February 2020 Errata

Decking Selection Tables

Com Commercial Grade

W_{FR} Maximum factored uniform load W_{FR} (kPa)*

Span m	D.Fir-L Thickness, mm 38[1]	64	89	Hem-Fir Thickness, mm 38[1]	64	89	S-P-F Thickness, mm 38[1]	64	89	Northern Thickness, mm 38[1]	64	89
1.0	24.0			26.3			28.3			18.2		
1.2	16.6			18.3			19.6			12.6		
1.4	12.2			13.4			14.4			9.29		
1.6	9.36			10.3			11.0			7.11	22.5	
1.8	7.39			8.13			8.72	27.6		5.62	17.8	
2.0	5.99	18.9		6.59	20.8		7.07	22.3		4.55	14.4	
2.2	4.95	15.6		5.44	17.2		5.84	18.5		3.76	11.9	
2.4	4.16	13.1		4.57	14.5		4.91	15.5		3.16	9.99	20.7
2.6	3.54	11.2		3.90	12.3		4.18	13.2	27.4		8.51	17.6
2.8	3.05	9.65	20.0	3.36	10.6	22.0		11.4	23.6		7.34	15.2
3.0		8.41	17.4		9.25	19.2		9.92	20.6		6.39	13.2
3.2		7.39	15.3		8.13	16.8		8.72	18.1		5.62	11.6
3.4		6.55	13.6		7.20	14.9		7.73	16.0		4.98	10.3
3.6		5.84	12.1		6.42	13.3		6.89	14.3		4.44	9.20
3.8		5.24	10.9		5.77	11.9		6.19	12.8		3.98	8.25
4.0		4.73	9.80		5.20	10.8		5.58	11.6		3.60	7.45
4.2			8.89			9.78			10.5			6.76
4.4			8.10			8.91			9.56			6.16
4.6			7.41			8.15			8.75			5.63
4.8			6.81			7.49			8.03			5.17
5.0			6.27			6.90			7.40			4.77
5.2			5.80			6.38			6.84			4.41
5.4			5.38			5.92			6.35			4.09
5.6			5.00			5.50			5.90			3.80
5.8			4.66			5.13			5.50			3.54

$W_{\Delta R}$ Maximum specified uniform load for L/240 deflection $W_{\Delta R}$ (kPa)

Span m	D.Fir-L 38[1]	64	89	Hem-Fir 38[1]	64	89	S-P-F 38[1]	64	89	Northern 38[1]	64	89
1.0	17.8			17.8			15.3	~~14.8~~		11.3	~~15.5~~	
1.2	10.3			10.3			8.87			6.54	~~10.9~~	
1.4	6.47			6.47			5.59			4.12		
1.6	4.34			4.34			3.74			2.76	15.5	
1.8	3.04			3.04			2.63	14.8		1.94	10.9	
2.0	2.22	12.5		2.22	12.5		1.92	10.8		1.41	7.94	
2.2	1.67	9.37		1.67	9.37		1.44	8.09		1.06	5.96	
2.4	1.28	7.22		1.28	7.22		1.11	6.23		0.82	4.59	12.4
2.6	1.01	5.68		1.01	5.68		0.87	4.90	13.2		3.61	9.71
2.8	0.81	4.55	12.2	0.81	4.55	12.2		3.93	10.6		2.89	7.78
3.0		3.70	9.94		3.70	9.94		3.19	8.58		2.35	6.32
3.2		3.04	8.19		3.04	8.19		2.63	7.07		1.94	5.21
3.4		2.54	6.83		2.54	6.83		2.19	5.90		1.62	4.34
3.6		2.14	5.75		2.14	5.75		1.85	4.97		1.36	3.66
3.8		1.82	4.89		1.82	4.89		1.57	4.22		1.16	3.11
4.0		1.56	4.19		1.56	4.19		1.35	3.62		0.99	2.67
4.2			3.62			3.62			3.13			2.30
4.4			3.15			3.15			2.72			2.00
4.6			2.76			2.76			2.38			1.75
4.8			2.43			2.43			2.10			1.54
5.0			2.15			2.15			1.85			1.37
5.2			1.91			1.91			1.65			1.21
5.4			1.70			1.70			1.47			1.08
5.6			1.53			1.53			1.32			0.97
5.8			1.38			1.38			1.19			0.88

[1] Thinner decking may result from remanufacturing. Loads are based on 36 mm thick decking and may be increased when the actual decking thickness is 38 mm.

* Where decking is used to support roof loads, the maximum spans for decking may be limited by the NBC roof point load requirements. Assuming specified dead load of 0.5 KPa, the maximum calculated spans based on maximum moment resistance for 36 mm decking, in accordance with NBC roof point load requirements, are 2.1 m for D.Fir-L, 2.3 m for Hem-Fir, 2.4 m for SPF and 1.6 m for Northern Species. Maximum calculated spans for 38 mm thick decking are 2.3 m for D.Fir-L, 2.5 m for Hem-Fir, 2.6 m for SPF and 1.8 m for Northern Species.

