Steel Wood Course Hybrid
Module 2: Design of Timber

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Hybrid Course Outline

Week	Planned Content
1	Module 1 Introduction – Limit states
2	Module 2 Tension
3	Module 3 Compression
4	Module 4 Corrosion and Software
5	Module 6 Beams
6	Module 7 Beam Columns and Plate Girders
7	Module 7 Steel Connections
8	Module 8 Composite Steel Design
9	Module 1a Introduction to Timber
10	Module 2a Design of Timber 2

This module series can be used with 3rd/4th year steel design providing these listed topics are discussed.

The module series takes approximately 4 50 minute lectures. It is based on the construct of :

***Chorlton, B., Mazur, N** and Gales, J. (June 2019) Incorporating Timber Education into Existing Accredited Engineering Programs. 10th Canadian Engineering Education Association's Annual Conference., Ottawa, Canada. 8pp.*

This second module is not intended to be included with second year curriculum as it requires knowledge of limit states.

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Typical Timber Undergraduate Course Outline

Introduction
- Wood as a green building material
- History of wood structures

Physical and mechanical properties of wood
- Molecular and cell structure
- Physical properties
- Mechanical properties

Structural wood products & structural forms

Strength and modification factors
- Specified strength of wood, size, use, species and grades
- Modification Factors
- Shrinkage calculation
- Modification factors

Fire safety

Design Process
- Limit States Design – Ultimate & Serviceability Limit States

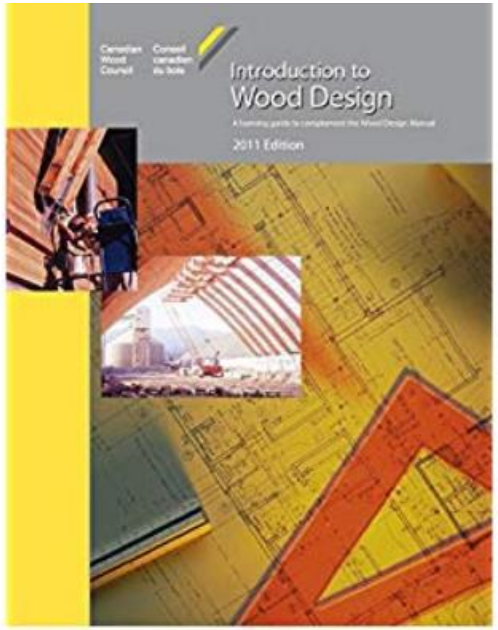
Design of Tension Members

Design of Compression Members

Design of Bending Members

Combined bending and axial load

Connections



Purple we will cover in Module 2.
Blue is covered elsewhere in Steel Design.
Red covered elsewhere in Module 1.

Diaphragms are not covered herein

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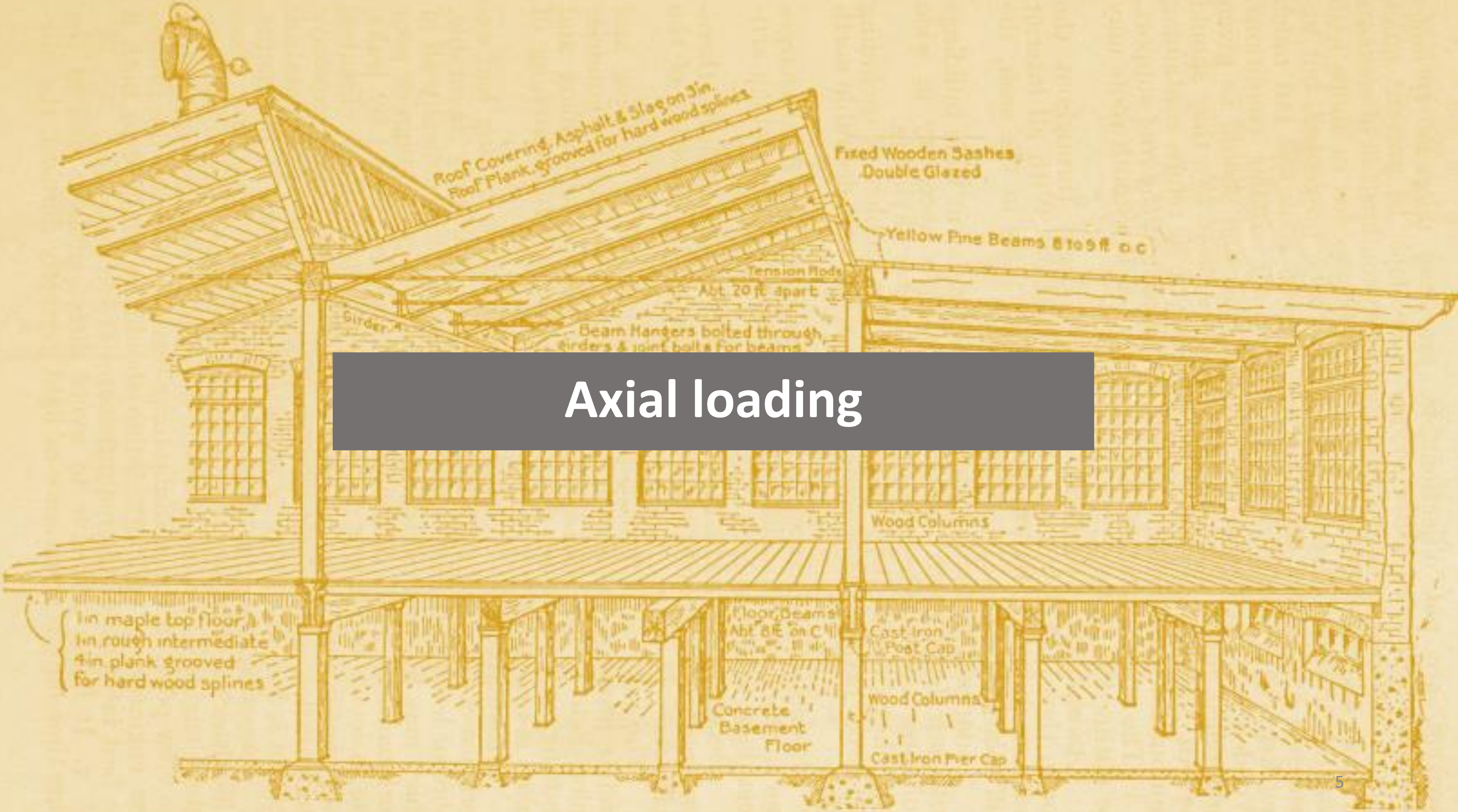
Axial loading



Sawn Lumber (Tension)

Tensile resistance parallel to the grain can be taken as:

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

Where:

$\phi = 0.9$

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

f_t = specified strength in tension parallel to grain, MPa (CSA O86 Tables 6.3.1A, 6.3.1B, 6.3.1C, 6.3.1D, 6.3.2 and 6.3.3)

A_n = net area of cross section, mm^2

K_{Zt} = size factor in tension (CSA O86 Table 6.4.5)

Recall regarding modification and specified strength factors :

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	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	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.

Glulam Members (Tension)

The factored tensile resistance , Tr, parallel grain is calculated as the lesser of :

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

or

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

Where:

$\phi = 0.9$

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

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

f_{tn} = specified strength in tension parallel to grain at the net section, MPa (CSA O86 Table 7.3)

f_{tg} = specified strength in tension parallel to grain at the gross section, MPa (CSA O86 Table 7.3)

A_n = net area of cross section, mm^2

A_g = gross area of cross section, mm^2

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

Sawn lumber (Compression)

Not as simple as timber. Slenderness factor, Kc, is used to relate the slenderness to load capacity. It is based on a cubic expression (Rankine-Gordon). Reliability is set to 0.8 is based on a nominal load eccentricity of 5% of the member dimension in the direction of buckling (averaged). Slenderness ratio Cc, is restricted to a max of 50 (if violated add supports or increase size).

$$P_{rd} = \phi F_c A K_{Zcd} K_{Cd}$$

$$P_{rb} = \phi F_c A K_{Zcb} K_{Cb}$$

where

ϕ = 0.8

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

f_c = specified strength in compression parallel to grain, MPa (CSA O86 Tables 6.3.1A, 6.3.1B, 6.3.1C, 6.3.1D, 6.3.2 and 6.3.3)

A = area of the cross-section, mm^2

K_{Zc} = size factor

$K_{Zcd} = 6.3(dL_d)^{-0.13} \le 1.3$ for buckling in direction of d

$K_{Zcb} = 6.3(bL_b)^{-0.13} \le 1.3$ for buckling in direction of b

K_C = slenderness factor

$$K_{Cd} = \left[1.0 + \frac{F_C K_{ZCd} C_{Cd}^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

for buckling in direction of d

$$K_{Cb} = \left[1.0 + \frac{F_C K_{ZCb} C_{Cb}^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

for buckling in direction of b

where

$$C_{cd} = \frac{K_e L_d}{d}$$

$$C_{cb} = \frac{K_e L_b}{b}$$

K_e = effective length factor given in CSA O86 Table A6.5.6.1

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

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

= 0.82 E for MSR lumber

= 0.75 E for MEL lumber

Glulam (compression)

For cases where compression is alone, 16c-E Douglas Fir or and 12c-E in spruce pine is recommended. Where combined with bending effects. It is generally most economical to use bending grades in either of the above group combinations.

The wood design manual contains column selection tables that assist in determining appropriate column sections.

Factored compressive resistance parallel to grain, P_r, may be calculated as

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

where

ϕ = 0.8

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

f_c = specified strength in compression parallel to grain, MPa (CSA O86 Table 7.3)

A = cross-sectional area, mm^2

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

Z = member volume, m^3

K_C = slenderness factor

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

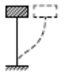
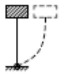
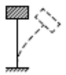
where

E_{05} = modulus of elasticity for design of glulam columns, defined as $E_{05} = 0.87\,E$ (the value used for modulus of elasticity for design of long columns is taken as a 5th percentile value because compressive resistance of a column is an ultimate limit state)

Effective length

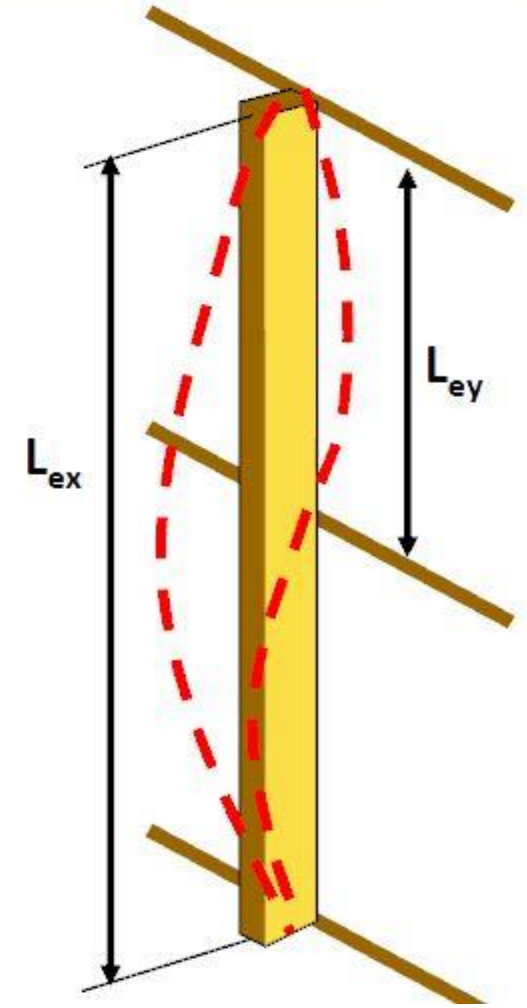
TABLE 8.1

K_e factor for columns (from CSA O86 Table A6.5.6.1)

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, 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 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 nor 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 should be measured from the outer surface of the base or cap plate. The effective length factor K_e is required to 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 should be used.



Considerations for CLT (compression walls)

CLT panels are more commonly being used for wall applications to resist axial loads in the plane of the panel. The contribution of the orientation of the layers of the panel are very important as most contribution comes from those parallel to applied load. Those perpendicular are generally neglected (see CL 8.2.4). The effective thickness is proportional to the effective radius of gyration (calculated only for contributing planes). Design is similar to sawn lumber, in that slenderness effects are still at play but calculated differently however these are limited now to 43 and not 50 as before.

Considerations for CLT (compression walls)

The effective wall thickness for a CLT panel is equivalent to $\sqrt{12}r_{eff}$ since CSA 086 requires layers in CLT walls with laminations oriented perpendicular to the applied axial load to be ignored. The ratio $L_e/\ r_{eff}$ is limited to less than 150 and therefore, the slenderness ratio is limited to approximately 43.

The size factor K_{Zc} is calculated similar to sawn lumber except the effective wall thickness $\sqrt{12}r_{eff}$ is substituted for the member thickness because the layers with laminations oriented perpendicular to the axial load are neglected. Slenderness factor K_C is also based on the same calculation approach for sawn lumber.

The factored compressive resistance of CLT panels under axial load is calculated as:

$$P_r = \phi F_c A_{eff} K_{Zc} K_C$$

where

ϕ = 0.8

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

f_c = specified strength in compression parallel to grain of the laminations oriented parallel to the axial load, MPa (CSA O86 Table 8.2.4)

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

K_{Zc} = size factor for compression

$$= 6.3\left(\sqrt{12}\, r_{eff}\, L\right)^{-0.13} \leq 1.3$$

Considerations for CLT (compression walls)

The factored compressive resistance of CLT panels under axial load is calculated as:

$$P_r = \phi F_c A_{eff} K_{Zc} K_C$$

where

K_C = slenderness factor

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

where

E_{05} = modulus of elasticity for design of compression members, only for the laminations oriented parallel to the axial load, MPa (CSA O86 Clause 8.2.4)

$$C_c = \frac{L_e}{\sqrt{12}\, r_{eff}}$$

where

L_e = effective length

$= K_e L$

K_e = effective length factor for compression members (Table 8.1)

L = height of the panel, mm

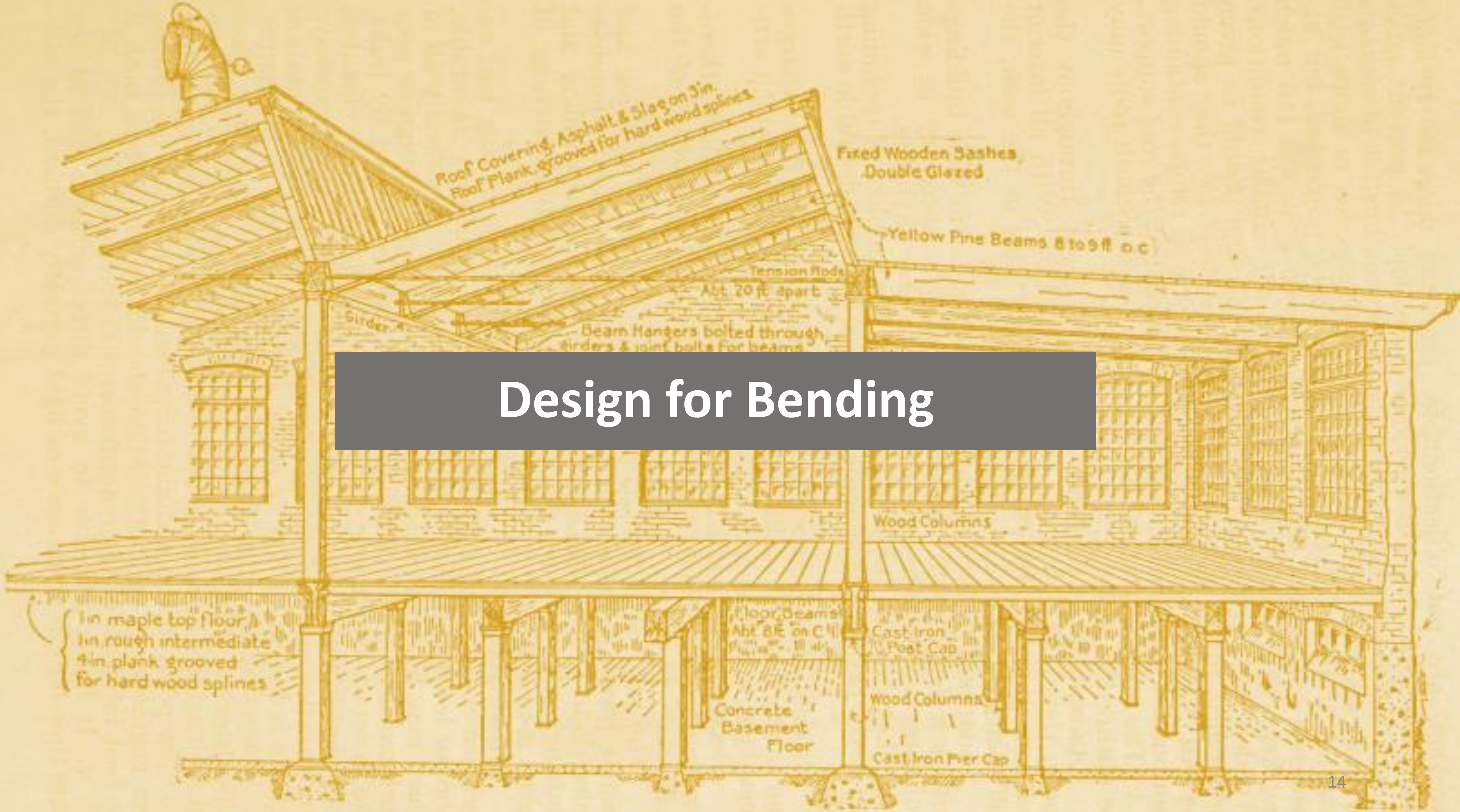
r_{eff} = effective radius of gyration, mm

$$= \sqrt{\frac{I_{eff}}{A_{eff}}}$$

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

Design for Bending



Sawn Lumber / Glulam / CLT

Governed through six limit states which require consideration. These are

- Moment (bending) resistance (ultimate)
- Shear resistance (Ultimate)
- Bearing with compression perpendicular to the grain (Ultimate) – refer to Module 6 for details on how to consider this.
- Deflection (service)
- Vibration (service)

Bending theory

Bending about the principal axis

$\sigma = M\,y\,/\,I$

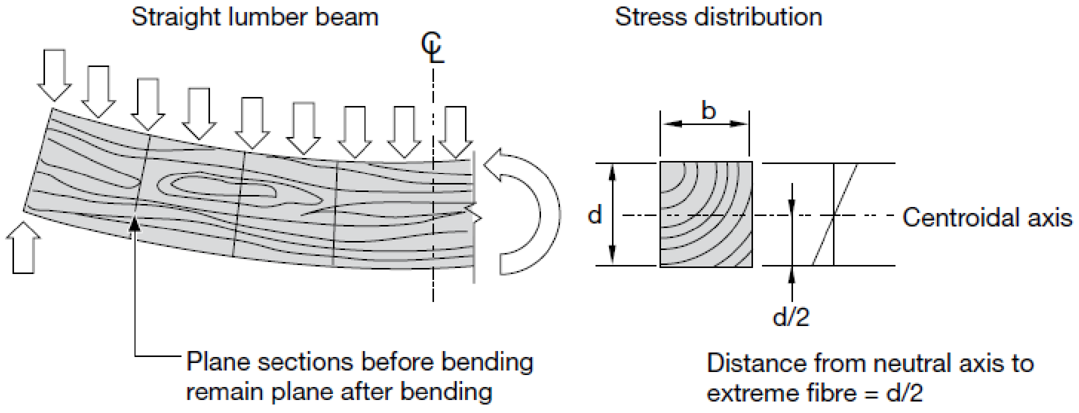
where

σ = unit bending stress at a given fibre

M = bending moment at the section

y = distance of the given fibre from the neutral axis

I = moment of inertia of the section about the neutral axis



where S = section modulus = I / y and

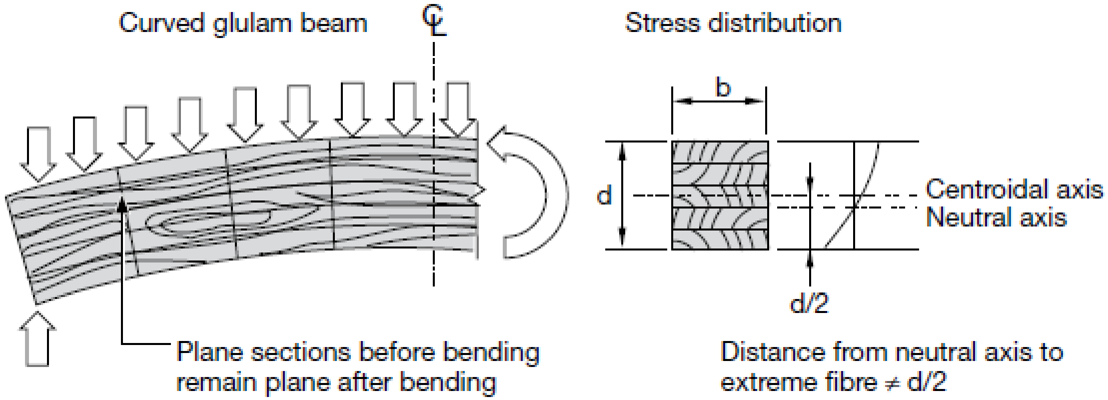
$$M = \sigma_{\max} S$$

Bending members are subjected to out-of-plane loading applied perpendicular to the longitudinal axis of beam or plane of panel

Applications:

- Joists, beams and purlins
- Sheathing and decking

Bending theory



Similar to sawn lumber, glulam bending stress is assumed to be proportional to strain and is directly proportional to the distance of the neutral axis

Sawn Lumber (bending resistance)

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

where

$\phi = 0.9$

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

f_b = specified strength in bending, MPa (CSA O86 Tables 6.3.1A, 6.3.1B, 6.3.1C, 6.3.1D, 6.3.2 and 6.3.3)

K_D = load duration factor (CSA O86 Table 5.3.2.2)

K_H = system factor (CSA O86 Table 6.4.4)

K_{Sb} = service condition factor in bending (CSA O86 Table 6.4.2)

K_T = treatment factor (CSA O86 Table 6.4.3)

S = section modulus = $\left[\frac{b\,d^2}{6}\right]$ mm^3

K_{Zb} = size factor in bending (CSA O86 Table 6.4.5)

K_L = lateral stability factor

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}	Compression perpendicular to grain, K_{Zcp}	Compression parallel to grain, K_{Zc}	All other properties
				All	All	All	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

While considering bending resistance, lateral stability also needs consideration (buckling)

Lateral stability explanation

$$M_{crit} = \frac{\pi}{L_e}\sqrt{EI_y\,G\,J}$$

where
EI_y = bending stiffness in the lateral direction
GJ = St. Venant torsional rigidity
L_e = effective length

The effective length depends on the unsupported length of the beam and the loading and support conditions. The unsupported length is the length between those supports that prevent the beam from buckling. (the supports which keep the compression fibres restrained from lateral movement

Sawn Lumber (Shear)

$$V_r = \phi F_v \left[\frac{2A_n}{3} \right] K_{Zv}$$

where

ϕ = 0.9

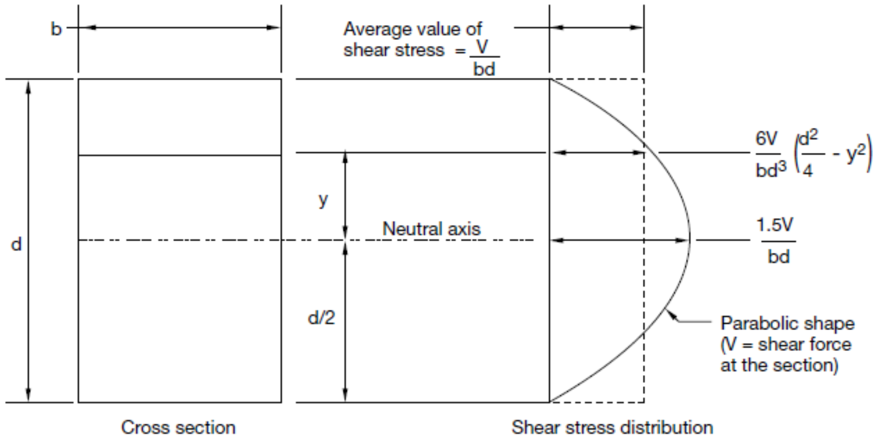
$F_v = f_v (K_D K_H K_{Sv} K_T)$

f_v = specified strength in shear, MPa (CSA O86 Tables 6.3.1A, 6.3.1B, 6.3.1C, and 6.3.1D
Note: For MSR and MEL lumber, specified strengths in shear are not grade-dependent and are taken from Table 6.3.1A for the appropriate species.)

A_n = net area of cross section, mm^2

K_{Zv} = size factor in shear (CSA O86 Table 6.4.5)

Loads are considered to be distributed down through the member in compression perpendicular to the grain. In this case longitudinal shear stress is parabolically distributed over the depth of the member. At neutral axis, the shear stress is 1.5x the average shear stress.



Effect of all loads acting within a distance from a support equal to depth of member shall be ignored.

Glulam (bending resistance)

The factored bending moment resistance, M_r, of glued-laminated timber members is the lesser of M_{r1} and M_{r2} calculated as:

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

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

where

ϕ = 0.9

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

f_b = specified strength in bending, MPa (CSA O86 Table 7.3)

K_{Zbg} = size factor for glulam beams

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

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

d = beam depth

L = length of the glulam beam from point of zero moment to zero moment

K_L = lateral stability factor

K_X = curvature factor (K_X = 1.0 for straight members; and $K_X \leq 1$ for curved member)

S = section modulus

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.*

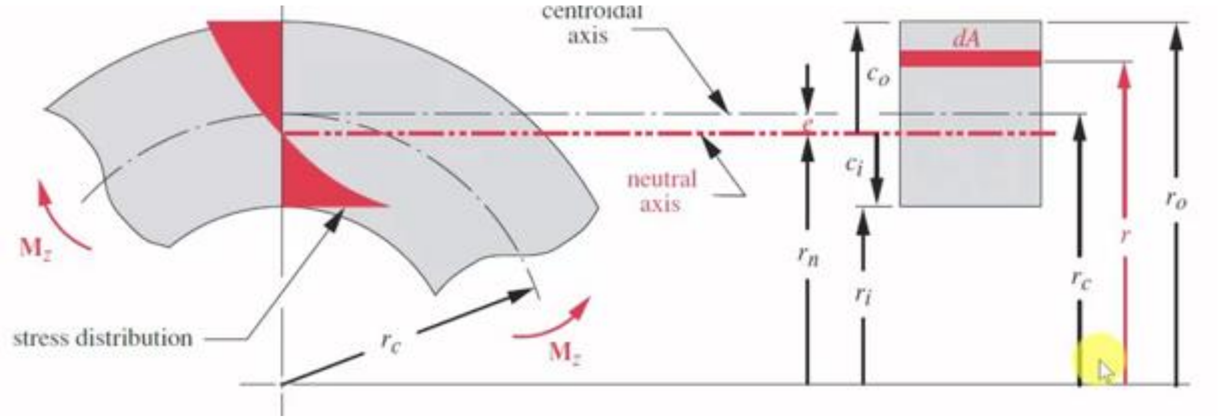
While considering bending resistance, lateral stability also needs consideration (buckling)

Assumptions for treating as unity are the same for sawn lumber, but if outside these bounds direct calculation is required

For curvature, refer to Kx in 7.5.6.5.2

Glulam (Curvature effect-Kx)

K_X is required to account for the shifting of neutral axis, creating a larger extreme fibre stress than when the NA is located at the centroid for straight beam.



Pearson, 2011

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.*

Glulam (Curvature effect-Kx)

Glulam (lateral stability - slenderness ratio)

7.5.6.4 Calculation of lateral stability factor, K_L

7.5.6.4.1 Unsupported length, ℓ_u

When no additional intermediate support is provided, the unsupported length, ℓ_u, shall be the distance between points of bearing or the length of the cantilever. When intermediate support is provided by purlins so connected that they prevent lateral displacement of the compressive edge of the bending member, the unsupported length shall be taken as the maximum purlin spacing, a (see Table 7.5.6.4.3).

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

Glulam (lateral stability)

Recall:

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

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

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

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

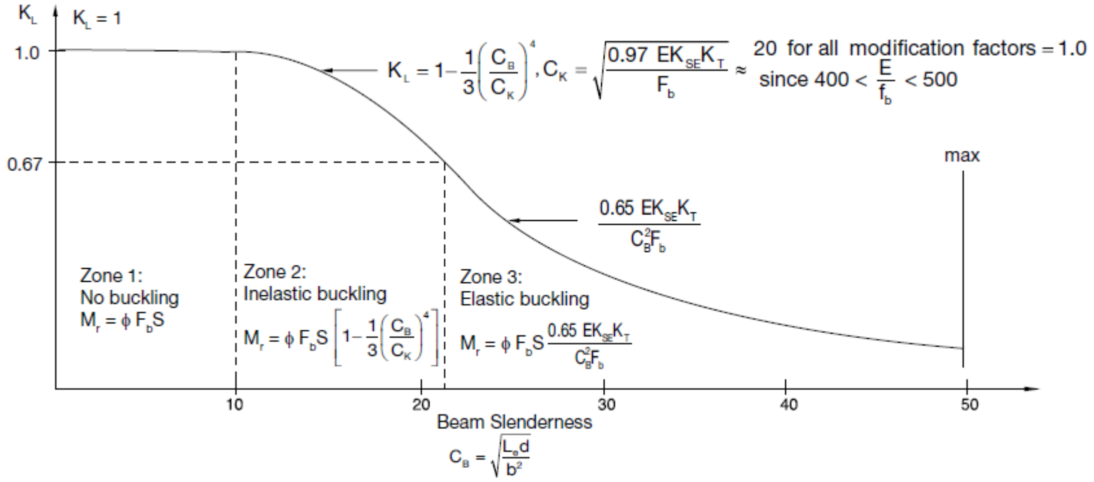
where

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

where

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

K_X = curvature factor (Clause 7.5.6.5.2)



Glulam (Shear- no notch)

Simplified approach (*Volume <2m³*)

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

where

ϕ = 0.9

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

f_v = specified strength in shear (CSA O86 Table 7.3), MPa

$A_g = b \times d$

= gross cross-sectional area, mm^2

Tables 7.5.7.5 a-e give direct values for Cv. If the loading condition is not in these tables. This value needs to be calculated directly

Advanced approach (*Volume > 2m³ or simplified method does not satisfy required resistance*)

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 m³ in volume, the factored shear resistance may be calculated using the equation in Item (b).*

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

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

where

ϕ = 0.9

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

where

f_v = specified strength in shear, MPa (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).*

Glulam (Shear- no notch)

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

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

Glulam (Shear- no notch)

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

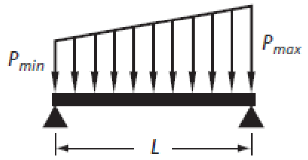
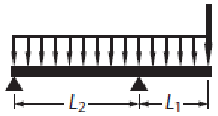
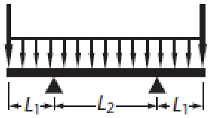
Type of loading	P_{min}/P_{max} 0.0	0.2	0.4	0.6	0.8	1.0
	3.40	3.55	3.63	3.67	3.69	3.69

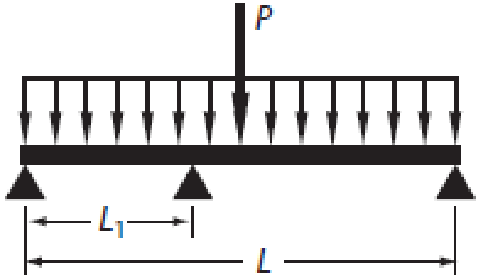
Table 7.5.7.5C
Shear load coefficient, C_V, for cantilevered beams

Beam type and loading	L_1/L_2	r^* 0.0	0.5	2.0	10.0 and over
 $L = L_1 + L_2$	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
 $L = L_2 + 2L_1$	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

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

Glulam (Shear- no notch)

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

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

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

Glulam (Shear- no notch)

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, ℓ_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:

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

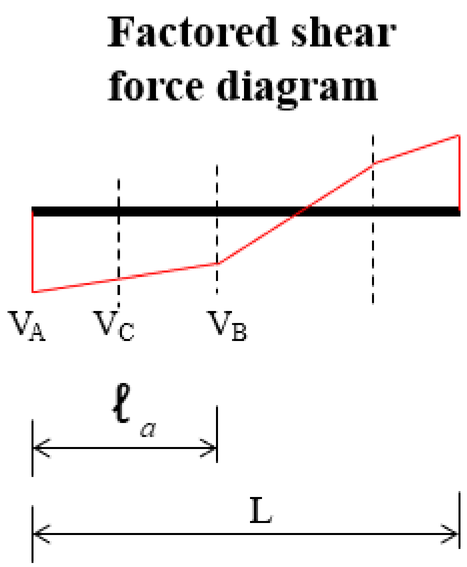
where
W_f = the total of all factored loads applied to the beam, N

- (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

The procedure involves a number of steps and requires prior determination of the shear force diagram of the beam



Glulam (Shear- with notch)

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)

A_n = $b(d - d_n)$ = net cross-sectional area of member, mm² (Clause 7.5.4)
A_g = $b \times d$ = gross cross-sectional area of member, mm²
b = member width, mm
d = member depth, mm
d_n = notch depth, mm (which shall not exceed $0.25d$)
e_c = length of notch, mm, from inner edge of closest support to farthest edge of notch

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.

