
**Penetration factor, $J_P$, for split rings and shear plates used with lag screws**

| | | Penetration of lag screw into member receiving point (number of shank diameters) | | | | |
|---|---|---|---|---|---|---|
| | | Species | | | | |
| 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<br>Minimum | 5<br>3.5 | 7<br>4 | 7<br>4 | 8<br>8 | 1.00<br>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 12.3.6C.

(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 $\theta$ to grain:

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

where

$\phi$ = 0.6

$P_u = p_u(K_D K_{SF} K_T)$

where

$p_u$ = lateral strength resistance parallel to grain, kN (Table 12.3.6A)

$J_F = J_G J_C J_T J_O J_P$

where

$J_G$ = factor for a group of fasteners (Tables 12.2.2.3.4A and 12.2.2.3.4B)

$J_C$ = minimum configuration factor (Clause 12.3.3 and Tables 12.3.3A to 12.3.3C)

$J_T$ = thickness factor (Table 12.3.4)

$J_O$ = factor for connector orientation in grain

= 1.00 for side grain installation

= 0.67 for end grain and all other installations

$J_P$ = factor for lag screw penetration (Clause 12.3.5 and Table 12.3.5)

$Q_u = q_u(K_D K_{SF} K_T)$

where

$q_u$ = lateral strength resistance perpendicular to grain, kN (Table 12.3.6B)

**Table 12.3.6A**
**Lateral strength resistance parallel to grain, $p_u$, of timber connector unit, kN**

| | Split rings | | Shear plates | |
|---|---|---|---|---|
| **Species** | **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.*

Δ

**Table 12.3.6B**
**Lateral strength resistance perpendicular to grain, $q_u$, of timber connector unit, kN**

| | Split rings | | Shear plates | |
|---|---|---|---|---|
| Species | 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 |

**Table 12.3.6C**
**Maximum factored strength resistance per shear plate unit, kN**

| | | 4 in shear plate | |
|---|---|---|---|
| Type of load | 2-5/8 in shear plate | 3/4 in bolt | 7/8 in bolt |
| Washers provided — no bearing on 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

#### 12.4.1.1 Resistance

In Clause 12.4, bolt or dowel connection resistance is based on design to resist possible yielding and brittle failure modes.

#### 12.4.1.2 Holes

Bolt holes in wood shall be accurately aligned and drilled not less than 1.0 mm and not more than 2.0 mm larger than the bolt diameter. Prebored holes in dowel connections shall have a diameter not greater than the dowel diameter.

#### 12.4.1.3 Multiple members

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.

### 12.4.2 Material

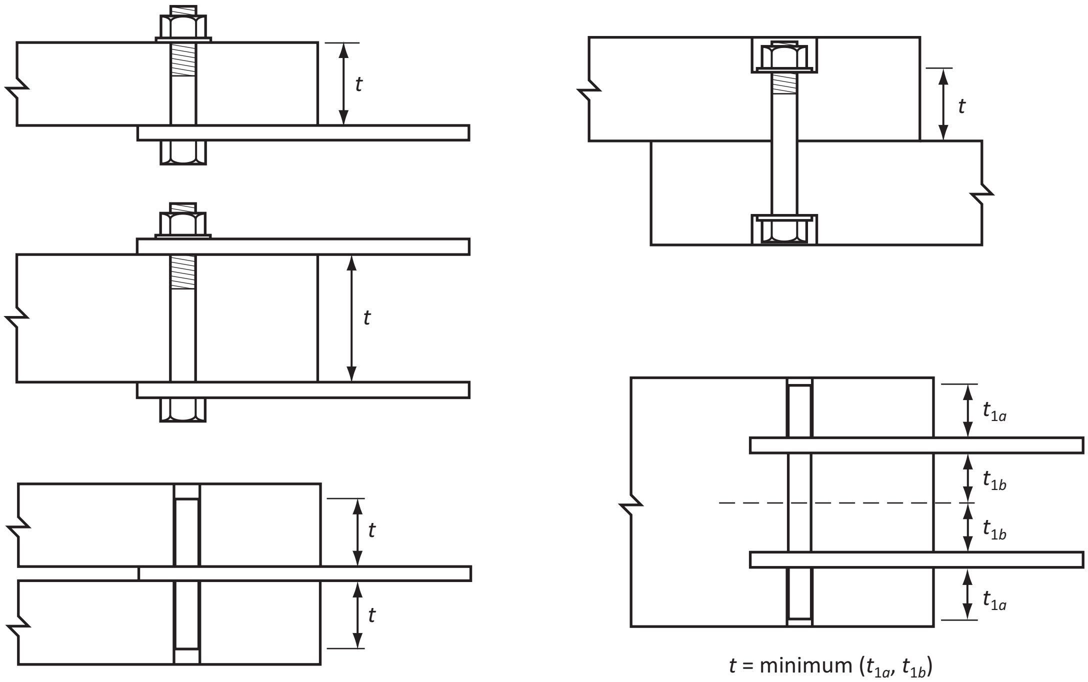
#### 12.4.2.1 Fastener material properties

The design requirements and data specified in Clause 12.4 apply to connections that use a metallic bolt or dowel whose material properties are referenced by a Canadian material design standard.

### 12.4.2.2 Wood

The resistance of wood members in a connection shall be determined in accordance with Clause 12.4.4. In a connection with two wood members, the wood thickness of the side members shall be the lesser of the thinnest piece or the smallest bearing length shown in Figure 12.4.2.2.

**Note:** *To avoid eccentricity in loading, side pieces should be of the same thickness.*




Δ **Figure 12.4.2.2**
**Thickness of wood member**

### 12.4.2.3 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 may be used.

### 12.4.2.4 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 of sufficient strength to resist the applied loads.

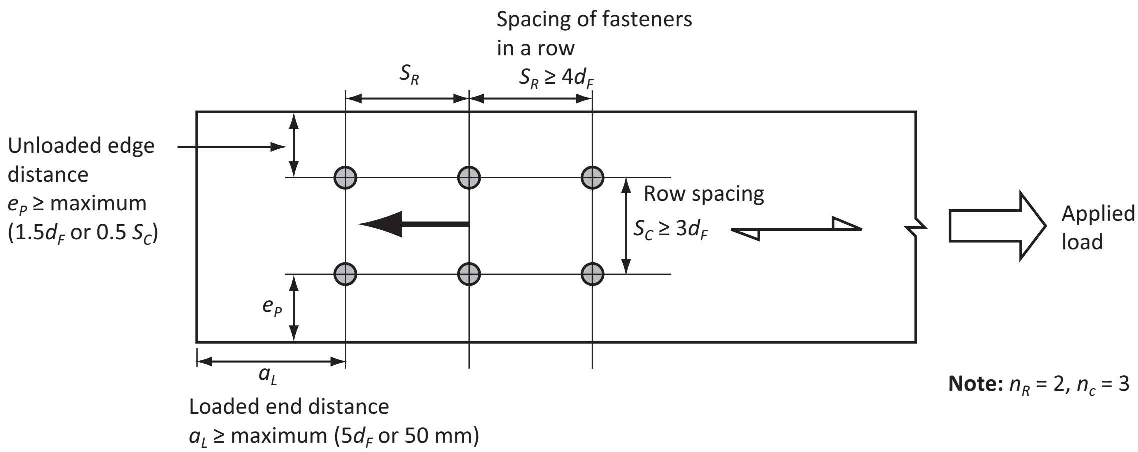
## 12.4.3 Placement of fasteners in connections

### Δ 12.4.3.1 Load component parallel to grain

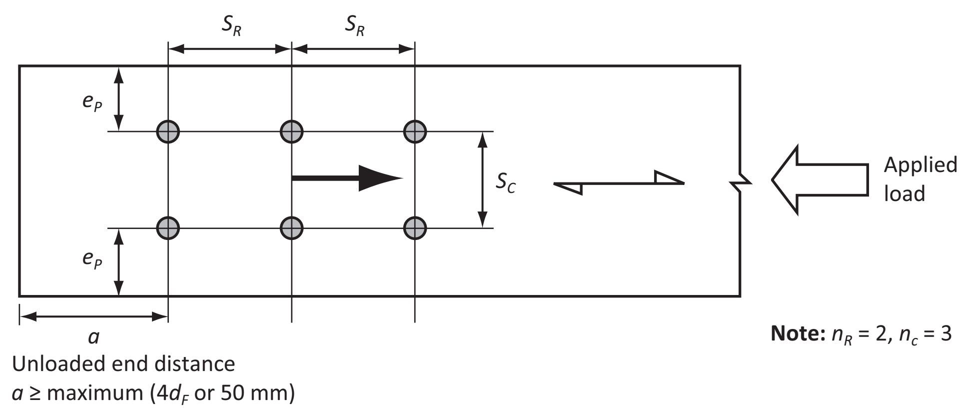
In a group of fasteners where the load component is acting parallel with the direction of the grain, including those in the panel face of CLT, the minimum spacings measured from centres of fasteners shall be as follows (see Figure 12.4.3.1):

(a) spacing of fasteners in a row, $S_R$: four bolt or dowel diameters;
(b) row spacing, $S_C$: three bolt or dowel diameters;
(c) loaded end distance, $a_L$: five bolt or dowel diameters or 50 mm, whichever is greater;

(d) unloaded end distance, $a$: four bolt or dowel diameters or 50 mm, whichever is greater; and
(e) unloaded edge distance, $e_P$: 1.5 bolt or dowel diameters or half the spacing perpendicular to grain, whichever is greater.




**a) Member in tension**



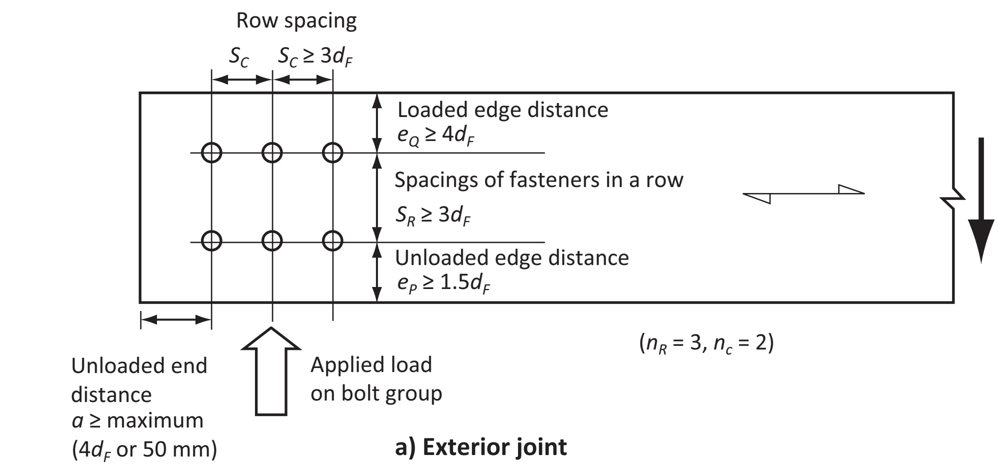

**b) Member in compression**

**Legend:**
$S_R$ = spacing of fasteners in row
$S_C$ = row spacing
$a$ = unloaded end distance
$a_L$ = loaded edge distance
$e_p$ = unloaded edge distance

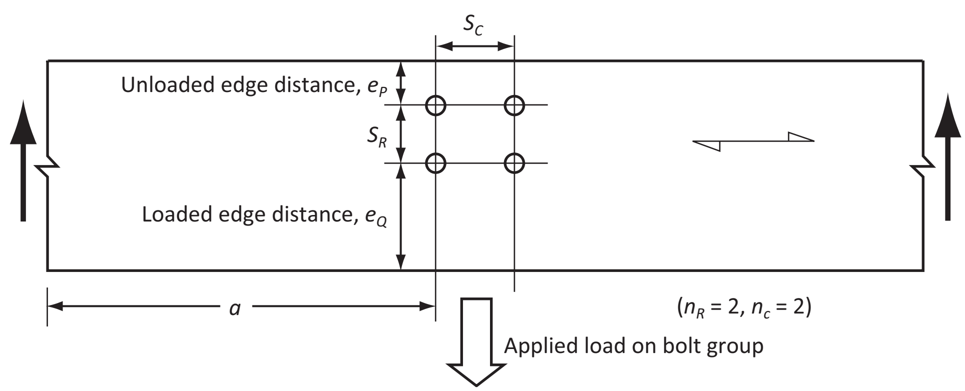
Δ **Figure 12.4.3.1**
**Placement of bolts and dowels in a connection loaded parallel to grain**

## Δ 12.4.3.2 Load component perpendicular to grain

In a group of fasteners where the load component is acting perpendicular to the direction of the grain, including those in the panel face of CLT, the minimum spacings measured from centres of fasteners shall be as follows (see Figure 12.4.3.2):
(a) spacing of fasteners in row, $S_R$: three bolt or dowel diameters;
(b) row spacing, $S_C$: three bolt or dowel diameters;
(c) unloaded end distance, $a$: four bolt or dowel diameters or 50 mm, whichever is greater;
(d) loaded edge distance, $e_Q$: four bolt or dowel diameters; and
(e) unloaded edge distance, $e_P$: 1.5 bolt or dowel diameters.




**a) Exterior joint**




**b) Interior joint**

**Legend:**
$S_R$ = spacing of fasteners in row
$S_C$ = row spacing
$a$ = unloaded end distance
$e_Q$ = loaded edge distance
$e_p$ = unloaded edge distance

Δ **Figure 12.4.3.2**
**Placement of bolts and dowels in a connection loaded perpendicular to grain**

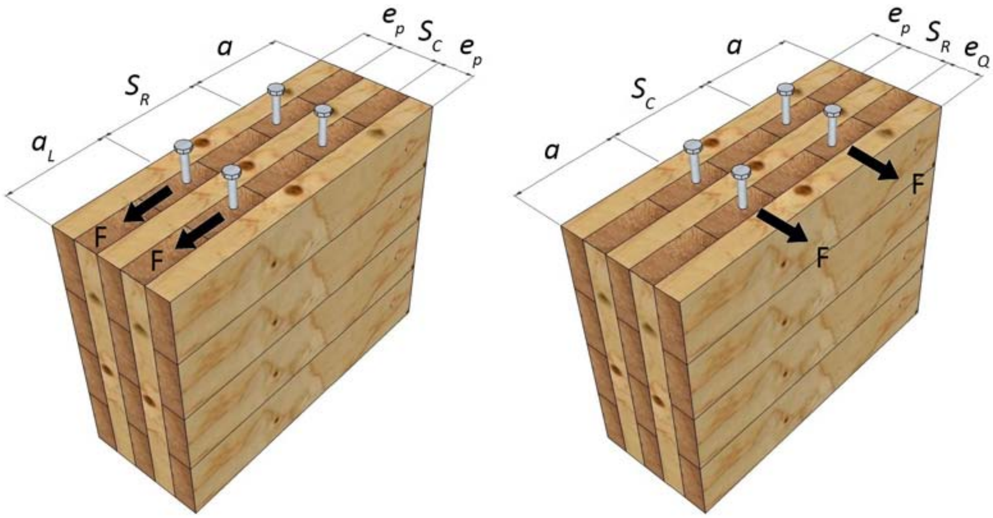
## Δ 12.4.3.3 Maximum distance perpendicular to grain

Except for connections in CLT, a single steel splice plate shall not be used for rows of bolts when the distance between the two outer rows exceeds 125 mm.

## Δ 12.4.3.4 Placement of fasteners in panel edge of CLT

In a group of fasteners installed in panel edge of CLT, the minimum spacings measured from centres of fasteners shall be as follows (see Figure 12.4.3.3):
(a) spacing of fasteners in row, $S_R$: four bolt or dowel diameters;
(b) row spacing, $S_C$: three bolt or dowel diameters;
(c) loaded end distance, $a_L$: five bolt or dowel diameters or 50 mm, whichever is greater;
(d) unloaded end distance, $a$: four bolt or dowel diameters or 50 mm, whichever is greater;

(e) loaded edge distance, $e_Q$: five bolt or dowel diameters; and
(f) unloaded edge distance, $e_P$: 1.5 bolt or dowel diameters or half the row spacing, whichever is greater.

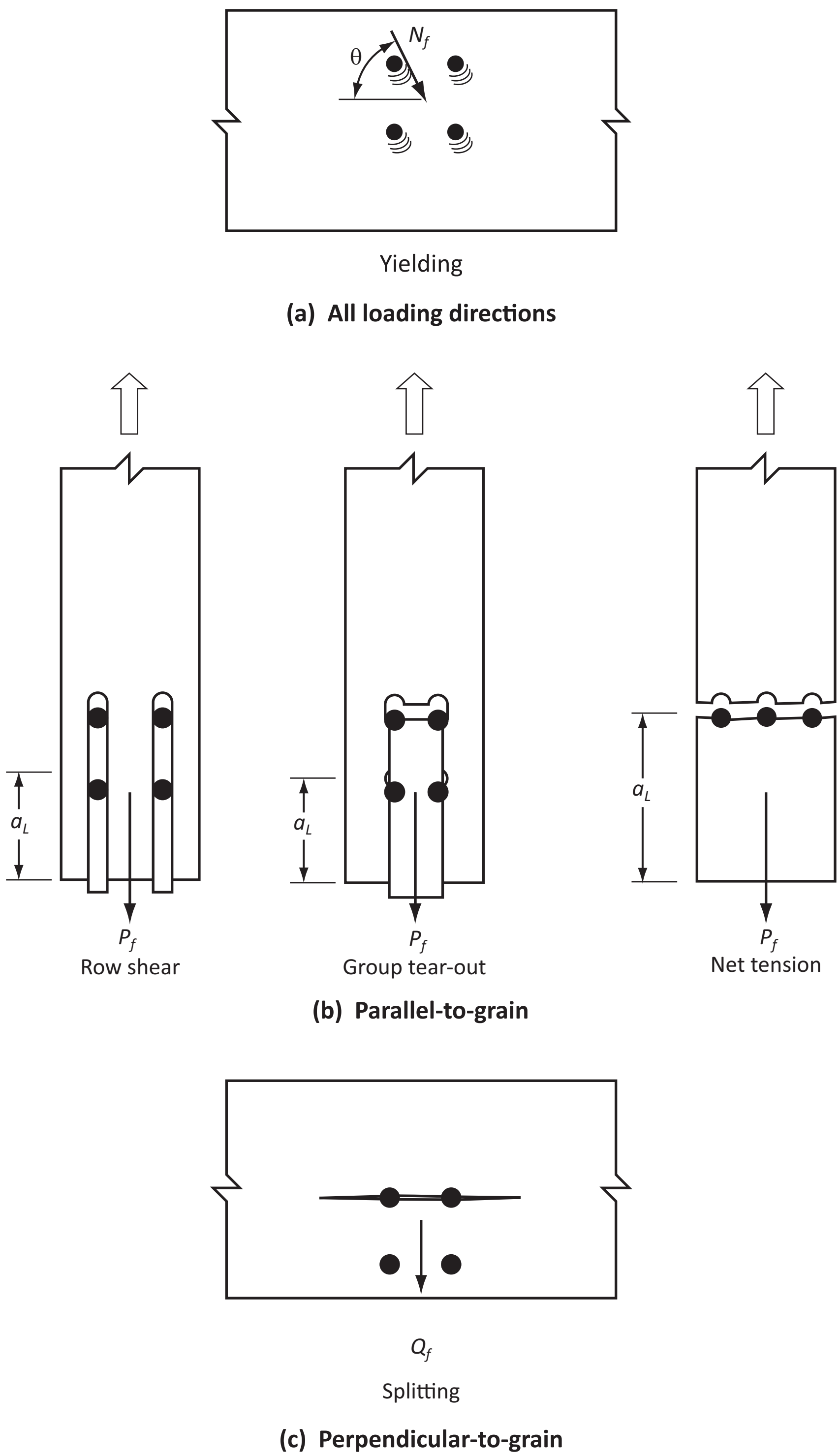



Δ **Figure 12.4.3.3**
**Placement of bolts and dowels in panel edge of CLT**

## 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 12.4.4.1.




**Note:** *A yielding failure can occur if the force is applied parallel to grain, perpendicular to grain, or at an angle to grain.*

Δ

**Figure 12.4.4.1**
**Potential failure modes**

### 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 (Clause 12.4.4.3)

(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 (Clause 12.4.4.4)

$PG_{rT}$ = factored group tear-out resistance (Clause 12.4.4.5)

$TN_{rT}$ = factored net tension resistance (Clause 12.4.4.6)

(c) $Q_f \leq Q_r$

where

$Q_f$ = factored load perpendicular to grain

$Q_r$ = $QS_{rT}$

where

$QS_{rT}$ = factored splitting resistance (Clause 12.4.4.7)

(d) for loading at an angle to grain, $\theta$:

$$N_f \leq \frac{P_r\, Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

$\theta$ = angle between the applied load and the grain

### 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 due to yielding is a function of the number of shear planes in the joint and shall be greater than $N_f$.

$$N_r = \phi_y\, n_u\, n_s\, n_F$$

where

$\phi_y$ = resistance factor for yielding failures

= 0.8

$n_u$ = unit lateral yielding resistance, N (Clause 12.4.4.3.2)

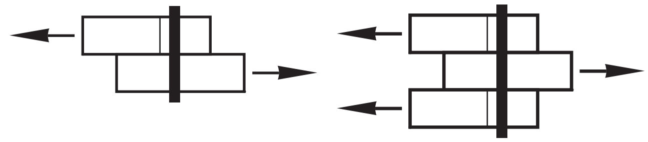
$n_s$ = number of shear planes in the connection

$n_F$ = number of fasteners in the connection

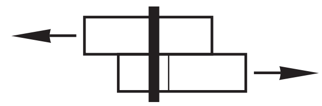
#### 12.4.4.3.2 Unit lateral yielding resistance

The unit lateral yielding resistance, $n_u$ (per shear plane), 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.