For L/180 deflection limit based on total load:

$$E_S I_{REQ'D} = 180\left[\frac{5wL^3}{384}\right] = 180\left[\frac{5 \times 19.7 \times 7500^3}{384}\right]$$

$$= 19500 \times 10^9 \text{ N•mm}^2 \text{ (governs stiffness calculation)}$$

For L/360 deflection limit based on live load:

$$E_S I_{REQ'D} = 360\left[\frac{5w_L L^3}{384}\right] = 360\left[\frac{5 \times 9.7 \times 7500^3}{384}\right]$$

$$= 19200 \times 10^9 \text{ N•mm}^2$$

From Beam Selection Tables for glulam try 130 × 646 mm:

M'_r = 208 kN•m

M_r = lesser of $M'_r K_L$ or $M'_r K_{Zbg}$

K_L = 1.0 (compression edge laterally supported) *Governs*

$$K_{Zbg} = \left(\frac{130}{130}\right)^{\frac{1}{10}}\left(\frac{610}{646}\right)^{\frac{1}{10}}\left(\frac{9100}{7500}\right)^{\frac{1}{10}} = 1.01$$

M_r = 208 kN•m > 191 kN•m *Acceptable*

For beam volumes < 2 m^3, V_r may be used as a simplified method of checking shear resistance.

beam volume = 0.630 m^3

V_r = 101 kN < 102 kN *Not Acceptable*

For beams < 2 m^3, W_r may be used to check beam shear capacity.
For beam volumes ≥ 2 m^3, W_r must be used to check beam shear capacity.
Calculate W_r and compare to W_f to check shear resistance:

W_r = $418 \times (7.5)^{-0.18}$ = 291 kN

W_f = 27.1 × 7.5 = 203 kN *Acceptable*

$E_S I$ = 36200×10^9 N•mm^2 > 19500×10^9 N•mm^2 *Acceptable*

Note: Verify acceptable bearing capacity as per Chapter 6

Use 130 × 646 mm 20f-E D.Fir-L glulam.

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Beam Selection Tables

140 mm Sawn Timbers

Species	Size (b x d) mm	Select Structural M_r kN•m	Select Structural V_r kN	Select Structural E_SI ×10^9 N•mm^2	No.1 Grade M_r kN•m	No.1 Grade V_r kN	No.1 Grade E_SI ×10^9 N•mm^2	No.2 Grade M_r kN•m	No.2 Grade V_r kN	No.2 Grade E_SI ×10^9 N•mm^2
D.Fir-L	140 x 140	9.79	22.9	384	7.38	22.9	336	3.21	22.9	304
	140 x 191	18.2	31.3	976	13.7	31.3	854	5.98	31.3	772
	140 x 241	28.5	36.4	1960	23.1	36.4	1960	13.2	36.4	1550
	140 x 292	38.4	40.5	3490	31.1	40.5	3490	17.7	40.5	2760
	140 x 343	48.2	43.2	5650	39.0	43.2	5650	22.2	43.2	4470
	140 x 394	57.2	44.7	8560	46.4	44.7	8560	26.4	44.7	6780
Hem-Fir	140 x 140	7.28	18.3	320	5.46	18.3	288	2.41	18.3	256
	140 x 191	13.5	25.0	813	10.2	25.0	732	4.48	25.0	650
	140 x 241	21.2	29.2	1630	17.1	29.2	1630	9.81	29.2	1310
	140 x 292	28.6	32.4	2900	23.0	32.4	2900	13.2	32.4	2320
	140 x 343	35.8	34.6	4710	28.9	34.6	4710	16.6	34.6	3770
	140 x 394	42.5	35.7	7140	34.3	35.7	7140	19.7	35.7	5710
S-P-F	140 x 140	6.80	18.3	272	5.14	18.3	240	2.25	18.3	208
	140 x 191	12.6	25.0	691	9.56	25.0	610	4.18	25.0	528
	140 x 241	19.9	29.2	1390	16.1	29.2	1390	9.22	29.2	1060
Northern	140 x 140	6.42	15.3	256	4.82	15.3	224	2.09	15.3	192
	140 x 191	12.0	20.9	650	8.96	20.9	569	3.88	20.9	488
	140 x 241	18.7	24.3	1310	15.8	24.3	1310	8.64	24.3	980

191 mm

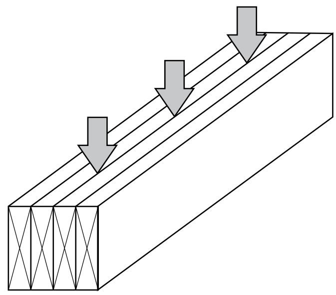
Species	Size (b x d) mm	Select Structural M_r kN•m	Select Structural V_r kN	Select Structural E_SI ×10^9 N•mm^2	No.1 Grade M_r kN•m	No.1 Grade V_r kN	No.1 Grade E_SI ×10^9 N•mm^2	No.2 Grade M_r kN•m	No.2 Grade V_r kN	No.2 Grade E_SI ×10^9 N•mm^2
D.Fir-L	191 x 191	24.9	42.7	1330	18.8	42.7	1160	8.15	42.7	1050
	191 x 241	36.5	49.7	2670	27.6	49.7	2340	12.0	49.7	2120
	191 x 292	52.4	55.2	4760	42.5	55.2	4760	24.2	55.2	3760
	191 x 343	65.7	59.0	7710	53.3	59.0	7710	30.3	59.0	6100
	191 x 394	78.1	61.0	11700	63.2	61.0	11700	36.0	61.0	9250
Hem-Fir	191 x 191	18.5	34.1	1110	13.9	34.1	998	6.11	34.1	887
	191 x 241	27.2	39.8	2230	20.4	39.8	2010	8.99	39.8	1780
	191 x 292	39.0	44.2	3960	31.4	44.2	3960	18.0	44.2	3170
	191 x 343	48.9	47.2	6420	39.4	47.2	6420	22.6	47.2	5140
	191 x 394	58.0	48.8	9740	46.8	48.8	9740	26.8	48.8	7790
S-P-F	191 x 191	17.3	34.1	943	13.0	34.1	832	5.71	34.1	721
	191 x 241	25.3	39.8	1890	19.1	39.8	1670	8.38	39.8	1450
Northern	191 x 191	16.3	28.5	887	12.2	28.5	776	5.30	28.5	665
	191 x 241	24.0	33.1	1780	18.0	33.1	1560	7.79	33.1	1340