Glulam (Shear- with notch)

7.5.7.4.1 Longitudinal shear resistance of residual member above notch

Tension side notches not exceeding 0.25*d* 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.

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

ϕ = 0.9

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

where

f_f = specified fracture shear strength at a notch, MPa

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

where

b_{eff} = effective lamination width (mm)

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

K_{Sf} = service condition factor for fracture shear

K_T = treatment factor

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

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

$\alpha = 1 - (d_n/d)$

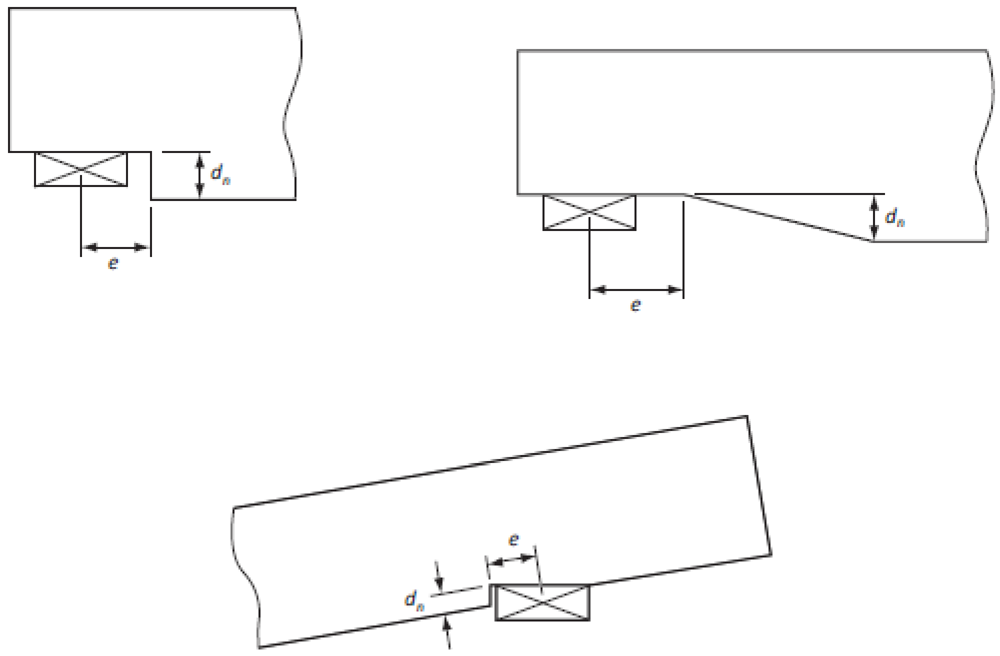
where

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)



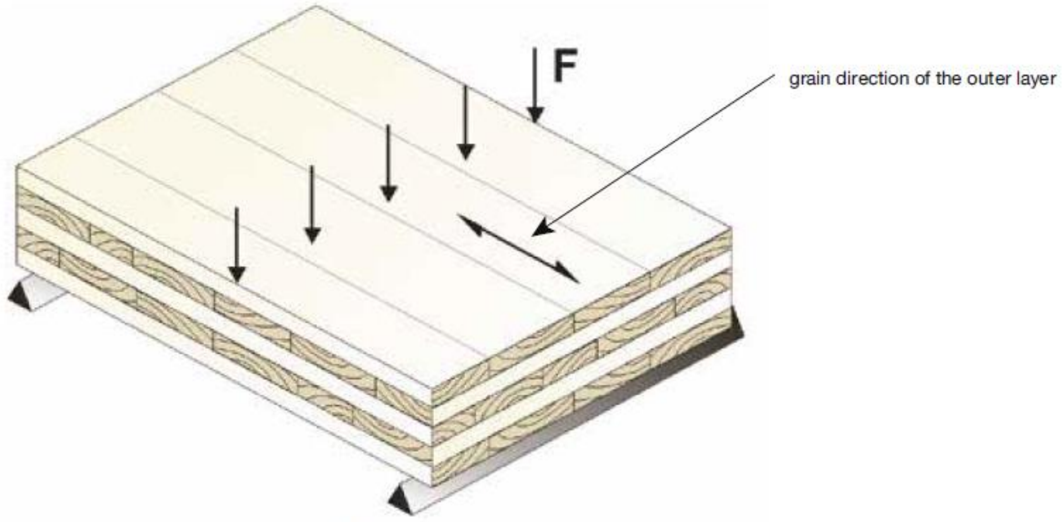
Legend:

d_n = depth of notch

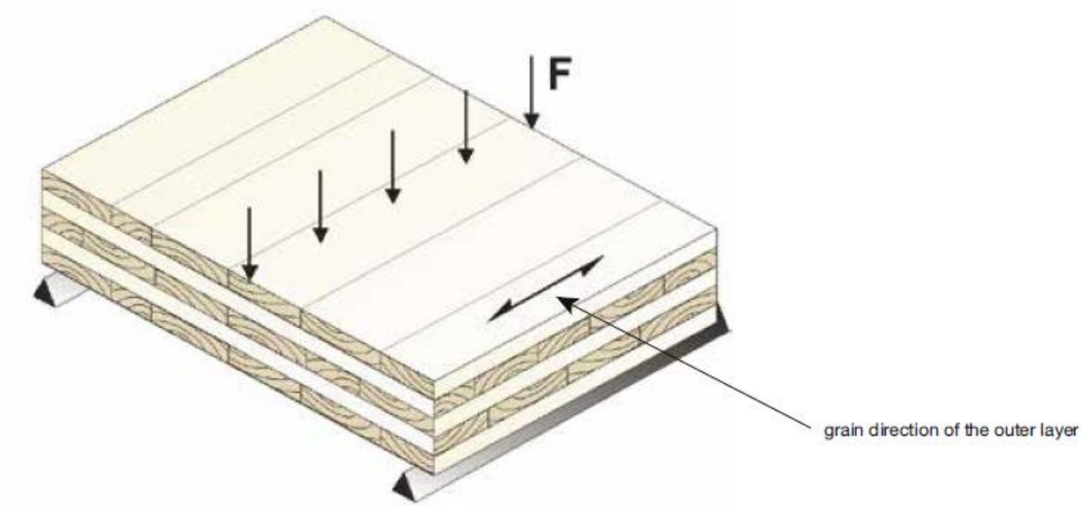
e = length of notch

Figure 6.5.5.3.2
Determination of length and depth of notch

Cross Laminated Timber



Bending in major strength direction

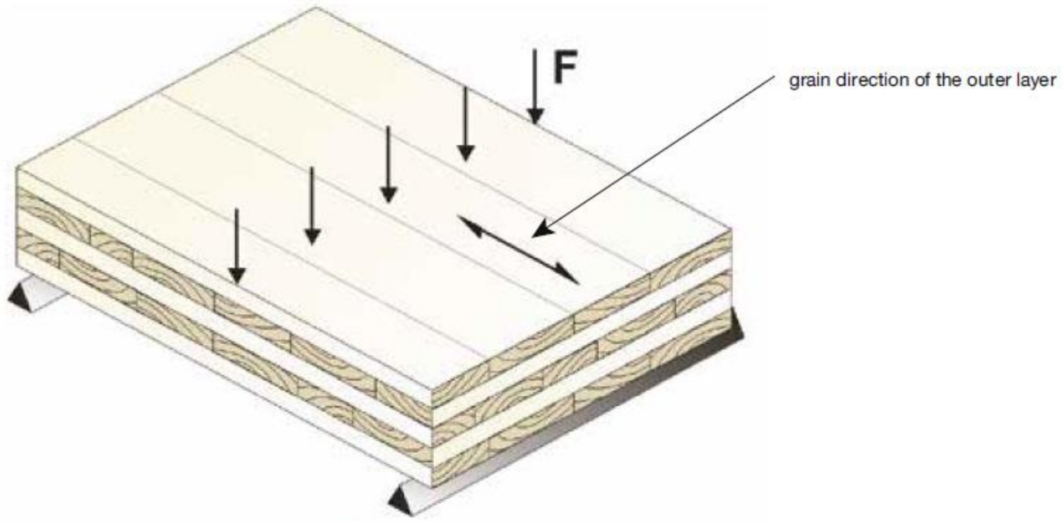


Bending in minor strength direction

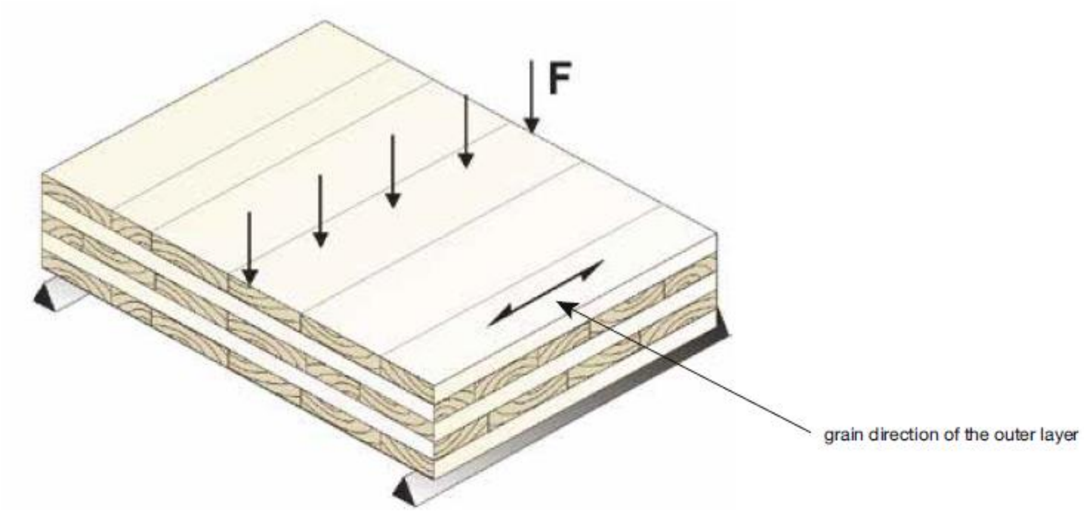
Out of plane bending design is prescribed in O86 and based on panels made in accordance to ANSI/APA PRG 320 standard with alternating orthogonal layers. These panels may be subjected to bending in either the major or minor direction. The major direction refers to the length wise or parallel orientation of the grain of the limitations in he outer layers of he panel.

Longitudinal layers refer to laminations oriented parallel to the major strength direction, else they are transvers layers.

Cross Laminated Timber



Bending in major strength direction



Bending in minor strength direction

For short or intermediate spans, moment and shear will govern, as the spans increase though deflection and serviceability effects will govern the design.

Mechanical properties of CLT panels are directionally dependent. Higher strength laminations are placed in the longitudinal layers. Bending is placed with the grain direction of the longitudinal layers parallel to the direction of the induced bending stress.

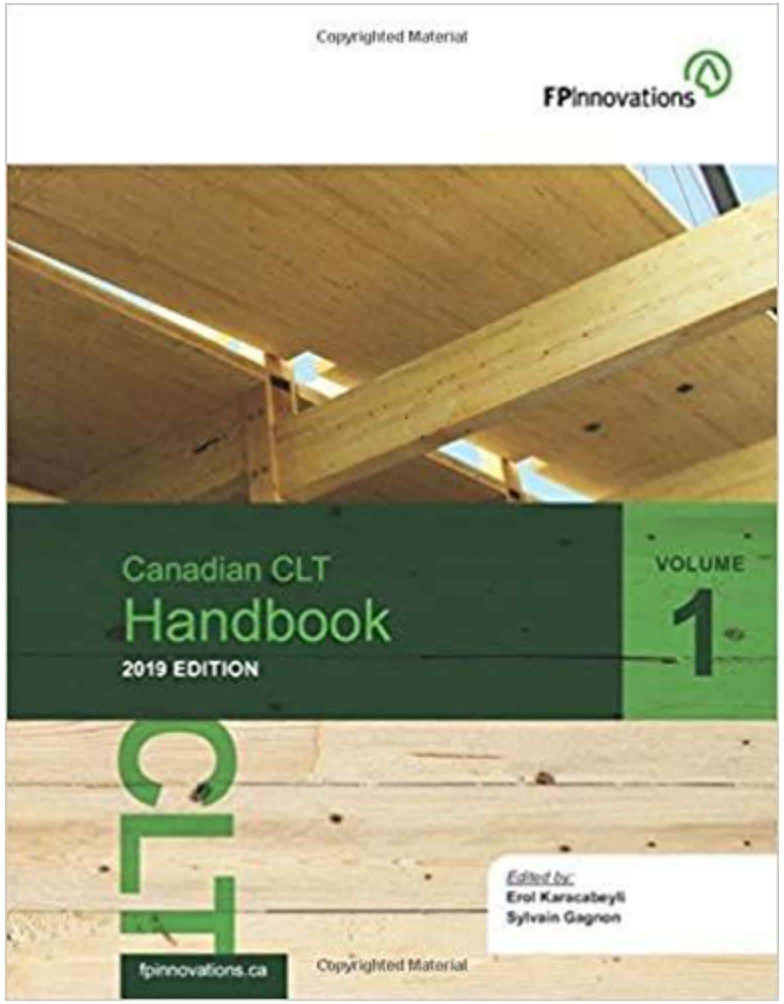
Cross Laminated Timber

Analytical methods aimed to determine the basic mechanical properties and stress distributions are provided in the FPInnovations CLT handbook.

Methodologies therein describe the shear analogy method which is considered the most accurate.

The methodology characterizes the a CLT panel as two virtual beams. Beam A captures the inherent flexural stiffness of individual layers along their own neutral axes, whereas Beam B includes additional flexural stiffness of individual layers when measured against the panel neutral axis and the panel shear stiffness.

The twin beams are assumed to be connected by infinitely rigid members so equal deflection is obtained. The distributions between the beams oof shear and bending are based on the continuity of the two beams. The procedure is similar though not identical to the concept of deflection compatibility in composite beam (steel) theory



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Cross Laminated Timber – Shear Analogy Method

(i) For the major strength direction (Figure 7.13):

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

where

ϕ = 0.9

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

f_b = specified bending strength of laminations in the longitudinal layers, MPa (CSA O86 Table 8.2.4)

K_D = load duration factor (CSA O86 Table 5.3.2.2)

K_H = system factor

= 1.0

K_{sb} = service condition factor for bending

= 1.0

K_T = treatment factor (CSA O86 Clause 8.3.3)

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

$(EI)_{eff,y}$ = effective bending stiffness of the panel for the major strength direction, N•mm² (calculation details provided below)

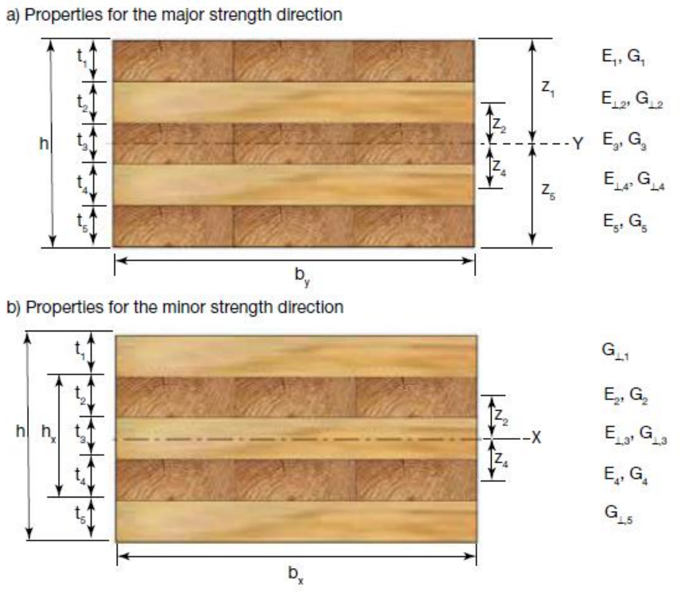
E = specified modulus of elasticity of laminations in the longitudinal layers, MPa (CSA O86 Table 8.2.4, Figure 7.13)

h = thickness of the panel, mm (Figure 7.13)

$K_{rb,y}$ = 0.85

FIGURE 7.13

Properties of a typical five-layer CLT panel



(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

Cross Laminated Timber – Shear Analogy Method

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.*

Cross Laminated Timber – Shear Analogy Method

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

Cross Laminated Timber – Shear Analogy Method

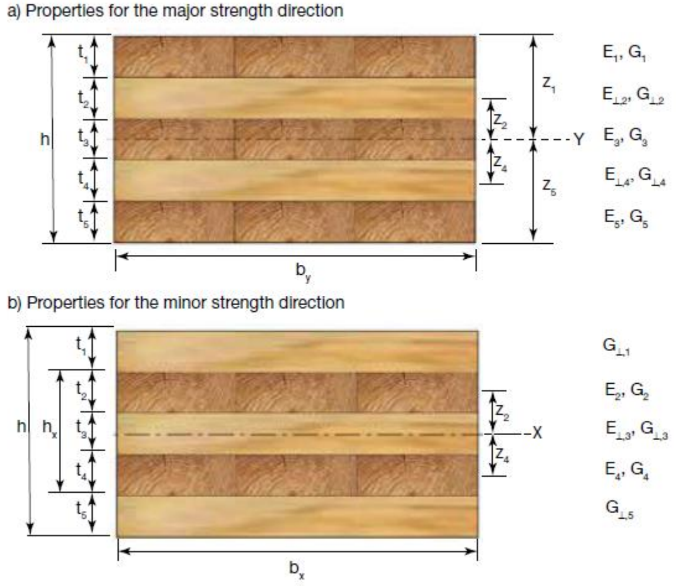
Rolling shear is affected by the cross-layer density, lamination thickness, moisture content and sawing configuration. It is important to consider as it accounts for the shear stress acting on the radial –tangential plane perpendicular to the grain. It basically is the rolling fibres on to of one another as can be seen below

(Source: Figure 8.2.4, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

Cross Laminated Timber – Shear Analogy Method

FIGURE 7.13

Properties of a typical five-layer CLT panel



(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

(i) For the major strength direction

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

where

b_y = width of the panel for the major strength direction, mm (Figure 7.13)

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

where

E = longitudinal modulus of elasticity, as provided in CSA O86 Table 8.2.4

$E_\perp$ = transverse modulus of elasticity, taken as 1/30 of longitudinal modulus of elasticity E

n = number of layers in the panel

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

z_i = distance between the centre point of the i-th layer and the panel neutral axis, mm (Figure 7.13)

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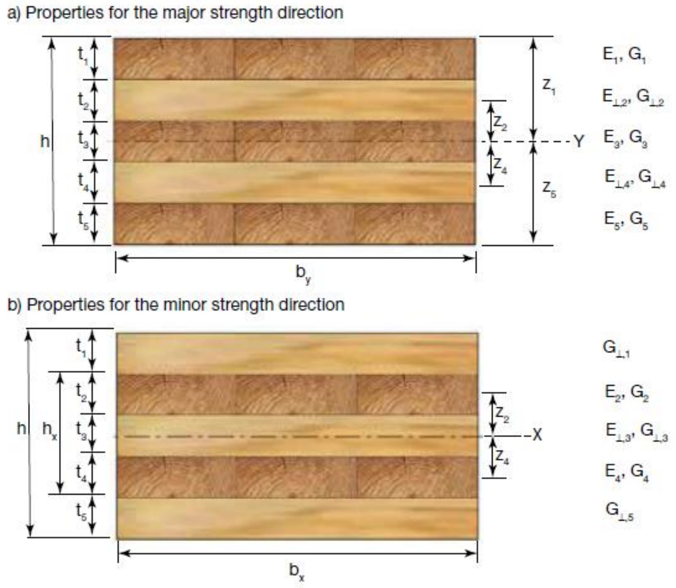
Cross Laminated Timber – Shear Analogy Method

FIGURE 7.13

Properties of a typical five-layer CLT panel



(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

For CLT panels with alternating orthogonal layers, the effective shear stiffness, $(GA)_{eff}$, affects the shear deflection and is calculated as follows:

(i) For the major strength direction

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

where

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

where

G = shear modulus, as provided in CSA O86 Table 8.2.4

$G_{\perp}$ = rolling shear modulus, estimated as G/10

h = thickness of the panel (Figure 7.13), mm

Cross Laminated Timber – Shear Analogy Method

(ii) For the minor strength direction (Figure 7.13):

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

where

f_b = specified bending strength of laminations in the transverse layers, MPa (CSA O86 Table 8.2.4)

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

$(EI)_{eff,x}$ = effective bending stiffness of the panel for the minor strength direction, N•mm² (calculation details provided below)

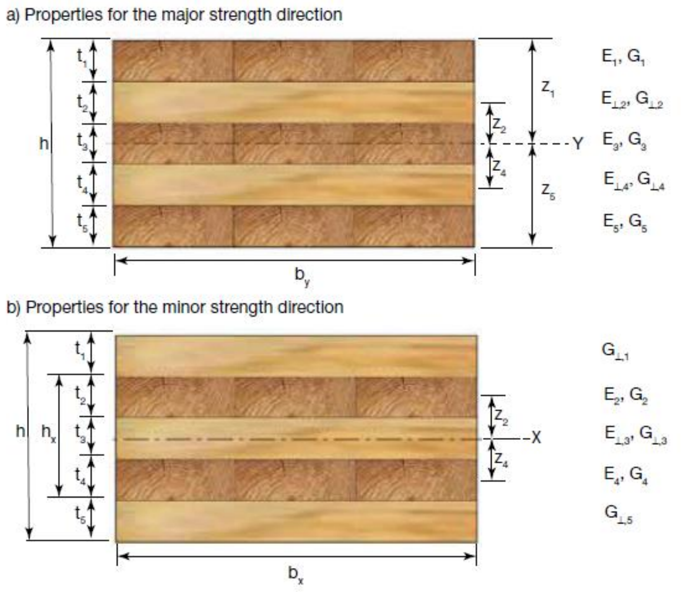
E = specified modulus of elasticity of laminations in the transverse layers, MPa (CSA O86 Table 8.2.4, Figure 7.13)

h_x = thickness of the panel without the outer longitudinal layers, mm (Figure 7.13)

$K_{rb,x}$ = 1.0

FIGURE 7.13

Properties of a typical five-layer CLT panel

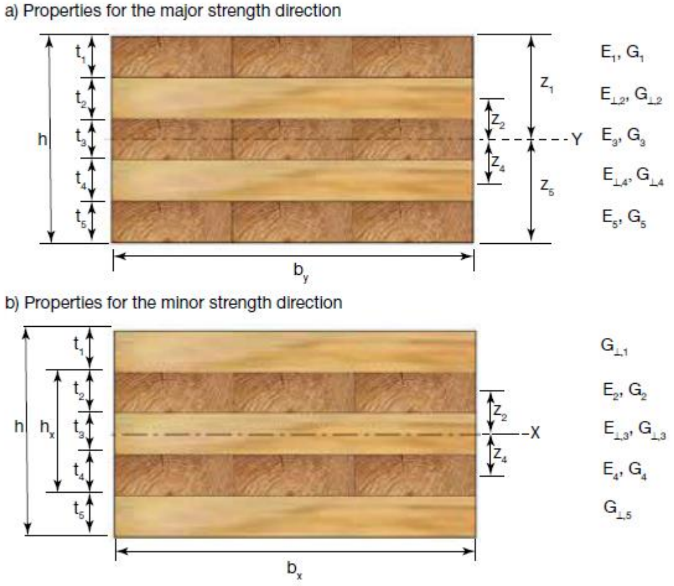


(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

Cross Laminated Timber – Shear Analogy Method

FIGURE 7.13

Properties of a typical five-layer CLT panel



(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

(ii) For the minor strength direction

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

where

b_x = width of the panel for the minor strength direction, mm (Figure 7.13)

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

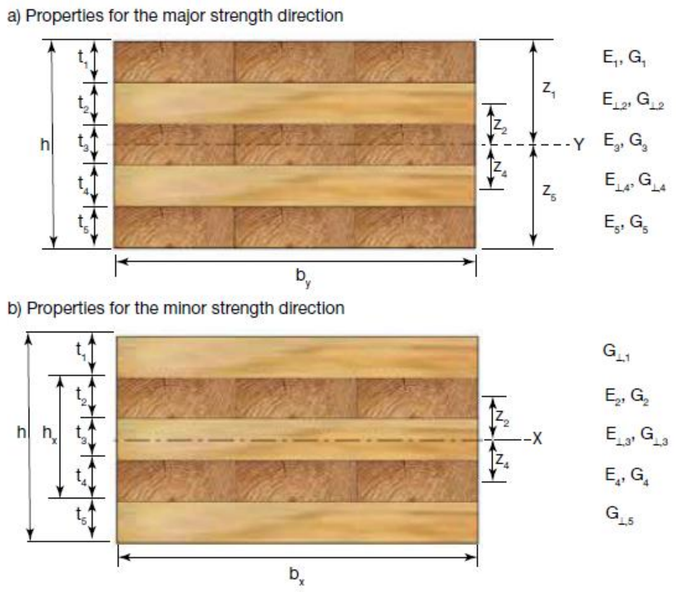
= E for laminations in the transverse layers, MPa

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

Cross Laminated Timber – Shear Analogy Method

FIGURE 7.13

Properties of a typical five-layer CLT panel



(Source: Figure 8.4.3.2, CSA O86-14 – Engineering design in wood. © 2014 Canadian Standards Association)

(ii) For the minor strength direction

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

where

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

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Cross Laminated Timber – Shear Analogy Method

The factored shear resistance, V_r, is directionally dependent and is calculated as follows:

(i) For the major strength direction:

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

where

$\phi = 0.9$

$F_s = f_s(K_D\ K_H\ K_{Sv}\ K_T)$

f_s = specified strength in rolling shear of laminations in the transverse layers, MPa (CSA O86 Table 8.2.4)

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

(ii) For the minor strength direction:

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

where

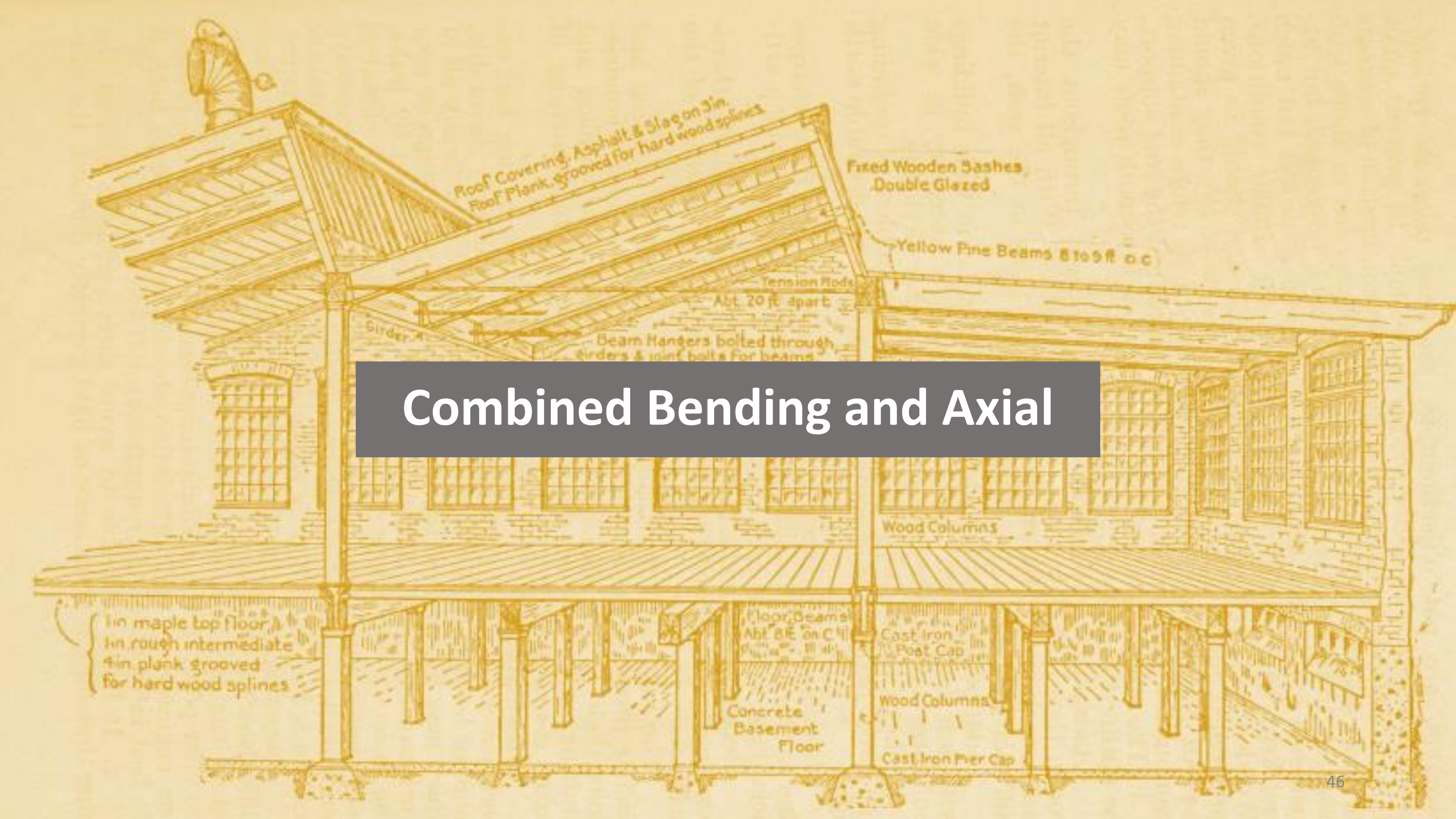
$\phi = 0.9$

$F_s = f_s(K_D\ K_H\ K_{Sv}\ K_T)$

f_s = specified strength in rolling shear of laminations in the longitudinal layers, MPa (CSA O86 Table 8.2.4)

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

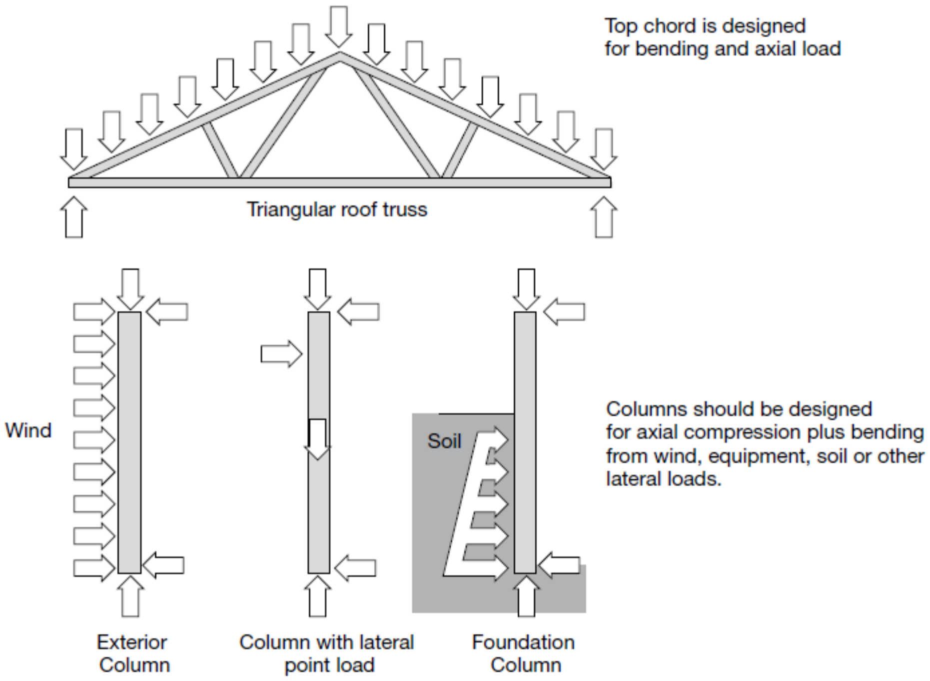
Rolling shear strength of wood is low when compared to the longitudinal shear strength. For this reason , only the rolling shear strength is used for the shear resistance. Other types of lumber such as sawn or EWPs, this is not the case.



Combined Bending and Axial

Combined Bending and Axial Loading

Many structural systems have an element of combined loading effect. This may be from eccentric loading, or members with lateral load conditions such as that from wind. Inclined columns will also have this effect that needs to be considered.



Combined Bending and Axial Loading

For most members combined bending can be considered following this formula:

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

where

P_f = factored compressive axial load

P_r = factored compressive load resistance parallel to grain calculated in accordance with the requirements of CSA O86

M_f = factored bending moment

M_r = factored bending moment resistance calculated in accordance with the requirements of CSA O86

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

$$= \frac{\pi^2 E_{05} K_{SE} K_T I}{(K_e L)^2}$$

where

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

= 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

= 0.87E for glulam

K_{SE} = service condition factor

K_T = treatment factor

$K_e L$ = effective length in the direction of the applied bending moment

Combined Bending and Axial Loading

For most sawn lumber truss members combined bending can be considered following this formula:

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

where

K_M = bending capacity modification factor (CSA O86 Table 6.5.13.5)

For studs used in preserved wood foundations:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] \leq 1$$ (CSA O86 Clause A6.5.12.6)

where

M_f = maximum factored bending moment on stud

Note that the K factors used for each variable may be different in the calculation for studs. Additional some loads may be directionally dependent based on the applied eccentricity.