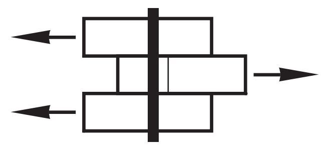
(a) $f_1 \, d_F \, t_1$



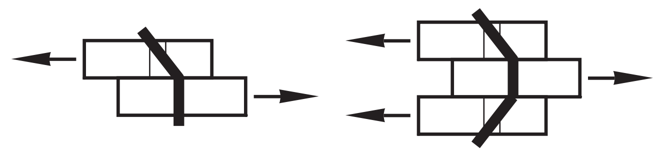
(b) $f_2 \, d_F \, t_2$



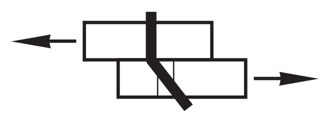
(c) $\frac{1}{2} f_2 \, d_F \, t_2$



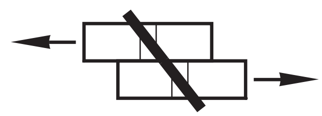
(d) $f_1 d_F^{\,2} \left( \sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$



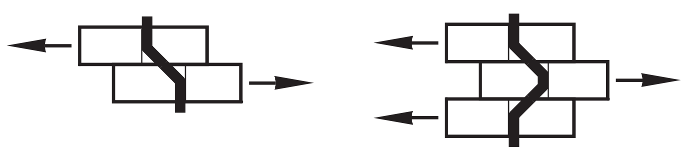
(e) $f_1 d_F^{\,2} \left( \sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$



(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

$f_1, f_2$ = 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

$d_F$ = diameter of fastener, mm

$t_1, t_2$ = member thickness or dowel-bearing length in accordance with Clause 12.4.2.2, mm

$f_y$ = yield strength of fastener in bending, in accordance with Clause 12.4.4.3.3.3, MPa

## Δ 12.4.4.3.3 Embedment strength

### 12.4.4.3.3.1 Wood-based materials

For fasteners installed in a wood-based material at any angle relative to the grain and for fasteners installed in the panel face of CLT, the embedment strength $f_{i\theta}$, in MPa, shall be calculated as follows:

$$f_{i\theta} = \frac{f_{ip} \; f_{iQ}}{f_{ip} \; \sin^2\theta + f_{iQ} \; \cos^2\theta} \left( K_D K_{SF} K_T \right)$$

where

$f_{i\theta}$ = embedment strength of member *i* for a fastener bearing at angle $\theta$ 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 (Table A.12.1)

$J_X$ = adjustment factor for connections in CLT

= 0.9 for CLT

= 1.0 in all other cases

$f_{iQ}$ = embedment strength for a fastener bearing perpendicular to grain ($\theta$ = 90°)

= 22 $G$ (1– 0.01$d_F$)

$\theta$ = 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 12.2.1.6

$K_T$ = treatment factor in accordance with Clause 12.2.1.8

For fasteners installed in panel edge of CLT, the embedment strength in any loading direction, $f_{i\theta}$, shall be not 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-based materials

The embedment strength of non-wood-based materials, in MPa, shall be taken as follows:

(a) for steel:

$K_{sp}\ (\phi_{steel}/\phi_y)\ f_u$

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., for*
*(a) ASTM A36/A36M steel, $f_u$ = 400 MPa;*
*(b) CSA G40.21 steel, Grades 300W and 350W, $f_u$ = 450 MPa; and*
*(c) cold-formed light gauge steel, Grade SS 230, $f_u$ = 310 MPa.*

$\phi_{steel}$ = 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

$\phi_y$ = resistance factor for yielding failures in wood members in connections with bolts and dowels

= 0.8

(b) for concrete or masonry: 125

### 12.4.4.3.3.3 Dowel or bolt yield strength

The yield strength of a dowel or bolt, $f_y$, in MPa, shall be taken as follows:

(a) ASTM A307, SAE J429 Grade 2 bolts and dowels: 310; or

(b) other CSA- or ASTM-compliant bolts and dowels:

$$\frac{f_{ym} + f_{um}}{2}$$

where

tensile $f_{ym}$ (yield strength of bolts and dowels) and $f_{um}$ (ultimate strength of bolts and dowels) are obtained from applicable material standards

### Δ 12.4.4.4 Parallel-to-grain row shear resistance

The total factored shear resistance of a connection shall be calculated as the sum of the factored row shear resistance of the wood members resisting the load, as follows:

$PR_{rT} = \Sigma\ (PR_{ri})$

The total factored row shear resistance of fasteners in a wood member *i* shall be calculated as follows:

$PR_{r\,i} = \phi_w \, PR_{ij\,min} \, n_R$

where

$\phi_w$ = 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 strength in shear for member *i*, MPa

= specified longitudinal shear values in Tables 6.3.1A, 6.3.1C, 6.3.1D, and 7.3, or 0.6 × the specified longitudinal shear values in Table 6.3.1B

$K_{ls}$ = factor for member loaded surfaces (see Figure 12.4.4.4)

= 0.65 for side member

= 1 for internal member

$t$ = member thickness, mm

$n_c$ = number of fasteners in row *j* of member *i*

$a_{cr\,i}$ = minimum of $a_L$ and $S_R$ for row *j* of member *i*, mm (see Figure 12.4.3.1)

$K_{Sv}$ = service condition factor for longitudinal shear (Tables 6.4.2 and 7.4.2)

The row shear resistance of each member shall be checked separately. For CLT, the row shear failure need not be considered.

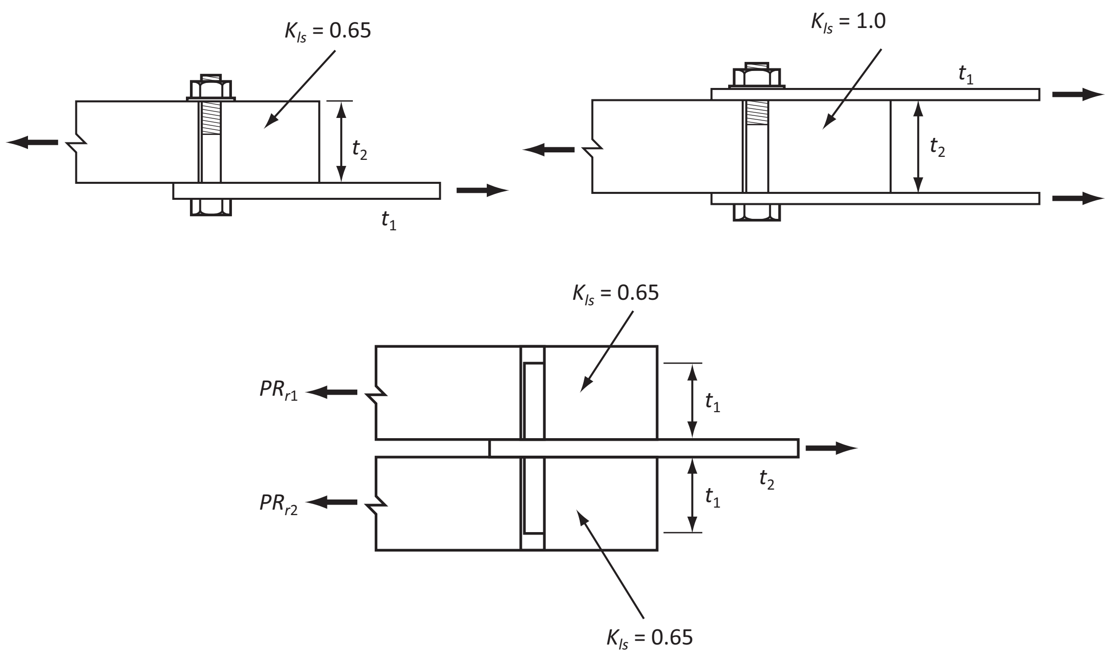



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**Figure 12.4.4.4**
**Factor for member loaded surfaces, $K_{ls}$**

### Δ 12.4.4.5 Parallel-to-grain group tear-out resistance

The total factored group tear-out resistance 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 total factored group tear-out resistance of fasteners in a wood member $i$ with $n_R$ rows shall be calculated as follows:

$$PG_{ri} = \phi_w\,[(PR_{i1} + PR_{inR})/2 + f_t\,(K_D\,K_{St}\,K_T)\,A_{PGi}]$$

where

$\phi_w$ = resistance factor for brittle failures
= 0.7

$PR_{i1}$ = shear resistance along row 1 of member $i$ bounding the fastener group, N
= $1.2\,f_v\,(K_D\,K_{Sv}\,K_T)\,K_{ls}\,t\,n_c\,a_{cr\,1}$

$PR_{inR}$ = shear resistance along row $n_R$ of member $i$ bounding the fastener group, N
= $1.2\,f_v\,(K_D\,K_{Sv}\,K_T)\,K_{ls}\,t\,n_c\,a_{cr\,nR}$

$f_t$ = specified strength in tension of member $i$, MPa
= specified tension parallel to grain values in Tables 6.3.1A, 6.3.1C, 6.3.1D, specified net tension parallel to grain values in Table 7.3, or 0.65 × the specified tension parallel to grain values in Table 6.3.1B

$K_{St}$ = service condition factor for tension parallel to grain (Tables 6.4.2 and 7.4.2)

$K_{Sv}$ = service condition factor for longitudinal shear (Tables 6.4.2 and 7.4.2)

$A_{PGi}$ = critical perpendicular net area between rows 1 and $n_R$ of member $i$, mm$^2$

The group tear-out resistance of each member in tension shall be checked separately. For CLT, group tear-out failure need not be considered. Group tear-out failure need not be considered if the load is applied in such a way that the member is in compression (see Figure 12.4.4.1).

### 12.4.4.6 Net tension resistance

#### 12.4.4.6.1 General

The total factored net tension resistance of the wood members loaded parallel to grain at a group of fasteners shall be calculated as follows:

$$T_{NrT} = \Sigma T_{Nri}$$

$T_{Nri}$ of member $i$ at a group of fasteners shall be determined using Clause 6.5.9 for sawn lumber and Clause 7.5.11 for glulam.

#### 12.4.4.6.2 Area deducted

The cross-sectional area deducted from the gross cross-section shall not be greater than 25% of the member gross area.

### Δ 12.4.4.7 Perpendicular-to-grain splitting resistance

The total factored splitting resistance of a connection shall be calculated as the sum of the splitting resistance of the wood members resisting the load and shall be the following:

$$QS_{rT} = \Sigma QS_{ri}$$

The factored perpendicular-to-grain splitting resistance of wood member $i$, N, shall be calculated as follows:

$$QS_{ri} = \phi_w QS_i\,(K_D\,K_{SF}\,K_T)$$

where
$\phi_w$ = resistance factor for brittle failures
= 0.7

$$QS_i = 14\ t\sqrt{\frac{d_e}{1-\frac{d_e}{d}}}$$

where
$t$ = thickness of member, mm
$d_e$ = effective depth of member, mm
= $d - e_p$
where
$d$ = depth of member, mm
$e_p$ = unloaded edge distance, mm

The splitting resistance need not 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.

**Notes:**
**(1)** *See Clause 12.2.1.5 for member shear resistance based on effective depth.*
**(2)** *See CWC* Commentary on CSA O86 *for additional information on calculation of splitting resistance in CLT.*

### 12.4.5 Axial resistance
Bolts shall be of adequate size to resist the load component parallel to their axis. Washers or plates of adequate thickness and size to resist the factored load component parallel to the axis of the bolt shall be installed. Dowels shall not be designed to resist axial loads.

### 12.4.6 Combined lateral and axial resistance
The resistance of a bolt 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 for drift pin joints in Clause 12.5 are based on round mild steel bolt stock that has a diameter of 16 to 25 mm and meets the requirements of CSA G40.21 or ASTM A307.

### 12.5.2 Prebored holes
Holes in timber shall be prebored 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 prebored holes with minimum damage to the wood.

### 12.5.4 Drift pin length

#### 12.5.4.1
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 12.5.5 shows a typical drift pin connection in timber cribwork.

### 12.5.4.2
Drift pin connections shall be used only where gravity or mechanical restraint prevents axial tension stress in the drift pins.

## 12.5.5 Size and placement of drift pins in connections

### 12.5.5.1
Pin diameter shall not be greater than one-tenth of the width of the timbers to be connected.

### 12.5.5.2
End and edge distances shall be at least 2-1/2 times the pin diameter.

### 12.5.5.3
Spacing between pins in a row and between rows of pins shall be at least four times the pin diameter.

## 12.5.6 Lateral resistance
The factored lateral strength of a drift pin connection, $P_r$, $Q_r$, or $N_r$, shall be greater than or equal to the effect of the factored loads, 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 $\theta$ 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 (Clause 12.4.4.4)

$PG_{r\,T}$ = factored group tear-out resistance (Clause 12.4.4.5)

$T_{Nr\,T}$ = factored net tension resistance (Clause 12.4.4.6)

$QS_{r\,T}$ = factored splitting resistance (Clause 12.4.4.7)

At any shear plane between two overlapping timbers, as shown in Figure 12.5.5, only two pins shall be counted as resisting the shear force.

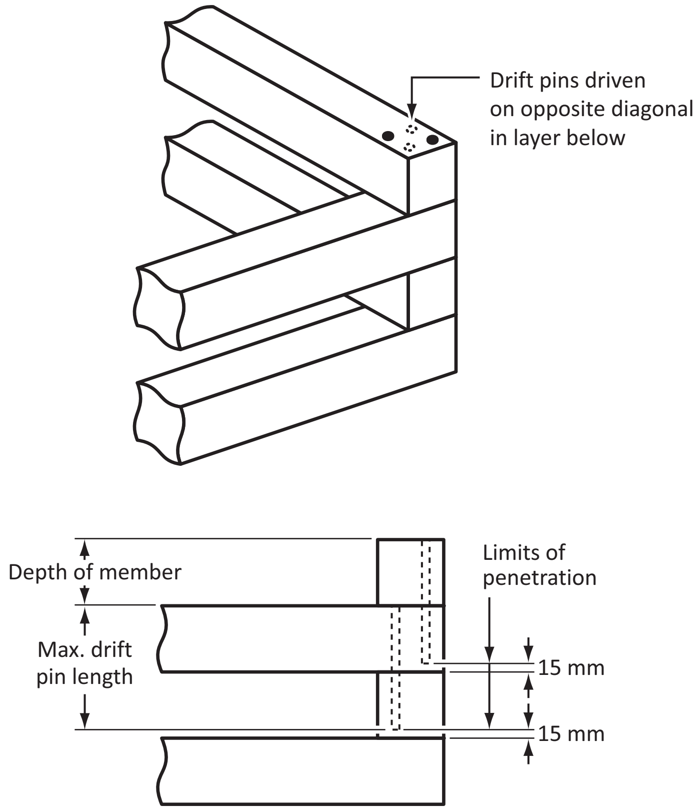



**Figure 12.5.5**
**Placement of drift pins**

## 12.6 Lag screws

### 12.6.1 General

#### 12.6.1.1

The design requirements specified in Clause 12.6 for lag screw connections are based on the use of lag screws of material that meets the requirements of ASME B18.2.1 and uses steel that meets or exceeds the properties of 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

For the purpose of specifying the resistances of lag screw connections, the values calculated in this Standard shall apply to lag screws loaded in withdrawal or laterally in a two-member connection.

### 12.6.2 Placement of lag screws in connections

#### 12.6.2.1 Lag screw holes

Lag screw holes in wood members shall meet the following requirements:

(a) The lead hole for the shank shall have the same diameter as the shank and the same depth as the penetration length of the unthreaded shank.

(b) The lead hole for the threaded portion shall have a diameter equal to 65 to 85% of the shank diameter for dense hardwoods, 60 to 75% of the shank diameter for Douglas Fir-Larch species, and

40 to 70% of the shank diameter for less dense species. The larger percentage figure in each range shall apply to screws of the greater diameters. The length of the lead hole shall be at least equal to the length of the threaded portion.

**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,* $G \le 0.5$*, provided that edge distances, end distances, and spacing are sufficient to prevent unusual splitting (see the* CWC Commentary on CSA O86*).*

## 12.6.2.2 Lag screw assembly

The threaded portion of the screw shall be inserted by turning with a wrench or other suitable 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.*

## 12.6.2.3 Facilitating lag screw assembly

Soap or other non-petroleum based lubricant may be used on the screws or in the lead hole to facilitate insertion and prevent damage to the screw.

## 12.6.2.4 Net section of wood member at lag screw connections

The net section for lag screw connections shall be the same as for connections with bolts of a diameter equal to the shank diameter of the lag screw used (see Clause 12.4.4.6.2).

## ∆ 12.6.2.5 Spacing of lag screws in a row

In a row of lag screws aligned with direction of load, including those in the panel face of CLT, regardless of direction of grain, and measured from the centres of lag screws, spacing shall be as follows (see Figure 12.6.2):

(a) for parallel-to-grain loading: minimum spacings shall be four times the lag screw shank diameter; and

(b) for perpendicular-to-grain loading: spacing between lag screws in a row perpendicular to grain shall be limited by the spacing requirements of the attached member or members (whether of wood loaded parallel to grain or metal), but shall be not less than three shank diameters.

## 12.6.2.6 Row spacing

### 12.6.2.6.1

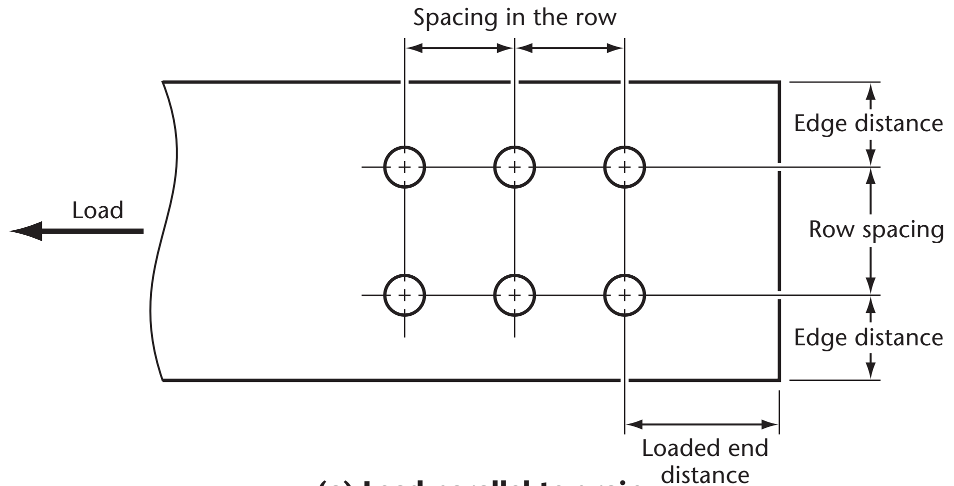
For parallel-to-grain loading, the spacing between rows shall be not less than twice the lag screw shank diameter.

### 12.6.2.6.2

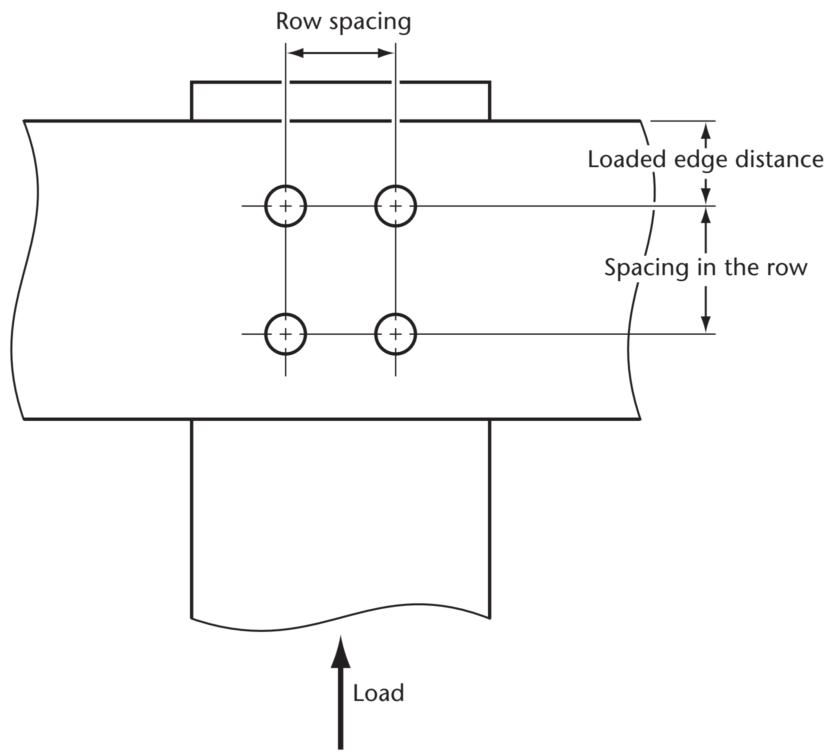
For perpendicular-to-grain loading, the spacing between rows shall be at least 2-1/2 times the lag screw shank diameter for a member thickness-to-diameter ratio of 2 and five times the lag screw shank diameter for member thickness-to-diameter ratios of 6 or more. For ratios between 2 and 6, the minimum spacing shall be obtained by linear interpolation.

### 12.6.2.6.3

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.




**(a) Load parallel to grain**




**(b) Load perpendicular to grain**

**Figure 12.6.2**
**Placement of lag screws in connections**

## 12.6.2.7 End distance

The end distance shall be at least
(a) seven times the lag screw shank diameter or 50 mm, whichever is greater, for the loaded end; or
(b) four times the lag screw shank diameter or 50 mm, whichever is greater, for the unloaded end.

### 12.6.2.8 Edge distance
For members loaded perpendicular to grain, the loaded edge distance shall be at least four times the lag screw shank diameter and the unloaded edge distance shall be at least 1-1/2 times the lag screw shank diameter. For members loaded parallel to grain, the edge distance shall be at least 1-1/2 times the lag screw shank diameter or half the distance between rows of lag screws, whichever is greater.

### Δ 12.6.2.9 Placement of lag screws in panel edge of CLT
For lag screws in panel edge of CLT, the minimum fastener spacings, end and edge distances as specified in Clause 12.4.3.4, shall be used.

