Preface

The Canadian Wood Council (CWC) is Canada's authoritative source of technical information and tools for wood design and construction.

Wood construction in mid-rise residential and non-residential buildings is booming, and provisions in the 2020 National Building Code are opening new opportunities for taller mass timber construction. Many innovative construction projects across the country are using materials such as cross-laminated timber and other engineered wood products for reasons of versatility, safety, and beauty. Expanded use of Canadian wood as a renewable and carbon-effective resource for construction, will also help ensure a more sustainable future for all of us.

The ninth edition of the *Wood Design Manual* is to help the design community – architects, engineers, specification writers, teachers and students of these disciplines – to design traditional and innovative wood structures with efficiency, economy and safety.

The CWC would like to thank the individuals who were instrumental in the original development of this manual: Stephen J. Boyd, Quaile Engineering Ltd., and Gary C. Williams, Timber Systems Ltd.

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President & CEO

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Errata available at:
www.cwc.ca/publication-type/manuals

*For information on CWC's other design tools, visit the CWC web site at www.cwc.ca

For CLT, a linear interaction equation is applicable:

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

where:

$P_{E,v}$ = Euler buckling load in the plane of the applied bending moment taking into account shear deformation (N)

$$= \frac{P_E}{1+\frac{P_E}{(GA)_{eff,f,0}}}$$

P_E = Euler buckling load for CLT in the plane of the applied moment accounting for only the layers with laminations parallel to the axial load (N)

$(GA)_{eff,f,0}$ = effective shear stiffness of the CLT panel in the major strength direction accounting for all layers (see Table 2.11 in Section 2.9 for $(GA)_{eff,f,0}$ values)

When checking the interaction equation for bending and compression, the compressive resistance P_r is calculated as if only compressive loads are present. Therefore, it is always based on buckling in the weak direction.

The factored resistances P_r, T_r and M_r may be found in the appropriate sections of this manual as indicated in Table 5.1. In addition, selection tables for stud walls and CLT walls subjected to combined lateral wind loading and axial loading are provided in Section 5.2 and Section 5.4, respectively.

Table 5.1
Factored resistance for combined bending and axial loads

Material	P_r	T_r	M_r
Sawn Lumber	Stud Wall Selection Tables, Section 3.2	Tension Member Selection Tables, Section 4.2	Joist Selection Tables, Section 2.3
Sawn Timber	Column Selection Tables, Section 3.3	Tension Member Selection Tables, Section 4.2	Beam Selection Tables, Section 2.5
20f-EX, 24f-EX Glulam	Column Selection Tables, Section 5.3	Tension Member Selection Tables, Section 4.2	Beam Selection Tables, Section 2.5
CLT	Wall Panel Selection Tables, Section 3.5	Not Applicable	Panel Selection Tables, Section 2.9

Column Selection Tables (combined compression and uniaxial bending)

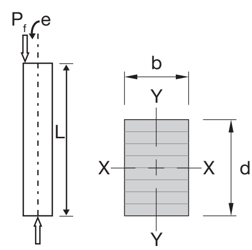
80 mm Glulam (D.Fir-L 24f-EX)

d (mm)	114						152					
L m	e=0 P_{rx} kN	P_{ry} kN	e=d/6 P'_r kN	e=d/2 P'_r kN	e=b/6 P'_r kN	e=b/2 P'_r kN	e=0 P_{rx} kN	P_{ry} kN	e=d/6 P'_r kN	e=d/2 P'_r kN	e=b/6 P'_r kN	e=b/2 P'_r kN
2.0	155	99.7	81.8	56.7	71.0	42.1	250	133	109	75.6	94.6	56.1
2.5	121	65.5	57.5	44.7	50.6	35.8	218	87.3	76.7	59.6	67.4	47.8
3.0	91.3	43.3	39.7	33.5	36.2	28.7	184	57.8	53.0	44.7	48.3	38.3
3.5	68.0	29.4	27.8	24.7	26.1	22.1	151	39.2	37.0	32.9	34.8	29.4
4.0	50.7	20.6	19.8	18.2	19.1	16.9	122	27.5	26.4	24.3	25.4	22.5
4.5	38.2						97.3					
5.0	29.2						77.7					
5.5	22.7						62.4					
6.0							50.5					
6.5							41.3					
7.0							34.0					
7.5							28.3					