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Combined Bending and Axial Loading

For CLT combined bending can be considered following this formula:

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

vhere

P_f = factored compressive axial load

P_r = factored compressive load resistance under axial load, calculated in accordance with CSA O86 Clause 8.4.5.4

M_f = factored bending moment

M_r = factored bending moment resistance, calculated in accordance with CSA O86 Clause 8.4.3

P_{Ev} = Euler buckling load in the plane of the applied bending moment adjusted for shear deformation

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

P_E = Euler buckling load in the plane of the applied bending moment in accordance with CSA O86 Clause 7.5.12, where E_{05} and I_{eff} are determined accounting only for the layers with laminations oriented parallel to the axial loac (CSA O86 Clauses 8.2.4 and 8.4.5.3)

$(GA)_{eff}$ = effective shear stiffness for out-of-plane bending accounting for all layers (CSA O86 Clause 8.4.3.2)

Recall the importance for consideration in the effects of shear with CLT!

Connections

Types of Fasteners

Covered here:

- Shear plates and split rings
- Nails and spikes
- Bolts
- Wood screws
- Drift pins
- Lag screws
- Timber rivets
- Truss plates
- Joist hangers

Right photo from Chris Naum of a timber mill showing a connection circa 1871. Left photo of the Scarborough Library glulam connection circa 2019.

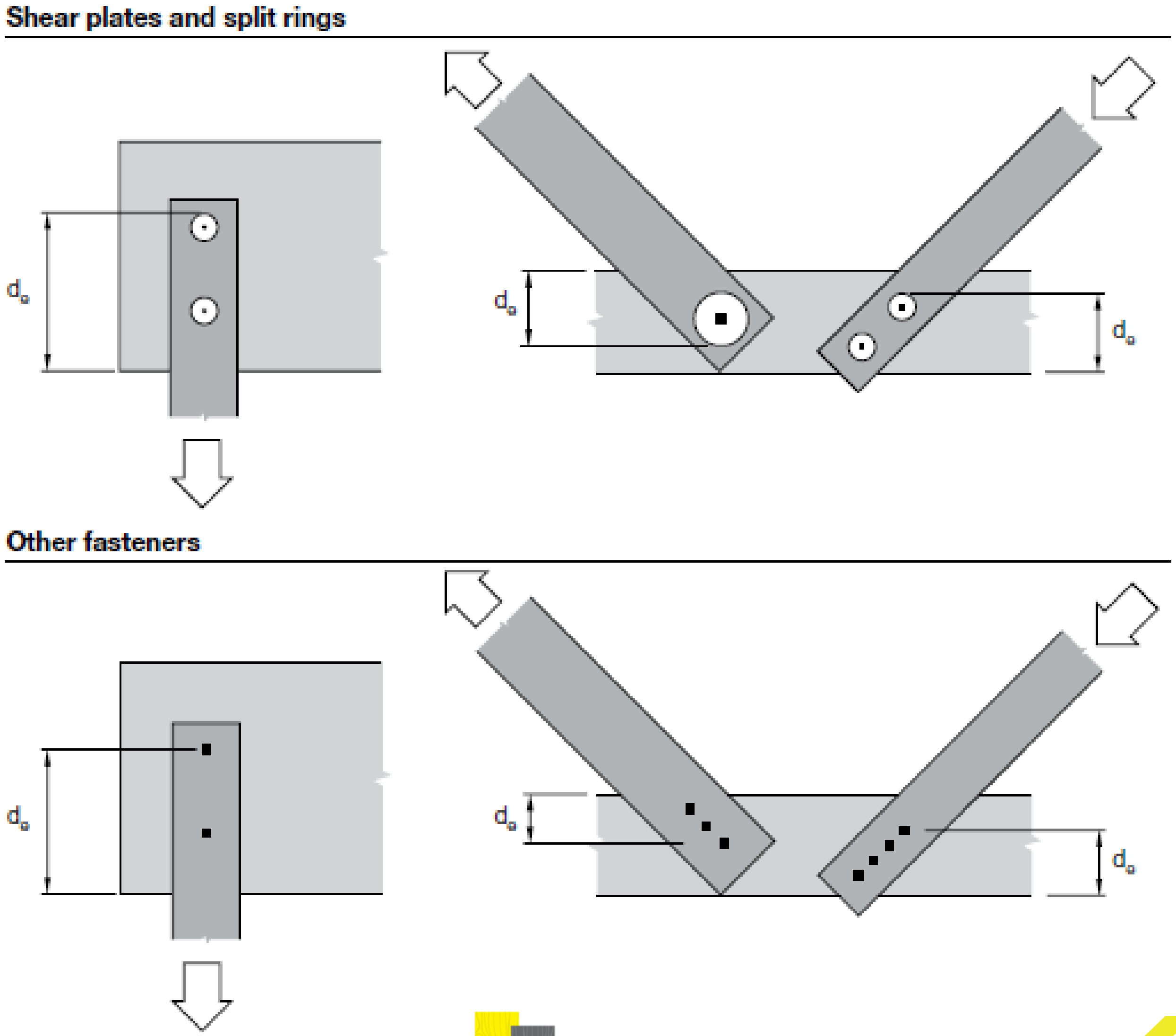
Modern connections can be made through advancements in precision cutting.



Shear Depth (d_e)

Particular to a wood member loaded at an angle. This results in a shear component that induces tensile stresses perpendicular to the grain of the wood. Stress is greatest at the boundary of the fasteners. we need to limit the total depth to an effective one.

The shear force to utilize is from the shear force diagram.



Consideration for Moisture (Service Condition)

Wood is weaker when wet so wood connections used in wet service conditions will have less resistance. When fasteners are installed in wood that will dry in service or where wood moisture contents fluctuate seasonally, shrinkage damage may occur. In fastenings, shrinkage damage may result in splits at the connections.

A service condition factor, Ksf reflects that wood is generally weaker when wet, and that shrinkage can occur when installed from drying. This may split at the fastener weakening the connector unless accounted for. Shrinkage in wood is greater perpendicular to the grain direction and splitting will result from wood's low tensile resistance perpendicular to the grain.

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°

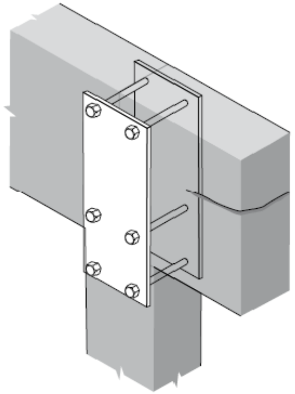
Legend:
A = a single fastener or single row parallel to grain with steel splice plates
B = a single row parallel to grain with wood splice plates, two rows parallel to grain not more than 127 mm apart with a common wood splice plate, or multiple rows with separate wood or steel splice plates for each 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).

Consideration for Moisture (Service Condition)

Splitting may be prevent with detailing to these conditions:

- Assemble wood structures at a moisture content that will be used in service conditions
- Use nails or slender bolts which are ductile
- One line of fasteners
- Minimize fasteners perpendicular to the grain
- Separate splice plates for each row of fasteners
- Slot splice plates perpendicular to the grain where practical to allow the wood to move.

Splitting occurs when wood is restrained from moving across the grain

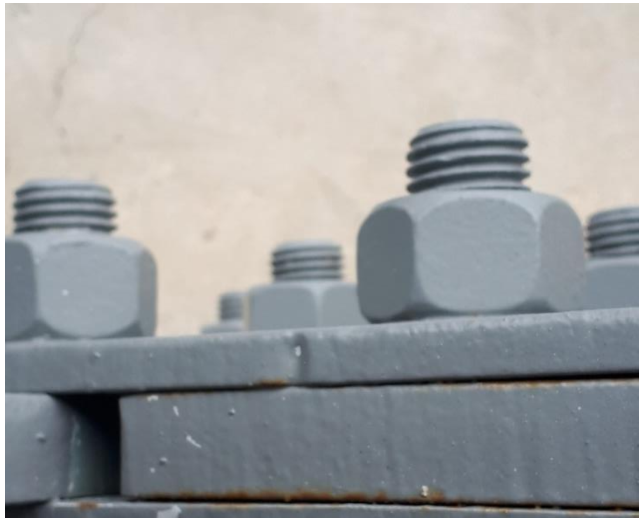


Load Duration

Assume the same behaviour as the wood itself for Kd

Treatment (preservative)

If the treatment contains copper it will be reactive (corrosive risk) for metallic connections so care needs to be taken for preservative treatment of exterior connections outside. Fire protection preservatives mechanisms can also effect the strength of he timber.



Geometry of Fasteners

Early theories supported the total strength of a joint with fastenings (small ductile, and widely spaced) is the product of the single fastener times the number of fasteners. Though more modern tests have indicated that the strength is lower. For spilt rings, shear plates and lag screws the load carrying capacity per fastener decreases with the number of fasteners in a row parallel to the load, but there is no reduction for the number of rows. A group modification factor (Jg), is provided below for split rings, shear plates and lag screws.

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
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

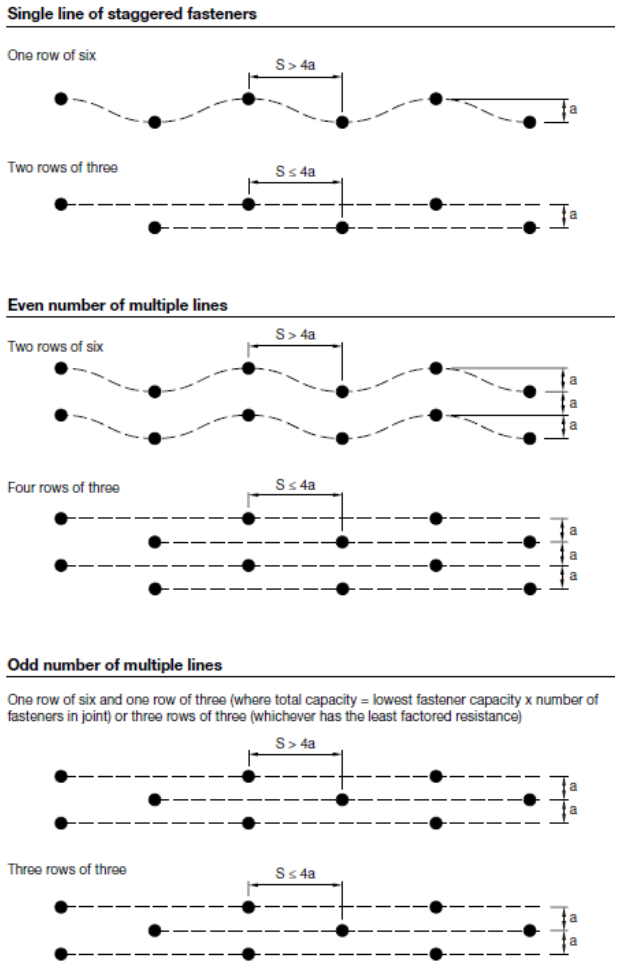
*Area ratio = A_m/A_s
where
A_m = gross cross-sectional area of main member, mm^2
A_s = sum of gross cross-sectional area of steel side plates, mm^2

Geometry of Fasteners

The capacity of a bolted connection is dependent on the number of bolts in a row, number of rows and the geometry (*right for interpretation especially when staggered lines are considered).*

Truss plates and joist hanger are developed empirically from tests, the group effect is direct incorporated into the resistance values. Likewise timber rivet connections.

Smaller joints made of ductile nails and spikes the group effect is ignored.



Net Area Reductions

$A_N = A_G - A_R$

For;
- split rings
- shear plates
- bolts
- lag screws
- drift pins

where:
A_G = gross cross-sectional area
A_R = area removed due to drilling, boring, grooving or other means
A_R must not exceed $0.25\ A_G$

Area reduction due to bolt, lag screw or drift pin holes is equal to the diameter of the hole multiplied by the thickness of the member.

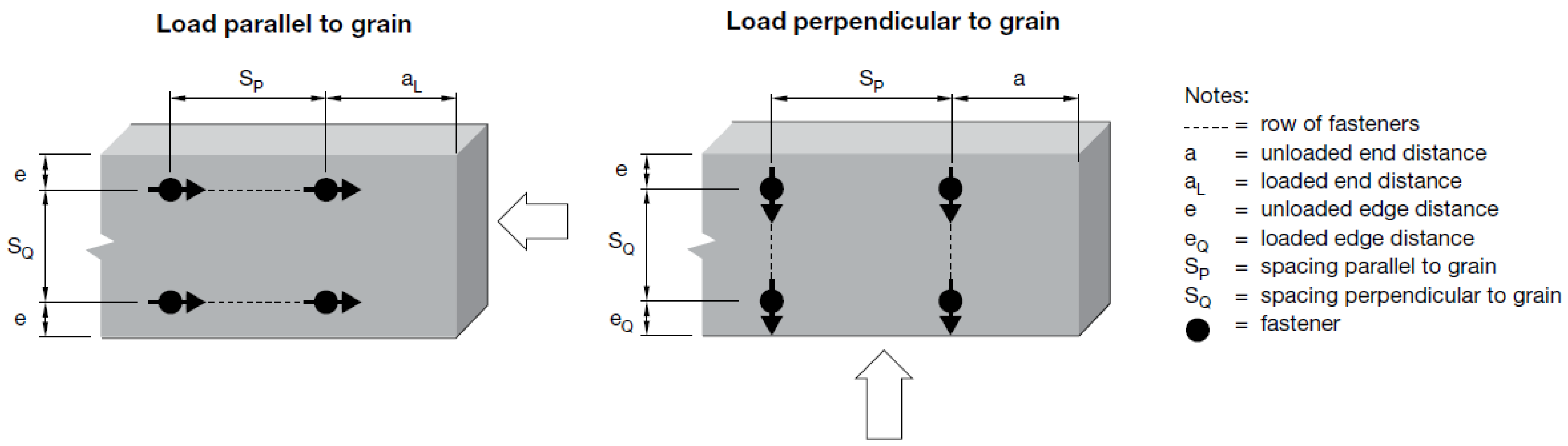
For **staggered rows of fasteners**, adjacent fasteners must be considered to occur at the same cross-sectional plane when the centre-to-centre spacing along the grain is less than the following values:
2 x fastener diameter for split rings and shear plates
8 x fastener diameter for bolts, lag screws or drift pins

Net Area Reductions

Area removed for shear plate and split ring joints ($mm^2 \times 10^3$)

Connector	Size in.	Bolt size in.	Number of faces with connector	Member thickness mm 38	64	80	89	130	140	175	191	215	241
Split rings	2-1/2	1/2	1	1.11	1.48	1.71	1.84	2.43	2.57	3.07	3.30	3.64	4.02
			2	1.68	2.05	2.28	2.41	3.00	3.14	3.64	3.87	4.21	4.58
	4	3/4	1	1.97	2.51	2.84	3.02	3.87	4.07	4.79	5.12	5.62	6.15
			2	3.16	3.70	4.02	4.21	5.06	5.26	5.98	6.31	6.81	7.34
Shear plates	2-5/8	3/4	1	1.31	1.85	2.18	2.36	3.20	3.41	4.13	4.46	4.96	5.49
			2	1.84	2.37	2.70	2.89	3.73	3.94	4.66	4.99	5.50	6.02
	4	3/4	1	2.12	2.65	2.98	3.17	4.01	4.22	4.94	5.27	5.76	6.30
			2	–	3.99	4.32	4.50	5.34	5.55	6.27	6.60	7.10	7.63
	4	7/8	1	2.19	2.80	3.18	3.40	4.38	4.61	5.45	5.83	6.40	7.02
			2	–	4.09	4.47	4.68	5.66	5.89	6.73	7.11	7.68	8.30

Geometry of Fasteners



The arrangement of fasteners in a connection is defined in terms of end distance, edge distance and spacing as shown. A row is defined as one or more bolts, lag screws, shear plates or split rings aligned in the direction of the load.

Loading

The lateral resistance of most fasteners depends upon the angle of load to grain. The angle is 0° when the load is parallel to grain, and 90° when the load is perpendicular to grain. Loading at an intermediate angle to grain may be calculated from the following empirical equation:

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

N_r = factored resistance at an angle θ to grain
P_r = factored resistance parallel to grain
Q_r = factored resistance perpendicular to grain
θ = angle between grain direction and direction of load
$N_{r;}$ may also be determined from Design Handbook (*Table 7.1)*

Design Tables (7.1-Handbook)

Angle of load to grain θ°	Q_r/P_r 0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
0	1	1	1	1	1	1	1	1	1
5	0.97	0.98	0.99	0.99	0.99	1	1	1	1
10	0.89	0.93	0.96	0.97	0.98	0.99	0.99	1	1
15	0.79	0.86	0.91	0.94	0.96	0.97	0.98	0.99	1
20	0.68	0.79	0.85	0.90	0.93	0.95	0.97	0.99	1
25	0.58	0.71	0.79	0.85	0.89	0.93	0.96	0.98	1
30	0.50	0.63	0.73	0.80	0.86	0.90	0.94	0.97	1
35	0.43	0.57	0.67	0.75	0.82	0.88	0.92	0.96	1
40	0.38	0.51	0.62	0.71	0.78	0.85	0.91	0.96	1
45	0.33	0.46	0.57	0.67	0.75	0.82	0.89	0.95	1
50	0.30	0.42	0.53	0.63	0.72	0.80	0.87	0.94	1
55	0.27	0.39	0.50	0.60	0.69	0.78	0.86	0.93	1
60	0.25	0.36	0.47	0.57	0.67	0.76	0.84	0.92	1
65	0.23	0.34	0.45	0.55	0.65	0.74	0.83	0.92	1
70	0.22	0.33	0.43	0.53	0.63	0.73	0.82	0.91	1
75	0.21	0.31	0.42	0.52	0.62	0.71	0.81	0.91	1
80	0.20	0.31	0.41	0.51	0.61	0.71	0.80	0.90	1
85	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1
90	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1

Note:
Determine the resistance at an angle to grain N_r as follows:
a) Calculate Q_r/P_r = Perpendicular to grain resistance/Parallel to grain resistance.
b) Select coefficient X for given Q_r/P_r ratio and angle θ.
c) $N_r = X P_r$

Other Considerations

Notching

joints that cause tension perpendicular to grain stresses should be avoided. This occurs for example in cases where simple beams have been notched on the tension side at the supports. In this case, tension perpendicular to grain stresses can be caused by prying action caused by secondary moments.

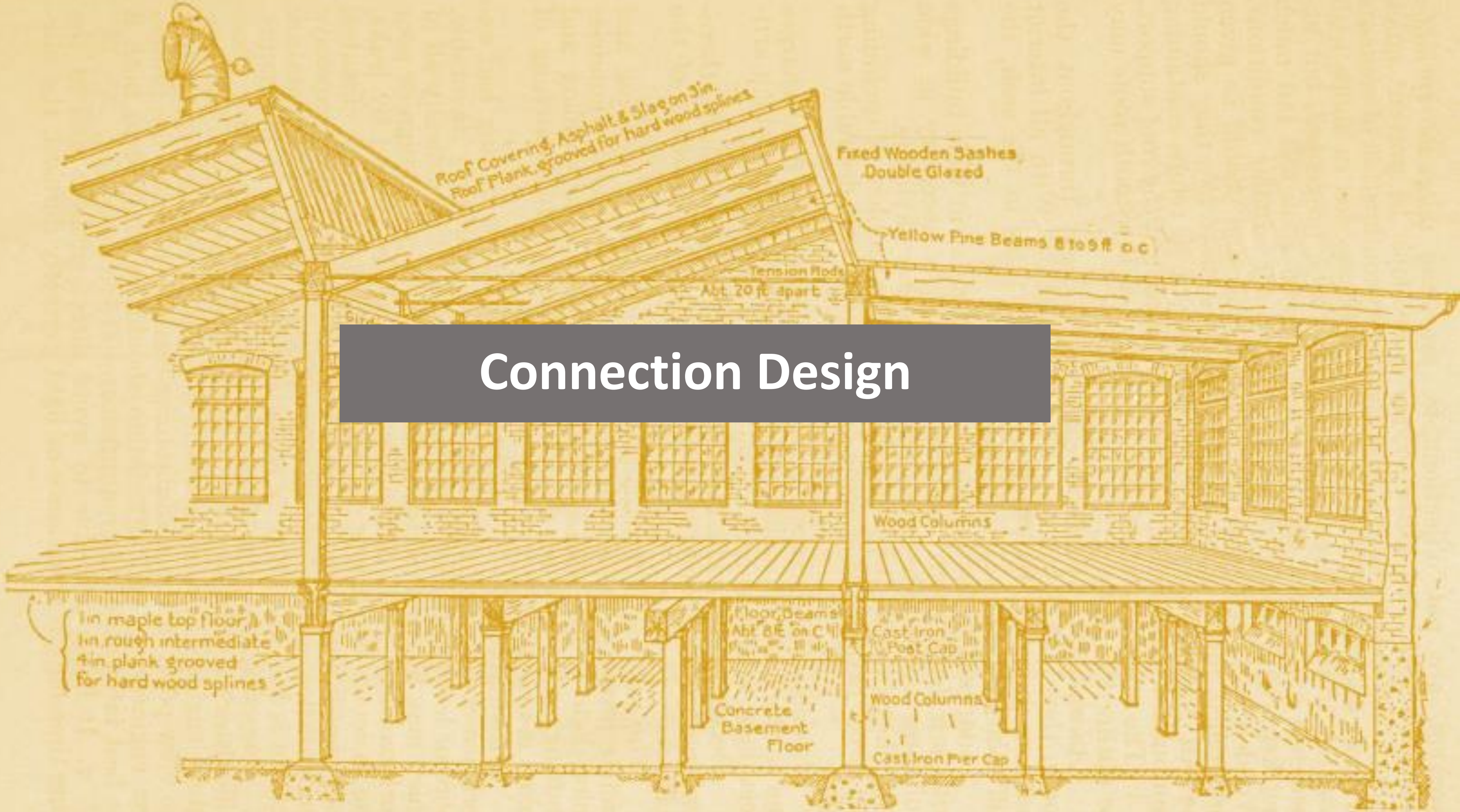
Decay Prevention

Moisture barriers, flashing and other protective features should be used to prevent moisture or free water from being trapped. Adequate site drainage must always be provided, all metals should be protected against corrosion by use of corrosive-resistant metals or resistant coatings or platings.

Lateral Restraint at Supports

Lateral restraint to prevent displacement and rotation must be provided at points of bearing for all beams with a depth-to-width ratio greater than 2.5.

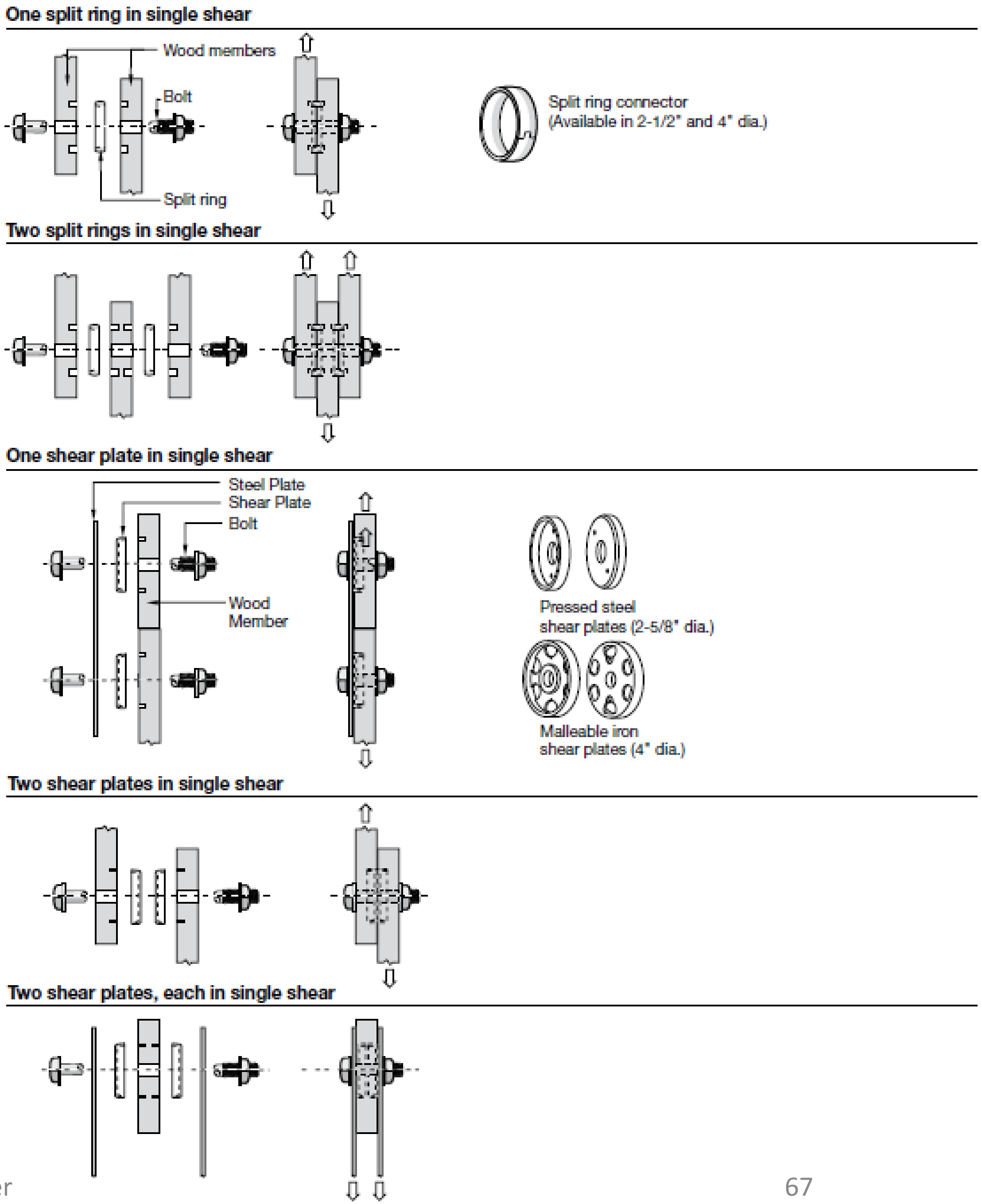
Connection Design



Split Rings and Shear Connectors

Round steel connectors installed in grooved wood. Split Rings are mainly for wood to wood connection. Formed through identical grooves in matching faces of two members. Shear plates are best adapted for wood steel, but can be used in wood wood connections. Shear plates transfer shear through the fastener (bolt or lag screw) through the center of the connector.

Split rings and shear plate joints are used more generally in heavy timber or glulam structures where there are large loads to be resisted (trusses, purlin to beam, column to foundation, arch peaks, or bridges).

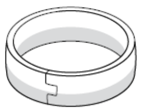


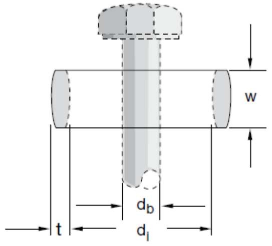
Split Rings and Shear Connectors

Timber connector types and sizes

Split ring connector

Steel ring with tongue and groove division; section of ring is wedge shaped for easy insertion in specially cut grooves.



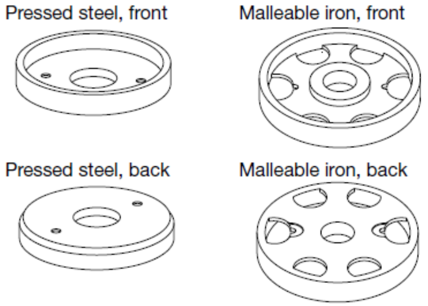


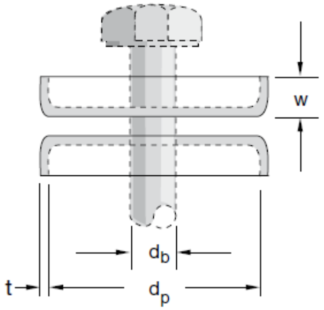
	Split ring dimensions	
	2-1/2" mm	4" mm
Inside diameter at centre when closed, d_i	63.5	101.6
Thickness of steel at centre, t	4.1	4.9
Depth of steel, w	19.0	25.4

Shear plates

For wood to wood joints, two shear plates are placed back to back in connection.

For wood to metal joints, a single shear plate is placed in wood member.





	Shear plate dimensions		
	2-5/8"	4"	
		3/4" bolt	7/8" bolt
	mm	mm	mm
Diameter of plate, d_p	66.5	102.1	102.1
Diameter of bolt hole, d_b	20.6	20.6	23.9
Thickness of plate, t	4.3	5.1	5.1
Depth of flange, w	10.7	15.7	15.7

Design of Split Rings and Shear Connectors

Factored lateral Strength resistance parallel to the grain

$$P_r = \phi P_u n_F J_F$$

Factored lateral Strength resistance perpendicular to the grain

$$Q_r = \phi Q_u n_F J_F$$

ϕ = 0.6

$P_u = p_u (K_D K_{SF} K_T)$

p_u = lateral strength resistance parallel to grain, kN (CSA O86 Table 12.3.6A)

$Q_u = q_u (K_D K_{SF} K_T)$

q_u = lateral strength resistance perpendicular to grain, kN (CSA O86 Table 12.3.6B)

K_D = load duration factor

K_{SF} = service condition factor (CSA O86 Table 12.2.1.6)

K_T = fire-retardant treatment factor (CSA O86 Clause 12.2.1.8)

n_F = number of fastening units

$J_F = J_G J_C J_T J_O J_P$

J_G = factor for groups of fastenings (CSA O86 Tables 12.2.2.3.4A and B)

J_C = minimum configuration factor (CSA O86 Tables 12.3.3A, B, and C)

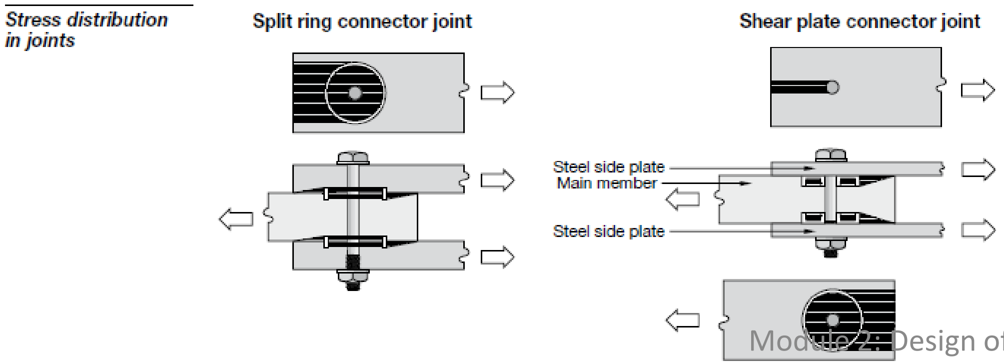
J_T = thickness factor (CSA O86 Table 12.3.4)

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 (CSA O86 Table 12.3.5)



Design of Split Rings and Shear Connectors

The capacity of a split ring is dependent upon the combined action of the bolt and ring. The capacity of the connector unit of the shear plat e is limited by the shear strength of the bolt.

Table 12.3.6A
Lateral strength resistance parallel to grain, p_w, 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.*

Table 12.3.6B
Lateral strength resistance perpendicular to grain, q_w, 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

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
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

Design of Split Rings and Shear Connectors

If lag screws are used instead of bolts, resistance is dependent on the depth of penetration (table 12.3.5 O86) gives requirements. Specific detailing is required to utilise the full capacity of the connection. Holes must first be drilled and then installed.

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 Minimum	5 3.5	7 4	7 4	8 8	1.00 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.*

Bolts and Dowels

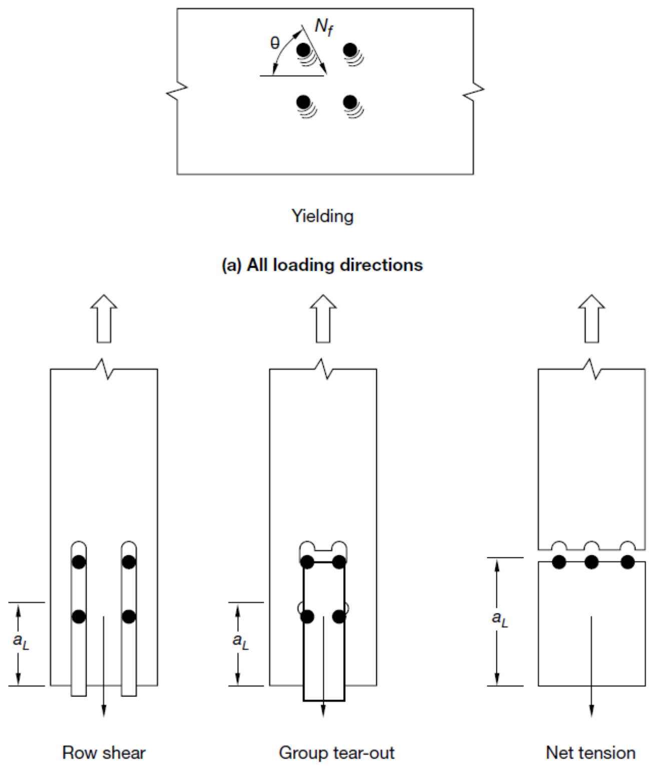
Design information is provided in the Wood Design Manual based on bolts and dowels conforming to the requirements of ASTM Standard A307 or SAE J429 Grade 2 bolts and dowels. The holes for bolts and dowels must meet the requirements of CSA O86. Bolts are required to have pre-drilled holes that are between 1 mm and 2 mm larger than the bolt diameter.