## 12.6.3 Penetration length of lag screws

### 12.6.3.1
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.

### 12.6.3.2
Unless the tensile resistance of the lag screw at the net (root) section is verified (Clause 12.6.5.2), the maximum length of threaded shank penetration into main member ($L_t$ in Clause 12.6.5.1) used to determine withdrawal resistance shall not exceed the following values:
(a) Douglas Fir-Larch: nine times shank diameter;
(b) Hem-Fir: ten times shank diameter;
(c) Spruce-Pine-Fir and Northern Species: 11 times shank diameter; and
(d) denser species: determined based on the tensile resistance of fasteners (Clause 12.6.5.2).

**Note:** *For lag screws loaded laterally, the same limits on the maximum length of penetration into main member ($t_2$ in Clause 12.6.6.1.2) may be used, because further increasing the length of a lag screw does not increase either the strength or stiffness of the connection (see the CWC* Commentary on CSA O86*).*

### 12.6.3.3
For lag screws loaded laterally, the minimum length of penetration into main member ($t_2$ in Clause 12.6.6.1.2) shall be not less than five times the shank diameter, $d_F$.

## 12.6.4 Side members

### 12.6.4.1 Wood side plates
The thickness of wood side plates shall be at least twice the shank diameter of the lag screw.

### 12.6.4.2 Steel side plates
The stresses induced in the steel side plate and at the bearing of the lag screw on the plate shall not exceed the resistance of the steel used.

## 12.6.5 Withdrawal resistance

### Δ 12.6.5.1
The factored withdrawal resistance, $P_{rw}$, of a group of lag screws in a connection shall be greater than or equal to the effect of the factored loads, as follows:

$$P_{rw} = \phi Y_w L_t n_F J_E$$

where

$\phi$ = 0.6

$Y_w$ = $y_w (K_D K_{SF} K_T)$

where

$y_w$ = basic 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 lag screw diameter, mm

$G$ = mean relative density of main member (Table A.12.1)

$J_X$ = 0.9 for CLT

= 1.0 in all other cases

$n_F$ = number of fasteners in the connection

$L_t$ = length of threaded shank penetration into main member, mm (Clause 12.6.3)

$J_E$ = 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.*

### 12.6.5.2

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.

**Note:** *See the CWC* Commentary on CSA O86*.*

## 12.6.6 Lateral resistance

### 12.6.6.1 Side grain

### Δ 12.6.6.1.1

The factored lateral strength resistance of a lag screw connection, $P_r$, $Q_r$, or $N_r$, shall be greater than or equal to the effect of the factored loads, 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 $\theta$ to grain:

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

where

$\phi$ = 0.6

$P_u = p_u(K_D K_{SF}\, K_T)$

where

$p_u$= lateral strength resistance for parallel-to-grain loading, $N$ (Clause 12.6.6.1.2)

$J_G$ = factor for a group of fasteners (Tables 12.2.2.3.4A and 12.2.2.3.4B)

$J_{PL}$ = factor for reduced penetration

= 0.625 for penetration of $5d_F$ and 1.0 for $8d_F$ (for intermediate values, interpolate linearly)

$Q_u = q_u(K_D K_{SF} K_T)$

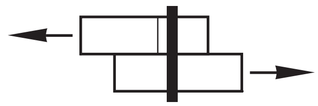
where

$q_u$ = lateral strength resistance for perpendicular-to-grain loading, N (Clause 12.6.6.1.2)

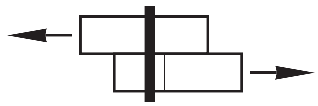
### Δ 12.6.6.1.2

The unit lateral strength resistances, $p_u$ or $q_u$, shall be taken as the smallest value determined as follows:

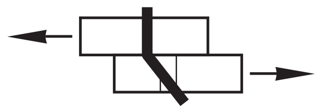
(a) $f_1 d_F t_1$



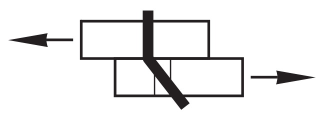
(b) $f_2 d_F t_2$



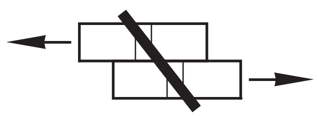
(c) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1 + f_2)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F}\right)$



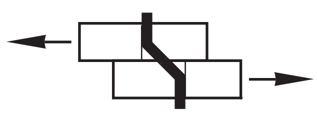
(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1 + f_2)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_2}{d_F}\right)$



(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

$d_F$ = lag screw diameter, mm

$f_2$ = embedment strength of main member, MPa

= $50G\,(1-0.01 d_F)\,J_x$ for parallel-to-grain loading

= $22G\,(1-0.01 d_F)$ for perpendicular-to-grain loading

where

$G$ = mean relative density (Table A.12.1)

$J_x$ = 0.90 for CLT

= 1.0 in all other cases

$t_2$ = length of penetration into main member, mm (Clause 12.6.3)

$f_y$ = lag screw yield strength, MPa

= 310 MPa for lag screws meeting SAE J429 Grade 1

$t_1$ = thickness of side member, mm (Clause 12.6.4)

$f_1$ = embedment strength of side member, MPa

For wood side plates:

$f_1 = 50G\,(1-0.01d_F)\,J_x$ for parallel-to-grain loading

$= 22G\,(1-0.01d_F)$ for perpendicular-to-grain loading

where

$J_x$ = 0.90 for CLT

= 1.0 in all other cases

For steel side plates:

$f_1 = K_{sp}(\phi_{steel}/\phi_{wood})\,f_u$

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) ASTM A36/A36M steel, $f_u$ = 400 MPa;*
*(b) CSA G40.21 steel, Grades 300W and 350W, $f_u$ = 450 MPa; and*
*(c) cold-formed light gauge steel, Grade SS 230, $f_u$ = 310 MPa.*

$\phi_{steel}$ = 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

$\phi_{wood}$ = resistance factor for wood members in connections with lag screws

= 0.6

### 12.6.6.2 End grain

The lateral resistance of lag screws inserted parallel to grain in the end grain of the main member shall be not 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 be not greater than one-half of the lateral side grain resistance for perpendicular-to-grain loading in the main member.

### Δ 12.6.6.3 Panel edge of CLT

The lateral resistance of lag screws inserted in panel edge of CLT shall be not 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.

### Δ 12.6.6.4 Connection deformation

Where the lateral deformation of lag screw connections is required for design, connection deformation may be estimated in accordance with Clause A.12.6.6.4.

## 12.7 Timber rivets

**Note:** *Timber rivets are also referred to as “glulam rivets”.*

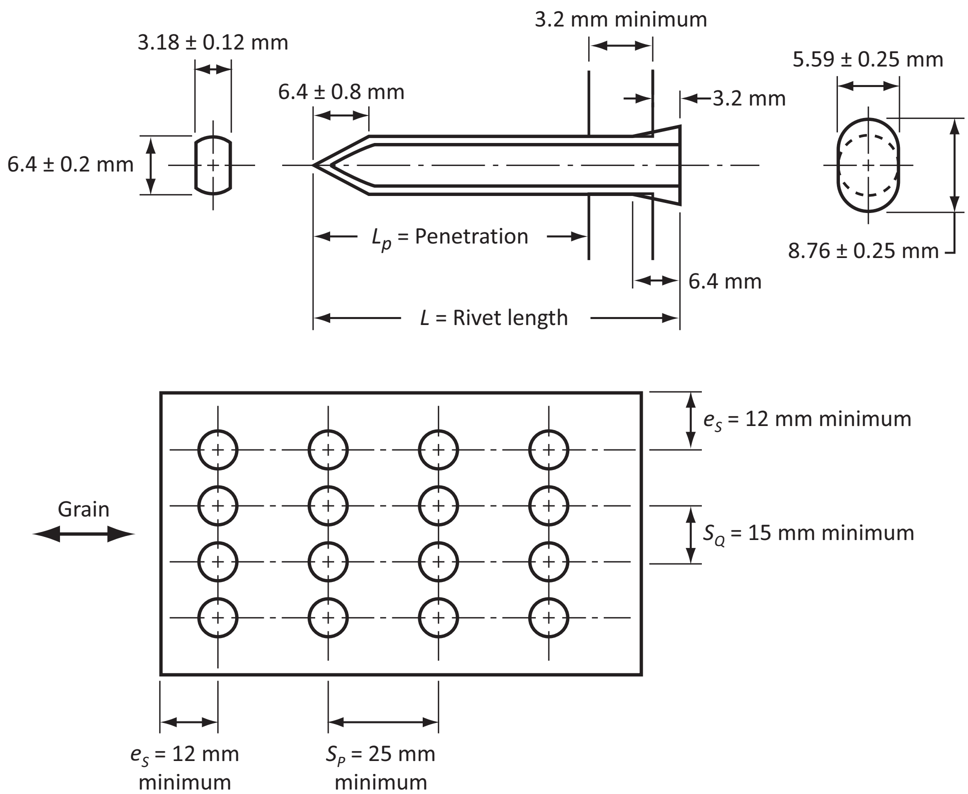
### Δ 12.7.1 General

### 12.7.1.1

The design methods and tabulated resistances specified in Clause 12.7 are for timber rivets that meet the following criteria:

(a) manufactured to have the following properties:
  (i) hardness: Rockwell C32-39;
  (ii) ultimate tensile strength: 1000 MPa, minimum; and
  (iii) finish: hot-dip galvanized; and

(b) used with steel side plates that
 (i) meet the requirements of CSA G40.21 or ASTM A36/A36M; and
 (ii) have dimensions as shown in Figure 12.7.1.1.




**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.*

**Figure 12.7.1.1**
**Steel side plates for timber rivets**

### 12.7.1.2
For wet service conditions, side plates shall be hot-dip galvanized.

### 12.7.1.3
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
Side plates shall have a cross-section adequate for resisting tension and compression forces, as well as buckling at critical sections, but shall be not less than 3.2 mm thick.

### 12.7.1.5
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:** *Timber rivets at the perimeter of the group should be driven first. Successive timber rivets should be driven in a spiral pattern from the outside to the centre of the group.*

### 12.7.1.6
The minimum spacing of rivets shall be 15 mm perpendicular to the grain and 25 mm parallel to the grain.

### 12.7.1.7
Minimum end and edge distances (Figure 12.7.1.7) shall be as specified in Table 12.7.1.7.

**Table 12.7.1.7**
**Minimum end and edge distances for timber rivet connections**

| | Minimum end distance, $a$, mm | | Minimum edge distance, $e$, mm | |
|---|---|---|---|---|
| Number of rivet rows, $n_R$ | Load parallel to grain | Load perpendicular to grain | Free 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 |

**Note:** *End and edge distances are shown in Figure 12.7.1.7.*

### 12.7.1.8
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 12.7.1.9 for connections with rivets driven from opposite faces of a wood member, the rivet length shall be such that the points do not overlap.

### 12.7.1.9
For connections where rivets are driven from opposite faces of a wood member such that their points overlap, the minimum spacing requirements of Clause 12.7.1.6 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 12.7.2, 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
(b) $S_p$ and $S_Q$ determined as the distances between the rivets at their points.

### 12.7.1.10
For wet fabrication conditions in sawn lumber, the maximum dimension perpendicular to grain over which a rivet group spans shall not exceed 200 mm.

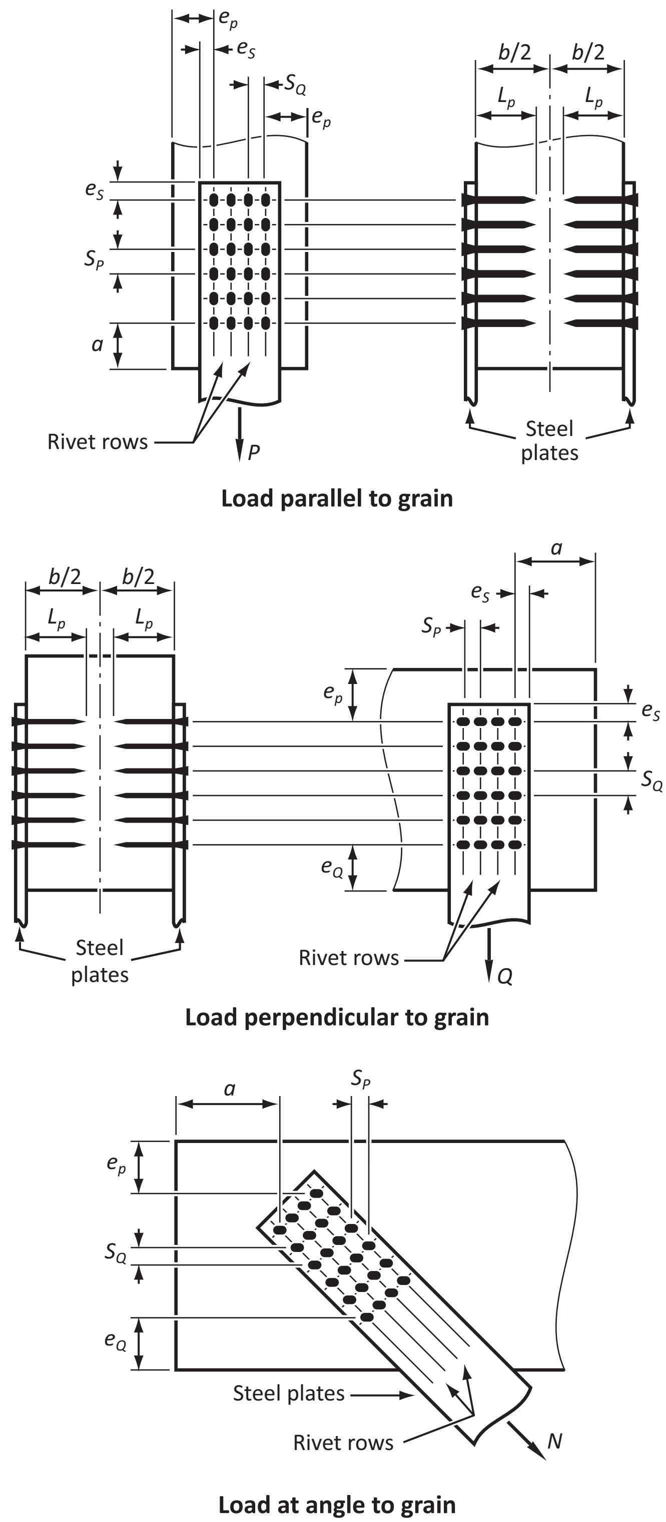



**Figure 12.7.1.7**
**End and edge distances for timber rivet connections**

## 12.7.2 Lateral resistance

### 12.7.2.1

A timber rivet joint (one 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 design of timber rivets loaded laterally is governed by either the ductile failure of the rivet or the brittle failure of the wood.*

### 12.7.2.2

For loading parallel to grain, the factored lateral strength resistance, $P_r$, of the joint shall be taken as follows:

$$P_r = \phi P_u H$$

where

$\phi$ = 0.6

$P_u$ = lateral resistance parallel to grain, kN (Clause 12.7.2.3)

$H$ = 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

### 12.7.2.3

The unit capacity per rivet joint parallel to grain, $P_u$, shall be calculated as the lesser of $P_y$ or $P_w$, as follows:

$P_y = (1.09 L_p^{0.32} n_R n_C)\, J_Y\, (K_{SF}\, K_T)$ for rivet capacity

$P_w = p_w\, (K_D\, K_{SF}\, K_T)$ for wood capacity

where

$L_p$ = length of penetration, mm (Figure 12.7.1.1)

= (overall rivet length) – (plate thickness) – 3.2

$n_R$ = number of rows of rivets parallel to 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

$p_w$ = lateral resistance parallel to grain, kN (Table 12.7.2.3), using wood member thickness for the member dimension in Table 12.7.2.3 for connections with steel plates on opposite sides and using twice the wood member thickness for the member dimension in Table 12.7.2.3 for connections having only one plate

**Note:** *As an alternative, $p_w$ may be calculated in accordance with Clause A.12.7.2.3.1.*

### 12.7.2.4

For loading perpendicular to grain, the factored lateral strength resistance, $Q_r$, of the joint shall be taken as follows:

$$Q_r = \phi Q_u H$$

where
$\phi$ = 0.6
$Q_u$ = lateral resistance perpendicular to grain, kN (Clause 12.7.2.5)
$H$ = material factor (Clause 12.7.2.2)

### 12.7.2.5
The unit capacity per rivet joint perpendicular to grain, $Q_u$, shall be calculated as the lesser of $Q_y$ or $Q_w$, 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 12.7.2.3, $q_w$, kN, is determined from Table 12.7.2.5A and $C_t$ is determined from Table 12.7.2.5B.

**Note:** *As an alternative, $q_w$ and $C_t$ may be calculated in accordance with Clause A.12.7.2.3.2.*

### 12.7.2.6
For loading at an angle to the grain, $\theta$, the factored lateral resistance of the joint, $N_r$, shall be taken as follows:

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

where $P_r$ is determined from Clause 12.7.2.2 and $Q_r$ is determined from Clause 12.7.2.4

### 12.7.2.7
When timber rivets are used in end grain, the factored lateral resistance of the joint shall be 50% of that for loading perpendicular to grain. When timber rivets are used in intermediate grain, the factored lateral resistance may be increased linearly from the value calculated for end grain, up to 100% of the applicable parallel or perpendicular to side grain value.

## 12.7.3 Withdrawal resistance

### 12.7.3.1
Timber rivets loaded in withdrawal shall only be permitted for dry service conditions for short-term and standard-term load durations.

### 12.7.3.2
The factored withdrawal resistance from the side grain, $P_{rw}$, of a timber rivet joint shall be taken as follows. $P_{rw}$ shall be greater than or equal to the effect of the factored loads

$P_{rw} = \phi\, Y_w\, L_p n_R n_C$

where
$\phi$ = 0.6
$Y_w$ = $y_w(K_{SF}\, K_T)$
where
$y_w$ = withdrawal resistance per millimetre of penetration, N/mm
= 13 for glulam
= 7 for sawn timber

and $L_p$, $n_R$, and $n_C$ are as specified in Clause 12.7.2.3.

# Table 12.7.2.3
## Values of $p_w$, kN, parallel to grain for timber rivet joints 40 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 25 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| | 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)*

## Table 12.7.2.3 (Continued)
## 40 mm rivets — Spacing: $S_p$ = 40 mm; $S_Q$ = 25 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| | 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)

## Table 12.7.2.3 (Continued)
## 40 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 15 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| | 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)*

## Table 12.7.2.3 (Continued)
## 65 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 25 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| 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 |
| 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)*

**Table 12.7.2.3 (Continued)**
**65 mm rivets — Spacing: $S_p$ = 40 mm; $S_Q$ = 25 mm**

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| | 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 |
| | 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)

**Table 12.7.2.3 (Continued)**
**65 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 15 mm**

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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)*

## Table 12.7.2.3 (Continued)
## 90 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 25 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| 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 |
| 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)

## Table 12.7.2.3 (Continued)
## 90 mm rivets — Spacing: $S_p$ = 40 mm; $S_Q$ = 25 mm

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 |
| 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 |
| 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)*

**Table 12.7.2.3 (Concluded)**
**90 mm rivets — Spacing: $S_p$ = 25 mm; $S_Q$ = 15 mm**

| Member dimension, mm* | Rivets per row, $n_C$ | Number 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 12.7.1.7 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.*

# Table 12.7.2.5A
# Values of $q_w$, kN, perpendicular to grain for timber rivet joints — Spacing: $S_p$ = 25 mm

| $S_Q$, mm | Rivets per row, $n_C$ | Number of rows, $n_R$ | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 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 |
| | 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 |

*(Continued)*

**Table 12.7.2.5A (Concluded)**

| $S_Q$, mm | Rivets per row, $n_C$ | Number of rows, $n_R$ 1 | 2 | 3 | 4 | 5 | 6 | 8 | 10 |
|---|---|---|---|---|---|---|---|---|---|
| 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 |

**Table 12.7.2.5B**
**Values of factor $C_t$**

| $e_p/[(n_c\text{-}1)s_Q]$ | $C_t$ | $e_p/[(n_c\text{-}1)s_Q]$ | $C_t$ |
|---|---|---|---|
| 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

#### Δ 12.8.1.1

The design requirements specified for truss plate connections in Clause 12.8 are for 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

Clause 12.8 does not apply to

(a) truss plates under corrosive conditions; and
(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
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
In cases where nail-on plates are used, the word "nail" should be read in place of "tooth" in Clause 12.8.

#### Δ 12.8.1.5
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
The thickness of members used in connections shall be not less than twice the tooth penetration.

#### 12.8.1.7
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
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
Truss plates shall not be considered effective in transferring compression loads at a joint.

#### Δ 12.8.2.3
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 12.8.2.3, angle $\rho$);
(c) direction of the applied load relative to the grain (Figure 12.8.2.3, angle $\theta$); and
(d) orientation of plates relative to the applied shear force.

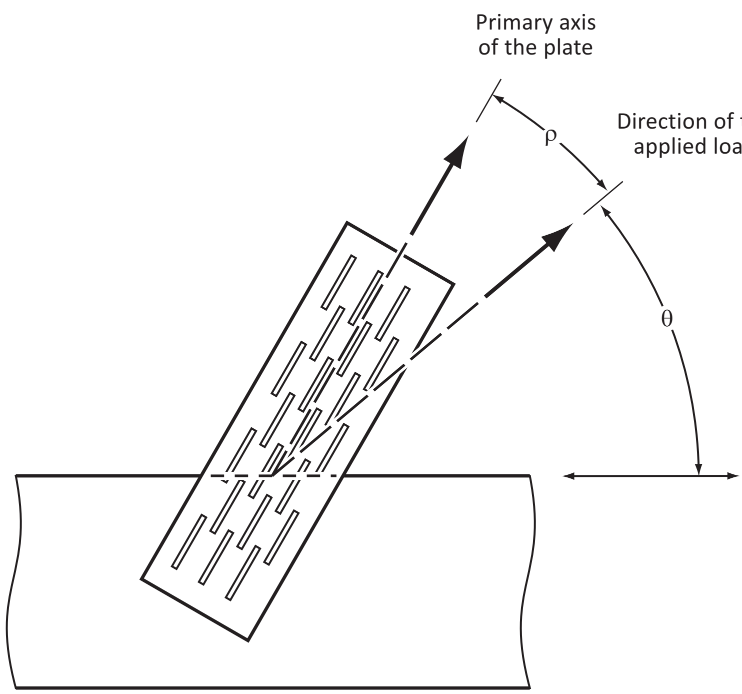
#### 12.8.2.4
The factored ultimate lateral resistance and the lateral slip resistance of the teeth shall be expressed in terms of the surface area of the plates.