d (mm)	190						228					
L m	e=0 P_{rx} kN	P_{ry} kN	e=d/6 P'_r kN	e=d/2 P'_r kN	e=b/6 P'_r kN	e=b/2 P'_r kN	e=0 P_{rx} kN	P_{ry} kN	e=d/6 P'_r kN	e=d/2 P'_r kN	e=b/6 P'_r kN	e=b/2 P'_r kN
2.0	337	166	136	94.4	118	70.1	419	199	162	111	142	84.1
2.5	312	109	95.8	74.5	84.3	59.7	400	131	114	87.4	101	71.7
3.0	281	72.2	66.2	55.9	60.3	47.8	372	86.5	78.7	65.5	72.3	57.4
3.5	247	49.0	46.2	41.2	43.5	36.8	337	58.6	54.9	48.2	52.1	44.1
4.0	210	34.3	32.9	30.3	31.7	28.0	301	41.1	39.1	35.6	38.0	33.6
4.5	178						266					
5.0	149						233					
5.5	125						202					
6.0	104						175					
6.5	87.6						151					
7.0	73.8						131					
7.5	62.5						113					
8.0	53.2						98.1					
8.5	45.5						85.3					
9.0	39.2						74.3					
9.5	33.9						65.0					
10.0							57.1					

Notes:

1. $P_r = \phi F_{cb} A K_{Zc} K_c$.
2. P_{rx} is the factored compressive resistance about the X-X axis. P_{ry} is the factored compressive resistance about the Y-Y axis. P'_r is the factored maximum combined compressive resistance for eccentric compression.
3. The tabulated values are based on columns loaded either concentrically or eccentrically at the top. When eccentrically loaded, the combined loading check is performed at the top and mid-height of the column.
4. Where values are not given, the slenderness ratio exceeds 50 (maximum permitted).
5. For eccentric compression about the X-X axis, the tabulated values are based on assuming the full member width b for calculating the size factor K_{Zbg} for the bending moment resistance M_r. Glulam members with width wider than 175 mm are typically maunfactured with multiple pieces in a lamination. In such cases, the width b for calculating K_{Zbg} is required to be the width of the widest piece of the lamination. Check with glulam supplier before applying the table.
6. L = unsupported length.



Nail and Spike Selection Tables

1S Single Shear, Sawn Lumber Main Member

Basic factored lateral resistance based on minimum penetration

Side plate thickness **ASTM A36 Steel** (mm)	Nail type	Nail length (in.)	Nail diameter (mm)	Minimum penetration (mm)	D.Fir-L $N'_r \cdot n_s$ (kN)	Hem-Fir $N'_r \cdot n_s$ (kN)	S-P-F $N'_r \cdot n_s$ (kN)	Northern $N'_r \cdot n_s$ (kN)
4.76	Common wire nails	1	1.83	9	0.315	0.298	0.275	0.230
4.76		1.25	2.03	10	0.384	0.363	0.335	0.283
4.76		1.5	2.34	12	0.504	0.476	0.440	0.374
4.76		1.5	2.52	13	0.580	0.548	0.506	0.431
4.76		1.75	2.64	13	0.633	0.599	0.553	0.470
4.76		2	2.84	14	0.726	0.687	0.634	0.540
4.76		2	2.87	14	0.741	0.701	0.647	0.550
4.76		2.25	2.87	14	0.741	0.701	0.647	0.550
4.76		2.25	2.95	15	0.780	0.738	0.681	0.579
4.76		2.5	3.25	16	0.934	0.884	0.816	0.694
4.76		2.5	3.33	17	0.977	0.925	0.853	0.726
4.76		2.75	3.33	17	0.977	0.925	0.853	0.726
4.76		3	3.66	18	1.16	1.10	1.02	0.864
4.76		3	3.76	19	1.22	1.16	1.07	0.908
4.76		3.25	3.76	19	1.22	1.16	1.07	0.908
4.76		3.5	4.06	20	1.41	1.33	1.23	1.04
4.76		3.5	4.11	21	1.44	1.36	1.25	1.07
4.76		4	4.88	24	1.95	1.85	1.70	1.45
4.76		4.5	5.26	26	2.22	2.10	1.94	1.65
4.76		4.5	5.38	27	2.31	2.19	2.02	1.72
4.76		5	5.74	29	2.58	2.44	2.25	1.92
4.76		5	5.89	29	2.70	2.55	2.35	2.00
4.76		5.5	6.2	31	2.94	2.78	2.56	2.18
4.76		5.5	6.4	32	3.09	2.93	2.70	2.30
4.76		6	6.65	33	3.29	3.11	2.87	2.44
4.76		6	7.01	35	3.58	3.39	3.12	2.66
4.76	Common spikes	4	6.4	32	3.09	2.93	2.70	2.30
4.76		6	7.62	38	4.07	3.85	3.55	3.02
4.76		8	8.23	41	4.56	4.31	3.98	3.38
4.76	Common spiral spikes	2.5	2.77	14	0.693	0.656	0.605	0.515
4.76		3	3.1	16	0.856	0.809	0.747	0.636
4.76		3.5	3.86	19	1.28	1.21	1.12	0.952
4.76		4	4.33	22	1.58	1.49	1.38	1.17
4.76		5	4.88	24	1.95	1.85	1.70	1.45

Notes: See page 375.