The bolt and dowel design provisions in CSA O86 address:

-Yielding of the bolt or dowel

-Wood failure mechanism around the connection including:

- Row shear parallel to grain along each fastener row
- Group tear-out of a connection with multiple rows of bolts loaded parallel to grain
- Net tension of member loaded parallel to grain, and
- Splitting of members loaded perpendicular to grain.



Sizing of Bolts and Dowels

Machine bolt and nut dimensions

Bolt diameter in.	Hex bolts height: Head mm	Hex bolts height: Nut mm	Width across: Flats mm	Width across: Corner mm
1/2	8	12	19	22
5/8	10	14	24	28
3/4	12	17	29	33
7/8	14	19	34	39
1	16	22	38	44
1-1/8	18	25	43	50
1-1/4	20	27	48	55
1-1/2	24	33	58	66
1-3/4	28	38	67	77
2	31	44	77	88

Machine bolts and nuts – threaded length and mass

Finished hex bolts	Minimum thread length (T) and mass (M) per 100, without nuts									
	ASTM A307 Bolt Diameter (in.)									
	1/2		5/8		3/4		7/8		1	
Bolt length under head in.	T mm	M kg	T mm	M kg	T mm	M kg	T mm	M kg	T mm	M kg
4	32	11.5	38	18.2	44	27.2	51	38.4	57	51.3
4-1/2	32	12.8	38	20.2	44	30.0	51	42.3	57	56.4
5	32	14.0	38	22.1	44	32.9	51	46.1	57	61.4
6	32	16.6	38	26.1	44	38.6	51	53.9	57	71.5
6-1/2	38	17.8	44	28.1	51	41.4	57	57.7	63	76.6
7	38	19.1	44	30.0	51	44.2	57	61.6	63	81.6
8	38	21.6	44	34.0	51	49.9	57	69.3	63	91.7
9	38	24.1	44	37.9	51	55.6	57	77.1	63	102
10	38	26.7	44	41.9	51	61.3	57	84.8	63	112
11	38	29.2	44	45.8	51	67.0	57	92.5	63	122
12	38	31.7	44	49.8	51	72.6	57	100	63	132
13	38	34.2	44	53.7	51	78.3	57	108	63	142
14	38	36.8	44	57.7	51	84.0	57	115	63	152
15	38	39.3	44	61.6	51	89.7	57	124	63	162
Hex nuts	–	1.7	–	13.3	–	15.4	–	18.6	–	12.8

Tension Resistance of Bolts and Dowels

Bolt diameter in.	Tensile resistance, T_r kN
1/2	31.5
5/8	49.2
3/4	70.8
7/8	96.4
1	126

Note:
T_r = $0.75\ \phi_b A_b F_u$ where $\phi_b = 0.80$ (from CSA S16)
A_b = area of bolt (mm^2)
F_u = minimum specified tensile strength = 414 MPa

Modification Factors of Bolts and Dowels

K_D = 1.0 when the loading is "standard" term
= 1.15 for short term loading (e.g. dead plus wind)
= 0.65 for permanent loads (e.g., dead loads alone)

K_{Dy} = 1.0 when the loading is "standard" term
= 1.06 for short term loading (e.g. dead plus wind)
= 0.65 for permanent loads (e.g., dead loads alone)

K_{SF} = 1.0 when the service conditions are "dry" and the lumber is seasoned (moisture content ≤ 19%) prior to fabrication

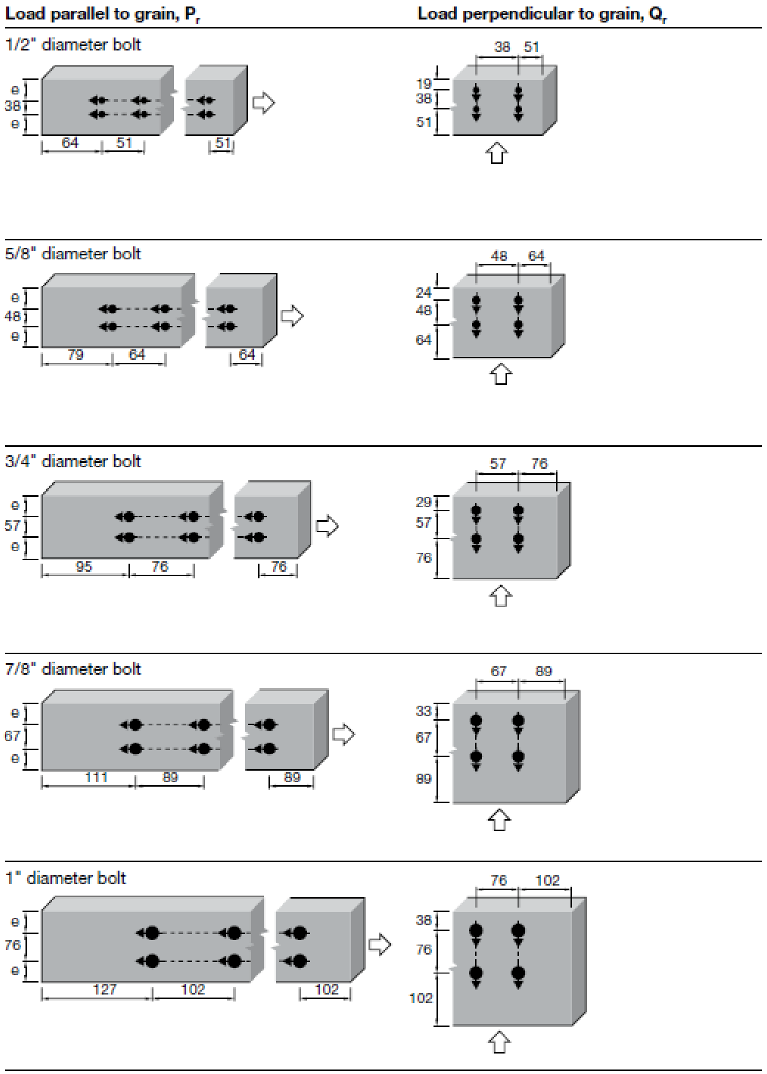
For other conditions, determine K_{SF} from Table 7.8.

K_{Sv} = 1.0 when the service conditions are "dry" or the least dimension of sawn lumber is over 89 mm
= 0.96 for sawn lumber in wet service conditions and the least dimension of sawn lumber is 89 mm or less
= 0.87 for glulam in wet service conditions

K_{St} = 1.0 when the service conditions are "dry" or the least dimension of sawn lumber is over 89 mm
= 0.84 for sawn lumber in wet service conditions and the least dimension of sawn lumber is 89 mm or less
= 0.75 for glulam in wet service conditions

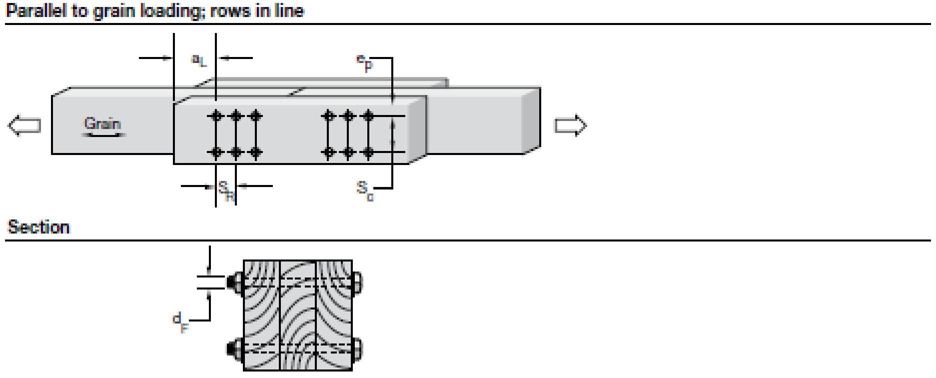
K_T = 1.0 when the wood is not treated with a fire retardant or other strength reducing chemical. For wood treated with a fire retardant or other strength reducing chemical see the Commentary for further guidance.

Minimum Spacing Bolts and Dowels



Notes:
1. e = the greater of 1.5 times bolt diameter or half of actual spacing between rows

Minimum spacing for bolts loaded parallel to grain



Edge Distance

$e_p \ge 1.5d_F$ or $S_C/2$ (whichever is greater)

End Distance

$a_L \ge 5d_F$ or 50 mm for members in tension
$a \ge 4d_F$ or 50 mm for member in compression

Proper end distance must also be provided for centre members

Spacing in Rows

$S_R \ge 4d_F$

Spacing Between Rows

$S_C \ge 3d_F$
(When $S_C \ge 125$ mm, separate splice plates are used for each row of fasteners.)

Design of Bolts and Dowels

Bolts can be hot dipped galvanized, stainless steel (including washers and nuts) or plated to provide resistance to corrosion.

Bolts are widely used in applications of heavy loads. They are seen in purlin to beam, beam to column, column to base, truss, arches, post and beam, pole-frame, bridges and marine structures (with care for corrosion considered.

There are a suite of equations provided from Clause 12.4.4.2 which capture the possible modes of failure. In this case the minimum resulting value is the selected lateral resistance.

Design of Bolts and Dowels

The factored lateral resistance depends upon the angle of load to grain and is calculated as follows;

(a) $N_f \leq N_r$

where

N_f = factored load on the connection

N_r = factored lateral yielding resistance

(b) $P_f \leq P_r$

where

P_f = factored load parallel to grain

P_r = factored resistance parallel to grain

= the lesser of factored row shear resistance PR_{rT}, factored group tear-out resistance PG_{rT}, or factored net tension resistance TN_{rT}

(c) $Q_f \leq Q_r$

where

Q_f = factored load perpendicular to grain

Q_r = QS_{rT}, factored perpendicular-to-grain splitting resistance

(d) For loading at an angle to grain, θ:

$$N_f \leq \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

where

θ = angle between the applied load and the grain

Design of Bolts and Dowels

Yielding resistance is a function of the number of shear planes in the connection it is based on the unit lateral yielding resistance which is determined based on the number of member connections;

(a) $N_f \leq N_r$

where

N_f = factored load on the connection

N_r = factored lateral yielding resistance

$$N_r = \phi_y n_u n_s n_F$$

where

ϕ_y = resistance factor for yielding

= 0.8

n_u = unit lateral yielding resistance, N

n_s = number of shear planes in the connection

n_F = number of fasteners in the connection

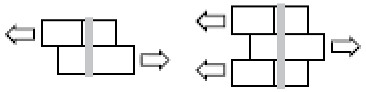
Design of Bolts and Dowels

n_u = unit lateral yielding resistance, N

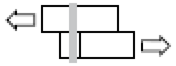
For two-member connections, only Items (a), (b) and (d) to (g) are considered valid.

For three-member connections, only Items (a), (c), (d) and (g) are considered valid.

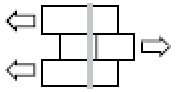
a) $f_1 d_F t_1$



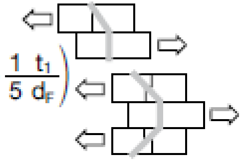
b) $f_2 d_F t_2$



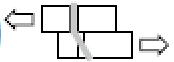
c) $\frac{1}{2} f_2 d_F t_2$



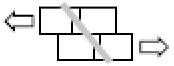
d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$



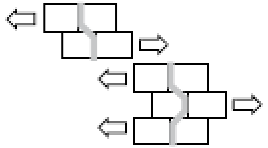
e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$



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) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}}$



f_1, f_2 = embedment strength of members 1 and 2, where member 1 is the side member, MPa

d_F = diameter of the fastener, mm

t_1, t_2 = member thickness or dowel bearing length, mm

f_y = yield strength of fastener in bending

= 310 MPa for ASTM A307, SAE J429 Grade 2 bolts and dowels

For wood member embedment strength:

$$f_{i\theta} = \frac{f_{iP} f_{iQ}}{f_{iP} \sin^2\theta + f_{iQ} \cos^2\theta} K_D K_{SF} K_T$$

where

$f_{i\theta}$ = embedment strength of member i for a fastener bearing at angle θ relative to the grain, MPa

f_{iP} = embedment strength for a fastener bearing parallel to grain (θ=0°)

= $50G(1-0.01d_F)\,J_x$

J_x = adjustment factor for connections in CLT

= 0.9 for CLT

= 1.0 for all other cases

f_{iQ} = embedment strength for a fastener bearing perpendicular to grain (θ=90°)

= $22G(1-0.01)d_F$

G = mean relative density (CSA O86 Table A12.1)

K_D = load duration factor

K_{SF} = service condition factor

K_T = treatment factor

For non-wood based materials, the embedment strength is taken as follows:

Steel: $K_{sp}\,(\phi_{steel}/\phi_y)\,f_u$, MPa

Concrete or Masonry: 125 MPa

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

ϕ_{steel} = resistance factor for steel plates in connections with bolts and dowels

= 0.8 for mild steel referenced in CSA S16

= 0.5 for cold-formed light gauge steel referenced in CSA S136

ϕ_y = 0.8 (resistance factor for yielding failures in wood members in connections with bolts and dowels)

Design of Bolts and Dowels

(b) $P_f \le P_r$

where

P_f = factored load parallel to grain

P_r = factored resistance parallel to grain

= the lesser of factored row shear resistance PR_{rT}, factored group tear-out resistance PG_{rT}, or factored net tension resistance TN_{rT}

Parallel-to-grain Row Shear Resistance

The total factored parallel-to-grain shear resistance of a joint is calculated as the sum of the factored row shear resistance of the wood members resisting load, as follows:

$$PR_{rT} = \Sigma(PR_{ri})$$

The total factored row shear resistance of fasteners in a wood member, i, is calculated as follows:

$$PR_{ri} = \phi_w PR_{ij\,min}\, n_R$$

where

ϕ_w = resistance factor for brittle failures

= 0.7

$PR_{ij\,min}$ = minimum row shear resistance of any row in the connection from PR_{i1} to PR_{inR}

n_R = number of fastener rows

PR_{ij} = shear resistance of fastener row j in member i, N

= $1.2\, f_v\, (K_D\, K_{Sv}\, K_T)\, K_{ls}\, t\, n_c\, a_{cr\,i}$

f_v = specified shear strength for member i, MPa

= CSA O86 Tables 6.3.1A, 6.3.1C, 6.3.1D, and 7.3, or 0.6 x f_v in Table 6.3.1B)

K_{ls} = 0.65 for side member

= 1.0 for internal member

t = member thickness, mm

n_c = number of fasteners in row j of member i

a_{cri} = minimum of a_L and S_R for row j of member i, mm

The capacity of each member in the joint must be checked separately for resistance to row shear failure.

Parallel-to-grain Group Tear-Out Resistance

The total factored group tear-out resistance of a joint is calculated as the sum of the factored group tear-out resistance of the wood members resisting 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 is calculated as follows:

$$PG_{ri} = \phi_w \left[\frac{PR_{i1} + PR_{inR}}{2} + f_t\,(K_D K_{St} K_T)\,A_{PGi}\right]$$

where

ϕ_w = 0.7

$PR_{i1} = 1.2\, f_v\, (K_D\, K_{Sv}\, K_T)\, K_{ls}\, t\, n_c\, a_{cr\,1}$

= shear resistance along row 1 of member i bounding the fastener group, N

$PR_{inR} = 1.2\, f_v\, (K_D\, K_{Sv}\, K_T) K_{ls}\, t\, n_c\, a_{cr\,nR}$

= shear resistance along row n_R of member i bounding the fastener group, N

f_t = specified strength in tension of member i, MPa

= CSA O86 Tables 6.3.1A, 6.3.1C, 6.3.1D and 7.3, or 0.65 × f_t in Table 6.3.1B

A_{pGi} = critical perpendicular net area between rows 1 and n_R of member i, mm^2

Group tear-out is not possible if the member end-distance is in compression. The capacity of each member must be checked separately.

Net Tension Resistance

The total factored net tension resistance of the wood members loaded parallel to grain at a group of fasteners is calculated as follows:

$$TN_{rT} = \Sigma TN_{ri}$$

TN_{ri} of member, i, at a group of fasteners is determined using the method outlined in Chapter 9.1. The cross-sectional area deducted from the gross cross-section must not be greater than 25% of the member gross area.

Recall from the module on Tension;

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

where

ϕ = 0.9

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

f_t = specified strength in tension parallel to grain, MPa (CSA O86 Tables 6.3.1A, 6.3.1B, 6.3.1C, 6.3.1D, 6.3.2 and 6.3.3)

A_n = net area of cross section, mm^2

K_{Zt} = size factor in tension (CSA O86 Table 6.4.5)

Design of Bolts and Dowels

(c) $Q_f \leq Q_r$

where

Q_f = factored load perpendicular to grain

Q_r = QS_{rT}, factored perpendicular-to-grain splitting resistance

Perpendicular-to-Grain Splitting Resistance

The total factored splitting resistance of a joint is calculated as the sum of the splitting resistance of the wood members resisting the load as follows:

$$QS_{r\,T} = \Sigma QS_{r\,i}$$

The factored perpendicular-to-grain splitting resistance of member i, N, is calculated as follows:

$$QS_{r\,i} = \phi_w QS_i (K_D K_{SF} K_T)$$

where

ϕ_w = 0.7

$$QS_i = 14t\sqrt{\frac{d_e}{1-\frac{d_e}{d}}}$$

t = member thickness, mm

d_e = d - e_p, effective member depth, mm

d = member depth, mm

e_p = unloaded edge distance, mm

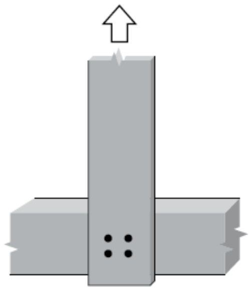
Design of Bolts and Dowels

Designing of bolts and dowels tends to be a trial and error process; Selection tables from 7.4 from the Hand book assists:

Single Shear, Wood Side Plates

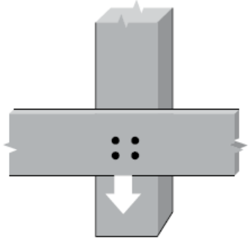
Parallel to grain loading

Type 1



Side plate parallel to grain loading. Member perpendicular to grain loading.

Type 2



Side plate perpendicular to grain loading. Member parallel to grain loading.

Type 3

Bolt and Dowel Selection Tables

1S Single Shear, Wood Side Plate

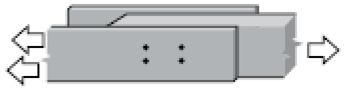
Factored lateral resistance - 1/2" Bolt or Dowel

			PR_F – Row shear resistance				G_{rT} – Group tear-out			
				Capacity per bolt or dowel				Tension resistance		
Species **Sawn lumber**	Side plate thickness mm	Member thickness mm	Spacing parallel to grain[1] mm	Type 1 Side Par. Main Par. kN	Type 2 Side Par. Main Perp. kN	Type 3 Side Perp. Main Par. kN	Group width[2] mm	Type 1 Side Par. Main Par. kN	Type 2 Side Par. Main Perp. kN	Type 3 Side Perp. Main Par. kN
D.Fir-L	38	38	51 60 ≥ 84	2.00 2.38 3.30	2.00 2.38 2.38	2.00 2.38 2.38	38 76 114	1.31 2.61 3.92	1.31 2.61 3.92	1.31 2.61 3.92
	38	89	51 85 ≥ 119	2.00 3.35 4.68	2.00 3.35 3.35	3.10 3.10 3.10	38 76 114	1.31 2.61 3.92	1.31 2.61 3.92	3.06 6.12 9.18
	38	140	51 102 ≥ 119	2.00 4.02 4.68	2.00 4.02 4.02	3.10 3.10 3.10	38 76 114	1.31 2.61 3.92	1.31 2.61 3.92	7.57 15.1 22.7
	38	191	51 102 ≥ 119	2.00 4.02 4.68	2.00 4.02 4.02	3.10 3.10 3.10	38 76 114	1.31 2.61 3.92	1.31 2.61 3.92	10.3 20.6 31.0
	89	89	51 ≥ 66	4.69 6.07	4.69 4.74	4.07 4.07	38 76 114	3.06 6.12 9.18	3.06 6.12 9.18	3.06 6.12 9.18
	89	140	51 ≥ 66	4.69 6.07	4.69 4.74	4.07 4.07	38 76 114	3.06 6.12 9.18	3.06 6.12 9.18	7.57 15.1 22.7
	89	191	51 ≥ 66	4.69 6.07	4.69 4.74	4.07 4.07	38 76 114	3.06 6.12 9.18	3.06 6.12 9.18	10.3 20.6 31.0

Design of Bolts and Dowels

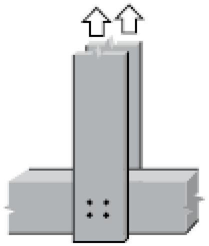
Designing of bolts and dowels tends to be a trial and error process; Selection tables from 7.4 from the Hand book assists:

Double Shear, Wood Side Plates



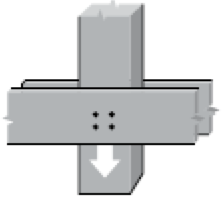
Parallel to grain loading

Type 4



Side plates parallel to grain loading. Member perpendicular to grain loading.

Type 5



Side plates perpendicular to grain loading. Member parallel to grain loading.

Type 6

Bolt and Dowel Selection Tables

2S Double Shear, Wood Side Plates

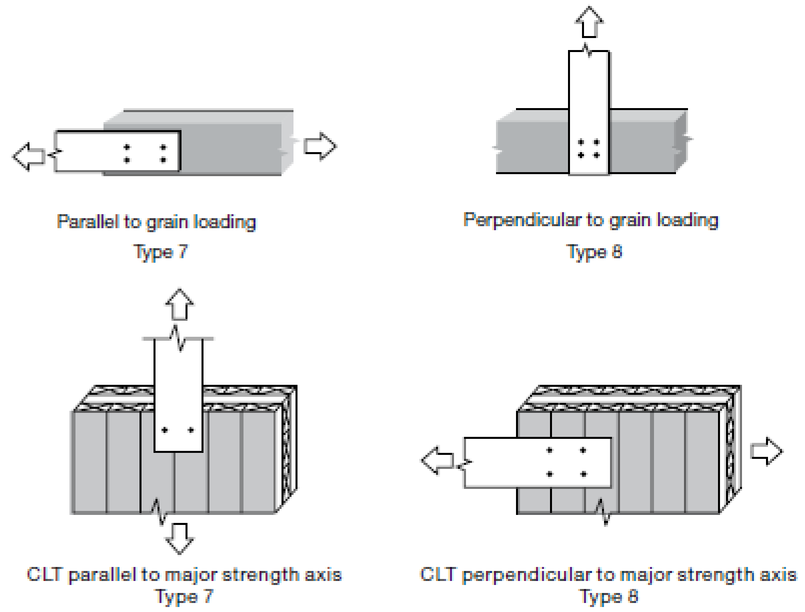
Factored lateral resistance - 1/2" Bolt or Dowel

Species **Sawn lumber**	Side plate thickness mm	Member thickness mm	PR_F – Row shear resistance: Spacing parallel to grain[1] mm	Capacity per bolt or dowel: Type 4 Side Par. Main Par. kN	Type 5 Side Par. Main Perp. kN	Type 6 Side Perp. Main Par. kN	G_{rT} – Group tear-out: Group width[2] mm	Tension resistance: Type 4 Side Par. Main Par. kN	Type 5 Side Par. Main Perp. kN	Type 6 Side Perp. Main Par. kN
D.Fir-L	38	38	51 102 ≥ 136	3.08 6.20 8.26	3.63 3.63 3.63	3.08 6.20 6.20	38 76 114	1.31 2.61 3.92	2.61 5.23 7.84	1.31 2.61 3.92
	38	89	51 102 ≥ 119	4.01 8.05 9.37	4.01 8.05 8.05	6.20 6.20 6.20	38 76 114	2.61 5.23 7.84	2.61 5.23 7.84	3.06 6.12 9.18
	38	140	51 102 ≥ 119	4.01 8.05 9.37	4.01 8.05 8.05	6.20 6.20 6.20	38 76 114	2.61 5.23 7.84	2.61 5.23 7.84	7.57 15.1 22.7
	38	191	51 102 ≥ 119	4.01 8.05 9.37	4.01 8.05 8.05	6.20 6.20 6.20	38 76 114	2.61 5.23 7.84	2.61 5.23 7.84	10.3 20.6 31.0
	89	89	51 57 ≥ 85	7.22 8.15 12.1	8.51 8.51 8.51	7.22 8.15 8.15	38 76 114	3.06 6.12 9.18	6.12 12.2 18.4	3.06 6.12 9.18
	89	140	51 ≥ 69	8.96 12.1	9.38 9.48	8.15 8.15	38 76 114	6.12 12.2 18.4	6.12 12.2 18.4	7.57 15.1 22.7
	89	191	51 ≥ 66	9.38 12.1	9.38 9.48	8.15 8.15	38 76 114	6.12 12.2 18.4	6.12 12.2 18.4	10.3 20.6 31.0

Design of Bolts and Dowels

Designing of bolts and dowels tends to be a trial and error process; Selection tables from 7.4 from the Hand book assists:

Single Shear, Steel Side Plates



Bolt Selection Tables

1S Single Shear, 6 mm Steel Side Plate

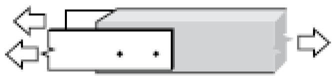
Factored lateral resistance based on A36 steel grade plates

Species Sawn lumber	Member thickness mm	1/2" bolt, Type 7 - Parallel to grain resistance, PR_F - Row shear: Bolt spacing parallel to grain[1] mm	1/2" bolt, Type 7, PR_F - Row shear: Capacity per bolt kN	1/2" bolt, Type 7, G_{rT} - Group tear-out: Group width[2] mm	1/2" bolt, Type 7, G_{rT} - Group tear-out: Tension resistance kN	1/2" bolt, Type 8, Q'_u: Perp. to grain yield resistance kN	5/8" bolt, Type 7 - Parallel to grain resistance, PR_F - Row shear: Bolt spacing parallel to grain[1] mm	5/8" bolt, Type 7, PR_F - Row shear: Capacity per bolt kN	5/8" bolt, Type 7, G_{rT} - Group tear-out: Group width[2] mm	5/8" bolt, Type 7, G_{rT} - Group tear-out: Tension resistance kN	5/8" bolt, Type 8, Q'_u: Perp. to grain yield resistance kN
D.Fir-L	38	51 ≥ 209	2.00 8.26	38 76 114	1.31 2.61 3.92	3.63	64 ≥ 252	2.50 9.90	48 95	1.66 3.32	4.38
	89	51 ≥ 92	4.69 8.50	38 76 114	3.06 6.12 9.18	5.67	64 ≥ 141	5.86 13.0	48 95	3.89 7.78	8.70
	140	51 ≥ 74	5.82 8.50	38 76 114	7.57 15.1 22.7	5.67	64 ≥ 114	7.28 13.0	48 95	9.62 19.2	8.70
	191	51 ≥ 54	7.95 8.50	38 76 114	10.3 20.6 31.0	5.67	64 ≥ 83	9.93 13.0	48 95	13.1 26.3	8.70
	241	≥ 51	8.50	38 76 114	13.0 26.1 39.1	5.67	64 ≥ 66	12.5 13.0	48 95	16.6 33.1	8.70
	292	≥ 51	8.50	38 76 114	15.8 31.6 47.4	5.67	≥ 64	13.0	48 95	20.1 40.1	8.70

Design of Bolts and Dowels

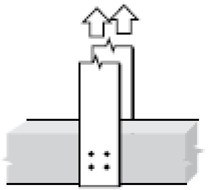
Designing of bolts and dowels tends to be a trial and error process; Selection tables from 7.4 from the Hand book assists:

Double Shear, Steel Side Plates



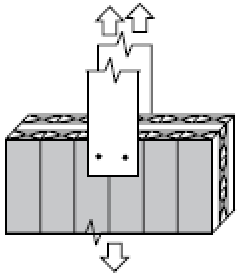
Parallel to grain loading

Type 9

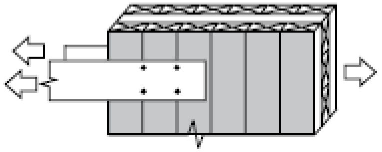


Perpendicular to grain loading

Type 10



CLT parallel to major strength axis
Type 9



CLT perpendicular to major strength axis
Type 10

Bolt Selection Tables

2S Double Shear, 6 mm Steel Side Plate

Factored lateral resistance based on A36 steel grade plates

		1/2" bolt Type 9 - Parallel to grain resistance				Type 10	5/8" bolt Type 9 - Parallel to grain resistance				Type 10
		PR_F - Row shear		G_{rT} - Group tear-out		Q'_u	PR_F - Row shear		G_{rT} - Group tear-out		Q'_u
Species **Sawn lumber**	Member thickness mm	Bolt spacing parallel to grain[1] mm	Capacity per bolt kN	Group width[2] mm	Tension resistance kN	Perp. to grain yield resistance kN	Bolt spacing parallel to grain[1] mm	Capacity per bolt kN	Group width[2] mm	Tension resistance kN	Perp. to grain yield resistance kN
D.Fir-L	38	51 ≥ 136	3.08 8.26	38 76 114	1.31 2.61 3.92	3.63	64 ≥ 164	3.85 9.9	48 95	1.66 3.32	4.38
	89	51 ≥ 120	7.22 17.0	38 76 114	3.06 6.12 9.18	8.51	64 ≥ 164	9.02 23.3	48 95	3.89 7.78	10.3
	140	51 ≥ 96	8.96 17.0	38 76 114	7.57 15.1 22.7	11.3	64 ≥ 148	11.2 26.1	48 95	9.62 19.2	16.1
	191	51 ≥ 71	12.2 17.0	38 76 114	10.3 20.6 31.0	11.3	64 ≥ 108	15.3 26.1	48 95	13.1 26.3	17.4
	241	51 ≥ 56	15.4 17.0	38 76 114	13.0 26.1 39.1	11.3	64 ≥ 86	19.3 26.1	48 95	16.6 33.1	17.4
	292	≥ 51	17.0	38 76 114	15.8 31.6 47.4	11.3	64 ≥ 71	23.4 26.1	48 95	20.1 40.1	17.4

Design of Bolts and Dowels

Designing of bolts and dowels tends to be a trial and error process; Selection tables from 7.4 from the Hand book assists:

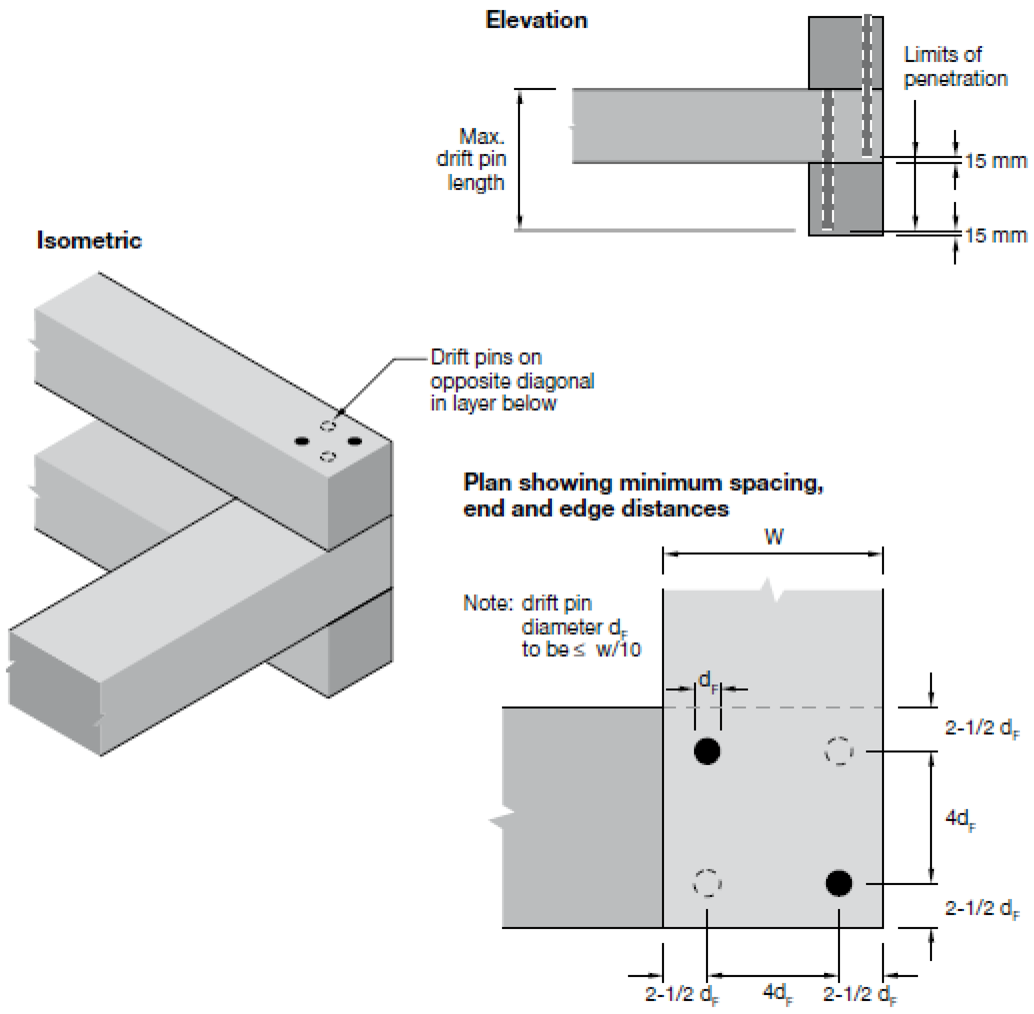
Table 7.10
Splitting resistance for members loaded perpendicular to grain, per mm thickness $Q'S_{rT,}$ (kN/mm)

Member depth	1/2" bolt		5/8" bolt		3/4" bolt		7/8" bolt		1" bolt	
	e_p	$Q'S_{rt}$	e_p	$Q'S_{rt}$	e_p	$Q'S_{rt}$	e_p	$Q'S_{rt}$	e_p	$Q'S_{rt}$
mm	mm	kN/mm	mm	kN/mm	mm	kN/mm	mm	kN/mm	mm	kN/mm
114	19	0.234	24	0.204	29	0.181	33	0.163	38	0.148
	57	0.105	57	0.105	57	0.105	57	0.105	57	0.105
140	19	0.292	24	0.256	29	0.229	33	0.207	38	0.190
	70	0.116	70	0.116	70	0.116	70	0.116	70	0.116
152	19	0.319	24	0.280	29	0.251	33	0.228	38	0.209
	76	0.121	76	0.121	76	0.121	76	0.121	76	0.121
184	19	0.391	24	0.345	29	0.310	33	0.283	38	0.260
	92	0.133	92	0.133	92	0.133	92	0.133	92	0.133
191	19	0.407	24	0.359	29	0.323	33	0.295	38	0.271
	96	0.135	96	0.135	96	0.135	96	0.135	96	0.135
228	19	0.490	24	0.433	29	0.391	33	0.358	38	0.330
	114	0.148	114	0.148	114	0.148	114	0.148	114	0.148
235	19	0.506	24	0.447	29	0.404	33	0.369	38	0.342
	118	0.150	118	0.150	118	0.150	118	0.150	118	0.150
241	19	0.519	24	0.459	29	0.415	33	0.380	38	0.351
	121	0.152	121	0.152	121	0.152	121	0.152	121	0.152
266	19	0.575	24	0.510	29	0.461	33	0.422	38	0.391
	133	0.160	133	0.160	133	0.160	133	0.160	133	0.160
286	19	0.620	24	0.550	29	0.497	33	0.456	38	0.423
	143	0.166	143	0.166	143	0.166	143	0.166	143	0.166
292	19	0.634	24	0.562	29	0.508	33	0.466	38	0.432
	146	0.167	146	0.167	146	0.167	146	0.167	146	0.167
304	19	0.661	24	0.586	29	0.530	33	0.487	38	0.451
	152	0.171	152	0.171	152	0.171	152	0.171	152	0.171
343	19	0.748	24	0.664	29	0.602	33	0.553	38	0.513
	172	0.181	172	0.181	172	0.181	172	0.181	172	0.181
380	19	0.832	24	0.739	29	0.670	33	0.616	38	0.572
	190	0.191	190	0.191	190	0.191	190	0.191	190	0.191
394	19	0.863	24	0.767	29	0.696	33	0.640	38	0.595
	197	0.195	197	0.195	197	0.195	197	0.195	197	0.195
418	19	0.917	24	0.815	29	0.740	33	0.681	38	0.633
	209	0.200	209	0.200	209	0.200	209	0.200	209	0.200
456	19	1.002	24	0.892	29	0.809	33	0.745	38	0.693
	228	0.209	228	0.209	228	0.209	228	0.209	228	0.209

Drift Pins

These are round steel dowels that do not have heads and are not threaded. The ends are shaped so that the pin can be inserted in a pre-drilled hole to minimize wood damage. Members are aligned and geld in place. Hole diameters are slightly undersized 1mm to give a tight fit.