### 12.8.2.5

Surface area 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 12.8.2.5. 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

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.

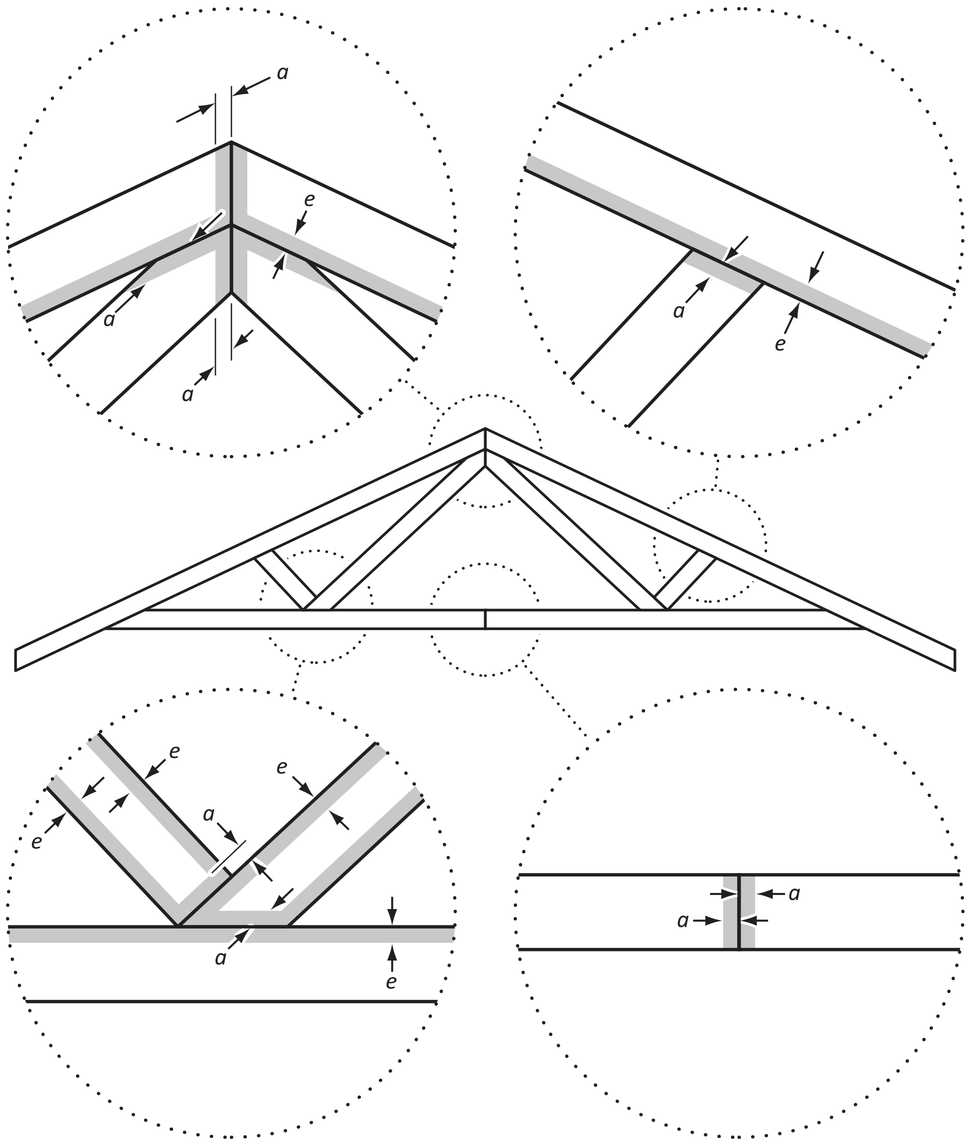



**Legend:**

$\theta$ = angle between the load direction and the grain direction

$\rho$ = angle between the load direction and the primary axis of the plate

**Figure 12.8.2.3**
**Truss plate, load, and grain orientation**




**Legend:**
$a$ = end distance
$e$ = edge distance

**Figure 12.8.2.5**
**End and edge distances for truss plates**

## 12.8.3 Factored resistance of truss plates

### 12.8.3.1

For the strength limit state, the factored resistances of truss plates shall be taken as follows:

(a) For the factored ultimate lateral resistance of the teeth:

$N_r = \phi N_u J_H$

where

$\phi$ = 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 (Clause 12.8.3.2)

$J_H$ = moment factor for heel connection (Table 12.8.3.1)

(b) For the factored tensile resistance of the plate:

$T_r = \phi t_p$

where
$\phi$ = 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
$\phi$ = 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 12.8.3.1**
**Moment factor for heel joints of pitched trusses, $J_H$**

| 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 taken 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:

$$n'_u = \frac{p'_u q'_u}{p'_u \sin^2\theta + q'_u \cos^2\theta}$$

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 $\theta$ and $\rho$:

| | $\theta$ | $\rho$ |
|---|---|---|
| $p_u$ | 0° | 0° |
| $q_u$ | 90° | 0° |
| $p'_u$ | 0° | 90° |
| $q'_u$ | 90° | 90° |

**Note:** *$\theta$ and $\rho$ are as shown in Figure 12.8.2.3.*

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

For the serviceability limit state, the adjusted lateral slip resistance of the teeth, $N_{rs}$, shall be taken as follows:

$$N_{rs} = N_s K_{SF}$$

where

$N_s$ = lateral slip resistance of the teeth (Clause 12.8.4.2)

### 12.8.4.2

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 $\theta$ and $\rho$:

| | $\theta$ | $\rho$ |
|---|---|---|
| $p_s$ | 0° | 0° |
| $q_s$ | 90° | 0° |
| $p'_s$ | 0° | 90° |
| $q'_s$ | 90° | 90° |

**Note:** *$\theta$ and $\rho$ are as shown in Figure 12.8.2.3.*

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 resistance values 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.

### 12.9.2 Connection configuration

#### Δ 12.9.2.1 Placement of fasteners in side grain
For connections nailed at 10% wood moisture content or greater, the minimum nail spacings in sawn lumber side plates and main members, including those in the panel face of CLT, shall be as specified in Table 12.9.2.1. Additional nails may be staggered on the intersection of diagonal lines drawn between rows of nails (see Figure 12.9.2.1).

**Note:** *When the moisture content of the wood is expected to be less than 10% at time of fabrication, the minimum spacings and/or end and edge distances should be increased and holes should be predrilled to avoid splitting.*

**Table 12.9.2.1**
**Minimum spacings for nails and spikes**

| | Minimum spacing (nail diameters) | |
|---|---|---|
| **Dimension*** | **Douglas Fir-Larch, Hem-Fir, and Western Cedar** | **Spruce-Pine-Fir and Northern Species** |
| *a* — Spacing parallel to grain | 20 | 16 |
| *b* — End distance parallel to grain | 15 | 12 |
| *c* — Spacing perpendicular to grain | 10 | 8 |
| *d* — Edge distance perpendicular to grain | 5 | 4 |

**See Figure 12.9.2.1.*

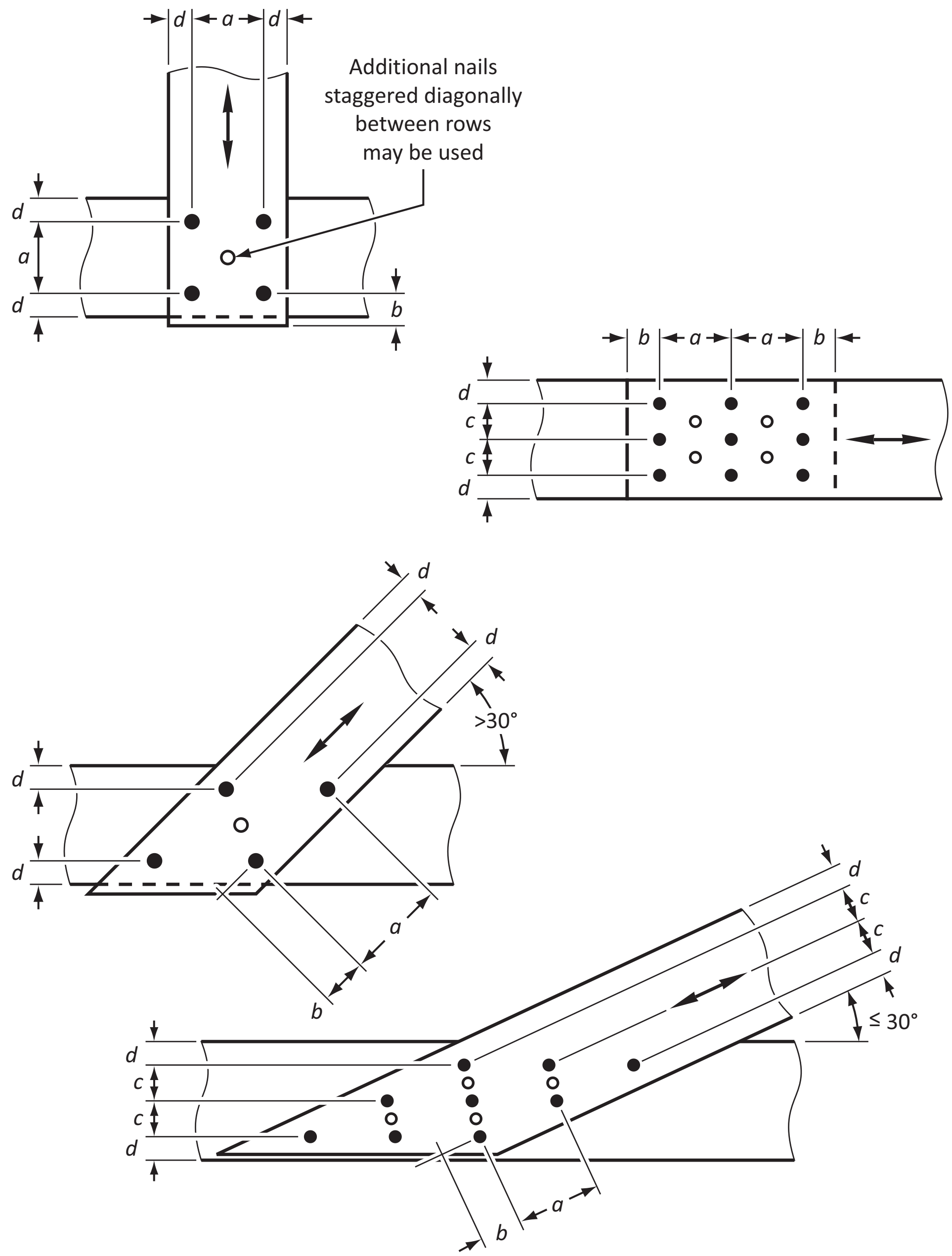
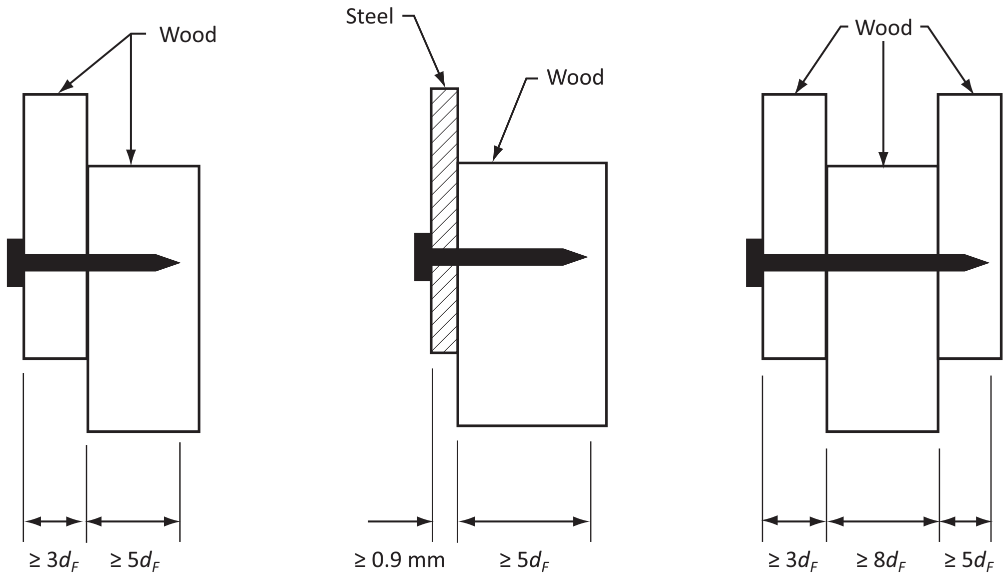



**Figure 12.9.2.1**
**Nail spacings for wood-to-wood connections**

## Δ 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 12.9.2.2.




**Note:** *$d_F$ = fastener shank diameter.*

**Figure 12.9.2.2**
**Penetration length and member thickness**

### Δ 12.9.2.3 Placement of fasteners in panel edge of CLT

In a group of fasteners installed in panel edge of CLT, the minimum spacings measured from centres of fasteners shall be as follows (see Figure 12.4.3.3):

(a) spacing of fasteners in row, $S_R$: ten fastener diameters;
(b) row spacing, $S_C$ : four fastener diameters;
(c) loaded end distance, $a_L$: twelve fastener diameters;
(d) unloaded end distance, $a$: seven fastener diameters;
(e) loaded edge distance, $e_Q$: six fastener diameters; and
(f) unloaded edge distance, $e_P$: three fastener diameters.

## 12.9.3 Connection design

### 12.9.3.1

The factored lateral resistance for nails and spikes driven perpendicular to the grain shall be calculated in accordance with Clause 12.9.4.

### 12.9.3.2

Where the lateral deformation of nailed or spiked wood-to-wood connections, including structural panel-to-lumber connections, is required for design, connection deformation may be estimated in accordance with Clause A.12.9.3.2.

### 12.9.3.3

The factored withdrawal resistance per millimetre of nail penetration when a nail is driven perpendicular to the grain shall be calculated in accordance with Clause 12.9.5.

### Δ 12.9.3.4

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 panel edge of CLT, precaution shall be taken to ensure that side grain penetration occurs.

### 12.9.4 Lateral resistance

#### Δ 12.9.4.1

The factored lateral strength resistance of the nail or spike connection, $N_r$, shall be taken as follows:

$N_r = \phi N_u n_F n_S J_F$

where

$\phi$ = 0.8

$N_u = n_u(K_D K_{SF} K_T)$

where

$n_u$ = unit lateral strength resistance, N (Clause 12.9.4.2)

$n_F$ = number of fasteners in the connection

$n_S$ = number of shear planes per nail or spike

$J_F = J_E\ J_A\ J_B\ J_D$

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

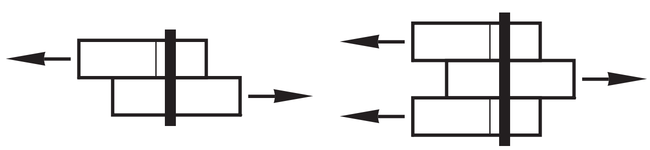
= 1.3 for nails and spikes used in diaphragm and shearwall construction

= 1.0 in all other cases

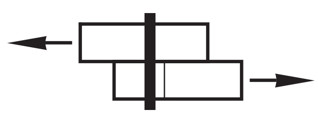
#### Δ 12.9.4.2

The unit lateral strength resistance, $n_u$ (per shear plane), 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 where nails fully penetrate all three members, only Items (a), (c), (d), and (g) shall be considered valid.

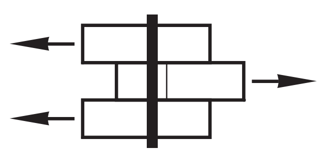
(a) $f_1 d_F t_1$



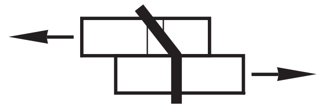
(b) $f_2 d_F t_2$

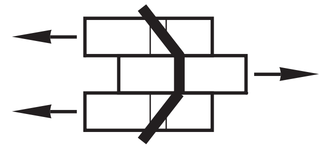


(c) $\frac{1}{2} f_2 d_F t_2$

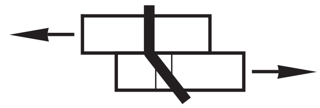


(d) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

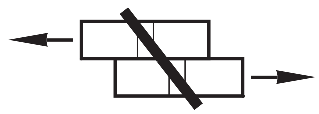




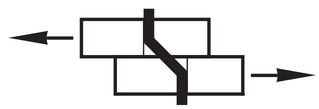
(e) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

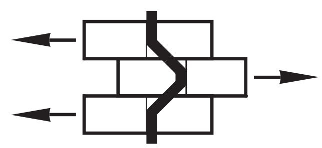


(f) $f_1 d_F^2 \frac{1}{5} \left( \frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$



(g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$





where

$t_1$ = head-side member thickness for two-member connections, mm
= minimum side plate thickness for three-member connections, mm (Clause 12.9.2.2)

$d_F$ = nail or spike diameter, mm

$f_2$ = embedment strength of main member, MPa
= $50\ G\ (1 - 0.01 d_F)\ J_x$

where
$G$ = mean relative density

**Note:** *Mean relative density values for wood members are provided in Table A.12.1.*

$J_x$ = 0.9 for CLT
= 1.0 in all other cases

$t_2$ = length of penetration into point-side member for two-member connections, mm
= centre member thickness for three-member connections, mm (Clause 12.9.2.2)

$f_3$ = embedment strength of main member where failure is fastener yielding, MPa
= $110\ G^{1.8}\ (1 - 0.01 d_F)\ J_x$

where
$J_x$ = 0.9 for CLT
= 1.0 in all other cases

$f_y$ = nail or spike yield strength, MPa
= $50\ (16 - d_F)$

$f_1$ = embedment strength of side member, MPa

For lumber and CLT:

$f_1$ = $50\ G\ (1 - 0.01 d_F)\ J_x$

where
$J_x$ = 0.9 for CLT
= 1.0 in all other cases

For structural panels:

$f_1$ = $104\ G\ (1 - 0.1 d_F)$

where
$G$ = 0.49 for DFP
= 0.42 for CSP and OSB

For steel:

$f_1 = K_{sp} (\phi_{steel} / \phi_{wood}) f_u$

where
$K_{sp}$ = 3.0 for mild steel referenced in CSA S16
= 2.7 for cold-formed light gauge steel referenced in CSA S136
$\phi_{steel}$ = 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
$\phi_{wood}$ = 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., for*
*(a) ASTM A36/A36M steel, $f_u$ = 400 MPa;*
*(b) CSA G40.21 steel, Grades 300W and 350W, $f_u$ = 450 MPa; and*
*(c) cold-formed light gauge steel, Grade SS 230, $f_u$ = 310 MPa.*

## Δ 12.9.5 Withdrawal resistance

### 12.9.5.1
Nails and spikes loaded in withdrawal may be used only for wind or earthquake loading.

### 12.9.5.2
The factored withdrawal resistance of the nail or spike connection, $P_{rw}$, shall be greater than or equal to the effect of the factored loads, as follows:

$P_{rw} = \phi Y_w L_p n_F J_A J_B$

where
$\phi$ = 0.6
$Y_w = y_w (K_{SF} K_T)$

where
$y_w$ = withdrawal resistance per millimetre of penetration into main member, N/mm
$= 16.4 d_F^{0.82} G^{2.2} J_x$

where
$d_F$ = nail diameter, mm
$G$ = mean relative density (Table A.12.1)
$J_x$ = 0.9 for CLT
= 1.0 in all other cases

$L_p$ = length of penetration into main 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 on the far side of a two-member connection
= 1.0 if not clinched or in three-or-more member connections

## 12.10 Joist hangers

### 12.10.1 General

#### 12.10.1.1

The design requirements for joist hangers specified in Clause 12.10 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 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 structural composite lumber.

#### 12.10.1.2

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

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.
(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 be sufficient to 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

Joist hangers shall be designed so that the effect of the factored loads is less than or equal to the factored resistance of the hanger.

#### 12.10.2.2

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 12.2.1.4.

### 12.10.3 Factored resistance of joist hangers

The factored resistance, $N_r$, of joist hangers shall be taken as follows:

$$N_r = \phi N_u$$

where

$\phi$ = 0.6

$N_u = n_u (K_D K_{SF} K_T)$

where

$n_u$ = ultimate resistance of the hanger

$K_D$ = the value determined in accordance with Clause 5.3.2, except that there shall not be an increase for short-term 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.*

## 12.11 Wood screws

### 12.11.1 General

The design requirements specified in this Standard are based on the use of wood screws that meet the requirements of ASME B18.6.1. Nominal diameters and minimum design yield strengths shall be as specified in Table 12.11.1.

**Table 12.11.1**
**Diameter and minimum yield strength of wood screws**

| | Gauge number | | | |
|---|---|---|---|---|
| | 6 | 8 | 10 | 12 |
| Diameter, mm* | 3.50 | 4.16 | 4.82 | 5.48 |
| Minimum yield strength, MPa† | 690 | 620 | 550 | 550 |

**For wood screw diameters greater than gauge 12, design in accordance with the lag screw requirements specified in Clause 12.6.*
*†Linear interpolation for yield strength may be used.*

### 12.11.2 Connection configuration

#### 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, $G$, nominal shank diameter, $d_S$, and root diameter, $d_R$, and are given in Table 12.11.2.1.

**Table 12.11.2.1**
**Lead hole diameter requirements**

| | Relative density | |
|---|---|---|
| **Loading** | $0.5 \le G \le 0.6$ | $G > 0.6$ |
| Lateral | $0.85d_S$ | $d_S$ |
| Withdrawal | $0.7d_R$ | $0.9d_R$ |

#### Δ 12.11.2.2 Spacing and edge and end distances in side grain

The minimum spacing and end and edge distances of wood screw connections shall be as specified in Clause 12.9.2.1 for nails and spikes.