2.8 Glued-Laminated Timber Panels

General

This section covers the design of Glued-Laminated Timber (GLT) panels used as flexural members. GLT is an engineered wood product manufactured by gluing together lumber laminations with a waterproof adhesive. GLT consists of multiple pieces of sawn lumber glued together and oriented such that the narrow faces of the laminations are normal to the direction of the load, as shown in Figure 2.8. When glulam members are loaded about their normal axis, the CSA O86 refers to this condition as “vertically glue-laminated”. GLT is manufactured to meet CSA Standard O122, *Structural Glued-Laminated Timber*, and the manufacturers of GLT must be certified in accordance with CSA Standard O177, *Qualification Code for Manufacturers of Structural Glued-Laminated Timber*.

Figure 2.8
GLT panel orientation



Availability

Most fabricators produce either D.Fir-L or Spruce-Pine GLT. Hem-Fir GLT may be available in certain areas. GLT fabricators can produce slightly different panel sizes and thicknesses. Most fabricators use 38 mm thick laminations for GLT. In addition to different section profiles (e.g., fluted), GLT is available in three appear¬ance grades. Information on appearance grades as well as other finishing and sealing data is provided in Chapter 11.

Specified Strengths

The design of vertically glued-laminated beams, also known as GLT, is addressed under Clause 7.5.3 of CSA O86. It states that GLT shall be designed as a built-up system of sawn lumber members of No.2 grade and the specified strengths may be multiplied by the corresponding system factors, K_H, for built-up beams. The GLT Panel Selection Tables are based on the specified strengths for sawn lumber from CSA O86 Table 6.3.1.A. Size factors for visually stress-graded lumber from CSA O86 Table 6.4.5 are conservatively determined based on the full “width” of the glulam (that is, the thickness or depth of panel) rather than the lamination width used in the glulam.

Systems

CSA O86 suggests the specified strengths may be multiplied by a system factor, K_H, equal to 1.1, when determining the factored moment and shear resistance of GLT.

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Two-way Bending Member Selection Tables

140 mm 191 mm

Sawn Timbers

Species	Size (b × d) mm	Select Structural M_{rx} kN•m	M_{ry} kN•m	E_SI_x ×10^9 N•mm^2	E_SI_y ×10^9 N•mm^2	No.1 M_{rx} kN•m	M_{ry} kN•m	E_SI_x ×10^9 N•mm^2	E_SI_y ×10^9 N•mm^2	No.2 M_{rx} kN•m	M_{ry} kN•m	E_SI_x ×10^9 N•mm^2	E_SI_y ×10^9 N•mm^2
D.Fir-L	140 × 140	9.79	9.79	384	384	7.38	7.38	336	336	3.21	3.21	304	304
	140 × 191	18.2	13.4	976	524	13.7	10.1	854	459	5.98	4.38	772	415
	140 × 241	28.5	14.6	1960	661	23.1	10.3	1960	595	13.2	5.89	1550	471
	140 × 292	38.4	16.2	3490	801	31.1	11.5	3490	721	17.7	6.54	2760	571
	140 × 343	48.2	17.3	5650	941	39.0	12.3	5650	847	22.2	6.99	4470	671
	140 × 394	57.2	17.9	8560	1080	46.4	12.7	8560	973	26.4	7.22	6780	770
	191 × 191	24.9	24.9	1330	1330	18.8	18.8	1160	1160	8.15	8.15	1050	1050
	191 × 241	36.5	29.0	2670	1680	27.6	21.8	2340	1470	12.0	9.50	2120	1330
	191 × 292	52.4	30.2	4760	2030	42.5	21.4	4760	1830	24.2	12.2	3760	1450
	191 × 343	65.7	32.2	7710	2390	53.3	22.8	7710	2150	30.3	13.0	6100	1700
	191 × 394	78.1	33.3	11700	2750	63.2	23.6	11700	2470	36.0	13.4	9250	1960
Hem-Fir	140 × 140	7.28	7.28	320	320	5.46	5.46	288	288	2.41	2.41	256	256
	140 × 191	13.5	9.93	813	437	10.2	7.45	732	393	4.48	3.29	650	349
	140 × 241	21.2	10.8	1630	551	17.1	7.66	1630	496	9.81	4.39	1310	397
	140 × 292	28.6	12.0	2900	668	23.0	8.51	2900	601	13.2	4.87	2320	481
	140 × 343	35.8	12.9	4710	784	28.9	9.08	4710	706	16.6	5.20	3770	565
	140 × 394	42.5	13.3	7140	901	34.3	9.39	7140	811	19.7	5.38	5710	649
	191 × 191	18.5	18.5	1110	1110	13.9	13.9	998	998	6.11	6.11	887	887
	191 × 241	27.2	21.5	2230	1400	20.4	16.1	2010	1260	8.99	7.12	1780	1120
	191 × 292	39.0	22.4	3960	1700	31.4	15.8	3960	1530	18.0	9.07	3170	1220
	191 × 343	48.9	23.9	6420	1990	39.4	16.9	6420	1790	22.6	9.68	5140	1430
	191 × 394	58.0	24.8	9740	2290	46.8	17.5	9740	2060	26.8	10.0	7790	1650
S-P-F	140 × 140	6.80	6.80	272	272	5.14	5.14	240	240	2.25	2.25	208	208
	140 × 191	12.6	9.27	691	371	9.56	7.01	610	328	4.18	3.07	528	284
	140 × 241	19.9	10.2	1390	468	16.1	7.20	1390	422	9.22	4.12	1060	322
	191 × 191	17.3	17.3	943	943	13.0	13.0	832	832	5.71	5.71	721	721
	191 × 241	25.4	20.1	1890	1190	19.2	15.2	1670	1050	8.39	6.65	1450	910
Northern	140 × 140	6.42	6.42	256	256	4.82	4.82	224	224	2.09	2.09	192	192
	140 × 191	12.0	8.76	650	349	8.96	6.57	569	306	3.88	2.85	488	262
	140 × 241	18.7	9.58	1310	441	15.8	7.07	1310	397	8.64	3.86	980	298
	191 × 191	16.3	16.3	887	887	12.2	12.2	776	776	5.30	5.30	665	665
	191 × 241	24.0	19.0	1780	1120	18.0	14.2	1560	980	7.79	6.17	1340	840