Nail and Spike Selection Tables

1S Single Shear, Sawn Lumber Main Member

Basic factored lateral resistance based on maximum penetration

				D.Fir-L		Hem-Fir		S-P-F		Northern	
Side plate thickness **ASTM A36 Steel** (mm)	Nail type	Nail length (in.)	Nail diameter (mm)	≥ 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)	≥ Maximum penetration (mm)	$N'_r \cdot n_s$ (kN)
4.76	Common wire nails	1	1.83	9	0.315	9	0.298	9	0.275	9	0.234
4.76		1.25	2.03	10	0.384	10	0.363	10	0.335	10	0.285
4.76		1.5	2.34	12	0.504	12	0.476	12	0.440	12	0.374
4.76		1.5	2.52	13	0.580	13	0.548	13	0.506	13	0.431
4.76		1.75	2.64	13	0.633	13	0.599	13	0.553	13	0.470
4.76		2	2.84	14	0.726	14	0.687	14	0.634	14	0.540
4.76		2	2.87	14	0.741	14	0.701	14	0.647	14	0.550
4.76		2.25	2.87	14	0.741	14	0.701	14	0.647	14	0.550
4.76		2.25	2.95	15	0.780	15	0.738	15	0.681	15	0.579
4.76		2.5	3.25	16	0.934	16	0.884	16	0.816	16	0.694
4.76		2.5	3.33	17	0.977	17	0.925	17	0.853	17	0.726
4.76		2.75	3.33	17	0.977	17	0.925	17	0.853	17	0.726
4.76		3	3.66	18	1.16	18	1.10	18	1.02	18	0.864
4.76		3	3.76	19	1.22	19	1.16	19	1.07	19	0.908
4.76		3.25	3.76	19	1.22	19	1.16	19	1.07	19	0.908
4.76		3.5	4.06	20	1.41	20	1.33	20	1.23	20	1.04
4.76		3.5	4.11	21	1.44	21	1.36	21	1.25	21	1.07
4.76		4	4.88	24	1.95	24	1.85	24	1.70	24	1.45
4.76		4.5	5.26	26	2.22	26	2.10	26	1.94	26	1.65
4.76		4.5	5.38	27	2.31	27	2.19	27	2.02	27	1.72
4.76		5	5.74	29	2.58	29	2.44	29	2.25	29	1.92
4.76		5	5.89	29	2.70	29	2.55	29	2.35	29	2.00
4.76		5.5	6.2	31	2.94	31	2.78	31	2.56	31	2.18
4.76		5.5	6.4	32	3.09	32	2.93	32	2.70	32	2.30
4.76		6	6.65	33	3.29	33	3.11	33	2.87	33	2.44
4.76		6	7.01	35	3.58	35	3.39	35	3.12	35	2.66
4.76	Common spikes	4	6.4	32	3.09	32	2.93	32	2.70	32	2.30
4.76		6	7.62	38	4.07	38	3.85	38	3.55	38	3.02
4.76		8	8.23	41	4.56	41	4.31	41	3.98	41	3.38
4.76	Common spiral spikes	2.5	2.77	14	0.693	14	0.656	14	0.605	14	0.515
4.76		3	3.1	16	0.856	16	0.809	16	0.747	16	0.636
4.76		3.5	3.86	19	1.28	19	1.21	19	1.12	19	0.952
4.76		4	4.33	22	1.58	22	1.49	22	1.38	22	1.17
4.76		5	4.88	24	1.95	24	1.85	24	1.70	24	1.45