### Δ 12.11.2.3 Placement of fasteners in panel edge of CLT

In a group of wood screws in panel edge of CLT, the minimum spacings, end and edge distances as specified in Clause 12.9.2.3 shall be used.

### Δ 12.11.2.4 Penetration length and member thicknesses

Penetration length and member thicknesses for wood-to-wood and steel-to-wood connections shall be as shown in Figure 12.9.2.2.

## 12.11.3 Connection design

### 12.11.3.1

The factored lateral resistance of wood screws driven perpendicular to the grain shall be calculated in accordance with Clause 12.11.4. The calculated lateral resistance shall apply to any angle of load to grain for wood.

Where the lateral deformation of a wood screw in wood-to-wood connections, including structural panel-to-lumber connections, is required for design, it may be estimated using the method for nail and spike connections specified in Clause A.12.9.3.2.

### Δ 12.11.3.2

The factored withdrawal resistance of a single wood screw per millimetre of threaded shank penetration when driven perpendicular to the grain shall be determined in accordance with Clause 12.11.5.

### Δ 12.11.3.3

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 panel edge of CLT, precaution shall be taken to ensure that side grain penetration occurs.

## 12.11.4 Lateral resistance

### 12.11.4.1

For two- or three-member connections, the factored lateral strength resistance of a wood screw connection shall be taken as follows:

$$N_r = \phi N_u n_F n_s J_A J_E$$

where

$\phi$ = 0.8

$N_u = n_u(K_D K_{SF} K_T)$

where

$n_u$ = unit lateral strength resistance, N (Clause 12.11.4.2)

$n_F$ = number of fasteners in the connection

$n_s$ = number of shear planes per screw

$J_A$ = 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

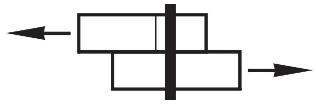
= 0.67 in end grain

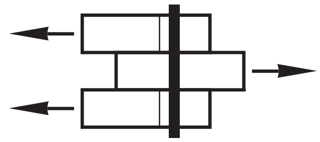
= 1.0 in all other cases

### Δ 12.11.4.2

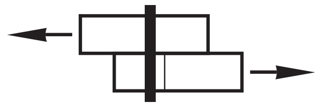
The unit lateral strength resistance, $n_u$ (N per shear plane), shall be taken as the smallest value calculated in accordance with Items (a) to (g). For two-member connections, only Items (a), (b), (d) to (f), and (g) are valid. For three-member connections, where screws fully penetrate all three members, only Items (a), (c), (d), and (g) are valid.

(a) $f_1 d_F t_1$

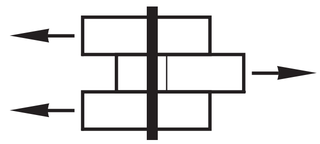




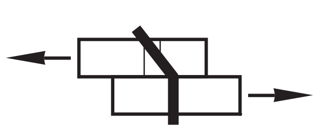
(b) $f_2 d_F t_2$

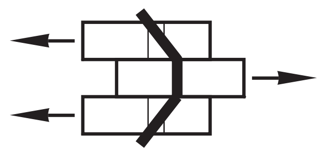


(c) $\frac{1}{2} f_2 d_F t_2$

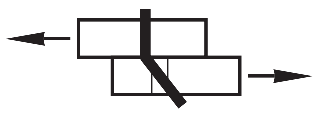


(d) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

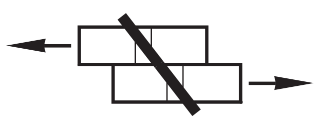




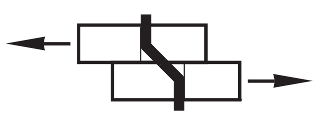
(e) $f_1 d_F^2 \left( \sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

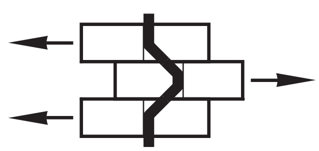


(f) $f_1 d_F^2 \frac{1}{5} \left( \frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$



(g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$





where

$d_F$ = wood screw diameter, mm (Table 12.11.1)

$t_1$ = head-side member thickness for two-member connections, mm
= minimum side plate thickness for three-member connections, mm (Clause 12.11.2.4)

$f_2$ = embedment strength of main member where failure is wood bearing, MPa
= $50\ G\ (1 - 0.01 d_F)\ J_x$

where

$G$ = mean relative density of wood member (see Table A.12.1)

$J_x$ = 0.9 for CLT
= 1.0 in all other cases

$t_2$ = length of penetration into point-side member for two-member connections, mm
= centre member thickness for three-member connections, mm (Clause 12.11.2.4)

$f_3$ = embedment strength of main member where failure is fastener yielding, MPa
$= 110\ G^{1.8}\ (1 - 0.01 d_F)\ J_x$

where
$J_x$ = 0.9 for CLT
= 1.0 in all other cases

$f_y$ = wood screw yield strength, MPa (Table 12.11.1)

$f_1$ = embedment strength of side member, MPa

For lumber and CLT:

$f_1 = 50\ G\ (1 - 0.01 d_F)\ J_x$

where
$J_x$ = 0.9 for CLT
= 1.0 in all other cases

For structural panels:

$f_1 = 104\ G\ (1 - 0.1 d_F)$

where
$G$ = 0.49 for DFP
= 0.42 for CSP and OSB

For steel:

$f_1 = K_{sp}\ (\phi_{steel} / \phi_{wood})\ f_u$

where
$K_{sp}$ = 3.0 for mild steel referenced in CSA S16
= 2.7 for cold-formed light gauge steel referenced in CSA S136

$\phi_{steel}$ = 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

$\phi_{wood}$ = 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) ASTM A36/A36M steel, $f_u$ = 400 MPa;*
*(b) CSA G40.21 steel, Grades 300W and 350W, $f_u$ = 450 MPa; and*
*(c) cold-formed light gauge steel, Grade SS 230, $f_u$ = 310 MPa.*

## 12.11.5 Withdrawal resistance

### 12.11.5.1 General

The factored withdrawal resistance shall be equal to the lesser of the factored screw withdrawal resistance of the main member (Clause 12.11.5.2) or the factored head pull-through resistance of the side member (Clause 12.11.5.3).

### Δ 12.11.5.2 Withdrawal resistance of main member

For a two-member connection connected with wood screws, the factored withdrawal resistance, $P_{rw}$, in N, of the main member shall be taken as follows:

$$P_{rw} = \phi\, Y_w\ L_{pt}\, n_F$$

where
$\phi$ = 0.6

$Y_w = y_w (K_D K_T K_{SF})$

where

$y_w$ = basic withdrawal resistance per millimetre of threaded shank penetration in main member, N/mm

$= 59 d_F^{0.82} G^{1.77} J_x$

where

$d_F$ = nominal wood screw diameter, mm (Table 12.11.1)

$G$ = mean relative density of main member (Table A.12.1)

$J_x$ = 0.9 for CLT

= 1.0 in all other cases

$L_{pt}$ = threaded length penetration in the main member, mm

$n_F$ = number of wood screws in the connection

For a connection with three members, the threaded length penetration shall be the maximum threaded length within any member other than the head-side member.

### Δ 12.11.5.3 Head pull-through resistance of side member

For connections with steel side plates, the factored head pull-through resistance, $P_{pt}$, in N, shall be taken as follows:

$$P_{pt} = 1.5 \phi\, t_1 d_w f_u n_F$$

For connections with lumber, glulam, CLT, or structural panel side plates, the factored head pull-through resistance shall be taken as follows:

$$P_{pt} = 65 \phi\, t_1 n_F K_D$$

where

$\phi$ = 0.4

$t_1$ = thickness of side plate, 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) ASTM A36/A36M steel, $f_u$ = 400 MPa;*

*(b) CSA G40.21 steel, Grades 300W and 350W, $f_u$ = 450 MPa; and*

*(c) cold-formed light gauge steel, Grade SS 230, $f_u$ = 310 MPa.*

$n_F$ = number of wood screws in the connection

# 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.*

### 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
The specified strengths for round timber piles shall be as specified in Table 13.3.

## 13.4 Modification factors
The modification factors specified in Clause 5.3.2.2 for duration of load and Clause 6.4.2 for service conditions shall apply to timber piling specified strengths. Other modification factors shall not apply.

## 13.5 Strength and resistance

### 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 adequate 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 adequate lateral support shall be designed in accordance with Clause 6.5.6, 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.

### 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 6.5.6. 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 6.5.10.

**Table 13.3**
**Specified strengths and modulus of elasticity for round timber piles, MPa**

| Species | Bending at extreme fibre, $f_b$ | 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 | 7 000 |
| Jack pine | 18.1 | 1.5 | 15.6 | 5.2 | 11.6 | 7 000 | 5 000 |
| Lodgepole or ponderosa pine | 14.2 | 1.0 | 13.2 | 5.2 | 9.7 | 7 000 | 5 000 |
| Red pine | 13.6 | 1.2 | 11.7 | 5.2 | 9.0 | 7 000 | 5 000 |

**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 group.*

# 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 specified strengths for select structural grade beams and stringers of the applicable species combination specified in Table 6.3.1C. The specified strengths for round eastern white cedar poles shall be 50% of the specified strengths for select structural grade beams and stringers of the species combination Northern Species.

## 14.4 Modification factors

The specified strengths for round poles shall be modified by the modification factors used for beams and stringers in Clause 6.

## 14.5 Strength and resistance

### 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.

### 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 adequate lateral support shall be designed in accordance with Clause 6.5.6, 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 6.5.6. 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 6.5.10.

#### 14.5.2.5 Constant non-rectangular cross-section

For non-rectangular compression members of constant section, $r\sqrt{12}$ shall be substituted for member width or depth in Clause 6.5.6, 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

Clause 15 specifies design requirements for proprietary structural wood products used in specific applications. The design procedures specified in Clause 15 are provided for the project engineer or building designer. The manufacturer's design values are usually generated by the manufacturer's engineer in accordance with Clause 16. Unless otherwise 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. For additional information on proprietary structural wood products in general, and prefabricated wood I-joists and structural composite lumber products in particular, see the CWC* Commentary on CSA O86*.*

## 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 mark of the appropriate certification organization should be included on the I-joists.*

### 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 specified strength and stiffness of prefabricated wood I-joists described in Clause 16 are applicable for use in dry service conditions, with $K_S = 1.0$.

#### 15.2.2.3 Treatment factor, $K_T$

The specified strength and stiffness 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, $K_H$, for prefabricated wood I-joists shall be taken as 1.0.

**Note:** *For additional information, see Appendix X1.4.1 of ASTM D5055.*

### 15.2.3 Strength and resistance

#### 15.2.3.1 Bending moment resistance

The factored bending moment resistance, $M_r$, of prefabricated wood I-joists shall be taken as follows:

$$M_r = \phi M_r' (K_D K_S K_H K_T) K_L$$

where

$\phi$ = 0.90

$M_r'$ = adjusted bending moment resistance

$K_S$ = service condition factor (Clause 15.2.2.2)

$K_H$ = system factor (Clause 15.2.2.4)

$K_L$ = lateral stability factor (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 taken as equal to 1.00 when lateral support is provided at points of bearing, in order 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.

**Note:** *For additional information on lateral stability, see the CWC* Commentary on CSA O86.

### 15.2.3.3 Notches

There shall be no notching or cutting of the flanges of prefabricated wood I-joists unless such details have been evaluated and shown to be acceptable on the basis of documented test data.

### 15.2.3.4 Shear resistance

The factored shear resistance, $V_r$, of prefabricated wood I-joists shall be taken as follows:

$$V_r = \phi V'_r\ (K_D\ K_S K_T)$$

where

$\phi$ = 0.90

$V'_r$ = shear resistance determined in accordance with the manufacturer's product evaluation report

Loads within a distance from the support equal to the depth of the member shall not be neglected.

**Note:** *Factored ultimate resistance values, $\phi V'_r$, are listed in the CCMC* Registry of Product Evaluations.

### 15.2.3.5 Reaction resistance

The factored end reaction resistance, $ER_r$, at the specified end bearing length of prefabricated wood I-joists shall be taken as

$$ER_r = \phi\ ER'_r\ (K_D K_S K_T)$$

where

$\phi$ = 0.90

$ER'_r$ = end reaction resistance determined in accordance with Clause 16 and published in the manufacturer's product evaluation report.

The factored intermediate reaction resistance, $IR_r$, at the specified intermediate bearing length of prefabricated wood I-joists shall be taken as

$$IR_r = \phi\ IR'_r (K_D K_S K_T)$$

where

$\phi$ = 0.90

$IR'_r$ = intermediate reaction resistance determined in accordance with Clause 16 and published in the manufacturer's product evaluation report

Neglecting loads within a distance from the support equal to the depth of the member shall not be permitted.

### 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.

**Note:** *For additional information, see the CWC* Commentary on CSA O86.

### 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. For additional information, see Clause 12.10 and the CWC* Commentary on CSA O86.

## 15.3 Structural composite lumber products

### 15.3.1 General
Structural composite lumber products 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:** *Evidence of compliance is typically a marking of a certification organization (CO).*

### 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 structural composite lumber products, provided that appropriate testing demonstrating the validity of the factor for use with the applicable structural composite lumber product has been conducted.

**Note:** *See the CWC* Commentary on CSA O86.

#### 15.3.2.2 Service condition factor, $K_S$
The specified strength and stiffness of structural composite lumber products described in Clause 15.3 are applicable for use in dry service conditions, with $K_S = 1.0$. If structural composite lumber products are to be used in other than dry service conditions, 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 specified strengths and stiffness described in Clause 15.3 are applicable to untreated structural composite lumber products, with $K_T = 1.0$. Treatment adjustments for specified strength and stiffness shall be based on the documented results of tests that take into account 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 structural composite lumber products used in a load-sharing system shall be 1.04. To qualify for this increase, structural composite lumber products 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 structural composite lumber products 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 structural composite lumber products shall be determined in accordance with the manufacturer's product evaluation report.

### 15.3.2.7 Lateral stability factor, $K_L$

The lateral stability factor, $K_L$, for structural composite lumber products shall be determined in accordance with Clause 6.5.4.2.

## 15.3.3 Strength and resistance

### 15.3.3.1 Bending moment resistance

The factored bending moment resistance, $M_r$, of structural composite lumber products shall be taken as follows:

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

where

$\phi$ = 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 (Clause 15.3.2.5)

$K_L$ = lateral stability factor (Clause 15.3.2.7)

### 15.3.3.2 Notches

Structural composite lumber 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.

### 15.3.3.3 Shear resistance

The factored shear resistance, $V_r$, of structural composite lumber products shall be taken as follows:

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

where

$\phi$ = 0.90

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

where

$f_v$ = specified shear strength of structural composite lumber products, MPa, determined in accordance with the manufacturer's product evaluation report

$A$ = cross-sectional area of member, mm$^2$

$K_{Zv}$ = 1.0

**Notes:**

**(1)** *For additional information on size factor in shear, $K_{Zv}$, see the CWC* Commentary on CSA O86*.*

**(2)** *For additional information concerning loads within a depth d of the support for shear design, see the CWC* Commentary on CSA O86.

### 15.3.3.4 Compressive resistance parallel to grain

#### 15.3.3.4.1 Effective length, $L_e$

Unless noted otherwise, the effective length, $L_e = K_e L$, shall be used in determining the slenderness ratio of structural composite lumber products in compression.

**Note:** *Recommended effective length factors for structural composite lumber products in compression are provided in Table A.6.5.6.1.*

#### 15.3.3.4.2 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

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

#### 15.3.3.4.3 Factored compressive resistance parallel to grain

The factored compressive resistance parallel to grain, $P_r$, of structural composite lumber products shall be taken as follows:

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

where

$\phi$ = 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 (Clause 15.3.3.5)

$K_{Zc}$ = 1.0

**Note:** *For additional information on size factor in compression parallel to grain, $K_{Zc}$, see the CWC* Commentary on CSA O86*.*

### 15.3.3.5 Slenderness factor, $K_C$

The slenderness factor, $K_C$, shall be taken 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

$E_{05} = 0.87E$

where

$E$ = specified modulus of elasticity, MPa, determined in accordance with the manufacturer's product evaluation report

### 15.3.3.6 Compressive resistance perpendicular to grain (bearing)

#### Δ 15.3.3.6.1 Maximum loads

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain determined in accordance with Clauses 15.3.3.6.2 to 15.3.3.6.3.

### Δ 15.3.3.6.2 Effect of all applied loads

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

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

where

$\phi$ = 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 (Clause 6.5.7.5)

$K_{Zcp}$ = size factor for bearing (Clause 6.5.7.4), except that $K_{Zcp}$ shall be equal to 1.0 in the flatwise (L-X plane, as defined in ASTM D5456) orientation

### Δ 15.3.3.6.3 Effect of loads applied near a support

#### 15.3.3.6.3.1 Factored compressive resistance perpendicular to grain

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

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

where

$\phi$ = 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$^2$ (Clause 15.3.3.6.3.2)

**Note:** *See Figure 6.5.7.3 and CWC* Commentary on CSA O86*.*

#### 15.3.3.6.3.2 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 shall be calculated as follows:

$$A'_b = b\left(\frac{L_{b1} + L_{b2}}{2}\right) \le 1.5b\left(L_{b1}\right)$$

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 taken as $1.5b$ ($L_{b1}$).*

### Δ 15.3.3.6.4 — *Deleted*

### 15.3.3.7 Compressive resistance at an angle to grain

The factored compressive resistance at an angle to grain shall be calculated in accordance with Clause 6.5.8, using the appropriate specified strengths and resistances for the proprietary grade of the structural composite lumber product.

### 15.3.3.8 Tensile resistance parallel to grain

The factored tensile resistance parallel to grain, $T_r$, of structural composite lumber products shall be taken as follows:

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

where

$\phi$ = 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 area, $mm^2$

$K_{Zt}$ = size factor in tension (Clause 15.3.2.6)

### 15.3.3.9 Resistance to combined bending and axial load

Members subject to combined bending and compressive or tensile axial loads shall be designed to satisfy the applicable interaction equation, as follows:

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

or

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

where

$P_f$ = factored compressive axial load

$P_r$ = factored compressive resistance parallel to grain calculated in accordance with Clause 15.3.3.4.3

$M_f$ = 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

$K_e$ = the effective length factor (Clause A.6.5.6.1)

$T_f$ = factored tensile axial load

$T_r$ = factored tensile resistance parallel to grain calculated in accordance with the requirements of Clause 15.3.3.8

**Note:** *See Clause A.6.5.10 for a more detailed interaction formula for combined bending and compressive loads.*

### 15.3.4 Serviceability limit states

The design of structural composite lumber products for serviceability limit states shall meet the requirements of 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 structural composite lumber 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 a proprietary structural composite lumber product shall meet the requirements of Clause 12.10.

#### 15.3.5.2 Other connections

The procedures and specified capacities for nails, wood screws, bolts, lag screws, timber rivets, shear plates, truss plates, and split rings in Clause 12 may be used for the design of connections for a proprietary structural composite lumber product when testing has demonstrated the validity of those procedures and specified capacities for use with that product.

**Notes:**

**(1)** *Design information for some fasteners, primarily nails and joist hangers, applicable for use with specific proprietary structural composite lumber products are listed in the CCMC* Registry of Product Evaluations*.*

**(2)** *In the absence of design information for specific fasteners in specific proprietary structural composite lumber products, connections for structural composite lumber 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 NRCC *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 16.2.3.6.

Prefabricated wood I-joists for use in accordance with Clause 16.2 shall meet the applicable requirements of Clause 16.2.

**Note:** *The mark of the appropriate certification organization should be included on the I-joists.*

### 16.2.2 Materials

#### 16.2.2.1 Flange materials

Clause 16.2 applies to flanges as specified in Clause 6 for sawn structurally graded lumber or Clause 7 for glued-laminated timber. Lumber not meeting the requirements of Clause 6 and structural composite lumber products 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 (Exterior Bond), or CSA O325.

**Note:** *See the CWC* Commentary on CSA O86.

#### 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 Standard or the CWC* Commentary on CSA O86.

### 16.2.3 Specified strengths and modulus of elasticity

#### 16.2.3.1 Specified strength parallel to grain, $f_a$

The specified strength parallel to grain, $f_a$, 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 (Clause 16.2.3.2)

$K_{LN}$ = length adjustment factor as specified in ASTM D5055 (see Clause A.16.2.3.1)

$K_{Zt}$ = the size factor for tension parallel to grain from Table 6.4.5 (applicable only to visually stress-graded lumber)

$f_c$ = specified strength in compression parallel to grain, MPa (Clause 16.2.3.3)

#### Δ 16.2.3.2 Specified strength in tension parallel to grain, $f_t$

The specified strength in tension parallel to grain, $f_t$, 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 7.3, respectively.

(b) When the flange material is not as described in Item (a), $f_t$ shall be taken as follows:

$$f_t = t\ K_r$$

where

$t$ = characteristic value for tension parallel to grain, MPa, as specified in Clause 16.2.3.7

$K_r$ = reliability normalization factor for bending and tension from Table 16.2.3.2

### Table 16.2.3.2
### Reliability normalization factor, $K_r$ (applicable to prefabricated wood I-joists and structural composite lumber products only)

| $CV_w$, %* | Bending and tension[1] | Compression parallel to grain[2] | Shear | | | Bearing reactions[4] |
|---|---|---|---|---|---|---|
| | | | Prefabricated wood I-joists | Structural composite lumber[3] | | 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.*
[1] *Applicable to structural composite lumber products in bending and tension. Also applicable to flanges of prefabricated wood I-joist flanges in compression as well as tension.*
[2] *Applicable to structural composite lumber products in compression parallel to grain.*
[3] *Structural composite lumber 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).*
[4] *Applicable to prefabricated I-joist reaction capacity at end bearing supports and intermediate bearing supports.*
**Note:** *See the CWC* Commentary on CSA O86.

## Δ 16.2.3.3 Specified strength in compression parallel to grain, $f_c$

The specified strength in compression parallel to grain 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 7.3, respectively.