Note:

All values are for single member application (K_H = 1.0).

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:

$\phi\, F_c$ = 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

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

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

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

$C_{cd} = \dfrac{K_e L_d}{d}$, $C_{cb} = \dfrac{K_e L_b}{b}$ (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 b or d (mm)

d = depth of member (mm)

b = thickness of member (mm)

Glulam

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

where:

$\phi\, F_c$ = 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 $\dfrac{K_e L_d}{d}$ or $\dfrac{K_e L_b}{b}$ ($C_c > 50$ is not permitted)

K_e = effective length factor, given in Figure 3.1

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

d = depth of member (mm)

b = thickness of member (mm)

The factored compressive resistance of CLT panels under axial load shall be calculated as follows:

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

where:

ϕ = 0.8

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

f_c = specified strength in compression parallel to grain of the laminations oriented parallel to the axial load, MPa

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

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

L = height of the panel, mm

The slenderness ratio, C_c, of CLT panels of constant rectangular cross-section shall not exceed 43 (i.e. $L_e/r_{eff} \leq 150$) and shall be calculated as follows:

$$C_c = \frac{L_e}{\sqrt{12}\, r_{eff}} \quad (C_c > 43 \text{ is not permitted})$$

where:

r_{eff} = $\sqrt{\dfrac{I_{eff}}{A_{eff}}}$

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

Example 2: Sawn Timber

Check the adequacy of the bottom chord member shown in Example 1 of Section 7.8. 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

Calculation

T_f = 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.

Example 3: Glulam

Design the bottom chord of a bowstring roof truss for the following conditions:

- factored tensile force = 250 kN (dead plus snow load)
- dry service condition
- untreated
- truss spacing exceeds 610 mm

Use 80 mm 14t-E Spruce-Pine glulam connected with a single row of 4" diameter shear plates with 3/4" diameter bolts.

5.1 General Information

This chapter contains design information for members that are subjected to combined bending and axial load. Combined bending and axial compression often occurs in glulam arches and the top chords of trusses, as well as in laterally loaded studs, columns and CLT walls. Members loaded in bending combined with tension are less common, and include the bottom chords of trusses or top chords of trusses subject to uplift loading. Permanent wood foundations are also subjected to combined loading, and design information is provided in Section 9.8.

Design

Members subjected to combined bending and axial compressive or tensile loads must be designed to satisfy the appropriate strength interaction equation. With the exception of CLT and specific truss configurations, the following interaction equations are applicable:

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

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

where:

P_f = factored axial compressive load

T_f = factored axial tensile load

M_f = factored bending moment

P_r = factored compressive resistance parallel to grain

T_r = factored tensile resistance parallel to grain

M_r = factored bending moment resistance

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

Additional details on applying the interaction equation to the design of stud walls for permanent wood foundations is given in Section 9.8 of this manual. Refer to clause 6.5.13 of CSA O86 for the interaction equation applicable to specific truss applications. Refer to the commentary for further information on the interaction equation for bi-axial bending.

For CLT, a linear interaction equation is applicable:

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

5

Combined Loads

Checklist: **Stud Walls** (Combined Loading)

To verify that the tabulated resistances given in the Stud Wall Selection Tables (Combined Loading) are appropriate for the structure being designed, the following questions should be asked:

1. Is the load duration "short term" (for the combined load condition)?
2. Is the service condition "dry"?
3. Is the material free of incisions and/or fire retardants?
4. Are the studs effectively pinned and laterally restrained at both ends and prevented from buckling about the weak axis by wall sheathing?
5. Does the stud wall meet the requirements for a Case 2 system (refer to page 33)?
6. Is the stud eccentricity less than or equal to 1/6 of the stud depth?