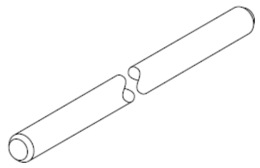
These are usually used where gravity or mechanical restraint prevents axial tension stress. These may also be used to anchor beams and columns.



Design of Drift Pins

The factored lateral resistance will be 60% of that of a bolt or dowel in single shear. At any shear plane with two overlapping members, a maximum of two drift pins can be considered as resisting shear forces.

Drift pins should be slightly shorter than the combined members depth so to prevent interface between the pins in successive layers as they are tightened.



Factored Lateral Strength

The factored lateral strength of a drift pin connection, P_r, Q_r, or N_r must be greater than or equal to the effects of the factored loads, as follows:

a) for parallel-to-grain loading, P_r is 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 is taken as $0.6\ QS_{r\,T}$;

c) for yielding resistance, N_r is taken as $0.6\ N_r$; and

d) for loads at angle θ to grain, the factored resistance on the joint, N_r, is 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

$P_{Gr\,T}$ = factored group tear out resistance

$T_{Nr\,T}$ = factored net tension resistance

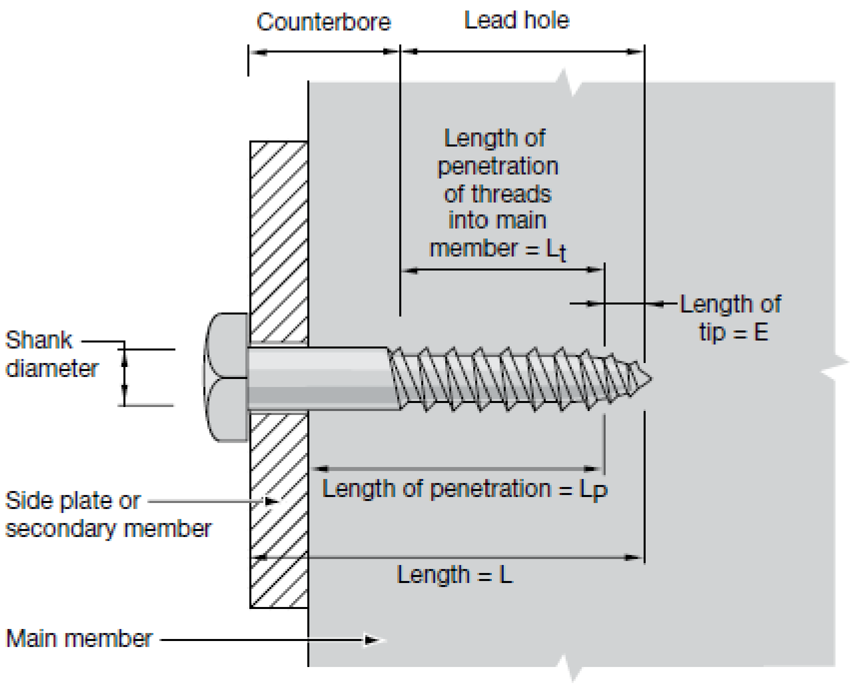
$QS_{r\,T}$ = factored splitting resistance

N_r = factored lateral yielding resistance

Lag Screws

Also known as lag bolts, are larger than screws, and installed in one face of the member. Shown right is a typical member. Very specific detailing is required in installation where a lead hole is made for the threaded portion and a counter bore for the unthreaded portion. The screw is turned with a wrench. If the procedures are not followed the load carrying capability is reduced.

They are used to anchor metal or wood in places where through bolts are undesirable or impractical. They may be loaded laterally or where withdrawal resistance is desired.



Drilling dimensions for lag screw hole

Drilling dimensions for lag screw holes	Counterbore	Lead hole Dense hardwoods	D.Fir-L	Hem-Fir, S-P-F Northern
Diameter	same as shank	65 to 85% of shank	60 to 75% of shank	40 to 70% of shank
Depth	same as shank	threaded length	threaded length	threaded length

Note: For lead hole diameter, the larger percentage applies to larger lag screws.

Spacing of Lag Screws

Same as bolted equations but with certain exceptions:

- For parallel to grain loading, minimum spacing between rows, S_C, may be 2 d_F, and
- For perpendicular to grain loading, minimum spacing between rows is :
  - $S_C \geq 2.5 d_F$ for L/d_F of 2 or less
  - $S_C \geq 5 d_F$ for L/d_F of 6 or greater

  Interpolation may be used for L/d_F ratios between 2 and 6

Design of Lag Screws

Yield Model (European) is used for the ultimate strength of a lag screw connection. The same yield equations as used for bolts are used. The embedding strengths are also the same, however modification factors are not incorporated (load, treatment, service conditions). Phi is also changed to 0.6 as opposed to 0.8 as is seen in bolts. Section 7.6 gives selection guidance in the handbook.

These resistancess are for members laterally loaded, for perpendicular, resistance is 2/3 the calculated value if wood plates are used, if steel , - 1/2.

f_1 = embedding strength of steel side plate, MPa

= $K_{sp} (\phi_{steel}/\phi_{wood})\ f_u$

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

= 400 MPa for ASTM A36/A 36M steel

= 450 MPa for CSA G40.21 steel, Grades 300W and 350W

= 310 MPa for cold-formed light gauge steel, Grade SS 230

ϕ_{steel} = resistance factor for steel plates

= 0.8 for mild steel referenced in CSA S16

= 0.5 for cold-formed light gauge steel referenced in CSA S136

ϕ_{wood} = resistance factor for wood members

= 0.6

Factored Lateral Strength Resistance

Factored lateral strength resistance parallel to the grain is calculated as

$$P_r = \phi P_u\ n_F\ J_G\ J_{PL}$$

Factored lateral strength resistance perpendicular to the grain is calculated as

$$Q_r = \phi Q_u\ n_F\ J_G\ J_{PL}$$

Factored lateral strength resistance for loads at an angle θ to the grain is calculated as

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

where

ϕ = 0.6

$P_u = p_u (K_D\ K_{SF}\ K_T)$

$Q_u = q_u (K_D\ K_{SF}\ K_T)$

J_G = factor for group of fasteners

J_{PL} = factor for reduced penetration

= 0.625 for penetration of 5d

= 1.0 for penetration of 8d or greater

Parameters p_u and q_u are determined in accordance with Clause 12.6.6.1.2 of CSA O86 in a manner similar to that used for bolts.

Design of Lag Screws

Withdrawal resistance also needs consideration. It is a function of the screw diameter and length, and specific gravity. The effective length in withdrawal is equal to the threaded length excluding the length of the tip. The screw should also be checked for resistance. These are the same criterion for ASTM A307 bolts.

Factored withdrawal resistance in side grain is calculated as

$$P_{rw} = \phi Y_w L_t n_F J_E$$

where

ϕ = 0.6

$Y_w = y_w (K_D K_{SF} K_T)$

y_w = basic withdrawal resistance per millimeter 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 (CSA O86 Table A.12.1)

J_x = 0.9 for CLT

= 1.0 for all other cases

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire-retardant treatment factor

L_t = length of threaded shank penetration into main member, mm

n_F = number of lag screws

J_E = end grain factor for lag screws

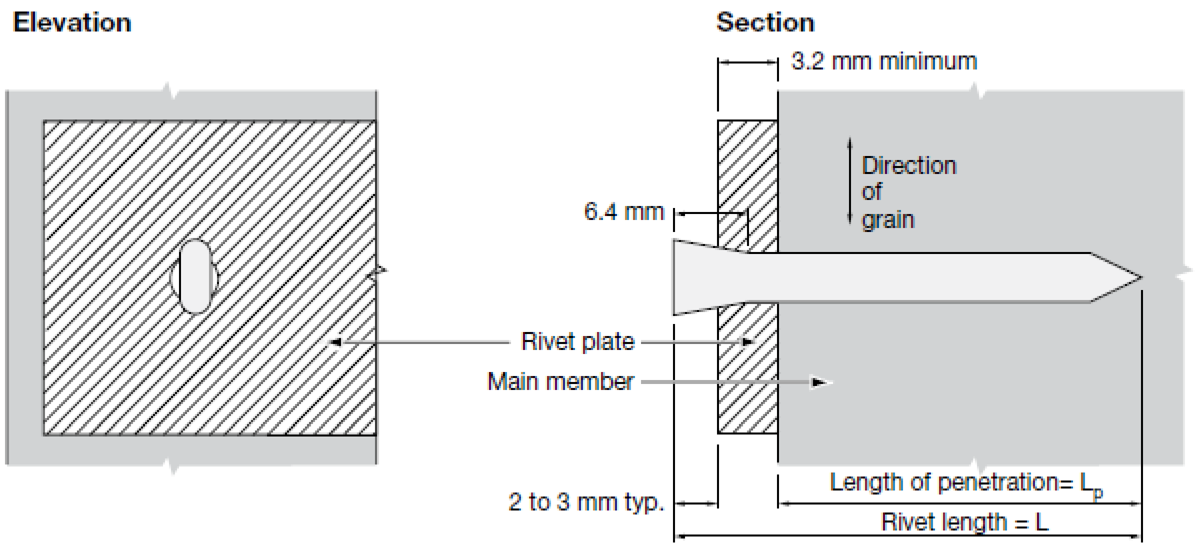
= 0.75 in end grain

= 0.67 in panel edge of CLT

= 1.0 for all other cases

Timber Rivets

Timber rivets were formerly known as glulam rivets. These are high strength fasteners that are now used for glulam and heavy timber construction as seen below,.



These are a favoured construction technique for glulam because they are stiffer and transfer greater loads, drilling is not required, simplifies fabrication as gross cross section can be relied on, easy to inspect ad favoured for as asethics reasons.

Rivets must be used with atleast 3.2mm thick steel plates.

Design of Timber Rivets

Usually found in truss connections, purlin beam connections, beam column connections, arch to base connections. Assembly is done on site to prevent damage.

It is a good alternative to lag screws because neither will penetrate the member full, and driving can be easier that turning. However, it requires more materials than lag screws would.

O86 Appendix A 12.7.2.3 gives formulas for these failure modes which is largely trail and error. To simplify the process, basic resistance values are tabulated for typical design configurations. When outside this, the appendix is used,

For timber rivet connections loaded parallel to the grain, three types of failures are considered:

1. Rivet yield accompanied by local crushing of the wood.
2. The wood fails in tension parallel to grain at the edge of the connection.
3. The wood fails in a plug shear mode around the rivets.

For timber rivet connections loaded perpendicular to the grain two types of failures are considered:

1. Rivet yield accompanied by local crushing of the wood.
2. Wood failure in tension perpendicular to grain at the edge of the connection.

Design of Timber Rivets

Factored lateral resistance parallel to the grain per rivet joint is calculated as

$$P_r = \phi P_u H$$

Factored lateral resistance perpendicular to the grain per rivet joint is calculated as

$$Q_r = \phi Q_u H$$

where

P_u = lesser of P_y and P_w

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

Q_u = lesser of Q_y and Q_w

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

n_R = number of rows of rivets parallel to direction of load

n_C = number of rivets per row

L_p = length of penetration of the rivet, mm
= overall rivet length - plate thickness - 3.2

p_w = lateral resistance parallel to grain (CSA O86 Clause 12.7.2.3 or Clause A.12.7.2.3.1)

q_w = lateral strength resistance for wood capacity perpendicular to grain loading (CSA O86 Table 12.7.2.5A or Clause A.12.7.2.3.2)

C_t = factor relating to edge distance (CSA O86 Table 12.7.2.5B or Clause A.12.7.2.3.2)

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire-retardant treatment factor

J_Y = side plate factor
= 0.8 for side plates between 3.2 mm and 4.7 mm in thickness
= 0.90 for plates between 4.7 mm and 6.4 mm in thickness
= 1.00 for side plates 6.4 mm and more in thickness

H = material factor
= 1.0 for Douglas Fir-Larch glulam
= 0.8 for Spruce-Pine glulam
= 0.5 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

Withdrawal resistance is calculated as

$$P_{rw} = \phi Y_w L_p n_R n_C$$

where

ϕ = 0.6

Y_w = $y_w (K_{SF} K_T)$

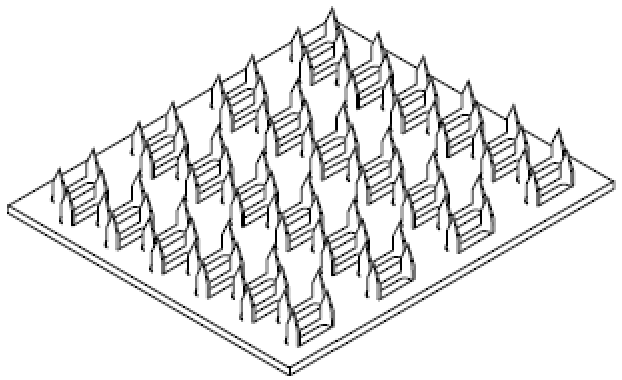
y_w = 13 N/mm for glulam
= 7 N/mm for sawn lumber

L_p, n_R and n_C are as defined for lateral loading.

Truss Plates

These are used to assemble dimensional lumber in roof and floor trusses. The are produced by punching 16-20mm gauge galvanized steel. The pattern of the teeth varies to the manufacturer.

These would be installed every joint of a light frame wood truss. The plate size would vary depending on the loads that need to be transferred. These would be embedded via a hydraulic press and by the manufacturer. So the design is largely proprietary as a consequence. Though this must follow specific standards (tightfitting, non deformed, direction normal to the surface of the lumber, fully embedded to a specific depth, and the quality of the lumber)



Truss plates must be designed so that the following criteria are satisfied:

1. Factored ultimate lateral resistance of the teeth, N_r > factored lateral load, N_f
2. Factored tensile resistance of the plate T_r > factored tensile force, T_f
3. Factored shear resistance of the plates V_r > factored shear force, V_f
4. Factored lateral slip resistance N_{rs} > specified lateral load, N_s

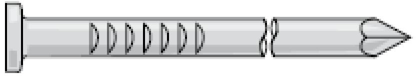
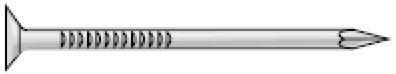
Nails and Spikes

Manufactured in many lengths and diameters (see text book figure 11.27). Nails are usually 1 to 6 inches and spikes are 4 inches to 14 inches. Nails are normally made of low or medium carbon steels or aluminum.

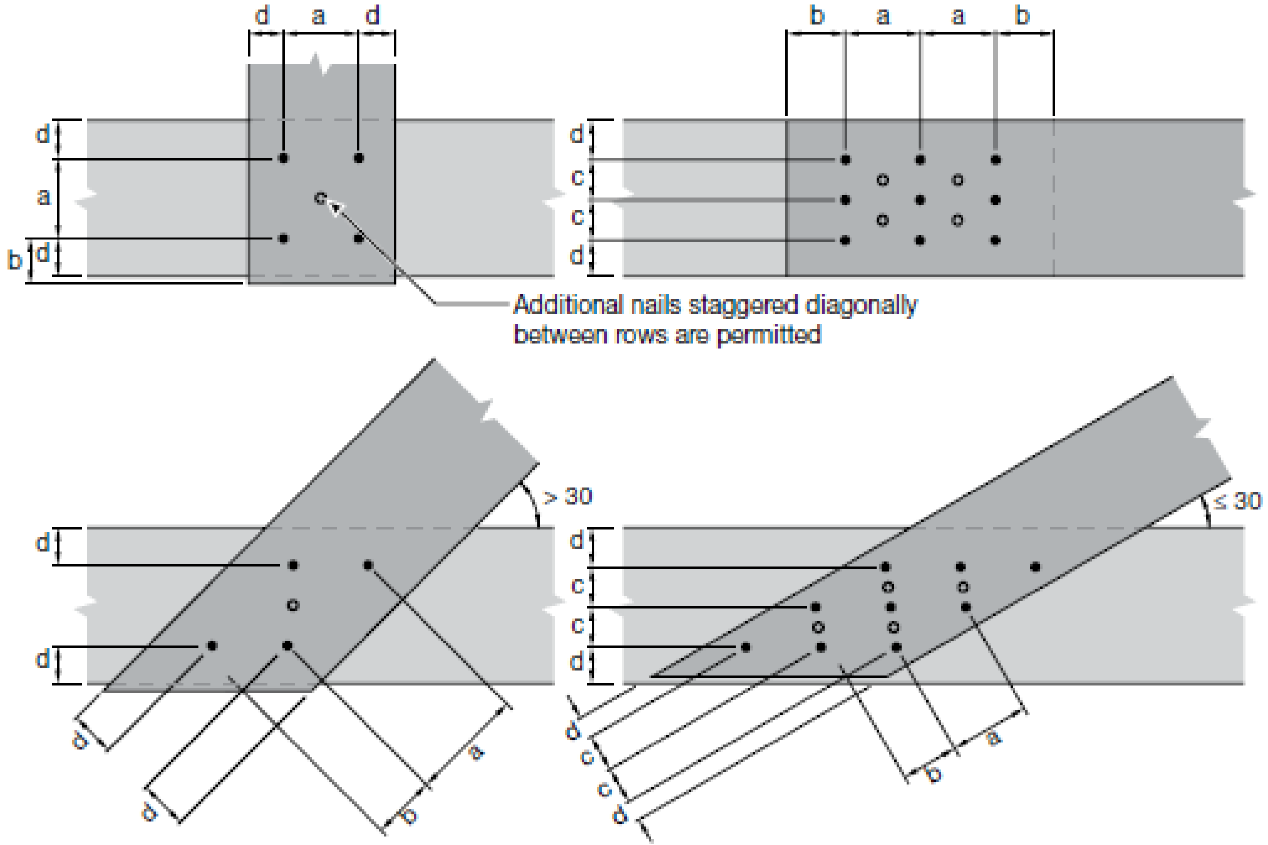
Splitting can be reduced by blunting the point of the nail before driving. A long diamond point may be easier to drive but it can split harder wood species. Uncoating in cedar species can cause staining of the wood galvanization is an alternative.

Nails are common in all types of construction, but seen mostly with wood frame, post and beam, heavy timber, shear walls, nailed gussets and panel assemblies.

FIGURE 11.27
Types of nails

Type of nail	Head	Shank	Point	Material	Finishes and coatings	Common lengths mm	in.
Common (Spike)	F	C, S	D	S, E	B	100 to 350 	4 to 14
Eavestrough (Spike)	Cs, F	C, S	D, N	S	B, Ghd	125 to 250 	5 to 10
Standard or Common	F	C, R, S	D	A, S, E	B, Ge	25 to 150 	1 to 6

Edge Distances for Nails and Spikes



Edge Distances for Nails and Spikes

- Fig 7.4 handbook

Minimum required spacing, end and edge distances for nails and spikes			D.Fir-L Hem-Fir				S-P-F Northern			
			Min. spacing parallel to grain	Min. end distance	Min. spacing perp. to grain	Min. edge distance	Min. spacing parallel to grain	Min. end distance	Min. spacing perp. to grain	Min. edge distance
Type	Length in.	Diameter mm	a mm	b mm	c mm	d mm	a mm	b mm	c mm	d mm
Common wire nails	1	1.83	37	28	19	10	30	22	15	8
	1.25	2.03	41	31	21	11	33	25	17	9
	1.5	2.34	47	36	24	12	38	29	19	10
	1.5	2.52	51	38	26	13	41	31	21	11
	1.75	2.64	53	40	27	14	43	32	22	11
	2	2.84	57	43	29	15	46	35	23	12
	2	2.87	58	44	29	15	46	35	23	12
	2.25	2.95	59	45	30	15	48	36	24	12
	2.5	3.25	65	49	33	17	52	39	26	13
	2.5	3.33	67	50	34	17	54	40	27	14
	2.75	3.33	67	50	34	17	54	40	27	14
	3	3.66	74	55	37	19	59	44	30	15
	3	3.76	76	57	38	19	61	46	31	16
	3.5	4.06	82	61	41	21	65	49	33	17
	3.5	4.12	83	62	42	21	66	50	33	17
	4	4.88	98	74	49	25	79	59	40	20
	4.5	5.26	106	79	53	27	85	64	43	22
	4.5	5.38	108	81	54	27	87	65	44	22
	5	5.74	115	87	58	29	92	69	46	23
	5	5.89	118	89	59	30	95	71	48	24
	5.5	6.2	124	93	62	31	100	75	50	25
	5.5	6.4	128	96	64	32	103	77	52	26
	6	6.66	134	100	67	34	107	80	54	27
	6	7.01	141	106	71	36	113	85	57	29
Common spikes	4	6.4	128	96	64	32	103	77	52	26
	6	7.62	153	115	77	39	122	92	61	31
	8	8.23	165	124	83	42	132	99	66	33
Common spiral nails	2-1/2	2.77	56	42	28	14	45	34	23	12
	3	3.1	62	47	31	16	50	38	25	13
	3-1/2	3.86	78	58	39	20	62	47	31	16
	4	4.33	87	65	44	22	70	52	35	18
	5	4.88	98	74	49	25	[illegible]	59	40	20

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Design of Nails and Spikes

O86 applies only for common round steel wire nails and spikes and common spiral nails spiralled to head as defined in CSA Standard B111 Wire Nails, Spikes and Staples.

Nails are usually specified by the type and length in inches. Nails are available in lengths from 1/2 to 6 inches and spikes range in size from 4 to 14 inches.

Factored lateral strength resistance is calculated as

$N_r = \phi N_u n_F n_s J_F$

where

ϕ = 0.8

$N_u = n_u(K_D K_{SF} K_T)$

n_u = unit lateral strength resistance pe nail or spike, N

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire retardant treatment factor

n_F = number of fasteners in the connection

n_s = number of shear planes per nail or spike

$J_F = J_E J_A J_B J_D$

J_E = factor for nailing into end grain

J_A = factor for toe nailing

J_B = factor for nail clinching

J_D = factor for diaphragm and shearwall construction

K_D = 1.0 when the loading is "standard" term
= 1.15 for short term loading (e.g., dead plus wind)
= 0.65 for permanent loads (e.g., dead loads alone)

K_{SF} = 1.0 when the service conditions are "dry" and the lumber is seasoned (moisture content ≤ 19%) prior to fabrication
= 0.8 for unseasoned lumber in dry service conditions under lateral loads
= 0.67 for wet service conditions under lateral loads, regardless of seasoning

K_T = 1.0 when the wood is not treated with a fire retardant or other strength-reducing chemical.

For wood treated with a fire retardant or other strength-reducing chemicals, see the Commentary for further guidance.

J_E = 1.0 when the nails are used in side grain
= 0.67 for end grain use

J_A = 1.0 when the nails are not used as toe nails
= 0.83 for toe nailing provided the nail is started at approximately 1/3 the nail length from the end of the piece and at an angle of 30° to the grain

J_B = 1.0 for unclinched nails in single shear, or nails in a multiple shear connection
= 1.6 when nails are in single shear and clinched on the far side
(Note: The clinched portion of the nail should be at least three nail diameters long.)

J_D = 1.0 when nails are not used as a fastener in shearwalls or diaphragms
= 1.3 when nails are used to fasten sheathing in shearwall and diaphragm construction

Design of Nails and Spikes

Unit Lateral Strength Resistance

The unit lateral strength resistance, n_u (per shear plane), is taken as the smallest value calculated in accordance with items (a) to (g) as follows:

For two-member connections, only Items (a), (b) and (d) to (g) are considered valid.

For three-member connections, where nails penetrate all three members and meet the minimum penetration requirements, only Items (a), (c), (d) and (g) are 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

d_F = nail or spike diameter, mm

f_2 = embedding strength of main member, MPa

= $50G\,(1 - 0.01d_F)\,J_x$

where

G = mean relative density

J_x = 0.9 for CLT

= 1.0 for 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

f_3 = embedment strength of main member where failure is fastener yielding, MPa

= $110G^{1.8}(1 - 0.01d_F)\,J_x$

f_y = nail or spike yield strength, MPa

= $50(16 - d_F)$

f_1 = embedment strength of side member, MPa

For lumber and CLT:

$f_1 = 50G(1 - 0.01d_F)\,J_x$

For structural panel side plates:

$f_1 = 104G(1 - 0.1d_F)$

where

G = 0.49 for DFP

= 0.42 for CSP and OSB

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

= 2.7 for cold-formed light gauge steel referenced in CSA S136

ϕ_{steel} = resistance factor for steel member in connections with nails and spikes

= 0.8 for mild steel referenced in CSA S16

= 0.4 for cold-formed light gauge steel referenced in CSA S136

ϕ_{wood} = resistance factor for steel member in connections with nails and spikes

= 0.8

f_u = specified minimum tensile strength of steel

= 400 MPa for ASTM A 36/A 36M

= 450 MPa for CSA G40.21, Grades 300W and 350W

= 310 MPa for cold-formed light gauge steel, Grade SS 230

Design of Nails and Spikes

Withdrawal resistance is based on the squeezing effect of broken and displaced fibres from the installation process. With time this will relax., and the depth, duration of loading and service conditions will become more important to the strength.

If nails are clinched, withdrawal resistance will increase by 60%. Nails in end grain have no resistance. Toe nails are partially resisted in the end grain of the wood, about 67% when installed in the side grain

Factored lateral withdrawal resistance is calculated as

$$P_{rw} = \phi \, Y_w \, L_p \, n_F \, J_A \, J_B$$

where

ϕ = 0.6

$Y_w = y_w \, (K_{SF} \, K_T)$

y_w = withdrawal resistance per mm of penetration, N/mm

$= 16.4 \, d_f^{0.82} \, G^{2.2} J_x$

where

d_F = nail or spike diameter, mm

G = mean relative density (CSA O86 Table A.12.1)

J_x = 0.9 for CLT

= 1.0 for all other cases

K_{SF} = service condition factor

K_T = fire-retardant treatment factor

L_p = penetration length into main member, mm

n_F = number of nails or spikes

J_A = factor for toe-nailing

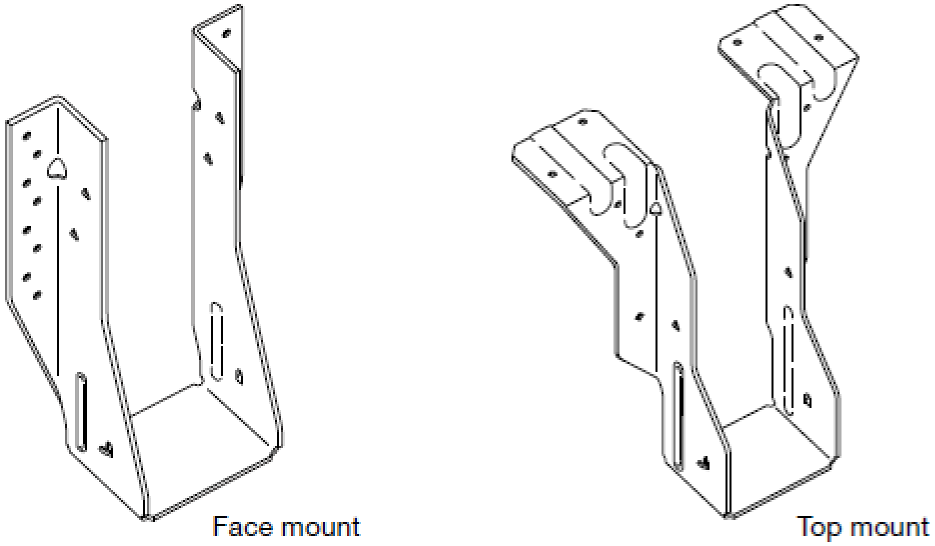
J_B = factor for nail-clinching

Design of Joist Hangers

Proprietary product generally made of 16 to 20 gauge galvanized steel. Most are face mount, but top mount is possible for heavier loads.

Installation is with the manufacturer's requirements. Where specifying a joist hanger, designers must ensure that the hanger is at least half the depth of the joist, and capable of providing lateral support; the hanger is fastened to both the header and joist. Detailing to splitting must be considered as well as lateral support.

Since joist hangers are proprietary, CSA O86 provides a method for determining design values but does not provide the design values in the Standard. Instead it requires that joist hangers be tested in accordance with ASTM Standard D 1761 *Standard Test Methods for Mechanical Fasteners in Wood*.



The factored resistance of joist hangers is calculated from the following formula:

$$N_r = \phi N_u$$

where

$\phi = 0.6$

$N_u = n_u (K_D K_{SF} K_T)$

n_u = ultimate resistance of the hanger. The value of n_u is taken from the CCMC evaluation report.

Design of Wood Screws

CSA O86 is applicable only for wood screws that meet the requirements of ASME B18.6.1 *Wood Screws*. Procedure is very similar to nails with small exceptions

For two- or three-member joints, the factored lateral strength resistance, N_r, of a wood screw connection is based on the same yield equation used for nail design. Yield strength values for wood screws are provided in Table 11.10.