(b) When the flange material is not as described in Item (a), $f_c$ shall be taken as follows:

$$f_c = c\,K_r$$

where

$c$ = characteristic value for compression parallel to grain, MPa, as specified in Clause 16.2.3.7

$K_r$ = reliability normalization factor for bending and tension from Table 16.2.3.2

### Δ 16.2.3.4 Specified shear capacity, $V_c$

The specified shear capacity, $V_c$, shall be taken as follows:

$V_c = v\, K_r$

where
$v$ = characteristic value for shear, N, as specified in Clause 16.2.3.7
$K_r$ = reliability normalization factor for shear for prefabricated wood I-joists from Table 16.2.3.2

### 16.2.3.5 Specified Reaction Capacities, $ER_c$, $IR_c$

The specified end reaction capacity, $ER_c$, at a specified bearing length shall be taken as

$ER_c = r_e\, K_r$

where
$r_e$ = the characteristic value for end reaction as defined in Clause 16.2.3.7, N
$K_r$ = reliability normalization factor for reaction for prefabricated wood I-joists from Table 16.2.3.2

The specified intermediate reaction capacity, $IR_c$, at a specified bearing length shall be taken as

$IR_c = r_i\, K_r$

where
$r_i$ = the characteristic value for intermediate reaction as defined in Clause 16.2.3.7, N
$K_r$ = reliability normalization factor for reaction for prefabricated wood I-joists from Table 16.2.3.2

### 16.2.3.6 Modulus of elasticity, $E$

The modulus of elasticity 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 7.3, 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 6.5.2.1 of ASTM D5055.

### Δ 16.2.3.7 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.

(b) The maximum characteristic reaction values, $r_e$ and $r_i$, for prefabricated wood I-joists shall be the end reaction ($r_e$) capacity and intermediate reaction ($r_i$) capacity, 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) structural composite lumber: 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
   $f_{c1}$ = assigned specified strength in compression parallel to grain as specified in Clause 6.3 for the same grade, species, and size as $f_{t1}$

$f_{t1}$ = closest assigned specified strength to $f_t$ in tension parallel to grain as specified in Clause 6.3 for the species and size tested in accordance with ASTM D5055

(ii) structural composite lumber: 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 resistance and strength

### 16.2.4.1 Adjusted bending moment resistance

The adjusted bending moment resistance, $M'_r$, of prefabricated wood I-joists shall be taken as

$$M'_r = f_a A_{net} Y$$

or

$$M'_r = M_{cv} K_r$$

where

$f_a$ = specified strength parallel to grain, MPa (Clause 16.2.3.1)

$A_{net}$ = net 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 16.2.3.7(e)

$K_r$ = reliability normalization factor for bending and tension (Table 16.2.3.2)

### 16.2.4.2 Adjusted shear resistance

The adjusted shear resistance, $V'_r$, of prefabricated wood I-joists shall be taken 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 16.2.3.4

Loads within a distance from the support equal to the depth of the member shall not be neglected. Adjustments to the shear design value near the support shall be substantiated by independent testing to the shear capacity criteria specified in ASTM D5055.

### 16.2.4.3 Adjusted Reaction Resistance

The adjusted end reaction resistance, $ER'_r$, of prefabricated wood I-joists shall be taken as

$$ER'_r = ER_c$$

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 16.2.3.5, N

The adjusted intermediate reaction resistance, $IR'_r$, of prefabricated wood I-joists shall be taken as

$$IR'_r = IR_c$$

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 16.2.3.5, 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 for stiffness calculations, $E_s$, shall be taken 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 taken as follows:

$$EI_w = \left(\sum B_a\right)\left(\frac{W_D^3}{12}\right)K_{SE}K_{TE}$$

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 taken as follows:

$$W_S = B_v\,K_{SG}K_{TG}$$

where

$E$ = specified modulus of elasticity (Clause 16.2.3.6)

$K_{SE}$ = 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 9.3A, 9.3B, or 9.3C, N/mm

$W_D$ = overall depth of the structural panel web, mm

$B_v$ = shear-through-thickness rigidity from Table 9.3A, 9.3B, or 9.3C, 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:** F*or axial web stiffness and web shear-through-thickness rigidity for products not covered by Table 9.3A, 9.3B or 9.3C 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 products

## 16.3.1 General

Structural composite lumber products 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.

Structural composite lumber products for use in accordance with Clause 16.3, including products subjected to secondary processing operations, shall meet the applicable requirements of Clauses 16.3.2 to 16.3.6.

**Note:** *Evidence of compliance is typically a marking of a certification organization (CO).*

## 16.3.2 Adhesives and binder systems

### 16.3.2.1 Adhesives

Adhesives used in the manufacture of structural composite lumber products shall be
(a) specific classes of adhesives conforming to CSA O112.6 or O112.7; or
(b) alternative adhesives conforming to the Standard, 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.

**Note:** *For additional information on adhesives, see the CWC* Commentary on CSA O86.

### 16.3.2.2 Binder systems

Binder systems shall demonstrate equivalent performance to the adhesives specified in Clause 16.3.2.1(a).

**Note:** *For additional information on equivalent adhesives, see the CWC* Commentary on CSA O86.

## 16.3.3 Specified strengths and modulus of elasticity

### 16.3.3.1 General

Specified strengths and modulus of elasticity for structural composite lumber products for use in accordance with this Standard shall be determined as specified in Clauses 16.3.3.2 to 16.3.3.7.

### 16.3.3.2 Specified bending strength, $f_b$

The specified bending strength, $f_b$, for structural composite lumber products shall be taken as follows:

$$f_b = F_B K_r$$

where

$F_B$ = characteristic value in bending, MPa, determined in accordance with ASTM D5456

$K_r$ = reliability normalization factor for bending and tension (Table 16.2.3.2)

### 16.3.3.3 Specified shear strength, $f_v$

The specified shear strength, $f_v$, for structural composite lumber products shall be taken as follows:

$$f_v = v_{SCL} K_r$$

where

$v_{SCL}$ = characteristic value in shear, MPa, determined in accordance with ASTM D5456

$K_r$ = reliability normalization factor for shear for structural composite lumber (Table 16.2.3.2)

### 16.3.3.4 Specified compression strength parallel to grain, $f_c$

The specified compression strength parallel to grain, $f_c$, for structural composite lumber products shall be taken 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 (Table 16.2.3.2)

### Δ 16.3.3.5 Specified compression strength perpendicular to grain, $f_{cp}$

The specified compression strength perpendicular to grain, $f_{cp}$, MPa, for structural composite lumber products 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 tension strength parallel to grain, $f_t$

The specified tension strength parallel to grain, $f_t$, for structural composite lumber products shall be taken as follows:

$$f_t = t_{SCL} K_r$$

where

$t_{SCL}$ = characteristic value in tension parallel to grain, MPa, as determined in accordance with ASTM D5456

$K_r$ = reliability normalization factor for bending and tension (Table 16.2.3.2)

#### 16.3.3.7 Specified modulus of elasticity

The specified modulus of elasticity, $E$, for structural composite lumber products shall be the mean modulus 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 structural composite lumber products shall be taken 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 structural composite lumber products shall be taken 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 taken as follows:

$$E_s = E\,(K_{SE}\,K_{TE})$$

where

$E$ = specified modulus of elasticity, MPa, as specified in 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 structural composite lumber products

The shear modulus or shear rigidity, $G_S$, for stiffness calculations shall be taken as follows:

$G_S = G\,(K_{SG}\,K_{TG})$

where
$G$ = specified shear modulus or shear rigidity, MPa, established by testing or as published in a recognized reference for the structural composite lumber product 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 structural composite lumber products

### 16.3.6 Connections for structural composite lumber

The procedures and specified capacities 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 a proprietary structural composite lumber product when testing has demonstrated the validity of those procedures and specified capacities for use with that product.

**Note:** *Design information for fasteners for use with proprietary structural composite lumber products, including specified capacities, 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 16.4.1.2.

#### 16.4.1.2

Truss plates shall be manufactured from galvanized sheet steel that conforms to Grade SQ230, SQ255, SQ275, HSLA I340, HSLA I410, HSLA II340, or HSLA II410 of ASTM A653/A653M and has the minimum properties specified in Table 16.4.1.2.

**Table 16.4.1.2**
**Minimum properties of steels used for truss plates**

| | Grade | | | | |
|---|---|---|---|---|---|
| | SQ230 | SQ255 | SQ275 | HSLA I340 or HSLA II340 | HSLA I410 or HSLA II410 |
| Ultimate tensile strength, MPa | 310 | 360 | 380 | 410 | 480 |
| Minimum yield, 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 $p_u$, $q_u$, $p'_u$, $q'_u$ shall be obtained from tests carried out in accordance with CSA S347 and calculated as follows:

$n_u = 0.8\ R_c\ B\ K_{mu}\ K_{pu}$

where

$n_u$ = ultimate lateral resistance values $p_u$, $q_u$, $p'_u$, $q'_u$

$R_c$ = the lesser of the following:

(a) $R_u$ (1 – 2.104 $CV$)

(b) 0.753 $R_u$

$R_u$ = average ultimate lateral resistance of the test data

$B$ = 1.44 – 2.18 $CV$, but not greater than 1.1

$CV$ = coefficient of variation of the test data

$K_{pu}$ = roller press modification factor for ultimate strength, determined in accordance with CSA S347

$K_{mu}$ = moisture response factor for ultimate strength

= 0.83 for sawn lumber to sawn lumber connections

= determined in accordance with CSA S347 for structural composite lumber to structural composite lumber connections, and sawn lumber to structural composite lumber connections.

**Note:** *where the moisture response factor for structural composite lumber to sawn lumber is less than 0.83, two values of $n_u$ will be calculated for sawn lumber connections, one value of $n_u$ for sawn lumber to sawn lumber connections, and one value of $n_u$ for sawn lumber to structural composite lumber 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:

$t_p = t_{uL}\ CF$

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 S347and calculated as follows:

$n_s = (R_s/1.2)\ K_{ps}\ K_{ms}$

where

$n_s$ = lateral slip values $p_s$, $q_s$, $p'_s$, $q'_s$

$R_s$ = 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 structural composite lumber to structural composite lumber connections, and sawn lumber to structural composite lumber connections

**Note:** *where the moisture response factor for sawn lumber to structural composite lumber is less than 0.83, two values of $n_s$ will be calculated for sawn lumber connections, one for sawn lumber to sawn lumber, and one for sawn lumber to structural composite lumber.*

## 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 structural composite lumber.

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.*

### 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 be not greater than 2% above the mean relative density values for the material tested. For manufactured wood products, the relative density shall be not 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.12.1.*

### 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) $N_{ult}\ CF_w$;

(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;

(b) $N_{ult\ steel}\ CF_s$.

#### 16.5.4.3

For testing where failure of the hanger steel does not occur, use the highest ultimate load per assembly in place of the $N_{ult\ steel}$ in Clause 16.5.4.1(b) and 16.5.4.2(b)

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 connector used to attach the hanger to the header calculated per Clause 12 using $f_{u\ spec}$ in place of $f_u$

$N_{r\ test}$ = factered lateral resistance of the connector 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

# 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 modification 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 a number of joists or rafters. System behaviour can be accounted for in single-member design by implementing system modification factors: $K_H$ for strength and $K_\Delta$ for serviceability. In this Standard, only $K_H$ has been quantified.

The system modification 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 fastener stiffness.

**Note:** *See the CWC* Commentary on CSA O86 *for further information.*

### 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 used 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 stud wall systems under wind load

Typical wood stud wall systems sheathed with wood panel products and 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 6.3.1A) derived from visually 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);
(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.

## Δ A.5.4.5 Floor vibration

Serviceability design of floor systems has traditionally been addressed by limiting the computed joist deflection under a uniform load. For some floor systems and end uses, traditional criteria have provided satisfactory performance. However, these criteria do not always result in floors satisfactory to occupants.

A point load deflection check was implemented in Part 9 of the NRCC *National Building Code of Canada* to address vibration performance of sawn lumber joist floors. While the point load deflection check has proved adequate for traditional sawn lumber joist floors, it does not adequately address the broad range of application variables that occur in engineered wood product floor systems. The span capabilities and optimization of engineered wood floor systems can merit a more refined analysis procedure. This can take the form of recommended maximum spans or calculation procedures. Users should exercise judgment in applying simplified criteria when attempting to limit objectionable floor vibrations in these systems.

On-going research to develop vibration design procedures for floors constructed with CLT panels is underway in Canada and internationally. As an interim measure, a design procedure for simply supported, bare floor system is provided in Clause A.8.5.3 to prevent excessive vibration caused by walking action of the occupants. Clause A.8.5.3 gives an empirical design method that has been found satisfactory for simple floors constructed from CLT panels. Additional simplified rules for multiple-span systems and floors with a medium-weight or heavy topping are also provided in A.8.5.3 as an interim measure until results from on-going research are available to refine these rules. This design procedure reflects the state of knowledge for this type of floor system. In the longer term the intent is to provide generalized guidance applicable to all types of wood floor systems.

In addition to making use of the guidance provided by the NRCC *User's Guide — NBC Structural Commentaries* (*Part 4 of Division B*), users should consult the CWC *Commentary on CSA O86* to assist in their assessment of floor vibration issues for their specific applications.

## Δ A.5.4.6 Building movements due to moisture content change

Most buildings are able to accommodate small amounts of movement due to moisture content change in wood members. However, if insufficient considerations are taken 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 calculations 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 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 located 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:

$$S = D \times (M_i - M_f) \times c$$

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 (Beam and Stringer and Post and Timber 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 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.6.5.2**
**Minimum dressed sizes of dimension lumber and timbers***

| | 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 |
| | 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 |

*(Continued)*

**Table A.6.5.2 (Concluded)**

| | Smaller dimension, mm | | Larger dimension, mm | |
|---|---|---|---|---|
| Item | Dry | Green | Dry | Green |
| Timbers | | 292 | | 292 |
| | | 343 | | 343 |
| | | 394 | | 394 |

**Sizes are rounded to the nearest whole millimetre and are based on CSA O141.*

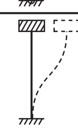
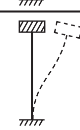
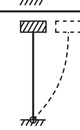
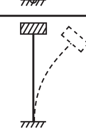
**Notes:**

**(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 in excess of 19%.*

## A.6.5.6.1 Effective length factor, $K_e$

**Table A.6.5.6.1**
**Minimum design values of effective length factor, $K_e$, for compression members**

| 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 |  |
| 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, Ke , shall be not 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.6.5.6.3 Spaced compression members

### A.6.5.6.3.1 General

Spaced compression members shall consist of two or more individual members joined with timber connectors and having spacer and end blocks as specified in Clauses A.6.5.6.3.2 to A.6.5.6.3.4.

### A.6.5.6.3.2 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.6.5.6.3.3, depending on the fixity factor assumed, are met. In compression members of trusses, a panel point that is stayed 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 be not 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.6.5.6.3.3 Fixity classes

The end fixity of spaced compression members shall be classified as one of the following in accordance with Figure A.6.5.6.3:

(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.6.5.6.3.4 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.6.5.6.3.

### A.6.5.6.3.5 Slenderness ratio

The slenderness ratio, $C_C$, of spaced compression members of uniform rectangular section shall not exceed 80 and shall be taken as follows:

$$C_C = \frac{\text{actual length between points of lateral support}}{\text{least dimension of an individual member}}$$

### A.6.5.6.3.6 Factored compressive resistance parallel to grain

The factored compressive load resistance, $P_r$, parallel to grain shall be taken as follows:

$$P_r = \phi\, F_c A K_c K_{Zc}$$

where

$\phi$ = resistance factor (Clause A.6.5.6.3.7)

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

where

$f_c$ = specified strength in compression parallel to grain, MPa (Tables 6.3.1A and 6.3.3 for sawn lumber and Table 7.3 for glulam)

$A$ = total cross-sectional area, mm$^2$

$K_C$ = slenderness factor (Clause A.6.5.6.3.7)

$K_{Zc}$ = 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

$L$ = column length associated with member dimension, mm

### A.6.5.6.3.7 Resistance factor, $\phi$, and slenderness factor, $K_C$

The resistance factor, $\phi$, and the slenderness factor, $K_C$, shall be taken as follows:

(a) $\phi$ = 0.80 for sawn lumber

= 0.90 for glued-laminated timber

(b) When $C_C$ does not exceed 10:

$K_C = 1.00$

(c) 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

$$C_K = \sqrt{\frac{0.76 E_{05} K_{SE} K_E K_T}{F_c}}$$

where

$E_{05}$ = applicable value from Tables 6.3.1A to 6.3.1D 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.6.5.6.3.3)

= 2.50 for condition A

= 3.00 for condition B

(d) When $C_C$ is greater than $C_K$ but does not exceed 80:

$$K_C = \frac{E_{05} K_{SE} K_E K_T}{k C_c^2 F_c}$$

where

$k$ = 1.8 for sawn lumber

= 2.0 for glued-laminated timber

### A.6.5.6.3.8 Design check of spaced compression members

The factored resistance determined by spaced compression member design shall be checked against the sum of factored resistances of individual members taken as simple compression members. In this check, $C_C$ shall be taken as follows:

$$C_C = \frac{\text{actual length between points of lateral support}}{\text{larger dimension of an individual member}}$$

The factored compressive resistance, $P_r$, shall be the smaller value obtained by Clause A.6.5.6.3.6 or A.6.5.6.3.8.

### A.6.5.6.3.9 Combined stresses

When axial compression in spaced compression members is combined with bending stresses, Clause 6.5.10 shall be used only if the bending is in a direction parallel to the larger dimension of the individual member.

**Table A.6.5.6.3**
**End block constants for spaced compression members, MPa**

| $C_C$* | D Fir-L | Hem-Fir | 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.*

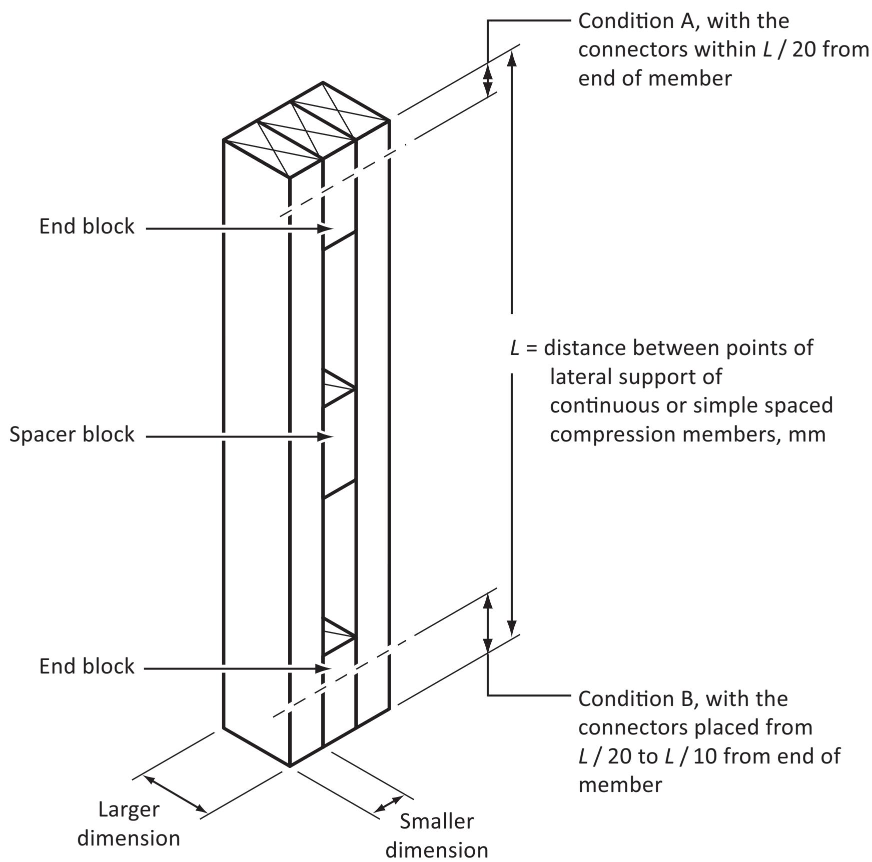



**Figure A.6.5.6.3**
**Spaced compression member (connector joined)**

### A.6.5.7 Compression perpendicular to grain

A relationship between mean compression perpendicular to grain strength 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 compression perpendicular to grain strength, 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 6.3.2 and 6.3.3 assign increased compression perpendicular to grain design values to specific grades of S-P-F or Hem-Fir machine-graded lumber, which have higher mean density than visually graded lumber of the same species (see Table A.12.1).

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.6.5.10 Detailed formula for combined bending and compressive loads

Members subject to combined bending and compressive loads may be designed using the alternative interaction equation:

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

where

$P_f$ = factored compressive axial load, N

$P_r$ = factored compressive load resistance parallel to grain, calculated in accordance with Clause 6.5.6, N

$M_f$ = factored bending moment, N•mm

$\Delta_L$ = 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.6.5.6.1

$M_r$ = factored bending moment resistance, calculated in accordance with Clause 6.5.4, N•mm

### A.6.5.12 Preserved wood foundations

#### A.6.5.12.1

Studs for preserved 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.6.5.12.6 to A.6.5.12.12 give conservative approximations of sufficient accuracy for practical construction. The dimensions used in the formulas are identified in Figure A.6.5.12.1.

#### A.6.5.12.2

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.

#### A.6.5.12.3

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.6.5.12.4
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.6.5.12.5
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 suitable systems to resist loads resulting from unequal pressure distribution.