If the answer to any of these questions is no, the selection tables should not be used directly. Instead, the designer should pick a trial section, determine the resistances in accordance with Sections 2.3 and 3.2, and recheck the interaction equation.

The Stud Wall Selection Tables (Combined Loading) provide values for concentric axial loading (e=0) and eccentric axial loading (e=d/6). The eccentric axial load values are appropriate for situations where the effect of the eccentric load and the effect of the wind load cause bending in the same direction. The designer is responsible for selecting the most appropriate eccentric load value to use. The concentric axial load case may also be used where the effect of the wind load and the effect of the eccentric axial load produce opposing bending.

If the stud is eccentrically loaded, considering the eccentricity of 1/6 the depth and using the selection table:

$P'_r = 16.7 > 13.5$ kN *Acceptable*

$w'_r = 1.45 > 0.70$ kN/m *Acceptable*

3. Check shear

$$V_f = \frac{0.7 \times 3}{2} = 1.05\ \text{kN}$$

$V_r = 10.8$ kN > 1.05 kN *Acceptable*

4. Check L/180 deflection based on wind load

$$E_s I_{REQ'D} = 180\left[\frac{5 w_{1/50} L^3}{384}\right] = 180\left[\frac{5 \times 0.375 \times 3000^3}{384}\right]$$

$$= 27.3 \times 10^9\ \text{N•mm}^2 < 82.5 \times 10^9\ \text{N•mm}^2$$ *Acceptable*

Use 38 x 140 mm No.1/No.2 S-P-F spaced at 600 mm.

5.3 CLT Walls

The CLT Wall Panel Selection Tables (Combined Loading) provide combinations of the factored compressive resistance P'_r and maximum factored lateral wind resistance w'_r that satisfy the interaction equation. The tabulated values are only valid for the conditions outlined in the CLT Walls (Combined Loading) checklist.

P'_r and w'_r represent pairs of values that define multiple points corresponding to $(P_f/P_r) + (M_f/M_r)P\Delta = 1.0$. For each wall height, values of P'_r ranging from 0.1 to 0.8 of the maximum compressive resistance P_r are listed with the corresponding maximum value of w'_r that reach failure. This load combination applies a short-term load duration to the resistance of the wall. The resistance without wind and the resistance to dead plus wind loads should also be verified. Deflection of the wall under specified loads, the shear capacity of the wall along with the bearing capacity of the supporting member should also be checked. The Wall Panel Selection Tables (Combined Loads) include resistance values $(EI)_{eff,y}$, $(GA)_{eff,zy}$, $V_{r,zy}$, and Q_r.

Checklist: **CLT Walls** (Combined Loading)

To verify that the tabulated resistances given in the Wall Panel Selection Tables (Combined Loading) are appropriate for the structure being designed, the following questions should be asked:

1. Is the load duration "short term" (K_D) for the combined load condition?
2. Is the service condition "dry" (K_S)? (CLT not permissible in wet service conditions).
3. Is the material free of strength-reducing chemicals (K_T)?
4. Is the wall eccentrically less than or equal to half of the wall thickness?
5. Is the wall effectively pinned at each end (K_e=1.0)?
6. Is the thickness of each ply 35 mm?

If the answer to any of these questions is no, then the wall panel selection tables should not be used. If the answer to questions 1 to 4 is no, deter-mine the resistances in accordance with Sections 2.9 and 3.5, and recheck the interaction equation. If the answer to questions 5 or 6 is no, calculate the capacity of the panel following the provisions in the CSA O86.

The Wall Panel Selection Tables (Combined Loading) provide values for concentric axial loading (e=0) and eccentric axial loading (e=d/6 and e=d/2). The eccentric axial load values are appropriate for situations where the effect of the eccentric load and the effect of the wind load cause bending in the same direction. The designer is responsible for selecting the most appropriate eccentric load value to use. The concentric axial load case may also be used where the effect of the wind load and the effect of the eccentric axial load produce opposing bending.

Example 1: CLT Wall

Design an exterior E2 stress grade CLT bearing wall subjected to lateral uniformly distributed wind load and concentrically applied dead and snow loads:

- wall height = 4.5 m
- specified vertical dead load = 50 kN/m
- specified vertical snow load = 100 kN/m
- specified wind pressure for strength calculations based on $q_{1/50}$ hourly wind pressure = 2.0 kPa
- dry service condition
- untreated

Calculation

Load Case 1: (1.25D + 1.5S + 0.4W)

P_f = total factored axial load per metre = (1.25 x 50) + (1.5 x 100) = 213 kN/m

w_f = total factored horizontal wind load per metre length of wall panel

= total factored horizontal wind pressure = 0.4 x 2.0 = 0.80 kPa

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

P_f = total factored axial load= (1.25 x 50) + (0.5 x 100)= 113 kN/m

For strength:

w_f = 1.4 x 2.0= 2.8 kPa

1. From the CLT Wall Selection Tables (Combined Loading) select a wall that satisfies the strength interaction equation for Load Case 1.