$N_r = \phi N_u n_F n_s J_A J_E$

$\phi = 0.8$

$N_u = n_u (K_D K_{SF} K_T)$

n_u = unit lateral strength resistance, N

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire retardant treatment factor

n_F = number of fasteners in the connection

n_s = number of shear planes per screw

J_A = toe screwing factor

J_E = factor for fastening into end grain

K_D = 1.0 when the loading is "standard" term
= 1.15 for short term loading (e.g., dead plus wind
= 0.65 for permanent loads (e.g., dead loads alone)

K_{SF} = 1.0 when the service conditions are "dry" and the lumber is seasoned (moisture content ≤ 19%) prior to fabrication
= 0.8 for unseasoned lumber in dry service conditions under lateral loads
= 0.67 for wet service conditions under lateral loads, regardless of seasoning

K_T = 1.0 when the wood is not treated with a fire retardant or other strength reducing chemical.
For wood treated with a fire retardant or other strength-reducing chemicals, see the Commentary for further guidance.

J_E = 1.0 when the screws are used in side grain
= 0.67 for end grain use.

J_A = 1.0 when the screws are not toe-screwed
= 0.83 for toe-screwing provided the screw is started at approximately 1/3 of the screw length from the end of the piece and at an angle of 30° to the grain.

Design of Wood Screws

The factored withdrawal resistance is taken as the lesser of the factored screw withdrawal resistance of the main member or the factored head pull-through resistance of the side member.

For a two-member joint connected with wood screws, the factored withdrawal resistance, P_{rw}, of the main member is taken as follows:

$$P_{rw} = \phi Y_w L_{pt} n_F$$

where

ϕ = 0.6

Y_w = $y_w (K_D K_T K_{SF})$

y_w = basic withdrawal resistance per millimetre of the threaded shank penetration in main member, N/mm

= $59 d_F^{0.82} G^{1.77} J_x$

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire retardant treatment factor

d_F = nominal wood screw diameter, mm

G = mean relative density of main member

J_x = 0.9 for CLT

= 1.0 for 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 is the maximum threaded length within any member other than the head-side member.

For connections with steel side plates, the factored head pull-through resistance, P_{pt}, is 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 is taken as follows:

$$P_{pt} = 65 \phi t_1 n_F K_D$$

where

ϕ = 0.4

t_1 = side plate thickness, mm

d_w = diameter of screw head, mm

f_u = specified minimum tensile strength of steel, MPa

= 400 MPa for ASTM A 36/A36M steel

= 450 MPa for CSA G40.21 steel, Grade 300W and 350W

= 310 MPa for cold-formed light gauge steel, Grades SS 230

n_F = number of wood screws in the connection

K_D = load duration factor

Design of Wood Screws

When using selection tables note the omissions of information and how the equations are re-arranged

For two- or three-member joints, the factored lateral strength resistance, N_r, of a wood screw connection is based on the same yield equation used for nail design. Yield strength values for wood screws are provided in Table 11.10.

$N_r = \phi N_u n_F n_s J_A J_E$

$\phi = 0.8$

$N_u = n_u (K_D K_{SF} K_T)$

n_u = unit lateral strength resistance, N

K_D = load duration factor

K_{SF} = service condition factor

K_T = fire retardant treatment factor

n_F = number of fasteners in the connection

n_s = number of shear planes per screw

J_A = toe screwing factor

J_E = factor for fastening into end grain

$$N'_r = N'_r n_s n_F K' J_F$$

Where:

$N'_r n_S$ are the factored lateral resistances given in the Screw Selection Tables

n_S = is the number of shear planes. (n_S is equal to 1 in the Screw Selection Tables)

n_F = number of screws

K' and J_F are composite modification factors given below

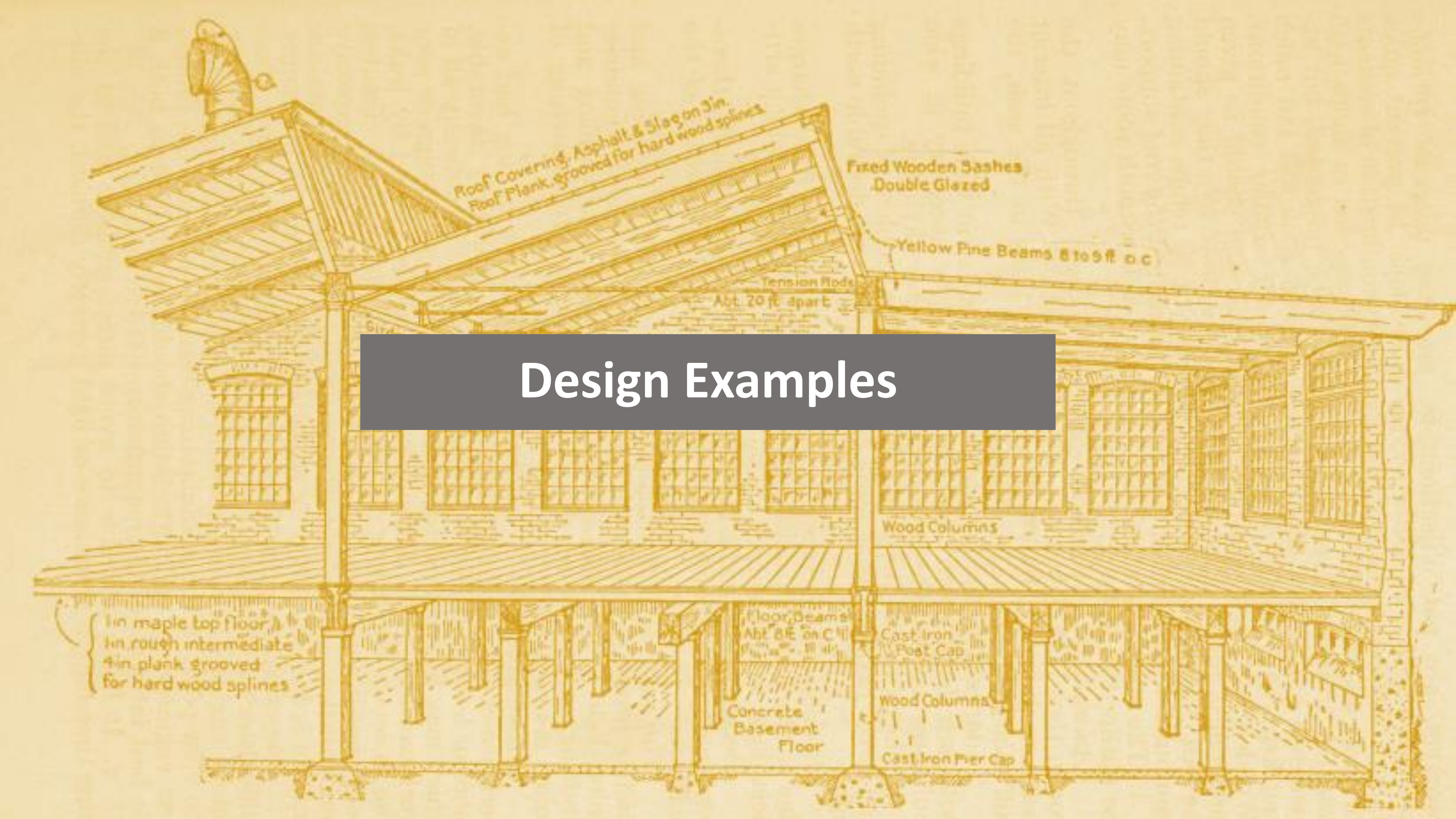
Screw Selection Tables

1S Single Shear, Sawn Lumber Main Member

Basic factored lateral resistance

Side plate thickness **Sawn lumber** (mm)	Screw gauge number	Screw length (in.)	Screw diameter (mm)	D.Fir-L Minimum penetration (mm)	D.Fir-L $N'_r \cdot n_s$ (kN)	Hem-Fir Minimum penetration (mm)	Hem-Fir $N'_r \cdot n_s$ (kN)	S-P-F Minimum penetration (mm)	S-P-F $N'_r \cdot n_s$ (kN)	North Minimum penetration (mm)	North $N'_r \cdot n_s$ (kN)
38	6	2.5	3.5	18	0.612	18	0.582	18	0.541	18	0.467
38	8	2.5	4.16	21	0.833	21	0.791	21	0.735	21	0.634
38	8	3	4.16	21	0.833	21	0.791	21	0.735	21	0.634
				4	1.07	24	1.02	24	0.947	24	0.798
				4	1.07	24	1.02	24	0.947	24	0.798
				7	1.33	27	1.25	27	1.14	27	0.949
				7	1.33	27	1.25	27	1.14	27	0.949
				Maximum enetration nm)	$N'_r \cdot n_s$ (kN)	≥Maximum penetration (mm)	$N'_r \cdot n_s$ (kN)	≥Maximum penetration (mm)	$N'_r \cdot n_s$ (kN)	≥Maximum penetration (mm)	$N'_r \cdot n_s$ (kN)
				6	0.718	26	0.681	26	0.632	26	0.543
				6	0.906	26	0.860	26	0.798	26	0.687
				2	1.02	33	0.973	34	0.914	36	0.805
38	10	2.5	4.82	26	1.10	26	1.04	26	0.969	26	0.816
38	10	3	4.82	36	1.28	36	1.23	37	1.15	38	0.979
38	12	3	5.48	38	1.55	38	1.45	38	1.33	38	1.11
38	12	4	5.48	41	1.60	41	1.51	43	1.40	45	1.20

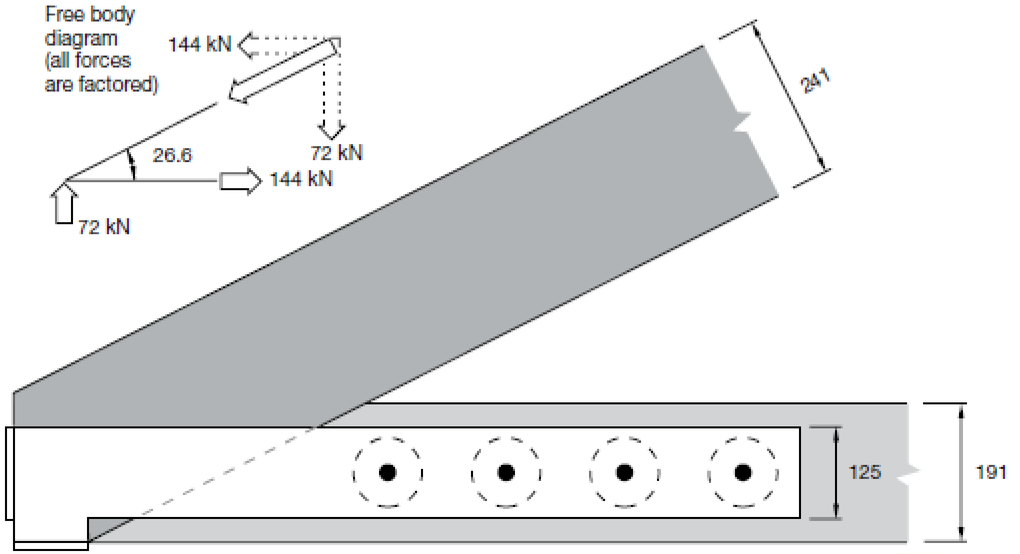
Design Examples



Example 1: Tension Member

Check the adequacy of the bottom chord member shown. The member is 140 x 191 mm No.1 grade D.Fir-L, connected with a single row of 4” diameter shear plates with 3/4” diameter bolts. The conditions are as follows:

- **factored tensile force = 144 kN (dead plus snow load)**
- dry service conditions
- untreated
- truss spacing exceeds 610 mm



Example 1: Tension Member

For sawn lumber or sawn timber:

$T_{rN} = \phi\ F_t\ A_n\ K_{zt}$ (N)

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

f_t = specified tensile strength (MPa)

A_n = net area of cross section (mm_2)

$\phi = 0.9$

For glulam:

$T_{rN} = \phi\ F_{tN}\ A_n$ (N)

$T_{rG} = \phi\ F_{tG}\ A_g$ (N)

$F_{tN} = f_{tN}\ (K_D\ K_H\ K_{St}\ K_T)$ (MPa)

$F_{tG} = f_{tG}\ (K_D\ K_H\ K_{St}\ K_T)$ (MPa)

f_{tN} = specified tensile strength at net section (MPa)

f_{tG} = specified tensile strength at gross section (MPa)

A_n = net area of cross section (mm_2)

A_g = gross area of cross section (mm_2)

$\phi = 0.9$

Example 1: Tension Member

Calculation

T_f = (factored tensile load)= 144 kN

Since the chord is connected with shear plates, the tensile resistance at net section must be checked.

From the Tension Member Selection Tables:

T_r = 185 kN > 144 kN *Acceptable*

Use 140 x 191 mm No.1 grade D.Fir-L.

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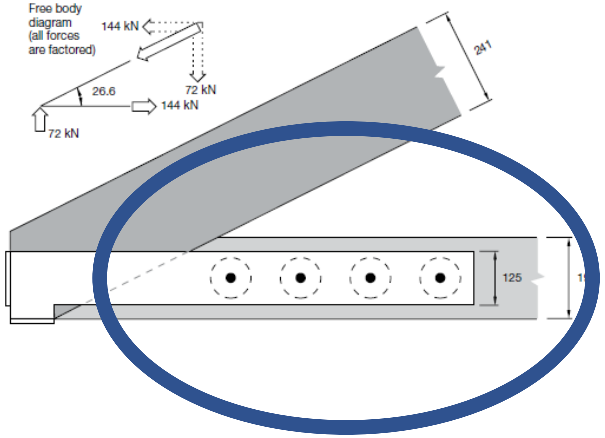
Tension Member Selection Tables

140 mm **Sawn Timber**

		Select Structural					No.1				
			T_{rN} for use with shear plates or split rings (kN)					T_{rN} for use with shear plates or split rings (kN)			
	Size (b × d)	T_r	2-5/8" SH PL		4" SH PL		T_r	2-5/8" SH PL		4" SH PL	
Species	mm	kN	1 row	2 rows	1 row	2 rows	kN	1 row	2 rows	1 row	2 rows
D.Fir-L	140 × 140	245	196				186	148			
	140 × 191	309	264		245		234	199		185	
	140 × 241	334	295	256	279		234	207	179	195	
	140 × 292	368	332	297	318		258	233	208	223	
	140 × 343	389	357	325	344	299	272	250	228	241	209
	140 × 394	397	369	340	357	317	278	258	238	250	222
Hem-Fir	140 × 140	181	145				138	110			
	140 × 191	228	195		181		173	148		137	
	140 × 241	247	218	189	207		174	153	133	145	
	140 × 292	272	246	220	235		191	173	154	165	
	140 × 343	288	264	241	255	221	202	186	169	179	156
	140 × 394	294	273	252	264	235	207	192	177	186	165
S-P-F	140 × 140	170	136				128	103			
	140 × 191	214	182		169		162	138		128	
	140 × 241	234	207	179	195		164	145	125	137	
Northern	140 × 140	161	128				122	97.1			
	140 × 191	202	172		160		153	131		121	
	140 × 241	217	192	166	181		154	136	118	128	



Example 2: Compression

Design columns for the following conditions:

- specified dead load = 2.0 kPa
- specified live load = 2.4 kPa
- tributary area = 25 m^2
- unbraced length = 5 m
- dry service conditions
- untreated
- column effectively pinned at both ends ($K_e = 1.0$)
- no eccentricity considered

Use No.1 D.Fir-L.

Example 2: Compression

Available sizes (sawn)

Grades	Column sizes for all grades (mm)
Select Structural, No.1, No.2	140 × 140
	191 × 191
	241 × 241
	292 × 292
	140 × 191
	191 × 241
	241 × 292

Available sizes (glulam)

Species and stress grades	Column thickness for all species and grades (mm)
D.Fir-L: 16c-E Spruce-Pine: 12c-E	80
	130
	175
	215
	265
	315
	365

Example 2: Compression

Sawn Timber

P_r is the lesser of:

$$P_{rd} = \phi F_c A K_{Zcd} K_{Cd} \text{ or } P_{rb} = \phi F_c A K_{Zcb} K_{Cb}$$

where:

ϕF_c = factored compressive resistance strength (MPa) given in Table 3.6

K_{Zc} = size factor

$K_{Zcd} = 6.3\,(dL_d)^{-0.13} \leq 1.3$ for buckling in direction of d

$K_{Zcb} = 6.3\,(bL_b)^{-0.13} \leq 1.3$ for buckling in direction of b

K_C = slenderness factor

$$K_{Cd} = \left[1.0 + \frac{F_c}{E'} K_{Zcd} C_{cd}^3\right]^{-1} \text{ for buckling in direction of d}$$

$$K_{Cb} = \left[1.0 + \frac{F_c}{E'} K_{Zcb} C_{cb}^3\right]^{-1} \text{ for buckling in direction of b}$$

F_c / E' = strength to stiffness ratio given in Table 3.7

$C_{cd} = \frac{K_e L_d}{d}$, $C_{cb} = \frac{K_e L_d}{b}$ (C_{cd} or C_{cb} > 50 is not permitted)

K_e = effective length factor, given in Figure 3.1

L_b, L_d = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

Example 2: Compression

Table 3.6

Factored compressive strength for sawn timbers $\phi\ F_c$ (MPa)[1]

Service conditions		Dry service			Wet service		
Load duration[2]		Std.	Perm.	Short term	Std.	Perm.	Short term
Species	Grade						
D.Fir-L	Sel Str	11.0	7.18	12.7	10.0	6.53	11.6
	No.1	9.76	6.34	11.2	8.88	5.77	10.2
	No.2	6.00	3.90	6.90	5.46	3.55	6.28
Hem-Fir	Sel Str	9.04	5.88	10.4	8.23	5.35	9.46
	No.1	8.00	5.20	9.20	7.28	4.73	8.37
	No.2	4.88	3.17	5.61	4.44	2.89	5.11
S-P-F	Sel Str	7.92	5.15	9.11	7.21	4.68	8.29
	No.1	6.96	4.52	8.00	6.33	4.12	7.28
	No.2	4.32	2.81	4.97	3.93	2.56	4.52
Northern	Sel Str	6.00	3.90	6.90	5.46	3.55	6.28
	No.1	5.36	3.48	6.16	4.88	3.17	5.61
	No.2	3.28	2.13	3.77	2.98	1.94	3.43

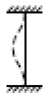
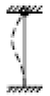
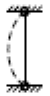
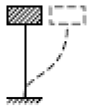
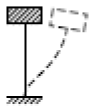
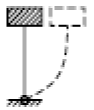
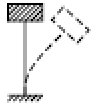
Example 2: Compression

Table 3.7

Table 3.7
Strength to stiffness ratio for sawn timbers
F_c / E′ (x10^{-6})

Service conditions		Dry service			Wet service		
Load duration[2]		Std.	Perm.	Short term	Std.	Perm.	Short term
Species	Grade						
D.Fir-L	Sel Str	49.3	32.0	56.7	44.9	29.2	51.6
	No.1	53.6	34.9	61.7	48.8	31.7	56.1
	No.2	35.7	23.2	41.1	32.5	21.1	37.4
Hem-Fir	Sel Str	46.1	30.0	53.0	42.0	27.3	48.3
	No.1	47.6	31.0	54.8	43.3	28.2	49.8
	No.2	31.7	20.6	36.4	28.8	18.7	33.2
S-P-F	Sel Str	47.1	30.6	54.2	42.9	27.9	49.3
	No.1	49.7	32.3	57.2	45.2	29.4	52.0
	No.2	34.3	22.3	39.4	31.2	20.3	35.9
Northern	Sel Str	39.0	25.3	44.8	35.5	23.0	40.8
	No.1	38.3	24.9	44.0	34.8	22.6	40.1
	No.2	29.3	19.0	33.7	26.7	17.3	30.6

Fig 3.1

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, 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 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 nor restrained against rotation at the other end	2.00	

Example 2: Compression

Glulam

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

where:

ϕF_c = factored compressive resistance strength (MPa) given in Table 3.8

K_{Zcg} = $0.68\,(Z)^{-0.13} \leq 1.0$, where Z = member volume in m^3

K_C = slenderness factor

$$= \left[1.0 + \frac{F_c}{E'} K_{Zcg} C_c^3\right]^{-1}$$

F_c / E' = strength to stiffness ratio given in Table 3.9

C_c = the greater of $\frac{K_e L_d}{d}$ or $\frac{K_e L_d}{b}$ ($C_c > 50$ is not permitted)

K_e = effective length factor, given in Figure 3.1

L_b, L_d = unsupported length associated with d or b (mm)

d = depth of member (mm)

b = thickness of member (mm)

Example 2: Compression

Calculation

Tributary area > 20 m_2

Therefore, live load reduction factor from Part 4 of the *NBC* is:

$$= 0.3 + \sqrt{\frac{9.8}{25}} = 0.926$$

Total factored load:

$P_f = (1.25 \times 2.0 + 1.5 \times 0.926 \times 2.4) \times 25$

$= 146$ kN

From Column Selection Tables select

191 x 191 mm:

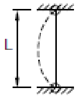
$P_r = 186$ kN > 146 kN *Acceptable*

Use 191 x 191 mm No.1 D.Fir-L column.

Column Selection Tables

DF-L No.1

Sawn Timbers



$K_e = 1.0$

		Square timbers				Rectangular timbers					
	b (mm)	140	191	241	292	140		191		241	
	d (mm)	140	191	241	292	191		241		292	
D.Fir-L No.1	L m	P_r kN	P_r kN	P_r kN	P_r kN	P_{rx} kN	P_{ry} kN	P_{rx} kN	P_{ry} kN	P_{rx} kN	P_{ry} kN
	2.0	198	393	629	915	288	270	499	496	756	763
	2.5	168	360	593	874	264	229	470	454	722	719
	3.0	138	324	556	834	238	189	440	409	688	673
	3.5	112	288	515	791	211	153	409	363	653	625
	4.0	89.5	251	474	747	184	122	375	317	617	574
	4.5	71.3	217	431	701	159	97.3	341	274	578	522
	5.0	57.0	186	389	653	136	77.8	308	235	539	471
	5.5	45.8	159	348	605	116	62.5	276	200	499	421
	6.0	37.1	135	310	557	99.0	50.6	245	170	460	375
	6.5	30.3	115	274	510	84.3	41.3	217	145	421	332
	7.0	25.0	98.0	242	465	71.8	34.1	192	124	384	294
	7.5		83.8	214	423	61.4		170	106	349	259
	8.0		71.9	189	383	52.7		150	90.7	316	229
	8.5		62.0	167	347	45.4		132	78.2	286	202
	9.0		53.6	147	313	39.3		117	67.7	259	178

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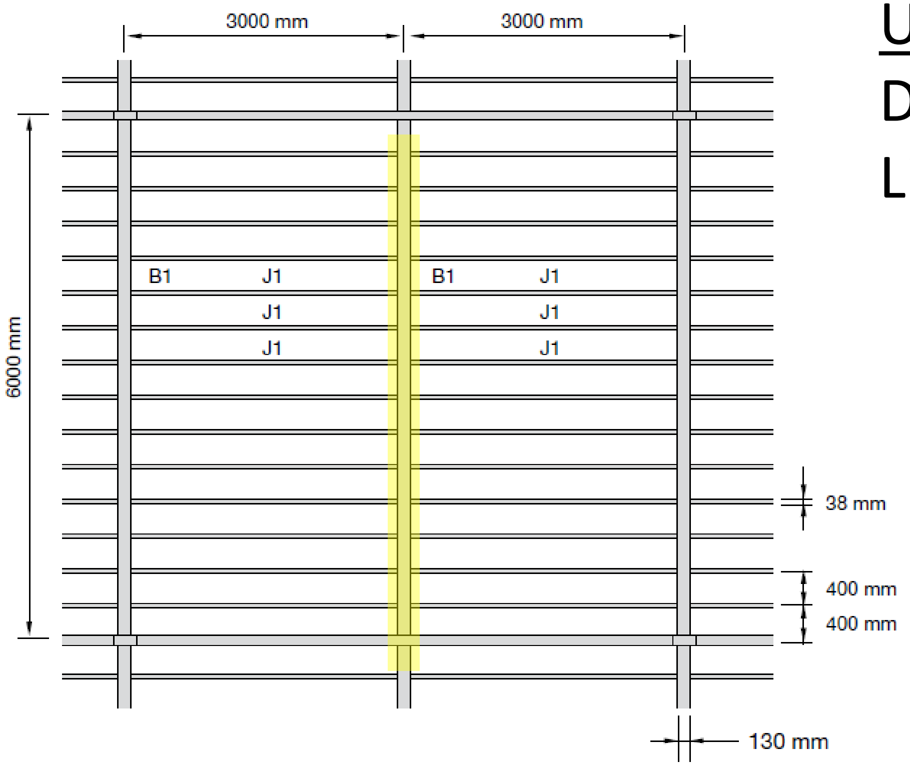
Example 3: Bending

Design the glulam beams for the flooring system shown using **20f-E** bending grade **Spruce Pine**. Because the joists are closely spaced, the loading on beam **B1** may be considered as **uniformly distributed**. Note beam tables need to be provided to students in this example.

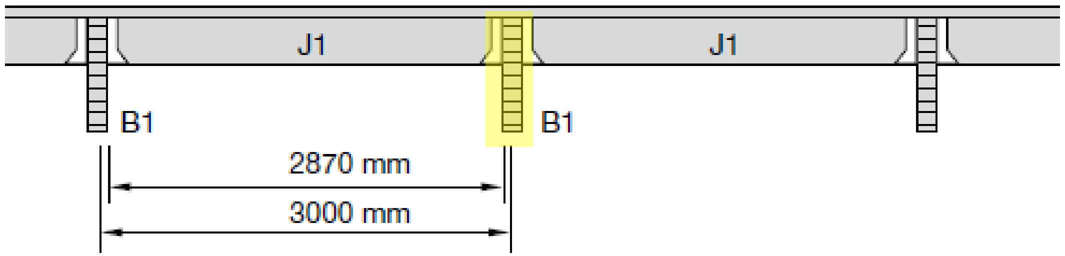


Plan

Unfactored loads

Dead load = 1.2 kPa (not including self weight)

Live load = 4.8 kPa



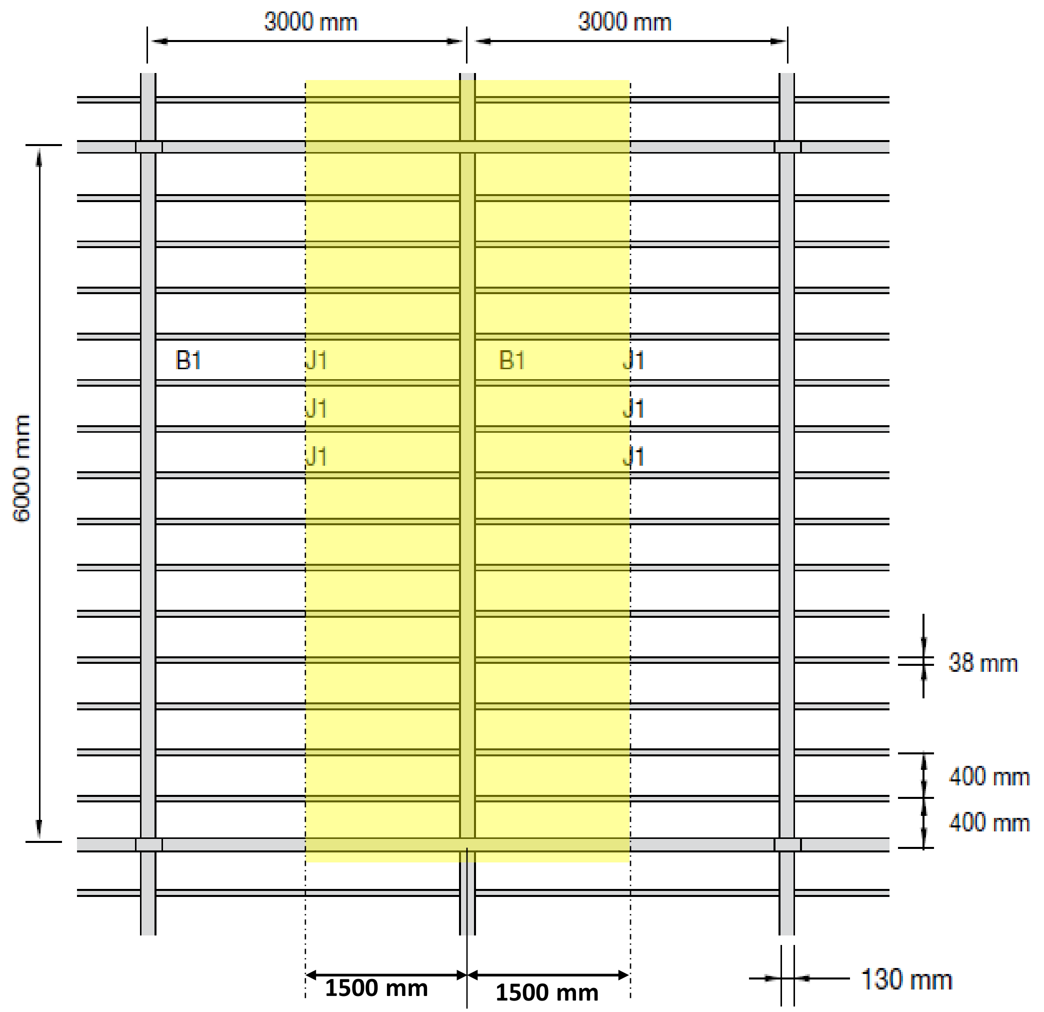
elevation

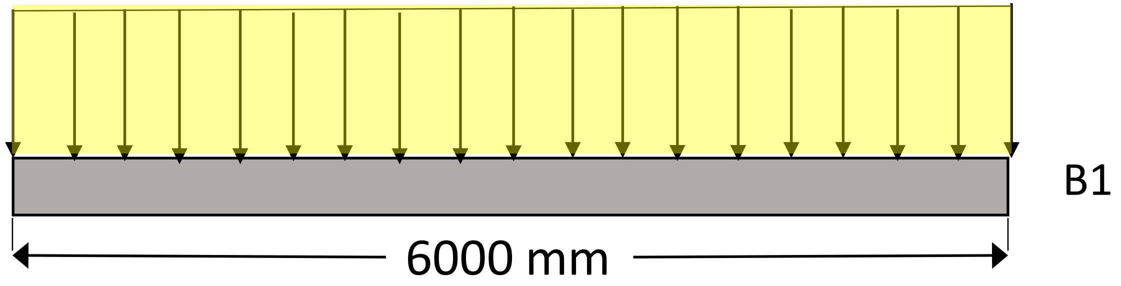
Example 3: Bending

1. Use the tributary area to convert the area loads into linear loads
2. Calculate load combinations and maximum factored shear and bending moment
3. Determine the standards modification factors
4. Collect the needed specified strengths for the type and grade
5. Use the beam selection tables to select a member
6. Calculate the size factor for bending and the lateral stability factor
7. Calculate bending and shear resistance
8. Check deflection

Example 3: Bending

Convert the area loads into uniformly distributed linear loads using the tributary width for B1





Tributary width for B1 =1.5m + 1.5m = 3.0m

Use the distance from center to center not edge to edge

Dead load (not including self weight)

$D = 1.2 \text{ kPa} * 3.0 \text{ m} = 3.6 \text{ kN/m}$

Estimated self weight = 0.4 kN/m

Total dead load $W_D = 3.6 + 0.4 = 4.0$ kN/m

Example 3: Bending

Live load W_L = 4.8 kPa x 3.0m = 14.4kN/m

Using the specified linear loads apply load case combination **2** to find the factored design load

Case	Load combination	
	Principal loads	Companion loads
1	1.4D	-
2	(1.25 or 0.9D) + 1.5L	1.0S or 0.4W
3	(1.25 or 0.9D) + 1.5S	1.0L or 0.4W
4	(1.25 or 0.9D) +1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

$$W_f = 1.25\ W_D + 1.5\ W_L = 1.25 \times 4.0 + 1.5 \times 14.4 = 26.6\ \text{kN/m}$$

Using the design load find the maximum factored shear force and the maximum factored bending moment

$$V_f = W_f\ L / 2 = 26.6\ \text{kN/m} \times 6\text{m} / 2 = 79.8\ \text{kN}$$

$$M_f = W_f\ L^2 / 8 = 26.6\ \text{kN/m} \times 6\text{m}^2 / 8 = 120\ \text{kN m}$$

Example 3: Bending

As no additional information was given assume 1.0 for all modification factors excluding K_L and K_{Zbg} which will be calculated later

$$K_D = K_H = K_{Sb} = K_{Sv} = K_{Scp} = K_{SE} = K_T = 1.0$$

As the beam is a straight member $K_X = 1.0$

Use **Table 7.3** of CSA O86 to find the specified strength for 20f-E Spruce- Pine Glulam

Notice note 2 at the bottom of the table

(2) Tabulated values are based on the following standard conditions:

(a) dry service conditions; and

(b) standard term duration of load.

If the modification factors did not equal one the values taken from the table would need to be multiplied to account for them

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

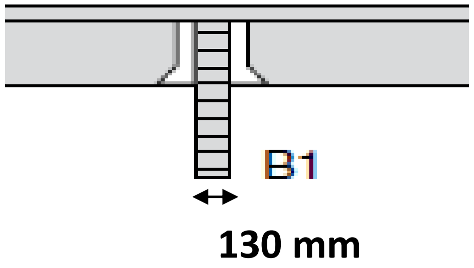
(b) standard term duration of load.