### A.6.5.12.6
Combined bending and compressive load effects may be evaluated using the following formula:

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

where

$P_f$ = factored compressive load on stud, N

$P_r$ = factored compressive resistance calculated in accordance with Clause 6.5.6.2.3, N

$M_f$ = maximum factored bending moment on stud, N•mm

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

$M_r$ = factored bending moment resistance calculated in accordance with Clause 6.5.4.1, N•mm

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

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.6.5.6.1

**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 $P_r$.*

### A.6.5.12.7
The value of $M_f$ in Clause A.6.5.12.6 is the maximum moment due to factored lateral load and may be calculated using the following formula, which is derived from recognized engineering formulas. At any point, $x$, above the floor the factored bending moment, $M_{fx}$ (N•mm), is

$$M_{fx} = \frac{w_f H^2}{6L}\left[(H-3a)\left(\frac{L-x}{H}\right) - L\left(\frac{H-Ax}{H}\right)^3\right]$$

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.6.5.12.1, mm

### A.6.5.12.8
The following formulas may be used to determine the maximum factored 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.6.5.12.8.*

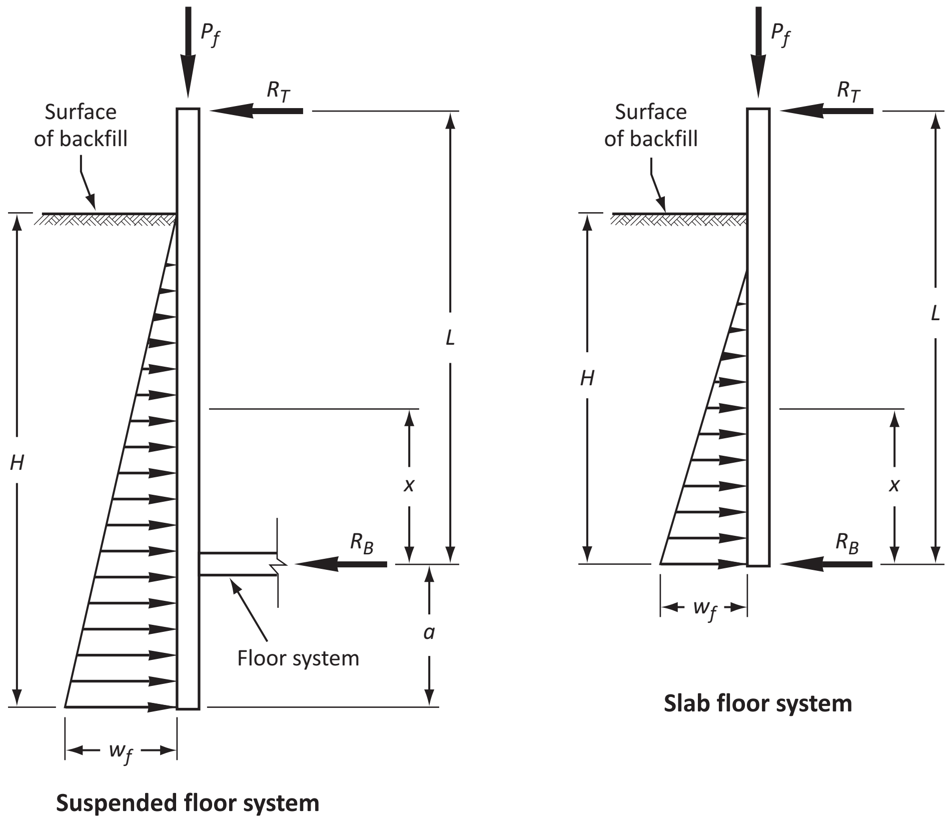
Δ

## Table A.6.5.12.8
## Moment and deflection coefficients, $K_m$ and $K_\Delta$, for typical values for $L$ and $a$*

| | Slab floor | | Suspended floor | | |
|---|---|---|---|---|---|
| | $K_\Delta \times 10^{15}$ | | $K_\Delta \times 10^{15}$ | | |
| Backfill height, $H$, mm | At point of maximum moment | At $x$ = 0.45$L$ | At point of maximum moment | At $x$ = 0.45$L$ | $K_m$† |
| 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,* $L$ = 2500 mm, *and* $a$ = 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).* $K_\Delta$ *at the point of maximum moment (fourth column) is zero where moment at the support governs.*




**Legend:**
$L$ = Stud length, mm
$x$ = Location of maximum moment, mm
$H$ = Height of backfill, mm

**Figure A.6.5.12.1**
**Dimensions and loading of foundation studs without surcharge**

## Δ A.6.5.12.9

The value of Δ can be calculated at any point, $x$, above the basement floor (see Figure A.6.5.12.1), using the following formula:

$$\Delta = \frac{w_f (L - x)}{360EILH} K_\Delta$$

where
$w_f$ = variable shown in Figure A.6.5.12.1, N/mm

$$K_\Delta = \left[10H^2 (H - 3a)(2L - x)x - 3(H - a)^5 + K_2\right]$$

where

$$K_2 = \frac{3L}{L - x}(H - a - x)^5 \quad \text{when } x \le H - a$$

$= 0$ when $x > H - a$

$E$ = modulus of elasticity of stud, MPa

$I$ = moment of inertia of stud, mm4

$H$, $L$, $a$ = variables shown in Figure A.6.5.12.1, mm

**Notes:**

**(1)** *$\Delta$ is normally calculated at the point where maximum moment occurs. If the maximum moment occurs at the support (due to cantilever effect), $\Delta$ = 0.*

**(2)** *Values of $K_\Delta$ for a range of backfill heights and typical wall dimensions are specified in Table A.6.5.12.8.*

## A.6.5.12.10

Maximum deflection may be calculated from Clause A.6.5.12.9 with $x = 0.45L$ and using specified rather than factored loads, to give a good approximation of the theoretical maximum deflection.

## A.6.5.12.11

Maximum longitudinal shear may be calculated from the following formulas derived from recognized engineering formulas and identified as the greatest value of $V_f$ :

(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

$V_f$ = factored shear force per stud, N

$w_f$ = variable shown in Figure A.6.5.12.1, 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.6.5.12.1, mm

## A.6.5.12.12

### A.6.5.12.12.1

The lateral restraint required at 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 top of stud, N

$w_f$ = variable shown in Figure A.6.5.12.1, N/mm

$H, L, a$ = variables shown in Figure A.6.5.12.1, mm

### A.6.5.12.12.2

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 bottom of stud, N

$w_f$ = variable shown in Figure A.6.5.12.1, N/mm

$H, L, a$ = variables shown in Figure A.6.5.12.1, mm

## A.6.5.13 Sawn lumber design for specific truss applications

### A.6.5.13.1 Scope

Fully triangulated — The modified design procedures of Clause 6.5.13* apply only to sawn lumber used in trusses where all of 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.6.5.13 and therefore are not considered part of the overall truss length.

**The modified design procedures of Clause 6.5.13 are based on research reported in C. Lum, E. Jones, and B. Hintz. 1996. Design of wood trusses for small buildings. In* Proceedings of the International Wood Engineering Conference*, Vol. 1. New Orleans.*

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.6.5.13 limits the applicability of Clause 6.5.13 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.6.5.13.

### Δ A.6.5.13.3 Compressive resistance parallel to grain

Effective length — Recommended effective length factors are specified in Table A.6.5.6.1. *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, provided that 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 sufficient to cover these cases (a pitch break is a point along the chord analogue line where the slope of the chord changes).

### A.6.5.13.4 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 6.5.7.1.

**Note:** *Testing to support this method of improving the bearing resistance is described in F. J. Keenan, N. S. Bulmanis, and H. A. Latos. 1983. Improving the bearing strength of supports of light wood trusses. Canadian Journal of Civil Engineering 10:306–312.*

### Δ A.6.5.13.5 Resistance to combined bending and axial load

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.

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 sufficient 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 6.5.13.5 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 6.5.13.5*, 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.

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 6.5.13.5 are based on W. Lau, J.D. Barrett, and F. Lam. 1995. Chord member design proposal. Vancouver: Department of Wood Science, University of British Columbia.*

# 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.7.5.5).

**Notes:**

**(1)** *Commercially available widths can vary.*

**(2)** *For widths greater than* 365 mm, *designers should check availability before specifying.*

**Table A.7.5.5**
**Minimum radius of curvature**

| Lamination thickness, mm | Minimum radius of curvature, mm — Tangent end* | Minimum radius of curvature, mm — Curved end |
|---|---|---|
| Standard | | |
| 38 | 8 400 | 10 800 |
| 19 | 2 800 | 3 800 |
| Non-standard | | |
| 35 | 7 400 | 9 500 |
| 32 | 6 300 | 8 500 |
| 29 | 5 600 | 7 300 |
| 25 | 4 600 | 6 200 |
| 16 | 2 300 | 3 000 |
| 13 | 1 800 | 2 200 |
| 10 | 1 200 | 1 400 |
| 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.*

# Δ A.8 Cross-laminated timber (CLT)

## A.8.5.2 Deflection of CLT panels

The maximum deflection of the CLT panel, $\Delta_{max}$, 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

$\Delta_{ST}$ = elastic deflection due to short term and/or standard term loads, without dead loads in combination

$\Delta_{LT}$ = 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, $\omega$, 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 bending stiffness, $(EI)_{eff}$, and the effective in-plane (planar) shear rigidity, $(GA)_{eff}$, as follows:

$$\Delta = \frac{5}{384}\frac{\omega L^4}{(EI)_{eff}} + \frac{1}{8}\frac{\omega L^2 \kappa}{(GA)_{eff}}$$

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:

$$\Delta = \frac{1}{48}\frac{PL^3}{(EI)_{eff}} + \frac{1}{4}\frac{PL\kappa}{(GA)_{eff}}$$

where

$\kappa$ = form factor

= 1.2 for rectangular cross-sections

**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. See the CWC* Commentary on CSA O86 *for more information.*

## 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:

$$l_v \leq 0.11\frac{\left(\frac{(EI)_{eff}}{10^6}\right)^{0.29}}{m^{0.12}}$$

where

$l_v$ = vibration-controlled span limit, m

$m$ = linear mass of CLT for a 1 m wide panel, kg/m

$(EI)_{eff}$ = effective bending stiffness for a 1m wide panel, N•mm$^2$ (see Clause 8.4.3.2)

**Note:** *The above equation assumes the floor*

*(a) has a single span with both ends simply supported (for CLT supported on beams refer to the CWC* Commentary on CSA O86 *for additional guidance); 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 "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.*
See *CWC Commentary on CSA O86* for background on assumptions and limitations.

# 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 9.3C for construction sheathing OSB that compiles with CSA O325 are consistent with values developed through in-grade testing and supported by various organizations.

Certification organizations need to 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.

Panel marks consist of an end use mark followed by a span mark (see Table A.9.2.2A), e.g., 2F24 or W16. Multiple panel marks may be shown on panels qualified for more than one end use, e.g., 1R24/2F16 or 2R48/2F24/1F24.

**Note:** *See Table A.9.2.2B.*

**Table A.9.2.2A**
**Panel marks for construction sheathing products (CSA O325)**

(a) Panel marks

| End use mark | Span marks | | | | | |
|---|---|---|---|---|---|---|
| | 16 | 20 | 24 | 32 | 40 | 48 |
| | Recommended framing member spacing, mm (in) | | | | | |
| | 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 |

**Note:** *Panels marked 1 are usually stiffer than panels marked 2.*

**Table A.9.2.2B**
**Relationship between panel marks and nominal thickness**

| | Nominal thickness, mm | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Panel mark | 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 taken 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$^2$

$\phi$ = 0.95

$X_B$ = buckling coefficient (Table A.10.6.3.4)

$s$ = clear spacing between longitudinals, mm

$\ell_p$ = span of stressed skin panel, mm

**Note:** *This Clause is not applicable to OSB compression flanges. Buckling coefficients have not been developed for OSB.*

**Table A.10.6.3.4**
**Buckling coefficient, $X_B$, for the plywood compression flange of stressed skin panels, kN•m**

| Unsanded plywood thickness, mm | Buckling coefficient, $X_B$ | |
|---|---|---|
| | 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 Fastener 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:

$$e_n = \left(\frac{0.013\ vs}{d_F^2}\right)^2$$

where

$v$ = maximum specified shear force per unit length along the diaphragm boundary or top of shearwall, N/mm

$s$ = nail spacing at panel edges of shearwalls or diaphragms, mm

For nails used in wood-based sheathing with green lumber, multiply calculated dry lumber $e_n$ values by 2.
For dry lumber only, $e_n$ for gypsum wallboard may be taken as 0.76 mm.

### A.11.7.1 Deflection of shearwalls in multi-storey buildings

(a) Deflection of blocked shearwall segments

The interstorey drift at the $i$-th level, mm, may be taken as follows:

$$\Delta_i^{storey} = \Delta_{b,i}^{storey} + \Delta_{s,i}^{storey} + \Delta_{n,i}^{storey} + \Delta_{a,i}^{storey}$$

And the total deflection at the $i$-th level, mm, may be taken as follows:

$$\Delta_i^{total} = \sum_{j=1}^{i} \Delta_j^{storey}$$

where

$\Delta_i^{storey}$ = interstorey drift at the $i$-th level

$\Delta_i^{total}$ = total deflection at the $i$-th level

$\Delta_{b,i}^{storey}$ = 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

$$\Delta_{b,i}^{storey} = \Delta_{b,i} + H_i \left(\sum_{j=1}^{i-1} \theta_j\right) = \Delta_{b,i} + H_i(\theta_1 + \theta_2 + \ldots + \theta_{i-1})$$

where

$\theta_i$ (or $\theta_j$) = the change in slope over the $i$-th (or the $j$-th) storey height subjected to bending, as shown in Figure A.11.7.1A, 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.11.7.1A, 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

$V_i$ = shear force in the shearwall at the $i$-th storey, N, is calculated as follows:

$V_i = \sum_{j=i}^{n} F_j = F_i + F_{i+1} + \ldots + F_n$

$M_i$ = overturning moment at level $i$ (at the top of $i$-th storey), N•mm, as shown in Figure A.11.7.1B, is calculated as follows:

$M_i = \sum_{j=i+1}^{n} V_j H_j$

where

$n$ = number of total 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

$E$ = elastic modulus of boundary member (vertical member at shearwall segment boundary), N/mm$^2$

$A$ = cross-sectional area of the boundary member, mm$^2$

$L_s$ = length of shearwall segment, mm

**Note:** *For a continuous rod system anchored at each floor level, the effective bending stiffness should be calculated.*

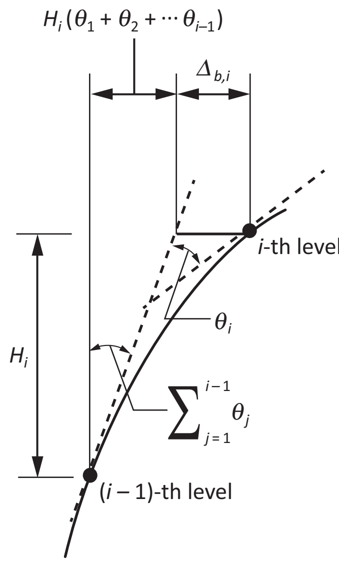



**Figure A.11.7.1A**
**$\theta_i$ & $\Delta_{b,i}$**

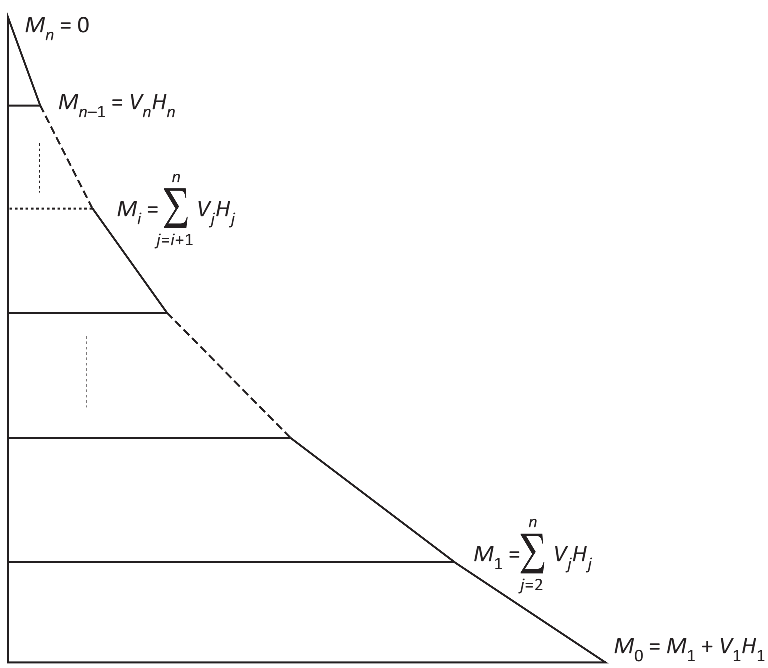



**Figure A.11.7.1B**
**Moment diagram**

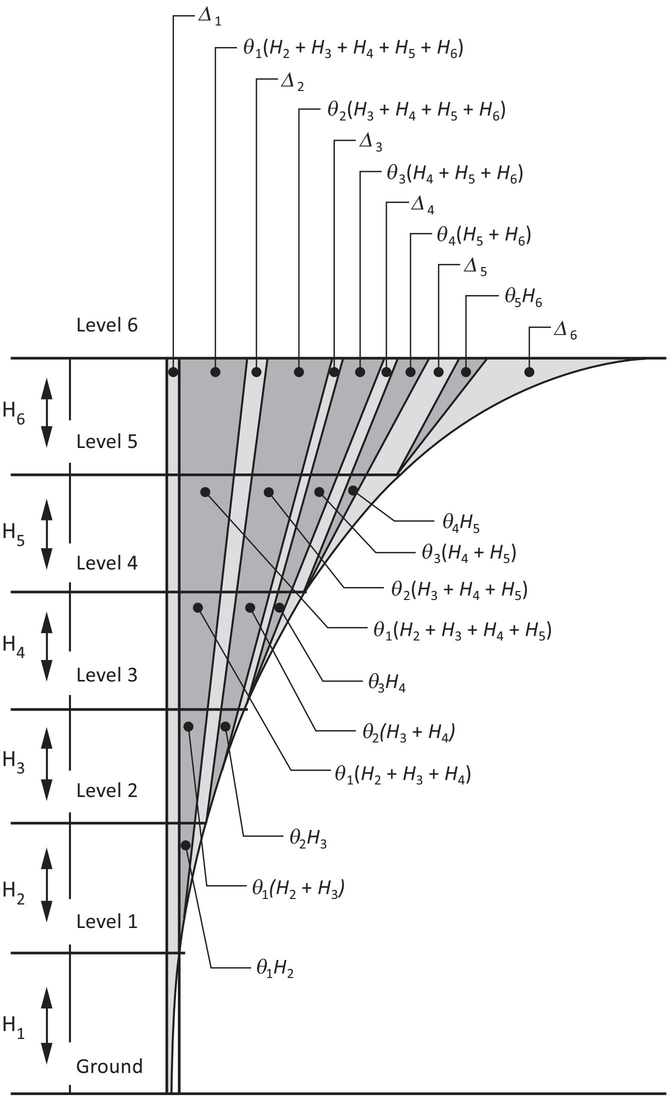



**Figure A.11.7.1C**
**Deflection due to bending**
(Source: APEGBC Technical and Practice Bulletin)

(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 9.3A, 9.3B, and 9.3C for wood-based panel sheathing. For gypsum wallboard, $B_v$ may be taken as 7000 N/mm)

(iii) Deflection due to nail slip

$\Delta_{n,i}^{storey} = \Delta_{n,i}$

where

$\Delta_{n,i} = 0.0025 H_i e_{n,i}$

where

$e_{n,i}$ = nail deformation at the $i$-th storey, mm (for wood-based sheathing, see Clause A.11.7. For gypsum wallboard, $e_n$ may be taken as 0.76 mm)

(iv) Deflection due to elongation in wall anchorage system

$$\Delta_{a,i}^{storey} = H_i\left(\sum_{j=1}^{i}\alpha_j\right) = H_i\alpha_i + H_i\left(\sum_{j=1}^{i-1}\alpha_j\right)$$

where

$$\alpha_i = \frac{(d_a)_i}{L_s}$$

where

$(d_a)_i$ = total vertical elongation of the wall anchorage system at the $i$-th storey

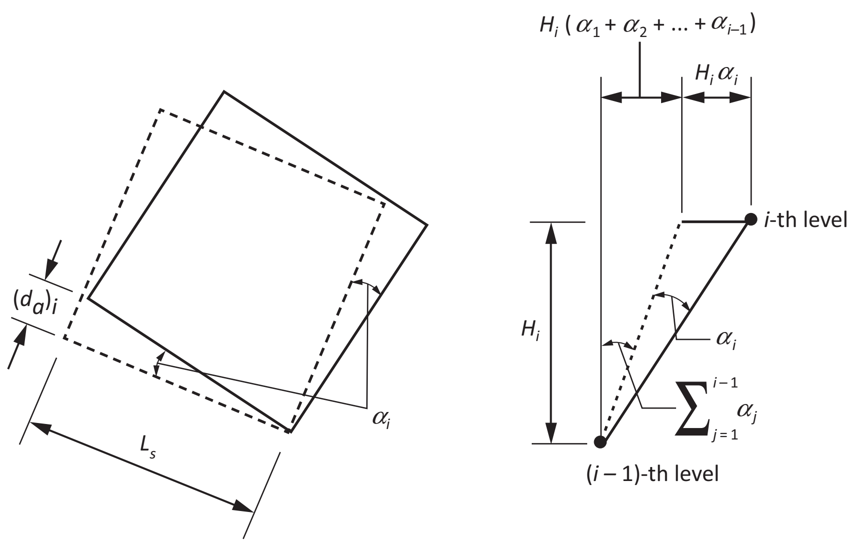



**Figure A.11.7.1D**
**Deflection due to vertical elongation of the wall anchorage system at the *i*-th storey**

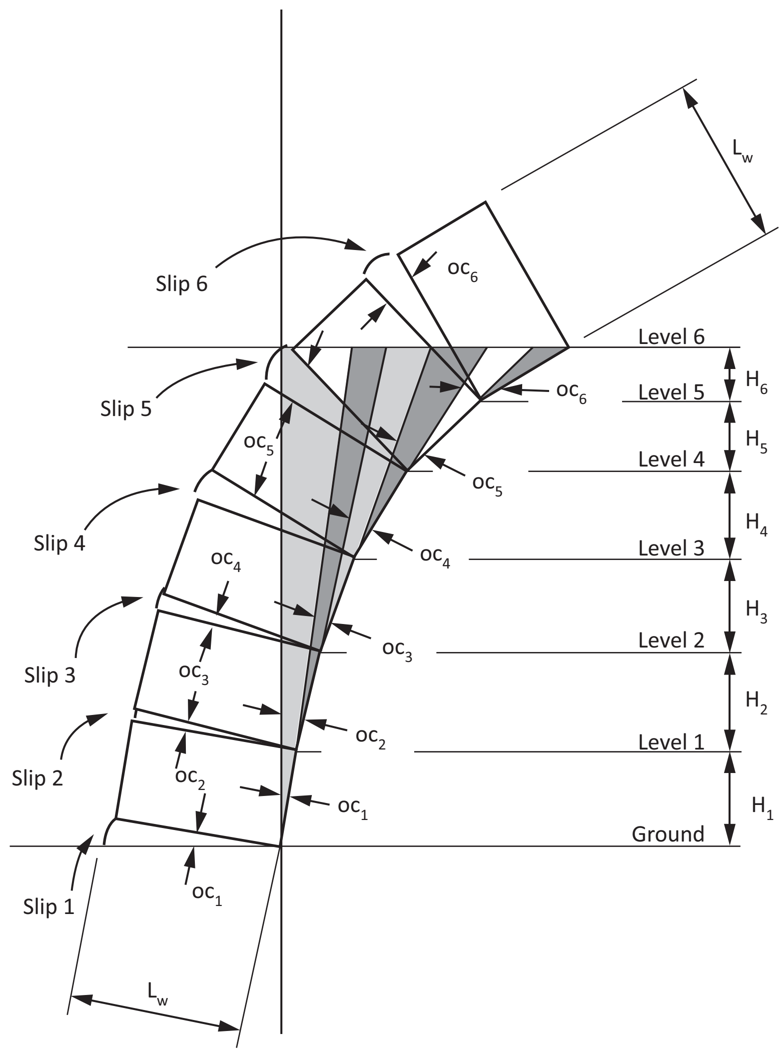



**Figure A.11.7.1E**
**Deflection due to vertical elongation of the wall anchorage system**
(Source: APEGBC Technical and Practice Bulletin)