   Select a 3-Ply E2 stress grade:

   P'_r = 228 > 213 kN/m *Acceptable*

   w'_r = 1.08 > 0.8 kPa *Acceptable*

2. Check interaction equation for Load Case 2

   P'_r = 114 > 113 kN/m *Acceptable*

   w'_r = 6.04 > 2.8 kPa *Acceptable*

3. Check shear

   $V_{r,zy}$ = 39.7 kN/m x 1.15 = 45.7 kN/m

V_f = w_f L / 2 = 2.8 kPa x 4.5 m / 2 = 6.3 kN/m < 45.7 kN/m *Acceptable*

4. Check L/180 Deflection limit based on serviceability wind load

w_s = 0.75 x 2.0 kPa = 1.5 kPa

Δ_{limit} = L/180 = 4500/180 = 25.0 mm

$(EI)_{eff,\,y}$ = 958 x 10^9 N•mm^2/m

$(GA)_{eff,\,zy}$ = 7.98 x 10^6 N/m

$$\Delta = \frac{5w_sL^4}{384(EI)_{eff,y}} + \frac{w_sL^2\kappa}{8(GA)_{eff,zy}}$$

$$= \frac{5 \times 1.5 \times 4500^4}{384 \times 958 \times 10^9} + \frac{1.5 \times 4500^2 \times 1.2}{8 \times 7.98 \times 10^6}$$

$$= 8.36 + 0.57 = 8.93 \text{ mm}$$

8.93 mm < 25 mm *Acceptable*

Note: Verify acceptable bearing capacity as per Chapter 6.

7.2 Nails and Spikes

General

Nails and spikes are manufactured in many different sizes and styles. However, design information provided in CSA O86 is applicable 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. The ASTM F1667 Standard Specification for Driven Fasteners: Nails, Spikes and Staples, is a widely accepted standard, and includes nail diameters that are not included in the CSA B111. The nail selection tables include common fasteners from both the CSA B111 and the ASTM F1667.

Common nails and spiral nails are used for general construction. However, spiral nails have greater withdrawal resistance than common nails and can also reduce splitting. Spikes are longer and thicker than nails and are generally used to fasten heavy pieces of timber.

Availability

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.

Design

Nails and spikes must be designed so that the factored lateral resistance is greater than or equal to the factored lateral load.
The factored lateral resistance for loading at any angle to grain is calculated from the following formula:

$$N_r = N'_r n_S n_F K' J_F$$

where:

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

n_S = the effective number of shear planes, (n_S is equal to 1 in the Nail Selection Tables)

n_F = number of nails or spikes

K' and J_F are composite modification factors given below

For nail or spike connections into CLT panels, the embedment strength equations are factored by J_x=0.9. The basic factored lateral resistances ($N'_r n_s$) for CLT selection tables incorporate J_x=0.9.

Figure 7.4 gives the minimum required end distance, edge distance and nail spacing.

Nails and spikes may be designed for withdrawal under wind or earthquake loading only. Design requirements for withdrawal are given in CSA O86 Clause 12.9.5. Withdrawal resistances for nails and spikes are given in Table 7.3.

Pre-drilled holes are recommended when nails greater than 4.88 mm in diameter are used. This will reduce the occurrence of splitting. These pre-drilled holes should roughly be equal to 75% of the nail diameter.

Calculation

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. 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 $N'_r\ n_s$ is equal to 0.544 kN.

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

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

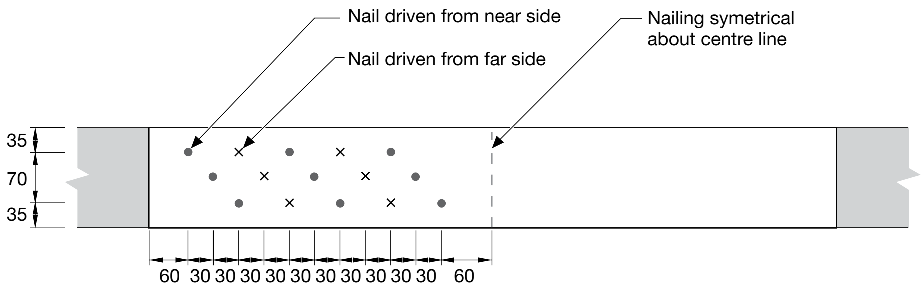
n_F required = 8/0.544 = 14.7

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

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

Therefore, use three rows of nails spaced at 35 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 (dimensions in mm). Note that the tensile resistance of the plywood and lumber members should also be checked.



Use fifteen 2-1/2" common nails.

Advantages

Timber rivets (commonly marketed as Griplam Nails) have many advantages over bolts and other connectors. These include:

- Timber rivet connections are much stiffer and transfer greater loads for a given connector area than any other timber fastener.
- The wood members need not be pre-drilled. This reduces shop fabrication time and costs. It also permits the integration of stock glulam beams into site-framed projects. Glulam fabricators are able to provide assistance in the design and sourcing of connection hardware.
- Ease of construction readily accommodates discrepancies on the site.
- The typically smaller connection size is considered a favourable feature by many architects.
- The ease of on-site installation of the rivet permits glulam and heavy timber trusses to be shipped knocked-down and assembled by the contractor.

Timber rivets may be obtained from fabricators.

Design

Timber rivet connections must be designed so that the factored resistance is greater than the factored lateral load.

This section provides design information and selection tables for the following two common timber rivet connections:

- double-sided splice plate loaded parallel to grain
- beam hanger loaded perpendicular to grain

For other configurations, refer to Clause 12.7.2 of CSA O86.