	Spruce-Lodgepole Pine-Jack Pine			
	20f-E	20f-EX	14t-E	12c-E
Bending moment (pos.), f_b	25.6	25.6	24.3	9.8
Bending moment (neg.), f_b	19.2	25.6	24.3	9.8
Longitudinal shear, f_v	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
Tension face bearing	5.8	5.8	5.8	5.8
Tension net section, f_{tn} (see Clause 7.5.11)	17.0*	17.0	17.9	17.0
Tension gross section, f_{tg}	12.7*	12.7	13.4	12.7
Tension perpendicular to grain, f_{tp}	0.51	0.51	0.51	0.51
Modulus of elasticity, E	10 300	10 300	10 700	9 700

f_b = 25.6 MPa f_V = 1.75 Mpa E=10300 MPa

Example 3: Bending

The plan design was created with an assumed width of b = 130 mm



Use the provided beam selection tables to find the depth of a 130 mm wide Spruce-Pine 20f-E member with

$$M'_r \geq M_f = 120 \text{ kN m}$$

While the 130 x 494 member has sufficient moment resistance shear is the governing value in this case

$$V_r \geq V_f = 79.8 \text{ kN}$$

A 130 x 608 member will be sufficient for both moment and shear

d=608 mm

130 mm Spruce-Pine 20f-E Stress Grade

Size (b × d) mm	M'_r kN•m	V_r kN	$W_r L^{0.18}$ kN•m$^{0.18}$	$E_S I$ ×10^9 N•mm^2
130 × 152	11.5	20.7	112	392
130 × 190	18.0	25.9	134	765
130 × 228	26.0	31.1	156	1320
130 × 266	35.3	36.3	177	2100
130 × 304	46.1	41.5	197	3130
130 × 342	58.4	46.7	217	4460
130 × 380	72.1	51.9	237	6120
130 × 418	87.2	57.1	256	8150
130 × 456	104	62.2	275	10600
130 × 494	122	67.4	294	13500
130 × 532	141	72.6	312	16800
130 × 570	162	77.8	330	20700
130 × 608	185	83.0	348	25100
130 × 646	208	88.2	366	30100
130 × 684	234	93.4	383	35700
130 × 722	260	98.6	401	42000
130 × 760	288	104	418	49000
130 × 798	318	109	435	56700
130 × 836	349	114	452	65200
130 × 874	381	119	469	74500
130 × 912	415	124	485	84600
130 × 950	451	130	502	95700

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Glulam Beam Example – Size Factor for Bending

With all three dimensions of the beam known the size factor for bending of a glulam member can be calculated

b=130 mm d=608 mm L=6000 mm

$$K_{Zbg} = (130/b)^{0.1} \times (610/d)^{0.1} \times (9100/L)^{0.1} \leq 1.3$$

$$K_{Zbg} = (130/130)^{0.1} \times (610/608)^{0.1} \times (9100/6000)^{0.1} = 1.04$$

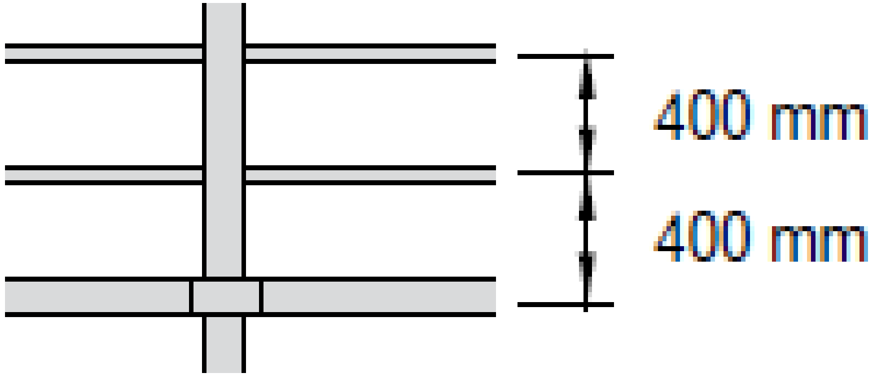
$$K_{Zbg} = 1.04 \leq 1.3 \text{ Okay}$$

Additionally volume needs to be calculated to verify if the simplified shear method can be used, as volume is less than 2 m^3 the simplified method can be used

$$V = b \times d \times L = 0.13 \times 0.608 \times 6.0 = 0.47\ m^3$$

Glulam Beam Example – Effective Length

Per the plan we can see the distance between supports L_u = 400 mm



Referring to **Table 7.5.6.4.3**

For a uniformly distributed load with no intermediate supports

The effective length

$$L_e = 1.92\ L_u = 1.92 \times 400 = 768 \text{ mm}$$

The slenderness ratio

$$C_B = (L_e\, d / b^2)^{0.5} = (768 \times 608 / 130^2)^{0.5}$$

$$C_B = 5.3$$

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	$1.11a$	$1.61\ell_u$

Glulam Beam Example – Lateral Stability Factor

Look to **Clause 7.5.6.4.4** for determining K_L

$C_B = 5.3 < 10$ $\quad\quad$ $K_L = 1.0$

The section modulus

$$S = b \times d^2 / 6 = 130 \times 608^2 / 6$$

$$S = 8.01 \times 10^6 \text{ mm}^3$$

The factored bending strength

$$F_b = f_b (K_D\ K_H\ K_{Sb}\ K_T) = 25.6\ (1 \times 1 \times 1 \times 1)$$

$$F_b = 25.6 \text{ MPa}$$

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

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

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

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

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)

Glulam Beam Example – Moment Calculations

The factored bending moment resistance

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

$M_{r1} = 0.9 \times 25.6 \times 8.01 \times 10^6 \times 1.04 \times 1.0 = 192\ \times 10^6$ N mm

M_{r1} =192 kN m $\geq M_f$ =120 kN m **Okay**

$M_{r2} = \phi\ F_b\ \ S\ \mathbf{K_L}\ K_X$

$M_{r2} = 0.9 \times 25.6 \times 8.01 \times 10^6 \times 1.0 \times 1.0 = 185\ \times 10^6$ N mm

M_{r2} = 185 kN m $\geq M_f$ =120 kN m **Okay**

Notice M_{r1} = 192 kN m > M_{r2} = 185 kN m , M_{r2} <u>governs</u>

Glulam Beam Example – Shear Calculations

The factored shear strength

$$F_v = f_v\,(K_D\;K_H\;K_{Sb}\;K_T) = 1.75\;(1 \text{ x } 1 \text{ x } 1 \text{ x}1)$$

$$F_v = 1.75 \text{ MPa}$$

The factored shear resistance

$$V_r = \phi\;F_v\,(2 \text{ x } b \text{ x } d\;/3)$$

$$V_r = 0.9 \text{ x } 1.75\;(2 \text{ x } 130 \text{ x } 608\;/3\;) = 83000 \text{ N} = 83.0 \text{ kN}$$

$V_r = 83.0$ kN $\geq$ $V_f = 79.8$ kN **Okay**

Glulam Beam Example – Deflection Calculations

The deflection limit for **floors** is suggested to be L/360 for the **specified** (unfactored) live loads . See **Table 2.1** of the WDM for different deflection criteria for different systems

$\Delta_{allowable} = 6000 /360 = 16.7$ mm

The flexural rigidity of the 130 x 608 beam

$E_s I = E \ (K_{SE} \times K_T)\ (b \times d^3 /12)$

$E_s I = 10300\ (1 \times 1)(130 \times 608^3 /12) = 25100 \times 10^9$ N mm^2

The deflection of the 130 x 608 beam

$\Delta = 5\ W_L\ L^4 /384\ E_s I$

$\Delta = (5 \times 14.4 \times 6000^4) / (384 \times 25100 \times 10^9)$

$\Delta = 9.68$ mm $< \Delta_{allowable} = 16.7$ mm **Okay**

Glulam Beam Example – Summary

In summary

M_{r1} = 192 kN m > M_{r2} = 185 kN m > M_f =120 kN m **Okay**

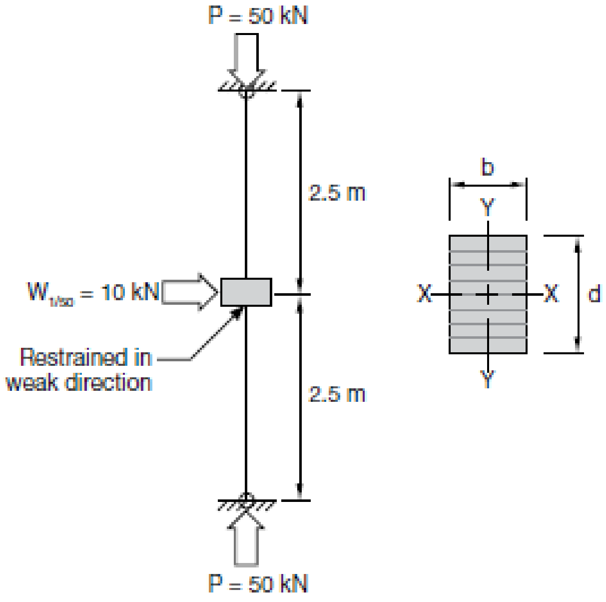
V_r = 83.0 kN ≥ V_f = 79.8 kN **Okay**

Δ = 9.68 mm < 16.7 mm **Okay**

Therefore a **130 x 608** beam is adequate for <u>bending moment</u>, <u>shear</u> and <u>deflection</u> additional <u>bearing</u> calculations would also be required

Example 4: Combined Loading

Design the glulam column shown below. Use untreated 20f-EX Spruce-Pine glulam in dry service conditions. The axial load consists of 10 kN dead load and 40 kN snow load. The specified wind load is 10 kN for strength calculations and 7.5 kN for serviceability calculations. The specified wind loads are based on the $q_{1/50}$ hourly wind pressure, wind importance factors, internal and external pressure coefficients and the tributary wind load area. **(To really practice this problem you do need the handbook)**



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Example 4: Combined Loading

Calculation

Load Case 1: (1.25D + 1.5S)

P_f = (1.25 x 10) + (1.5 x 40) = 72.5 kN

Load Case 2: (1.25D + 1.4W + 0.5S)

P_f = (1.25 x 10) + (0.5 x 40) = 32.5 kN

W_f = 1.4 x 10 = 14 kN

Load Case 3: (1.25D + 1.5S + 0.4W)

P_f = (1.25 x 10) + (1.5 x 40) = 72.5 kN

W_f = 0.4 x 10 = 4 kN

1. For Load Case 1 (standard term load)

Select initial trial section 130 x 152 mm:

$$C_{cx} = \frac{K_e L_d}{d} = \frac{1.0 \times 5000}{152} = 32.9$$ *Governs*

$$C_{cy} = \frac{K_e L_b}{b} = \frac{1.0 \times 2500}{130} = 19.2$$

Since buckling about X axis governs, Column Selection Tables may be used because $K_e L_d = L$.

Therefore, from Column Selection Tables:

$P_r = P_{rx}$ = 101 kN > 72.5 kN *Acceptable*

The 130 x 152 mm section is adequate for Load Case 1.

Example 4: Combined Loading

Calculation

Load Case 1: (1.25D + 1.5S)

P_f = (1.25 x 10) + (1.5 x 40) = 72.5 kN

Load Case 2: (1.25D + 1.4W + 0.5S)

P_f = (1.25 x 10) + (0.5 x 40) = 32.5 kN

W_f = 1.4 x 10 = 14 kN

Load Case 3: (1.25D + 1.5S + 0.4W)

P_f = (1.25 x 10) + (1.5 x 40) = 72.5 kN

W_f = 0.4 x 10 = 4 kN

Table 1.3 – Load combination of ultimate limit states

Case	Load combination	
	Principal loads	Companion loads
1	1.4D	-
2	(1.25 or 0.9D) + 1.5L	0.5S or 0.4W
3	(1.25 or 0.9D) + 1.5S	0.5L or 0.4W
4	(1.25 or 0.9D) +1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

1. For Load Case 1 (standard term load)

Select initial trial section 130 x 152 mm:

$$C_{cx} = \frac{K_e L_d}{d} = \frac{1.0 \times 5000}{152} = 32.9$$ *Governs*

$$C_{cy} = \frac{K_e L_b}{b} = \frac{1.0 \times 2500}{130} = 19.2$$

Since buckling about X axis governs, Column Selection Tables may be used because $K_e L_d = L$.

Therefore, from Column Selection Tables:

$P_r = P_{rx}$ = 101 kN > 72.5 kN *Acceptable*

The 130 x 152 mm section is adequate for Load Case 1.

Example 4: Combined Loading

130 mm Glulam

Spruce-Pine
20f-EX

L	d (mm) 114		152		190		228	
m	P_{rx} kN	P_{ry} kN	P_{rx} kN	P_{ry} kN	P_{rx} kN	P_{ry} kN	P_{rx} kN	P_{ry} kN
2.0	208	231	337	308	455	385	557	456
2.5	162	190	293	254	413	312	518	369
3.0	121	150	243	199	365	245	475	290
3.5	89.8	116	197	153	317	189	431	224
4.0	66.6	88.9	158	117	271	145	386	173
4.5	50.0	68.4	126	90.4	229	112	342	134
5.0	38.2	53.2	$\boxed{101}$	70.4	193	87.5	300	104
5.5	29.6	41.8	81.2	55.4	161	69.0	261	82.5
6.0		33.3	65.8	44.2	135	55.0	226	65.8
6.5		26.8	53.8	35.7	114	44.4	196	53.2
7.0			44.3		96.0		170	
7.5			36.9		81.3		147	
8.0					69.3		127	
8.5					59.3		111	
9.0					51.1		96.7	
9.5					44.2		84.6	
10.0							74.4	
10.5							65.6	
11.0							58.0	

Example 4: Combined Loading

2. Check 130 x 152 mm section for Load Case 2 (short term load)

a) Calculate P_r

For short term loads Column Selection Tables cannot be used.

$P_r = \phi F_{cb} A K_{Zcg} K_C$

ϕF_{cb} = 23.2 MPa from Table 5.2

A = 130 x 152 = 19760 mm^2

$K_{Zcg} = 0.68 (0.130 \times 0.152 \times 5.0)^{-0.13} = 0.92$

$$K_C = \left[1.0 + \frac{F_{cb}}{E'} K_{Zcg} C_c^3\right]^{-1}$$

$F_{cb} / E' = 92.4 \times 10^{-6}$ from Table 5.3

$K_C = [1.0 + 92.4 \times 10^{-6} \times 0.92 \times (32.9)^3]^{-1} = 0.25$

$P_r = 23.2 \times 19760 \times 0.92 \times 0.25 = 105$ kN

Table 5.2
Factored compressive strength for glulam subjected to combined bending and axial compression ϕF_{cb} (MPa)

	Dry service			Wet service		
	Load duration			Load duration		
Grade	Std.	Perm.	Short term	Std.	Perm.	Short term
D.Fir-L 24f-EX and **D.Fir-L 20f-EX**	24.2	15.7	27.8	18.1	11.8	20.8
Spruce-Pine 20f-EX	20.2	13.1	23.2	15.1	9.83	17.4

Notes:
1. $\phi F_{cb} = \phi f_{cb} K_D K_H K_{Sc} K_T$ (Refer to Clause 7.5.8.4 of CSA O86).
2. For fire-retardant treatment, multiply by a treatment factor K_T. See the Commentary for further guidance.
3. Standard term loading = dead loads plus snow or occupancy loads
   Permanent loading = dead loads alone
   Short term loading = dead plus wind loads

Example 4: Combined Loading

b) Determine M_r using Beam Selection Tables:
A review of the checklist for beams indicates that the tabulated M'_r values may be multiplied by 1.15 for short term loading. (Note that $K_{Zbg} = 1.22$ and $K_L = 1.0$)
$M_r = 1.15 \times 11.5 = 13.2$ kNm
$M_f = 17.5$ kNm > 13.2 kNm *Not Acceptable*

c) Select a larger section:
Try 130 x 228 mm section
$C_{cx} = 1.0 \times 5000 / 228 = 21.9$ *Governs*
$A = 130 \times 228 = 29640\ mm^2$
$K_{Zc} = 0.68\ (0.13 \times 0.228 \times 5.0)^{-0.13} = 0.87$
$K_C = [1 + 92.4 \times 10^{-6} \times 0.87 \times (21.9)^3]^{-1} = 0.54$
$P_r = 23.2 \times 29640 \times 0.87 \times 0.54 = 323$ kN
$M_r = 1.15 \times 26.0 = 29.9$ kNm

Note that since d / b < 4, the factor K_L may be taken as unity.
According to Table 2.10, $K_{Zbg} = 1.17$.

Example 4: Combined Loading

d) Calculate amplified bending moment and verify interaction equation:

$E_{05}I = 0.87 \times 1320 \times 10^9 = 1150 \times 10^9\ \text{N}\bullet\text{mm}^2$

$$P_E = \frac{\pi^2 E_{05} I}{(K_e L)^2} = \frac{\pi^2 1150\text{x}10^9}{(5000)^2} = 454\ \text{kN}$$

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{\frac{W_f L}{4}}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] = \left(\frac{32.5}{323}\right)^2 + \frac{\frac{14.0\text{x}5}{4}}{29.9}\left[\frac{1}{1-\frac{32.5}{454}}\right] = 0.64 \leq 1$$

e) Check shear

$V_f = 14/2 = 7.0$ kN

beam volume = 0.130 x 0.228 x 5.00 = 0.148 $m_3 < 2.0\ m_3$

Check V_r from the Beam Selection Tables.

V_r values may be multiplied by 1.15 for short term loading.

$V_r = 1.15 \times 31.1 = 35.6$ kN > 7.0 kN *Acceptable*

Example 4: Combined Loading

f) Check L/180 deflection limit based on serviceability wind load

$E_S I \quad = 1320 \times 10^9$ N•mm²

$$E_s I_{REQ'D} = 180\left[\frac{W_{1/50}L^2}{48}\right] = 180\left[\frac{7500 \times 5000^2}{48}\right]$$

$$= 703 \times 10^9 \text{ N•mm}^2 < 1320 \times 10^9 \text{ N•mm}^2$$

3. Check 130 x 228 mm for Load Case 3 (short term load)

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{\frac{W_f L}{4}}{M_r}\left[\frac{1}{1-\frac{P_f}{P_E}}\right] = \left(\frac{72.5}{323}\right)^2 + \frac{\frac{14.0 \times 5}{4}}{29.9}\left[\frac{1}{1-\frac{72.5}{454}}\right] = 0.25$$

Use 130 x 228 mm 20f-EX Spruce-Pine glulam.

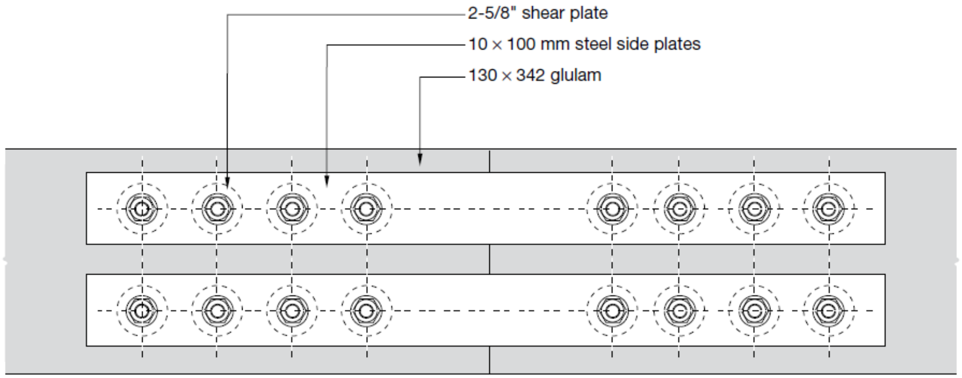
Example 5: Tension Splice

An arena roof is to be framed with heavy timber trusses. The bottom chords are 130 x 342 Douglas-fir glulam. The bottom chord splice is constructed with four 10 x 100 mm steel side plates and sixteen 2- 5/8” shear plates per side. There are two shear plates per 3/4” bolt arranged as two rows of four pairs per side.

a) What is the net section of the bottom chord?

b) What is the group modification factor?

c) The arena will be subject to changes in moisture content. What can be done to minimize splitting at the connection?



Example 5: Tension Splice

a) Net section

$A_{net} = A_{gross}$ – Area removed for connection

$= 130\text{ mm} \times 342\text{ mm} - 2(3.73 \times 10^3\text{ mm}^2)$

$= 37.0 \times 10^3\text{ mm}^2$

$A_{net} \geq 0.75A_{gross}$ (Acceptable)

Example 5: Tension Splice

b) Determine J_G

$A_m = 130\text{ mm} \times 342\text{ mm}$

$= 44\,500\text{ mm}^2)$

$A_s = 4(10\text{ mm} \times 100\text{ mm})$

$= 4000\text{ mm}^2$

$A_m/A_s = 11.1$

Four fasteners in a row

From CSA 086 Table 12.2.2.3.4B

$J_G = 0.95$

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)

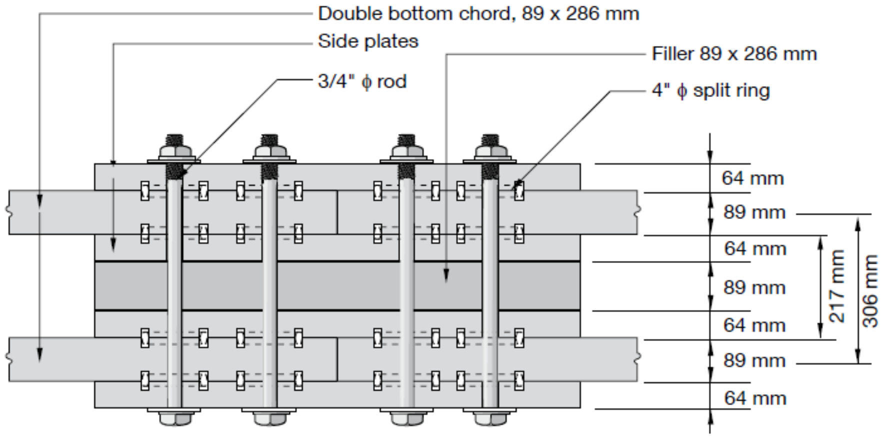
Example 5: Tension Splice

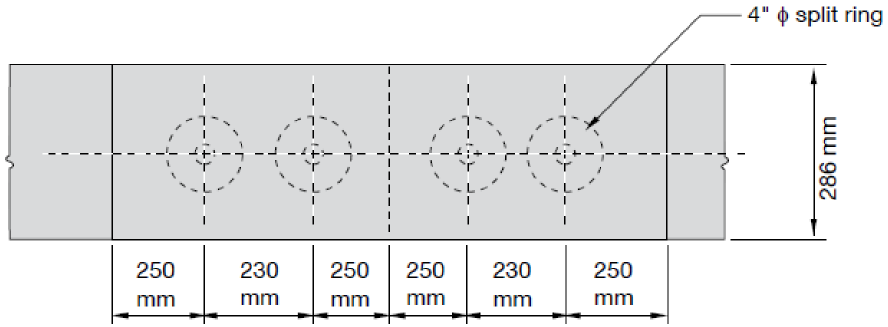
c) Details to prevent splitting:

- Use of separate splice plates for each row of fastenings
- Minimize spacing between rows for fastenings
- Maximize end distance in the connection
- Use a different detail which will only use one row of fastenings. For example: design joint with 4” diameter shear plates.

Example 6: Split Ring Tension Splice

The bottom chord of a truss consists of two 89 x 286 mm S-P-F lumber members spaced at 306 mm on centre. A tension splice for the chord consists of 64 x 286 mm spruce lumber side plates on both sides of each member, with split rings between each member and side plate, and a filler to close up the connection. The truss members will be unseasoned when assembled but will be used in a dry service location.





Example 6: Split Ring Tension Splice

Design a 4” diameter split ring connection for a tensile force of 160 kN due to factored dead and snow loads.

$$P_f = 160 \text{ kN}$$

Factored lateral strength resistance of 4” diameter split ring connection, parallel to grain, P_r, is given by:

$$P_r = \phi \, P_u \, n_F \, J_F$$

where

$P_u = \phi \, p_u \, (K_D \, K_{SF} \, K_T)$

$p_u = 45$ kN (CSA 086 Table 12.3.6A)

$P_u = 45 \times (1.0 \times 0.80 \times 1.0) = 36$ kN

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Example 6: Split Ring Tension Splice

$J_F = J_G\ J_C\ J_T\ J_O\ J_P$

where

J_G and J_C are assumed to be 1.0, but should be verified once the number of split rings and joint configuration has been determined.

J_T = 1.0 (CSA 086 Table 12.3.4 since thickness > 76 mm)

J_O = 1.0 (side grain installation)

J_P = 1.0 (bolts used and not lag screws)

$$J_F = J_G\ J_C\ J_T\ J_O\ J_P$$

$$= 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0$$

$$= 1.0$$

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)

Example 6: Split Ring Tension Splice

$P_r = 0.6 \times 36 \text{ kN} \times n_F \times 1.0$

$= 21.6\ n_F\ (P_r \text{ kN})$

$P_r \geq P_f$

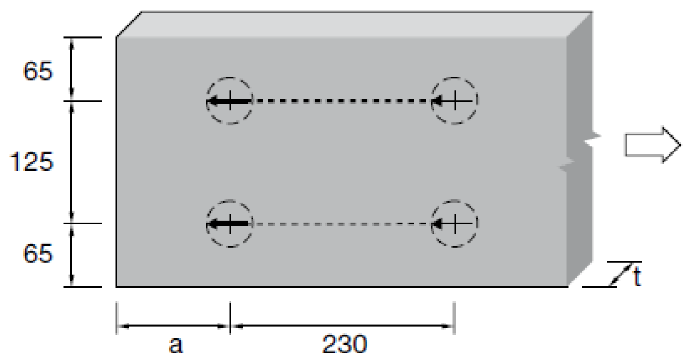
$21.6\ n_F \geq 160 \text{ kN}$

$n_F \geq 7.41$

Therefore, use eight, 4" split rings as shown
$J_G = 1.0$ (since two fasteners per row)

Adjust J_C, so that $n_F = 8.0$ and $P_r = 160$ kN
$J_C = 7.41/8.0$
$= 0.93$

Example 6: Split Ring Tension Splice



Angle of load to grain: θ = 0°
Angle of connector row to grain: β = 0°

Minimum edge distance = 65 mm
Minimum end distance = 245 mm for J_C = 0.93 (CSA 086 Table 12.3.3B)

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	θ = 0° to 90°	θ = 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

Example 6: Split Ring Tension Splice

Minimum spacing between connectors for $J_C = 0.93$ is, by interpolation (CSA 086 Table 12.3.3C)

$$125 + \left[\frac{0.93-0.75}{1.00-0.75}\right] \times [230 - 125] = 201 \text{ mm}$$

Therefore, use the following:

Edge distance

143 mm > 65 mm (Acceptable)

End distance

250 mm > 245 mm (Acceptable)

Spacing between connectors

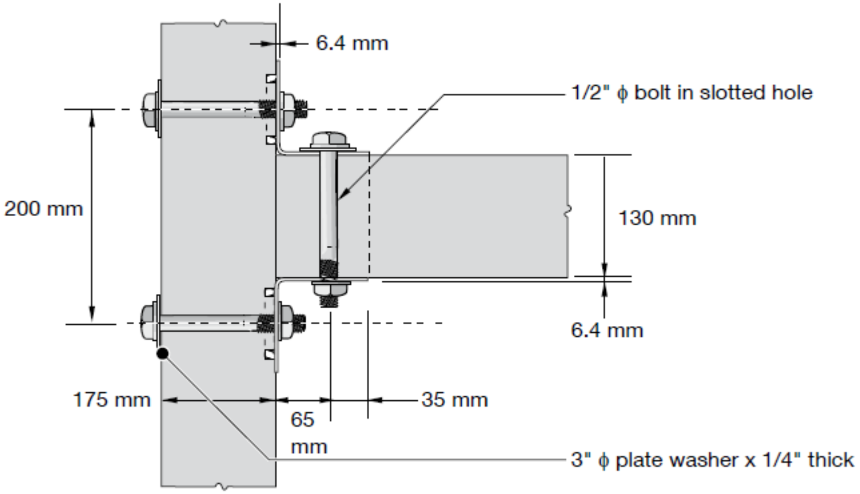
230 mm > 201 mm (Acceptable)

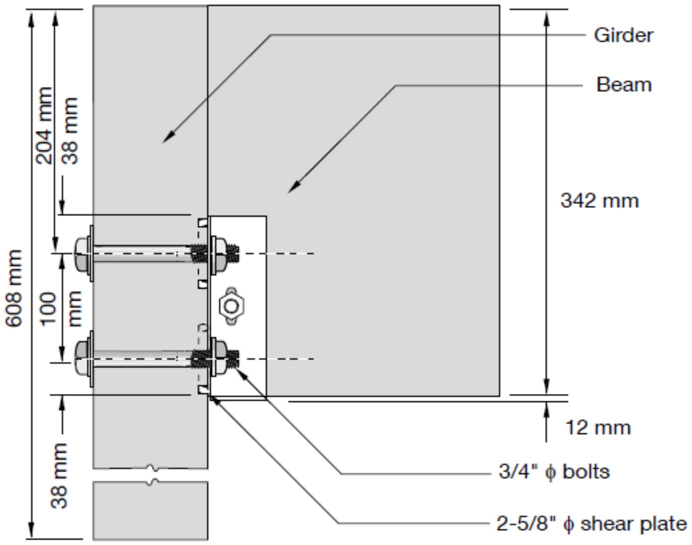
Table 12.3.3C
Timber connector spacing, mm, for values of J_C between 0.75 and 1.0

Angle of load to grain, θ°	Angle of connector row to grain, β°	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

Example 7: Shear Plate – Beam to Girder

A 130 x 342 mm Douglas fir glulam floor beam is supported by a hanger on a 175 x 608 mm Douglas fir glulam girder. The hanger is connected to the girder using shear plates, transferring specified dead and live loads of 7.5 kN and 22.5 kN respectively from the floor beam. Design the connection using 2-5/8” diameter shear plates.





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Example 7: Shear Plate – Beam to Girder

Factored load from the beam:

$Q_f = 1.25D + 1.5L$

$= 1.25(7.5) + 1.5(22.5)$

$= 43.1$ kN

Factored lateral strength resistance of 2-5/8" diameter shear plate perpendicular to grain,

$Q_r = \phi\, Q_u\, n_F\, J_F$

$Q_u = q_u(K_D\, K_{SF} K_T)$

$= 23$ kN $(1.0 \times 1.0 \times 1.0)$

$= 23$ kN

$J_F = J_G\, J_C\, J_T\, J_O\, J_P$

$= 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0$

$= 1.0$

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Example 7: Shear Plate – Beam to Girder

$Q_r = \phi\, Q_u\, n_F\, J_F$

$= 0.6 \times 23 \times n_F \times 1.0$

$= 13.8\, n_F$ (kN)

$Q_r \geq Q_f$

$13.8\, n_F \geq 43.1$ kN

$n_F \geq 3.1$

Table 12.3.6C
Maximum factored strength resistance per shear plate unit, kN

Type of load	2-5/8 in shear plate	4 in shear plate	
		3/4 in bolt	7/8 in bolt
Washers provided — no bearing on threaded portion of the bolt	$\boxed{18}$	32	43
When bearing can occur on the threaded portion of the bolt	16	28	38

Therefore, use four 2-5/8” diameter shear plates in two rows.