(b) Deflection of unblocked shearwall segments

The interstorey drift of an unblocked shearwall segment with wood-based panels at the *i*-th storey, may be taken 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 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.*

# A.12 Connections

## A.12.1 Scope

Δ **Table A.12.1**
**Relative density values**

| Visually 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 | | 8 300–11 700 MPa | V2, E1 | 0.42 |
| Northern Species | | | E3 | 0.35 |

**For species of MSR or MEL lumber other than S-P-F, use visually graded lumber values.*
*†The outer laminations of Hem-Fir glulam consist of Douglas Fir-Larch; for this reason, use the relative density of Douglas Fir-Larch for Hem-Fir glulam where the fasteners do not pass through the inner laminations. In all other cases, use the visually graded lumber Hem-Fir density values.*
**Notes:**
**(1)** *Relative density values are specified in this Table on a mean oven-dry mass and volume basis. In the connection equations, these values are modified to relate connection resistances to fifth percentile density values on a 15% moisture 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.*

### Δ A.12.6.6.4 Lateral deformation of lag screw connections

For a specified load, $P$, the lateral deformation of a lag screw connection may be taken as follows:

$$\Delta = \frac{P}{k d_F t_2 n_F}$$

where
$\Delta$ = lateral deformation, mm
$P$ = specified load on the connection, N
$k$ = lateral slip resistance of 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)
$= k_P\ k_Q / (k_P \sin^2\theta + k_Q \cos^2\theta)$ for $0° < \theta < 90°$ (angle-to-grain loading)

where
$G$ = mean relative density (Table A.12.1)
$J_Y$ = side plate factor
= 1.5 for steel side plates
$J_G$ = factor for a group of fasteners (Tables 12.2.2.3.4A and 12.2.2.3.4B)
$J_Q$ = perpendicular-to-grain load factor (Table A.12.6.6.4)
$K_{SF}$ = service condition factor (Table 12.2.1.6)

$d_F$ = lag screw diameter, mm

$t_2$ = length of penetration into main member, mm (Clause 12.6.3)

$n_F$ = number of lag screws in connection

**Note:** *Lag screws loaded laterally tend to reach a point of permanent deformation at about* 0.8 *to* 1.0 mm*. This limit should be avoided to prevent permanent deformation.*

Δ

## Table A.12.6.6.4
## Factor $J_Q$ for perpendicular-to-grain loading of lag screws

| Lag screw 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.12.7.2.3 Analysis of timber rivet (glulam 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 12.7.2.2.*

### A.12.7.2.3.1 Parallel-to-grain loading

In Clause 12.7.2.3, the lateral strength resistance for wood capacity, $p_w$, kN, per connection 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, $P_v$, determined 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 (Table 7.3)

$S_Q$ = rivet spacing perpendicular to grain, mm (Table 12.7.1.7)

$n_R$ = number of rows of rivets

$K_t$ = constant depending on $n_R$ and $n_C$ (Table A.12.7.2.3A)

where

$n_C$ = number of rivets per row

$\beta_t$ = constant depending on $S_p$, $S_Q$, and $n_R$ (Table A.12.7.2.3B)

where

$S_p$ = rivet spacing parallel to grain, mm (Table 12.7.1.7)

$E_t$ = constant depending on $b$ and $L_p$ (Table A.12.7.2.3C)

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

where

$f_v$ = longitudinal shear strength, MPa (Table 7.3)

$C_V = (\beta_1 + \beta_2\beta_3)^{-0.2}$

where

$$\beta_1 = \frac{1}{1300} L_P S_P (n_c - 1)\left[1 + 500 \exp^{-0.52(S_Q - 12.5)}\right]$$

$$\beta_2 = 0.5 \times 10^{-16} L_P \left[(n_c - 1) S_P\right]^3 \left[(n_R - 1) S_Q\right]^{4.5} \exp^{-\left(\frac{a-50}{100}\right)}$$

$$\beta_3 = \left(\frac{3\mu - 1}{2} + L_P \frac{1-\mu}{50}\right)^5 \left(1 - \exp^{\left(1.9 - 0.95\frac{b}{L_p}\right)}\right)^5$$

where

$a$ = end distance (Figure 12.7.1.7), mm

$\mu$ = 43.9 $[(n_C - 1)S_p]^{-0.4}$ $[(n_R - 1)S_Q]^{-0.2}$

$L_p$ = rivet penetration (Figure 12.7.1.1)

$K_v$ = constant depending on $n_R$ and $n_C$ (Table A.12.7.2.3D)

$\beta_v$ = constant depending on $S_p$ and $S_Q$ (Table A.12.7.2.3E)

$Y$ = 90.5 + 5.4 $L_p$

**Note:** *For cases where b* ≥ 175 mm, $S_p$ = 40 mm, $S_Q$ = 25 mm, *and* $L_p$ ≥ 55 mm, $P_v$ *will be greater than* $P_t$.

### A.12.7.2.3.2 Perpendicular-to-grain loading

The lateral strength resistance for wood capacity perpendicular to grain, $q_w$, and the value of $C_t$ (see Clause 12.7.2.5) may be taken 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 (Table 7.3)

$K_{tp}$ = constant depending on $n_R$ and $n_C$ (Table A.12.7.2.3F)

$\beta_{tp}$ = constant depending on $S_p$ and $S_Q$ (Table A.12.7.2.3G)

(b) $$C_t = \frac{1}{\beta_D}\left[\frac{e_p}{(n_c - 1)S_Q}\right]^{-0.2}$$

where

$\beta_D$ = constant depending on $e_p/(n_C - 1)S_Q$ (Table A.12.7.2.3H)

$e_p$ = unloaded edge distance, mm (Figure 12.7.1.7)

**Table A.12.7.2.3A**
**Values of $K_t$**

| Rivets per row, $n_C$ | Number 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 |

## Table A.12.7.2.3B
## Values of $\beta_t$

| $S_p$, mm | $S_Q$, mm | Number 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 |

## Table A.12.7.2.3C
## Values of $E_t$*

| 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, straight line interpolation or the following equation may be used to calculate $E_t$:*

$$E_t = \frac{2\alpha_t \gamma}{L_p}$$

*where*

$\alpha_t$ = 1.0 for $b \geq 6L_p$

$$= 1.0 + 0.155\left(3 - \frac{b}{2L_p}\right)^2 \text{ for } b < 6L_p$$

$\gamma$ = $90.5 + 5.4\,L_p$

# Table A.12.7.2.3D
# Values of $K_v$

| Rivets per row, $n_C$ | Number 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 |

# Table A.12.7.2.3E
# Values of $\beta_v$

| $S_p$, mm | $S_Q$, mm | Number 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 | 1.00 | 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 |

## Table A.12.7.2.3F
## Values of $K_{tp}$

| Rivets per row, $n_C$ | Number 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.12.7.2.3G
## Values of $\beta_{tp}$

| $S_Q$, mm | $S_p$, mm | Number 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 |

**Table A.12.7.2.3H**
**Values of $\beta_D$**

| $(n_c - 1)S_Q$ | $\beta_D$ | $(n_c - 1)S_Q$ | $\beta_D$ |
|---|---|---|---|
| 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, $\beta_D$ is given to three significant digits due to the sensitivity in this range.*

## A.12.9.3.2 Lateral deformation of nailed and spiked wood-to-wood connections

For specified loads, $P$, not greater than $n_u/3$, the lateral deformation of nailed and spiked connections may be estimated as follows:

$$\Delta = 0.5 d_F K_m \left(\frac{P}{n_u}\right)^{1.7}$$

where

$\Delta$ = lateral deformation, mm

$d_F$ = nail diameter, mm

$K_m$ = service creep factor (Table A.12.9.3.2)

$P$ = specified load per nail or spike, N

$n_u$ = unit lateral strength resistance, N (Clause 12.9.4.1)

**Table A.12.9.3.2**
**Service creep factor, $K_m$, for nail and spike connections**

| | Moisture condition | | |
|---|---|---|---|
| **Load duration class** | **Nailed dry, loaded dry** | **Nailed wet, loaded dry** | **Nailed wet, loaded wet** |
| Long term | 1.5 | 2.0 | 3.0 |
| Standard | 1.2 | 1.5 | 2.0 |
| Short | 1.0 | 1.2 | 1.5 |

## Table A.12.9.5.2
## Nail and spike characteristics

| Nail characteristics | | | |
|---|---|---|---|
| **Type** | **Length, in** | **Gauge, No.** | **Diameter,* mm** |
| Common wire nails | 1 | 15 | 1.83 |
| | 1-1/8 | 15 | 1.83 |
| | 1-1/4 | 14 | 2.03 |
| | 1-1/2 | 13 | 2.34 |
| | 1-3/4 | 12 | 2.64 |
| | 2 | 11-3/4 | 2.84 |
| | 2-1/4 | 11 | 2.95 |
| | 2-1/2 | 10 | 3.25 |
| | 2-3/4 | 10 | 3.25 |
| | 3 | 9 | 3.66 |
| | 3-1/4 | 9 | 3.66 |
| | 3-1/2 | 8 | 4.06 |
| | 4 | 6 | 4.88 |
| | 4-1/2 | 5 | 5.38 |
| | 5 | 4 | 5.89 |
| | 5-1/2 | 3 | 6.40 |
| | 6 | 2 | 7.01 |
| Common spikes | 4 | 3 | 6.40 |
| | 6 | 1 | 7.62 |
| | 7 | 1 | 7.62 |
| | 8 | 0 | 8.23 |
| | 9 | 0 | 8.23 |
| | 10 | 0 | 8.84 |
| | 12 | 0 | 8.84 |
| Common spiral nails | 2-1/2 | 10-1/2 | 2.77 |
| | 3 | 9-3/4 | 3.10 |
| | 3-1/4 | 9-3/4 | 3.10 |
| | 3-1/2 | 8 | 3.86 |
| | 4 | 7 | 4.33 |
| | 5 | 5 | 4.88 |

**In this table, "diameter" for spiral nails is the effective diameter based on the projected lateral width.*

# A.16 Proprietary structural products — Materials and evaluation

## A.16.2.3.1 Length-adjustment factor, $K_{LN}$, used for determining moment resistance of prefabricated wood I-joists

The length-adjustment factor is used by manufacturers to develop proprietary I-joist moment resistance values and is not intended as an adjustment factor for specific applications. $K_{LN}$ is taken from Section 6.3.1.5 of ASTM D5055 and is the lesser of 1.0 or the value calculated as follows:

$$K_{LN} = (\text{stress distribution adjustment factor})\left(\frac{L_1}{L}\right)^Z \leq 1.0$$

where

stress distribution adjustment factor

= the adjustment of specified strength parallel to grain, $f_a$, 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

$L$ = nominal I-joist span

= 18 times the joist depth

$Z$ = a coefficient (see Table A.16.2.3.1) that accounts for the variation in tensile strength of the flange material

**Table A.16.2.3.1**
***Z* for calculating the length-adjustment factor for prefabricated wood I-joists**

| 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 graded lumber.*

*‡Interpolation between tabular values may be used.*

# 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.*

Δ **(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.

### Δ 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* (NBC), 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 particular duration of fire exposure provided the reduced structural resistance of the element, after the specified exposure time, is greater than the specified load effects.

### Δ 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 loadbearing 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 actual 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 standard CAN/ULC-S101 fire-resistance test, additional analysis may be required. For example, a heat transfer analysis may be needed in order 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 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 *(NBC).*

### ∆ 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 6.5.6.4 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 massive member and in some cases the connections 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 plank decking and timber, Clause 7 for glued-laminated timber, Clause 8 for CLT and Clause 15.3 for structural composite lumber (SCL).

### B.2.2 Glued-laminated timber
When determining the fire resistance of glued-laminated timber bending elements as specified in Clause 7, it is important to use the proper bending strength. In order to use the specified strengths provided in Table 7.3, bending members rated as providing a 1-hour fire-resistance rating shall be manufactured to the specified layup except that a core lamination shall be removed, the tension zone moved inward, and the equivalent of one additional 38 mm (2 in. nominal) thick outer tension lamination added on the tension side for beams subjected to a positive bending moment and both sides (tension and compression) for beams subjected to both positive and negative bending moments. Bending members rated as providing a 1.5- or 2-hour fire-resistance rating shall be manufactured to the specified layup except that two core laminations shall be removed, the tension zone moved inward, and the equivalent of two additional 38 mm (2 in. nominal) thick outer tension lamination added on the tension side for beams subjected to a positive bending moment and both sides (tension and compression) for beams subjected to both positive and negative bending moments.

When the additional tension laminations are not substituted for core laminations, the actual material strength for the individual laminations under consideration shall be used.

### ∆ B.2.3 CLT
When determining the axial or moment resistance of CLT panels exposed to fire in accordance with Clause 8, 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.*

## ∆ B.3 Modification factors

### B.3.1 General
Modification factors in Clauses B.3.2 to B.3.9 shall be used in calculating the factored resistances of the reduced cross-section after the fire exposure duration using the appropriate equations, as specified in Clauses 6, 7, 8, and 15.

### B.3.2 Resistance factor, $\phi$
The resistance factor, $\phi$ shall be taken as 1.0.

### B.3.3 Load duration factor, $K_D$
The specified strengths shall be multiplied by a load duration factor, $K_D$, for short term load duration in accordance with Clause 5.3.2.

### B.3.4 System factor, $K_H$

The specified strengths shall be multiplied by a system factor, $K_H$, equal to 1.0.

### B.3.5 Size factor, $K_Z$

The specified strengths shall be multiplied, where applicable, by the size factor, $K_Z$, in accordance with Clauses 6.4.5, 7.5.6, 7.5.8, 7.5.9, 15.3.2.5, 15.3.2.6, and 15.3.3, based on the original cross-section dimensions.

### B.3.6 Lateral stability factor, $K_L$

The lateral stability factor, $K_L$, in accordance with Clauses 6.5.4.2 and 7.5.6.4 shall be determined based on the reduced cross-section after the fire exposure duration. In order to rely on intermediate lateral support, the members providing the 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 based on the reduced cross-section in accordance with Clause B.6.2. In order to rely on intermediate lateral support, the members providing the support shall have at least the same fire-resistance rating as that sought for the column.

### Δ B.3.8 Slenderness factor, $K_C$

The slenderness factor, $K_C$, shall be determined based on the reduced cross-section in accordance with Clause B.6.2.

### Δ B.3.9 Adjustment factor for fire resistance, $K_{fi}$

The specified strengths shall be multiplied by the adjustment factor for fire resistance, $K_{fi}$, as specified in Table B.3.9.

Δ **Table B.3.9**
**Adjustment factor for fire resistance, $K_{fi}$**

| Product | $K_{fi}$ |
|---|---|
| Timber and plank decking | 1.5 |
| Glued-laminated timber | 1.35 |
| Structural composite lumber | 1.25 |
| Cross-laminated timber<br>– V1-V2 stress grade<br>– E1-E3 stress grade | <br>1.5<br>1.25 |

**Note:** *Adjustment factor for fire resistance, $K_{fi}$, converts specified strength to mean strength.*

## Δ B.4 Char depth

### B.4.1 General

The charring rates in Table B.4.2 apply when wood is exposed to the standard fire exposure in accordance with CAN/ULC-S101. Clause B.4.3 shall be used to determine the char depth where heat transfer is one-dimensional, such as in solid wood walls and floors, or when corner rounding is explicitly considered for large rectangular cross-sections following Clause B.4.5. For CLT, the char depth shall be determined in accordance with Clause B.4.6. For all other cases, the char 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 in particular, corner rounding, which is not included in the method in Clause B.4.3.*

## Δ B.4.2 Design charring rates

The design charring rates, $\beta_o$ and $\beta_n$ , for wood and wood-based products are listed in Table B.4.2.

### Δ Table B.4.2 Design charring rates for wood and wood-based products, mm/min

| | $\beta_o$ | $\beta_n$ |
|---|---|---|
| Timber and plank decking | 0.65 | 0.80 |
| Glued-laminated timber | 0.65 | 0.70 |
| Structural Composite Lumber | 0.65* | 0.70* |
| Cross-laminated timber | 0.65 | 0.80 |

**Values are only applicable to wood-based structural composite lumber products.*

The charring rates in Table B.4.2 apply to cross-sections of wood products provided the residual minimum dimension is greater than 70 mm when heated on opposite sides.

**Note:** *When the residual minimum dimension of members heated on the opposite 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 residual cross-section to use.*

## B.4.3 One-dimensional char depth

The char depth for one-dimensional charring, $x_{c,o}$ (mm), shall be taken as follows:

$$x_{c,o} = \beta_o t$$

where

$\beta_o$ = one-dimensional charring rate, mm/min

$t$ = fire exposure duration, min

## B.4.4 Notional char depth

The char depth for notional charring, $x_{c,n}$ (mm), shall be taken as follows:

$$x_{c,n} = \beta_n t$$

where

$\beta_n$ = 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 separately when the char depth is determined in accordance with Clause B.4.4.*

## Δ B.4.6 Char depth for CLT

The one-dimensional charring rate ($\beta_0$) specified in Table B.4.2 and Clause B.4.3 shall be used when the char 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 depth surpasses the first bond line during the fire resistance period sought, the notional charring rate ($\beta_n$) specified in Table B.4.2 and Clause B.4.4 shall be used.

# B.5 Zero-strength layer

## Δ B.5.1 Zero-strength layer depth

A zero strength layer shall further reduce the cross-section (beyond the char depth determined in Clause B.4.3 or B.4.4) in order to account for a reduction in strength of the heated wood beyond the char front. This additional reduction in cross-section, $x_t$ (mm), depends on the fire exposure duration and shall be taken as follows:

$$x_t = \left(\frac{t}{20}\right) \times 7 \text{ (for } t < 20\text{)}$$

$$x_t = 7 \text{ (for } t \geq 20\text{)}$$

where
$t$ = fire exposure duration, min

# B.6 Resistance of reduced cross-section

## Δ B.6.1 General

Charring of all surfaces of an element directly exposed to fire shall be taken into account and, where relevant, any surfaces initially protected from fire exposure where wood charring could occur at some point during the required fire exposure duration. The reduced 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:** *Surfaces protected from fire exposure for the duration of the exposure can include the top surface of a beam protected by a fire-resistance-rated floor or roof assembly and may be considered to be protected from the fire.*

## Δ B.6.2 Reduced cross-section

The residual cross-section shall be the initial cross-section reduced by the char depth determined in accordance with Clause B.4 and the zero-strength layer depth determined in accordance with Clause B.5 on each exposed surface. Section properties shall be calculated using standard equations for area, section modulus, and moment of inertia using the reduced cross-sectional dimensions.

**Note:** *In some instances, such as the reduced cross-section of CLT, the location of the neutral axis must be determined prior to calculating the moment of inertia and section modulus.*

## Δ B.6.3 Modification of specified strengths

The specified strengths used in calculating the resistance of the member shall be multiplied by the adjustment factor for fire resistance, $K_{fi}$ , as specified in Table B.3.9. All other modification factors used in calculating the member resistance shall be taken as that used for design under ambient conditions with the exception of those listed in Clause B.3.

## Δ 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 particular duration of fire exposure if the reduced structural resistance of the element, after the specified exposure time, is greater than the specified gravity load effects.

## Δ 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;
(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.

### 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 off-set 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 elements to the wood structural elements 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 off-set 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 reduced cross-section of the wood element shall be considered to be properly protected.

**Note:** *Additional information can be found in the American Wood Council's Technical Report 10 and in Eurocode 5: Part 1-2.*

## Δ B.10 Plank decking

### Δ B.10.1 Double tongue-and-groove
Double tongued-and-grooved plank decking shall be designed as an assembly of wood beams fully exposed on one face.

### Δ B.10.2 Single tongue-and-groove or internal spline
Single tongued-and-grooved or single internal splined plank decking shall be designed as an assembly of wood beams fully exposed on one face, provided the loss in depth of the reduced cross-section has not yet reached the tongue-and-groove or internal spline.

### Δ B.10.3 Butt-jointed
Butt-jointed plank decking 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.

### Δ 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 plank decking protected by:
(a) tongued-and-grooved wood flooring not less than 19 mm thick laid crosswise or diagonally;
(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 elements 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.*

## Δ B.11 Fire performance criteria for evaluating separating function of building elements

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, shall be considered when required by the applicable building code.

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