Double-sided Splice Plate

The double-sided splice plate is shown in Figure 7.12 with design values tabu-lated on pages 486 to 492. It consists of a pair of steel splice plates fastened with timber rivets to both sides of the member. The rivets are loaded parallel to the grain and the total resistance of the connection (the sum of both sides) P_r is the lesser of:

$P_{ry} = P'_r K_{SF} K_T J_Y J_O H$ for rivet capacity

$P_{rw} = P'_r K_D K_{SF} K_T J_O H$ for wood capacity

where:

P'_r given in the Timber Rivet Selection Tables, is the capacity for two identical side plates and rivet groups;

K_T = treatment factor. The tabulated resistances of panel buckling strength assume no treatment (K_T = 1.0)

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb} \frac{\pi^2 t^2}{3000b} \left(B_{a,0} B_{a,90}{}^3\right)^{\frac{1}{4}}$$

where:

K_{pb} = panel buckling factor

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

where:

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

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

where:

a = larger dimension of panel, mm

b = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, see Tables 9.3A, 9.3B and 9.3C of O86, N/mm

$B_{a,90}$ = axial stiffness of panel 90° orientation, see Tables 9.3A, 9.3B and 9.3C of O86, N/mm

B_v = shear-through-thickness rigidity, see Tables 9.3A, 9.3B and 9.3C of O86, N/mm

t = panel thickness, mm

The tabulated values for panel buckling strength are based on the most commonly available plywood products or the minimum buckling strength for OSB panels of the same thickness. In the Selection Tables it is assumed the panel size is 2440 mm x 1220 mm, and the intermediate stud support is ignored for the panel buckling strength calculation.

For diaphragms constructed with diagonal lumber sheathing:

The factored shear resistance of diaphragms constructed with diagonal lumber sheathing is calculated as follows:

$$v_{rs} = \phi v_d J_D n_L$$

where:

$$\phi = 0.8$$

$$v_d = \frac{2}{3}\frac{N_u}{\sqrt{2}s}, \text{ kN/m}$$

Checklist: **Shearwalls**

To verify that the tabulated resistance values are valid for the shearwall being designed the following questions should be asked (the appropriate modification factor is given in brackets):

1. Is load duration "short term" (K_D)?
2. Is the service condition "dry" and are the framing members seasoned prior to fabrication (moisture content ≤ 19%) (K_{SF} and K_s)?
3. Is the material free of strength-reducing chemicals (K_T)?
4. Is the wood-based shearwall blocked (J_{us})?
5. Is there sufficient dead load to resist overturning or are hold-down connections provided to resist all of the overturning forces (J_{hd})?

If the answer to any of these questions is no, refer to the design procedure section. Otherwise, the Shearwall Selection Tables can be used directly.

The tabulated resistance values for shearwalls (per unit length), v_{rs} (kN/m), are based on the following formula and modification factors.

For shearwalls constructed with wood-based panels:

The tabulated lateral resistance of the sheathing-to-framing connection and the panel buckling strengths are provided in the Shearwall Selection Tables. The factored shear resistance value for wood-based structural panel shearwalls (per unit length), v_{rs} (kN/m) is the smaller of the two values.

(a) $v_{rs} = \phi \, v_d \, J_D \, n_s \, J_{us} \, J_s \, J_{hd}$

where:

ϕ = 0.8

v_d = N_u/s, for shearwall segment sheathed with plywood or OSB, kN/m

N_u = lateral strength resistance of sheathing-to-framing connection along panel edges, per fastener, N

= n_u (K_D K_{SF} K_T) for nails

where:

n_u = unit lateral strength resistance, N (Clause 12.9.4.2 of CSA O86)

K_D = load duration factor. The tabulated lateral resistances of shearwall are based on short term loading such as wind or earthquake loads (K_D = 1.15). For permanent load duration multiply the tabulated values by 0.565. For standard term loads multiply the tabulated values by 0.870.

K_{SF} = service condition factor for connections. The tabulated lateral resistances of shearwall are based on lumber that has been seasoned prior to fabrication (moisture content ≤ 19%) and

Design of the non-dissipative connection

Once it is established that the wall has sufficient capacity to resist the applied load, the dissipative connections (in this example discrete hold-downs and panel-to-panel connection) need to be capacity protected by ensuring that the non-dissipative connections (in this example angle brackets) are designed to remain elastic when the dissipative connectors reach their ultimate strength or target displacement. This can be achieved by considering the force-deflection curves provided by the manufacturer. The steps required to meet the design standard requirements are:

a) Determine the 95th percentile ultimate capacity of the dissipative connection

b) Considering whether the dissipative and non-dissipative connectors can be assumed as springs in series or parallel and taking into account the wall geometry, calculate the force induced in the non-dissipative connection when the dissipative connection reach their ultimate capacity

c) At the force level determined in b) check if the 5th percentile force-displacement of the non-dissipative connection still remain in the elastic range.

It is important to emphasize that whereas taking the minimum of the two wall strengths related to the CP and SW behaviour is sufficient and conservative for strength requirements for the design of the hold-down and the panel to panel connections, considerations for capacity based design necessitate taking the maximum of the wall strength related to the CP and SW behaviours calculated based on the 95th percentile of the hold-down and panel to panel strength.