Factored load per shear plate:

43.1 kN/4 = 10.8 kN

From CSA 086 Table 12.3.6C maximum factored strength resistance per shear plate is:

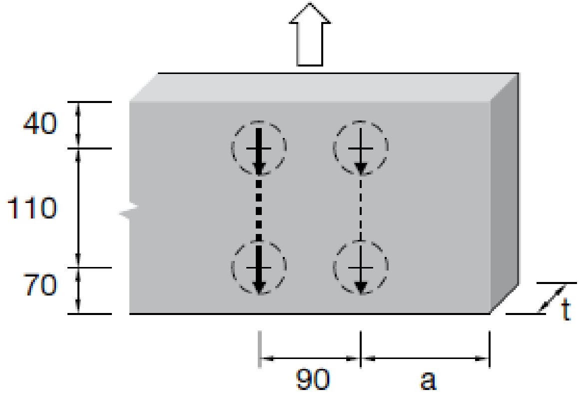
18 kN > 10.8 kN (Acceptable)

Example 7: Shear Plate – Beam to Girder

$J_G = 0.1$ (since two fasteners per row)

Adjust J_C so that $n_F = 4.0$ and $Q_r = 43.1$ kN

$J_C = 3.1/4 = 0.78$



Angle of load to grain: $\theta = 90°$

Angle of connector row to grain: $\beta = 90°$

Minimum loaded edge distance = 45 mm (for $J_C = 0.83$, CSA 086 Table 12.3.3A)

Minimum unloaded edge distance = 40 mm

Loaded edge distance is determined by the floor beam's depth:

$$608 – 342 + 38 = 304 \text{ mm}$$

Example 7: Shear Plate – Beam to Girder

Minimum spacing between connectors for $J_C = 0.78$ is, by interpolation (CSA 086 Table 12.3.3C)

$$90 + \left[\frac{0.78-0.75}{1.00-0.75}\right] \times [110 - 90] = 92 \text{ mm}$$

Therefore, use the following:

Loaded edge distance

304 mm > 45 mm (Acceptable)

Unloaded edge distance

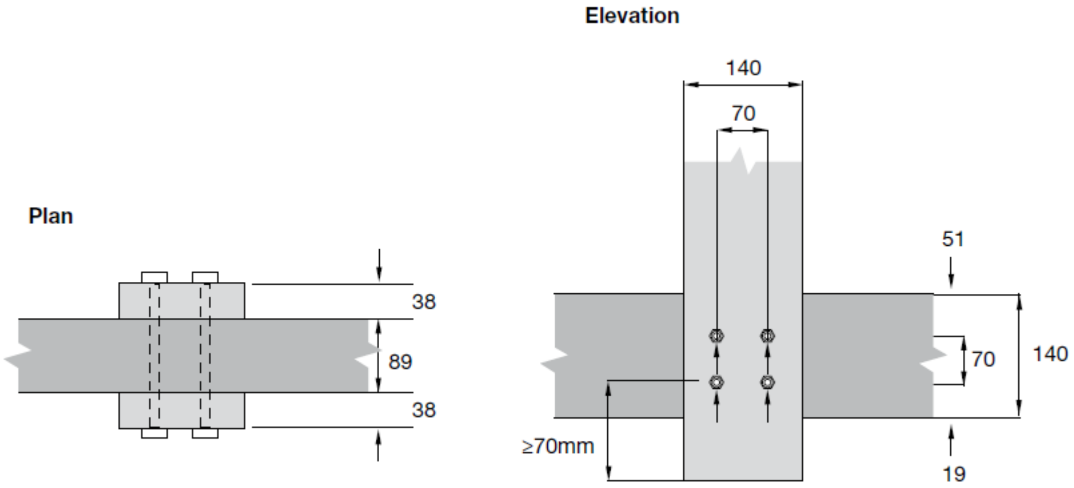
204 mm > 40 mm (Acceptable)

Spacing between connectors

100 mm > 92 mm (Acceptable)

Example 8: Bolts

Determine if two rows of 1/2” bolts are adequate for the attachment of the beam to the wooden tension member. The wood tension member consists of two 38 x 140 mm D.Fir-L No.2 grade, untreated sawn lumber members. The beam is 89 x 140 mm D.Fir-L No.2 grade, untreated sawn lumber. The total factored tension force is 20 kN. The load duration is standard, the material is seasoned, and the service conditions are dry.



Example 8: Bolts

Yielding Resistance

Factored yielding resistance of a bolted connection, N_r, is given by: $N_r = \phi_y n_u n_s n_F$

where

$\phi_y = 0.8$

n_u = unit lateral yielding resistance, N

$n_s = 2$ (number of shear planes in connection)

$n_F = 4$ (number of fasteners in connection)

Determine the unit lateral yielding resistance, n_u, for each shear plane of each bolt.

$d_F = 1/2$ inch (12.7 mm)

$f_y = 310$ MPa for ASTM A 307 bolts

Example 8: Bolts

$f_1 = f_{1P}K_DK_{SF}K_T$ for $\theta = 0°$ (parallel to grain loading) where

$f_{1P} = 50G\,(1 - 0.01\,d_F)\,J_x$

$G = 0.49$ (CSA 086 Table A.12.1)

$f_{1P} = 50 \times 0.49 \times (1 - 0.01 \times 12.7) \times 1.0$

$= 21$ MPa

$f_1 = 21 \times 1.0 \times 1.0 \times 1.0 = 21$ MPa

$f_2 = f_{2Q}K_DK_{SF}K_T$ for $\theta = 90°$ (perpendicular to grain loading) where

$f_{2Q} = 22G\,(1 - 0.01\,d_F)$

$G = 0.49$ (CSA 086 Table A.12.1)

$f_{2Q} = 22 \times 0.49 \times (1 - 0.01 \times 12.7)$

$= 9.4$ MPa

$f_2 = 9.4 \times 1.0 \times 1.0 \times 1.0 = 9.4$ MPa

$t_1 = 38$ mm and $t_2 = 89$ mm

Modification Factors

$K_D = 1.0$ (loading is standard term)

$K_{Dy} = 1.0$ (loading is standard term)

$K_{SF} = 1.0$ (service conditions are dry, material is seasoned)

$K_{Sv} = 1.0$ (dry service conditions)

$K_{St} = 1.0$ (dry service conditions)

$K_T = 1.0$ (wood members are not treated)

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	$\boxed{0.49}$
		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

Example 8: Bolts

For a three member connection, n_u will be the minimum of the following equations:

a) $n_u = f_1\, d_F t_1 = 21 \times 12.7 \times 38$

$= 10.3$ kN

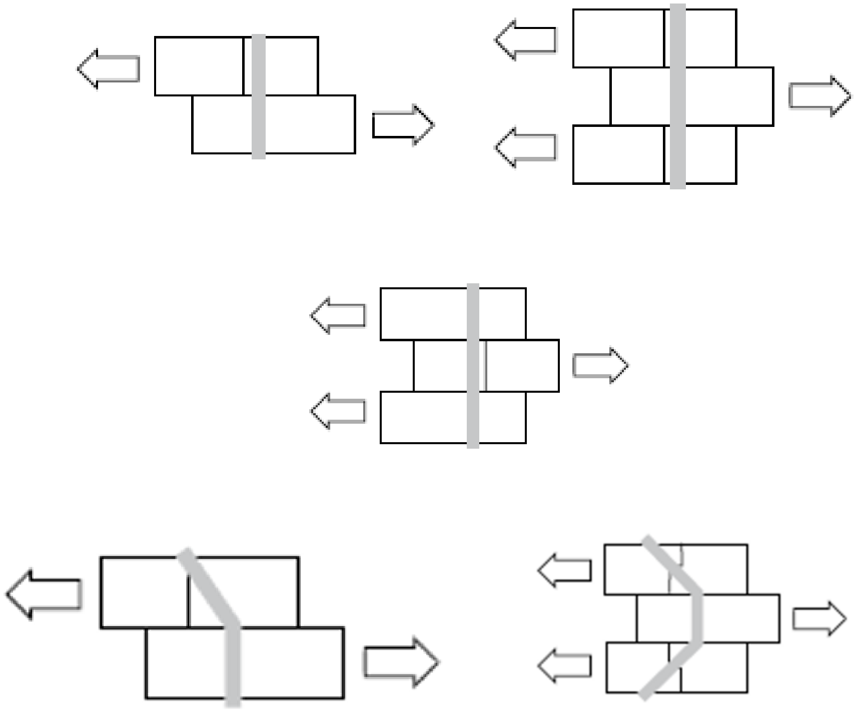
c) $n_u = \frac{1}{2} f_2\, d_F t_2 = \frac{1}{2} \times 9.4 \times 12.7 \times 89$

$= 5.32$ kN

d) $n_u = f_1 d_F^{\,2}\left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F}\right)$

$= 21 \times (12.7)^2 \times \left(\sqrt{\frac{1}{6}\frac{9.4}{(21+9.4)}\frac{310}{21}} + \frac{1}{5}\left(\frac{38}{12.7}\right)\right)$

$= 5.03$ kN



Example 8: Bolts

g) $n_u = f_1 d_F^2 \left(\sqrt{\frac{2}{3} \frac{f_2}{(f_1+f_2)} \frac{f_y}{f_1}} \right)$

$= 21 \text{ x } (12.7)^2 \text{x} \left(\sqrt{\frac{2}{3} \frac{9.4}{(21+9.4)} \frac{310}{21}} \right)$

$= 5.93 \text{ kN}$

Case d governs. Therefore,

$N_r = \phi_y \, n_u n_s n_F$

$= 0.8 \text{ x } 5.03 \text{ x } 2 \text{ x } 4$

$= 32.2 \text{ kN} > 20 \text{ kN}$ (Acceptable)

Example 8: Bolts

Row Shear Resistance

$PR_{rT} = \Sigma(PR_{ri})$

$(PR_{ri}) = \phi_w PR_{ij\,min}\, n_R$

where

$\phi_w = 0.7$

$n_R = 2$

$PR_{ij} = 1.2\, f_v\, (K_D K_{Sv} K_T) K_{ls} t\, n_c\, a_{cr\,i}$

$f_v = 1.9$ MPa (CSA 086 Table 6.3.1A)

$K_{ls} = 0.65$ (for side member)

$t = 38$ mm

$n_c = 2$

$a_{cr\,i} = 70$ mm

$PR_{ri} = 0.7 \times (1.2 \times 1.9 \times (1.0 \times 1.0 \times 1.0) \times 0.65 \times 38 \times 2 \times 70) \times 2 = 11.0$ kN

$PR_{rT} = PR_{r1} + PR_{r2} = 11.0 + 11.0 = 22.0$ kN > 20 kN (Acceptable)

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	$\boxed{1.9}$	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

Example 8: Bolts

Group Tear-Out Resistance

$PG_{rT} = \Sigma(PG_{ri})$

$Pr_{ri} = \phi_w\left[\frac{PR_{r1} + PR_{r2}}{2} + f_t\,(KDKStKT)APGi\right]$ where

$\phi_w = 0.7$

$PR_{11} = 1.2\ f_v\ (K_D K_{Sv} K_T) K_{ls} t\ n_c\ a_{cr\ 1}$

$= 1.2 \times 1.9 \times (1.0 \times 1.0 \times 1.0) \times 0.65 \times 38 \times 2 \times 70$

$= 7.88$ kN

$PR_{11} = PR_{12} = PR_{22} = 7.88$ kN

$f_t = 5.8$ MPa (CSA 086 Table 6.3.1A)

$A_{PGi} = 38 \times (70 – (12.7 + 2)) = 2100\ mm^2$

$PG_{ri} = 0.7 \times \left[\frac{7880+7880}{2} + [5.8(1.0 \times 1.0 \times 1.0) \times 2100]\right]$

$= 14.0$ kN

$PG_{rT} = PG_{r1} + PG_{r2}$

$= 14.0 + 14.0$

$= 28.0\ kN > 20\ kN$ (Acceptable)

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Example 8: Bolts

Net Tension Resistance

$TN_{rT} = \Sigma(TN_{ri})$

$TN_{ri} = \phi_w F_t A_n K_{Zt}$ where

$\phi_w = 0.9$

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

$f_t = 5.8$ (CSA 086 Table 6.3.1A)

$A_n = (140 - 2 \times (12.7 + 2)) \times 38$

$= 4.20 \times 10^3\ mm^2$

$A_n \geq 0.75A_g$ (Acceptable)

$K_{Zt} = 1.3$ (CSA 086 Table 6.4.5)

$TN_{ri} = 0.9 \times (5.8 \times 1.0 \times 1.0 \times 1.0 \times 1.0) \times 4.20 \times 10^3 \times 1.3$

$= 28.5$ kN

$TN_{rT} = TN_{r1} + TN_{r2}$

$= 57.0$ kN

P_r = minimum of PR_{rT}, PG_{rT} or TN_{rT}

$P_r = 22$ kN > 20 kN (Acceptable)

Example 8: Bolts

Splitting Resistance

$QS_{ri} = \phi\, QS_i\,(K_D K_{SF} K_T)$ where

$\phi_w = 0.7$

$QS_i = 14t\sqrt{\dfrac{d_e}{1-\frac{d_e}{d}}}$ where

$t = 89$ mm

$d = 140$ mm and $e_p = 19$

$d_e = d - e_p = 140 - 19 = 121$ mm

$QS_{ri} = 0.7 \times 14 \times 89\sqrt{\dfrac{121}{1-\frac{121}{140}}} \times (1.0 \times 1.0 \times 1.0)$

$QS_{rT} = 26.0$ kN > 20 kN (Acceptable)

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Example 8: Bolts

Beam Shear Resistance

For a group of fasteners, the effective shear depth d_e is measured from the extremity of the fastener group to the loaded edge of the member (CSA 086 Clause 12.2.1.5).

Therefore,

$$d_e = 70 + 51 + 12.7/2 = 127 \text{ mm}$$

The shear resistance at the location of each connection row is

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

where

$\phi_w = 0.9$

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

$f_v = 1.9$ MPa (CSA 086 Table 6.3.1A)

$A_n = 89 \times 127 = 11303 \text{ mm}^2$

$K_{Zv} = 1.55$ (linear interpolation using CSA 086 Table 6.4.5)

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Example 8: Bolts

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

$= 0.9 \times (1.9 \times 1.0 \times 1.0 \times 1.0 \times 1.0) \times 2/3 \times 11303 \times 1.55$

$= 20.0$ kN

The factored shear force per fastener row

$= 20/2$

$= 10$ kN

Therefore,

$V_r = 20.0$ kN $> V_f = 10$ Kn (Acceptable)

Example 8: Bolts

Spacing Requirements

Load Applied Parallel to Grain (outside Members)

Unloaded Edge Distance

$e_p \geq$ maximum of $1.5d_F$ or $0.5S_C$

$e_p = 35$ mm (Acceptable)

Unloaded End Distance

$a \geq$ maximum of $4d_F$ or 50 mm

$a \geq 51$ mm

Load End Distance

$a_L \geq$ maximum of $5d_F$ or 50 mm

$a_L \geq 70$ mm (Acceptable)

Spacing in a Row

$S_R \geq 4d_F$

$S_R = 70$ mm (Acceptable)

Spacing between Rows

$S_C \geq 3d_F$

$S_C = 70$ mm (Acceptable)

Example 8: Bolts

Load perpendicular to grain (centre member):

Loaded Edge Distance

$e_Q \geq 4d_F$

$e_Q = 51$ mm (Acceptable)

Unloaded End Distance

$e_Q \geq 1.5d_F$

$e_Q = 19$ mm (Acceptable)

Spacing in a Row

$S_R \geq 3d_F$

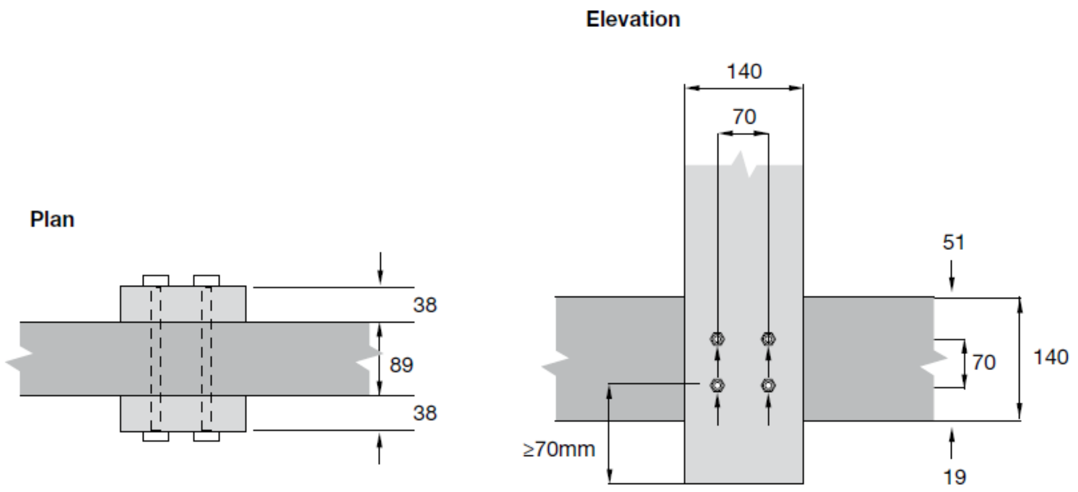
$S_R = 70$ mm (Acceptable)

Spacing between Rows

$S_C \geq 3d_F$

$S_C = 70$ mm (Acceptable)

Therefore, use four 1/2" bolts.



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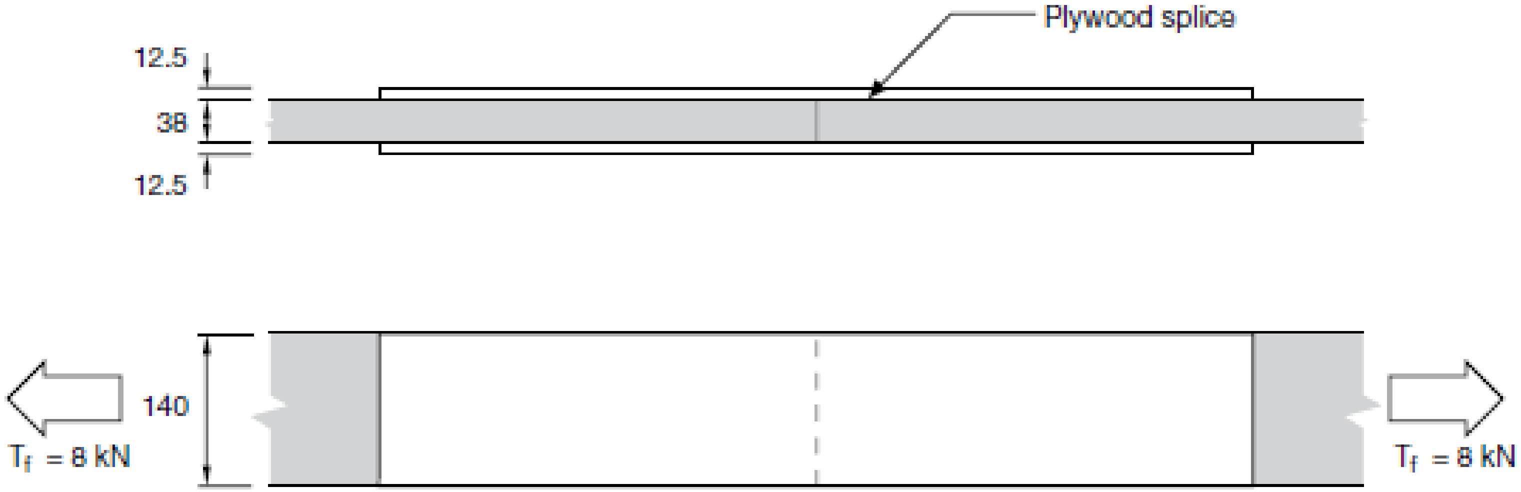
Example 9: Connections Nails and Screws

Determine the size and number of nails required for the plywood tension splice shown below (dimensions in mm). The lumber members are kiln dried, untreated 38 x 140 mm S-P-F and the plywood is 12.5 mm sheathing grade Douglas Fir. The factored load is 8 kN due to dead plus snow loads. Service conditions are dry.



Example 9: Connections Nails and Screws

Modification Factors

K_D = 1.0 (standard term load)

K_{SF} = 1.0 (dry service condition)

K_T = 1.0 (untreated)

J_E = 1.0 (nailed in side grain)

J_A = 1.0 (not toe nailed)

J_B = 1.0 (clinch factor does not apply)

J_D = 1.0 (not a shear wall or diaphragm)

Therefore,

K' = 1.0

J_F = 1.0

Example 9: Connections Nails and Screws

Try 2-1/2” common wire nails driven from alternate sides of the main member.
For double shear connections penetration into the point side member must be 5 nail diameters (15mm) or greater (guidance).
The connection cannot be considered double shear.

From the Nail Selection Tables:
Minimum penetration into the main member is 16 mm and corresponding
$N'_r\ n_s$ is 0.395 kN
When penetration into the main member is ≥ 30 mm, the corresponding
$N'_r\ n_s$ is 0.544 kN
Actual penetration is 38 mm, and therefore N'_r ns is equal to 0.544 kN.

Nail Selection Tables

1S **Single Shear, Sawn Lumber Main Member**

Basic factored lateral resistance based on maximum penetration

Side plate thickness **DFP** (mm)	Nail Type	Nail length (in.)	Nail diameter (mm)	**D.Fir-L** ≥Maximum penetration (mm)	**D.Fir-L** $N'_r \cdot n_s$ (kN)	**Hem-Fir** ≥Maximum penetration (mm)	**Hem-Fir** $N'_r \cdot n_s$ (kN)	**S-P-F** ≥Maximum penetration (mm)	**S-P-F** $N'_r \cdot n_s$ (kN)	**Northern** ≥Maximum penetration (mm)	**Northern** $N'_r \cdot n_s$ (kN)
9.5	Common wire nails	1	1.83	16	0.228	16	0.221	16	0.212	16	0.196
9.5		1.25	2.03	19	0.273	19	0.268	20	0.261	22	0.243
9.5		1.5	2.34	21	0.331	22	0.325	23	0.316	25	0.297
9.5		1.5	2.52	23	0.366	24	0.359	25	0.349	26	0.328
9.5		1.75	2.64	24	0.390	25	0.382	26	0.371	27	0.348
9.5		2	2.84	25	0.431	26	0.422	27	0.409	29	0.384
9.5		2	2.87	25	0.437	26	0.428	27	0.415	30	0.389
9.5		2.25	2.95	26	0.453	27	0.444	28	0.431	30	0.403
12.5		2.5	3.25	28	0.571	29	0.560	$\boxed{30}$	$\boxed{0.544}$	33	0.512
12.5		2.5	3.33	29	0.588	30	0.577	31	0.561	33	0.527
12.5		2.75	3.33	29	0.588	30	0.577	31	0.561	33	0.527
12.5		3	3.66	31	0.663	32	0.650	33	0.631	36	0.592
12.5		3	3.76	31	0.685	32	0.672	34	0.652	37	0.612
12.5		3.5	4.06	33	0.753	34	0.738	36	0.716	39	0.671

Example 9: Connections Nails and Screws

$N_r = N'_r \, n_s \, n_F \, K'J_F$

$N_r = 0.544 \times n_F \times 1 \times 1 = 0.544$ nF kN

n_F required = 8 (kn) / 0.544 = 14.7

Therefore, use fifteen 2-1/2” nails per side.

Determine minimum spacing, end distance and edge distance from Figure 7.4 (See right)):
minimum spacing perpendicular to grain c = 26 mm
minimum edge distance d = 13 mm

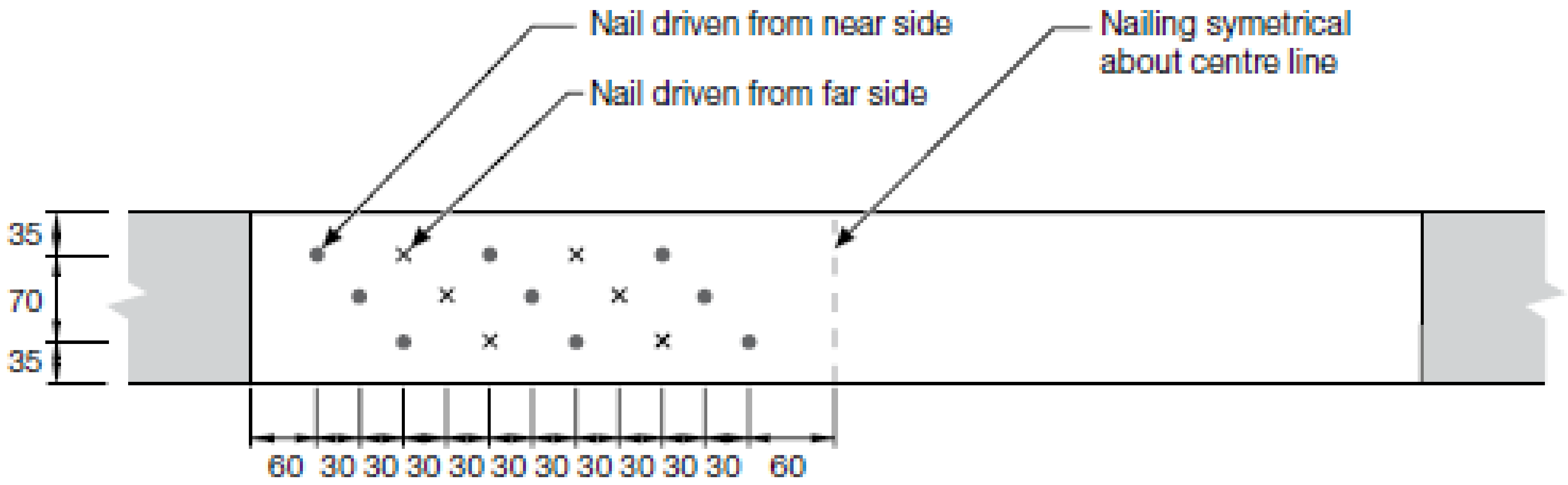
Therefore, use two rows of nails spaced at 70 mm, with 35 mm edge distance: minimum spacing parallel to grain a = 52 mm, use 60 mm
minimum end distance b = 39 mm, use 60 mm

The final connection geometry is shown below next slide (dimensions in mm). Note that the tensile resistance of the plywood and lumber members should also be checked.

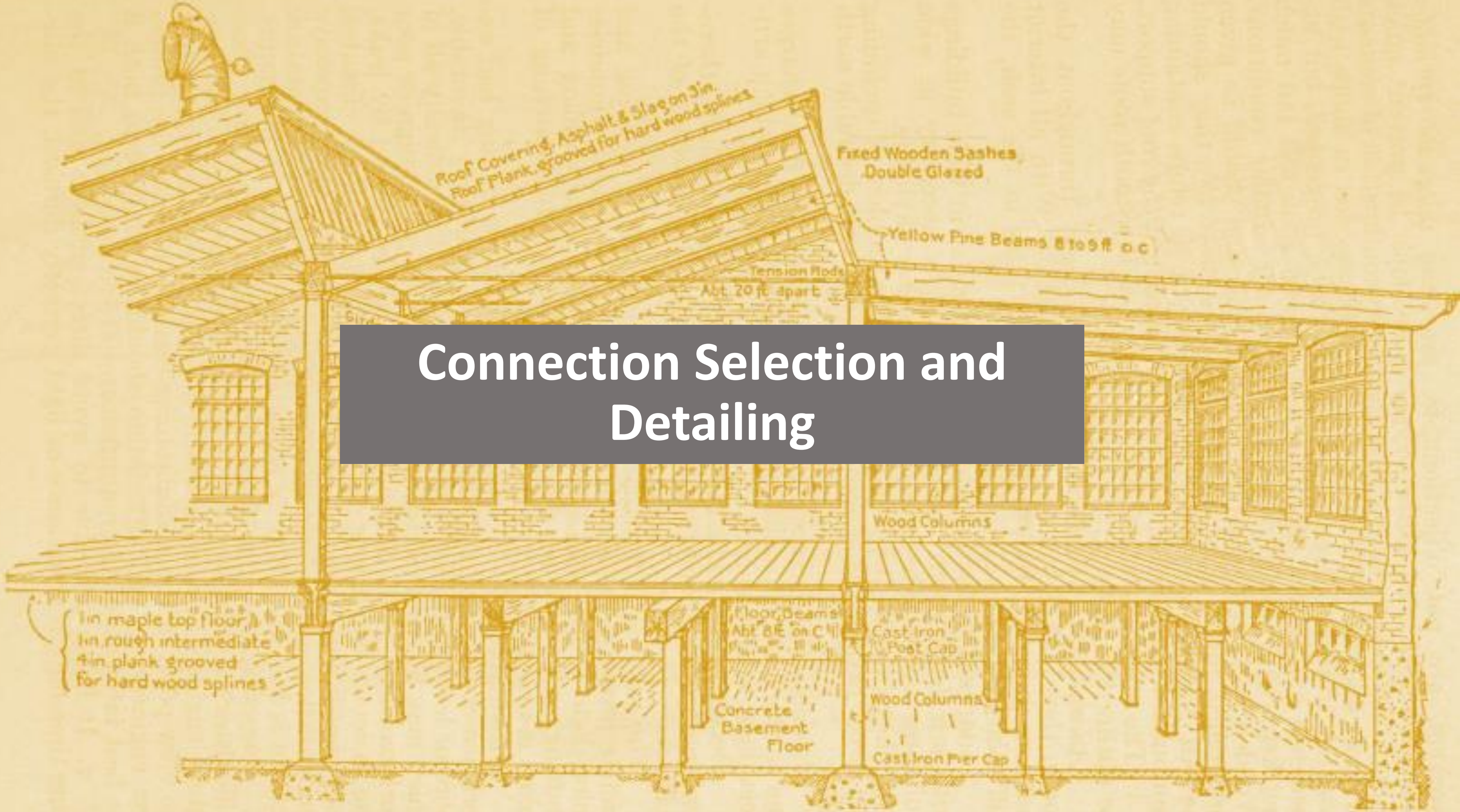
Minimum required spacing, end and edge distances for nails and spikes

Type	Length in.	Diameter mm	D.Fir-L Hem-Fir: Min. spacing parallel to grain a mm	Min. end distance b mm	Min. spacing perp. to grain c mm	Min. edge distance d mm	S-P-F Northern: Min. spacing parallel to grain a mm	Min. end distance b mm	Min. spacing perp. to grain c mm	Min. edge distance d mm
Common wire nails	1	1.83	37	28	19	10	30	22	15	8
	1.25	2.03	41	31	21	11	33	25	17	9
	1.5	2.34	47	36	24	12	38	29	19	10
	1.5	2.52	51	38	26	13	41	31	21	11
	1.75	2.64	53	40	27	14	43	32	22	11
	2	2.84	57	43	29	15	46	35	23	12
	2	2.87	58	44	29	15	46	35	23	12
	2.25	2.95	59	45	30	15	48	36	24	12
	2.5	3.25	65	49	33	17	52	39	26	13
	2.5	3.33	67	50	34	17	54	40	27	14
	2.75	3.33	67	50	34	17	54	40	27	14
	3	3.66	74	55	37	19	59	44	30	15
	3	3.76	76	57	38	19	61	46	31	16
	3.5	4.06	82	61	41	21	65	49	33	17
	3.5	4.12	83	62	42	21	66	50	33	17
	4	4.88	98	74	49	25	79	59	40	20
	4.5	5.26	106	79	53	27	85	64	43	22
	4.5	5.38	108	81	54	27	87	65	44	22
	5	5.74	115	87	58	29	92	69	46	23
	5	5.89	118	89	59	30	95	71	48	24
	5.5	6.2	124	93	62	31	100	75	50	25
	5.5	6.4	128	96	64	32	103	77	52	26
	6	6.66	134	100	67	34	107	80	54	27
	6	7.01	141	106	71	36	113	85	57	29
Common spikes	4	6.4	128	96	64	32	103	77	52	26
	6	7.62	153	115	77	39	122	92	61	31
	8	8.23	165	124	83	42	132	99	66	33
Common spiral nails	2-1/2	2.77	56	42	28	14	45	34	23	12
	3	3.1	62	47	31	16	50	38	25	13
	3-1/2	3.86	78	58	39	20	62	47	31	16
	4	4.33	87	65	44	22	70	52	35	18
	5	4.88	98	74	49	25	79	59	40	20

Example 9: Connections Nails and Screws



Connection Selection and Detailing



What connection do I use?

The handbook (Sect 7.12) gives detailing information regarding the construction of each connection with consideration to service conditions and the effects of notching.

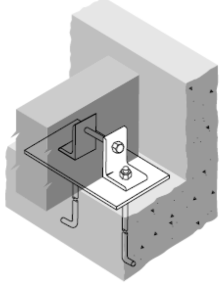
The handbook also provides detailing to the type of member connection or the hybrid connection with specific requirements for detailing.

Typical Connection Details

Beam to Masonry

Detail 7.1

This standard anchorage to a masonry wall resists both uplift and lateral forces. The bearing plate should be a minimum of 6 mm thick when required to distribute the load over a larger area of masonry. Clearance should be maintained between the wall and end of the beam to allow for ventilation.

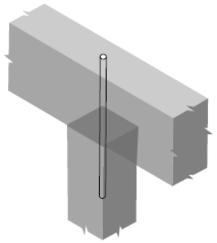


Typical Connection Details

Beam to Column

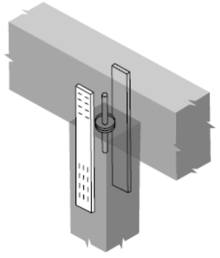
Detail 7.7

The simplest connection for a beam continuous over a column is a steel dowel driven through a hole drilled partially or fully through the beam. Rotation of the column is to be prevented at the base or by other framing.



Detail 7.8

Uplift is resisted by this standard beam-to-column connection. The shear plates and dowel are only included when lateral forces are to be resisted, in which case some form of bracing would be required. A separate bearing plate may be included when the cross section of the column does not provide adequate bearing for the beam.

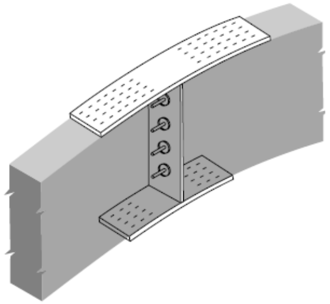


Typical Connection Details

Moment Splice

Detail 7.40

This connection detail provides for the maximum lever arm between splice plates. Direct thrust is transferred through a 3 mm pressure plate between the ends of the members to avoid end grain penetration of fibres. Shear forces are transferred through pairs of shear plates and 19 mm diameter by 225 mm long dowels in the member end faces. Lag screws or bolts together with shear plates may also be used with this connection.



End