To illustrate this approach consider Case B in Example 1. The governing case was chosen to be the SW behaviour because it provided the minimum capacity based on static equilibrium. In reality, when using simple equilibrium considerations, the designer cannot know whether this is the actual wall behaviour. For this reason and to be conservative, when implementing the concept of capacity based design, the CP behaviour should be considered as governing in this example. In other words, the non-dissipative connection need to designed for the highest calculated wall strength governing the dissipative connections. Assuming in Case B that the 95th percentile strength for r_c and r_h are 7kN and 70kN, respectively, the wall strength can now be calculated as follows:

CP behaviour: $$R_w = F = \frac{b}{h}(r_h + qb + nr_c) = \frac{1.4}{2.7}(70 + 8 \times 1.4 + 18 \times 7) = 107.4 \text{ kN}$$

SW behaviour: $$R_w = \mathrm{Min}\left(\frac{r_c \bullet n \bullet B}{h}, \frac{r_h \bullet B}{h} + \frac{q \bullet B^2}{2h}\right) = \mathrm{Min}\left(\frac{7 \times 18 \times 2.8}{2.7}, \frac{70 \times 2.8}{2.7} + \frac{8 \times 2.8^2}{2 \times 2.7}\right)$$

$$= \mathrm{Min}(130.7 \text{ kN}, 83.4 \text{ kN}) = 83.4 \text{ kN}$$

Using the magnitude of force for the CP behaviour (i.e. 107.4 kN) and considering the 5th percentile curve of the angle bracket connection, it should be verified that the behaviour if within the elastic range.

In cases where the non-dissipative connection is in parallel with the dissipative connection, the assumption of equal displacement is made and in that case it should be ensured that at the target displacement, the behaviour of the non-dissipative connection remains elastic.

Symbol	Meaning
G_{rT}	Factored Group Tear-out Resistance
h	Centre-to-centre Distance Between Chords, Thickness of CLT Panel
H	Truss Rise, Height, Material Factor, Lateral Earth Pressure Load, Height of Backfill
H_s	Height of Shearwall Segment (measured from the bottom of bottom plate to the top of top plate)
h_x	Thickness of CLT Panel Without Outer Longitudinal Layers
I	Moment of Inertia
I_A	Moment of Inertia at Apex (Chapter 9)
I_E	Importance Factor for Earthquake Loads
I_{eff}	Effective Out-of-plane Moment of Inertia of CLT Panels
I_S	Importance Factor for Snow Loads
I_W	Importance Factor for Wind Loads
J´	Composite Modification Factor
J_A	Factor for Toe Nailing/Screwing
J_B	Factor for Nail Clinching
J_C	Minimum Configuration Factor
J_D	Factor for Diaphragm and Shearwall Construction
J_E	Factor for Connecting into End Grain
J_f	Fastener Row Factor for Blocked Diaphragms
J_H	Moment Factor for Heel Connections of Pitched Trusses
J_{hd}	Hold-down Factor
J_O	Factor for Connector Orientation to Grain
J_P	Factor for Lag Screw Penetration
J_S	Side Plate Factor for 4 inch Diameter Shear Plates
J_s	Fastener Spacing Factor
J_T	Thickness Factor
J_{Tr}	Tension Factor for Group Tear-out Resistance
J_{ud}	Strength Adjustment Factor for Unblocked Diaphragms
J_{us}	Strength Adjustment Factor for Unblocked Shearwalls
J_Y	Side Plate Factor
K_Δ	Deflection Factor for Plank Decking
K´	Composite Modification Factor
K_B	Length of Bearing Factor
K_C	Slenderness Factor
K_{creep}	Creep Adjustment Factor

Symbol	Meaning
K_D	Load Duration Factor
K_{Dy}	Load Duration Factor for Yielding Resistance of Bolts or Dowels
K_e	Effective Length Factor
K_{fi}	Specified Strength Adjustment Factor for Fire Design
kg/m^2	Kilograms per Square Metre
kg/m^3	Kilograms per Cubic Metre
K_H	System Factor
K_L	Lateral Stability Factor
K_m	Service Creep Factor
kN	Kilonewton
K_N	Notch Factor
kN/m	Kilonewton per Metre
kN•m	Kilonewton Metre
kPa	Kilopascals
K_{pb}	Panel Buckling Factor
K_{rb}	Adjustment Factor for Bending Moment Resistance of CLT Panels
K_R	Radial Stress Factor
K_S	Service Condition Factor
K_{SF}	Service Condition Factor for Connections
K_{span}	Deflection Factor for Plank Decking Pattern
K_T	Treatment Factor
K_X	Curvature Factor
K_Z	Size Factor
K_Δ	Deflection Factor
L	Length, Unsupported Length, Span, Live Load, Diaphragm Span Perpendicular to Direction of Load
L'_b	Average Bearing Length
L_b	Unsupported Length in Direction of b, Bearing Length
L_d	Unsupported Length in Direction of d
L_D	Depth of Diaphragm Parallel to the Direction of Load
L_e	Effective Length
L_p	Length of Penetration of Fastener into Main Member
L_{pt}	Threaded Length of Penetration of Fastener into Main Member
l_p	Overall Panel Span in Direction of Webs
L_s	Length of the Shearwall Segment Parallel to the Applied Load
L_t	Length of Penetration of Threads, Length of Structure having Greatest Depth of Backfill