Nail and Spike Selection Tables

1S Single Shear, CLT Main Member

Basic factored lateral resistance based on minimum penetration

Side plate thickness **ASTM A36 Steel** (mm)	Nail type	Nail length (in.)	Nail diameter (mm)	Minimum penetration (mm)	**E2** and **V1** $N'_r \cdot n_s$ (kN)	**E1** and **V2** $N'_r \cdot n_s$ (kN)	**E3** $N'_r \cdot n_s$ (kN)
4.76	Common wire nails	1	1.83	9	0.290	0.249	0.207
4.76		1.25	2.03	10	0.356	0.305	0.254
4.76		1.5	2.34	12	0.472	0.404	0.337
4.76		1.5	2.52	13	0.546	0.468	0.390
4.76		1.75	2.64	13	0.598	0.513	0.427
4.76		2	2.84	14	0.690	0.592	0.494
4.76		2	2.87	14	0.704	0.605	0.504
4.76		2.25	2.87	14	0.704	0.605	0.504
4.76		2.25	2.95	15	0.741	0.639	0.532
4.76		2.5	3.25	16	0.887	0.773	0.644
4.76		2.5	3.33	17	0.928	0.810	0.675
4.76		2.75	3.33	17	0.928	0.810	0.675
4.76		3	3.66	18	1.10	0.964	0.813
4.76		3	3.76	19	1.16	1.01	0.857
4.76		3.25	3.76	19	1.16	1.01	0.857
4.76		3.5	4.06	20	1.33	1.16	0.991
4.76		3.5	4.11	21	1.36	1.19	1.01
4.76		4	4.88	24	1.85	1.62	1.38
4.76		4.5	5.26	26	2.11	1.84	1.57
4.76		4.5	5.38	27	2.20	1.92	1.63
4.76		5	5.74	29	2.45	2.14	1.82
4.76		5	5.89	29	2.56	2.23	1.90
4.76		5.5	6.2	31	2.79	2.43	2.07
4.76		5.5	6.4	32	2.94	2.56	2.18
4.76		6	6.65	33	3.13	2.73	2.32
4.76		6	7.01	35	3.40	2.97	2.52
4.76	Common spikes	4	6.4	32	2.94	2.56	2.18
4.76		6	7.62	38	3.87	3.37	2.87
4.76		8	8.23	41	4.33	3.78	3.21
4.76	Common spiral spikes	2.5	2.77	14	0.658	0.564	0.470
4.76		3	3.1	16	0.813	0.704	0.587
4.76		3.5	3.86	19	1.22	1.06	0.902
4.76		4	4.33	22	1.50	1.31	1.11
4.76		5	4.88	24	1.85	1.62	1.38

Notes: See page 388.

Nail and Spike Selection Tables

1S Single Shear, Cross-Laminated Timber Main Member

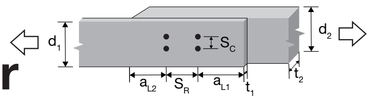
Basic factored lateral resistance based on maximum penetration

Side plate thickness **ASTM A36 Steel** (mm)	Nail type	Nail length (in.)	Nail diameter (mm)	**E2** and **V1** ≥ Maximum penetration (mm)	**E2** and **V1** $N'_r \cdot n_s$ (kN)	**E1** and **V2** ≥ Maximum penetration (mm)	**E1** and **V2** $N'_r \cdot n_s$ (kN)	**E3** ≥ Maximum penetration (mm)	**E3** $N'_r \cdot n_s$ (kN)
4.76	Common wire nails	1	1.83	9	0.299	10	0.261	10	0.222
4.76		1.25	2.03	10	0.365	11	0.318	11	0.271
4.76		1.5	2.34	12	0.478	12	0.417	12	0.355
4.76		1.5	2.52	13	0.551	13	0.481	13	0.409
4.76		1.75	2.64	13	0.601	14	0.525	14	0.446
4.76		2	2.84	14	0.690	14	0.602	15	0.512
4.76		2	2.87	14	0.704	15	0.614	15	0.522
4.76		2.25	2.87	14	0.704	15	0.614	15	0.522
4.76		2.25	2.95	15	0.741	15	0.647	15	0.550
4.76		2.5	3.25	16	0.887	16	0.774	17	0.659
4.76		2.5	3.33	17	0.928	17	0.810	17	0.689
4.76		2.75	3.33	17	0.928	17	0.810	17	0.689
4.76		3	3.66	18	1.10	18	0.964	18	0.820
4.76		3	3.76	19	1.16	19	1.01	19	0.862
4.76		3.25	3.76	19	1.16	19	1.01	19	0.862
4.76		3.5	4.06	20	1.33	20	1.16	20	0.991
4.76		3.5	4.11	21	1.36	21	1.19	21	1.01
4.76		4	4.88	24	1.85	24	1.62	24	1.38
4.76		4.5	5.26	26	2.11	26	1.84	26	1.57
4.76		4.5	5.38	27	2.20	27	1.92	27	1.63
4.76		5	5.74	29	2.45	29	2.14	29	1.82
4.76		5	5.89	29	2.56	29	2.23	29	1.90
4.76		5.5	6.2	31	2.79	31	2.43	31	2.07
4.76		5.5	6.4	32	2.94	32	2.56	32	2.18
4.76		6	6.65	33	3.13	33	2.73	33	2.32
4.76		6	7.01	35	3.40	35	2.97	35	2.52
4.76	Common spikes	4	6.4	32	2.94	32	2.56	32	2.18
4.76		6	7.62	38	3.87	38	3.37	38	2.87
4.76		8	8.23	41	4.33	41	3.78	41	3.21
4.76	Common spiral spikes	2.5	2.77	14	0.658	14	0.574	14	0.489
4.76		3	3.1	16	0.813	16	0.709	16	0.603
4.76		3.5	3.86	19	1.22	19	1.06	19	0.904
4.76		4	4.33	22	1.50	22	1.31	22	1.11
4.76		5	4.88	24	1.85	24	1.62	24	1.38

Notes: See page 388.

Bolt and Dowel Selection Tables

Type 1 1/2" Bolt or dowel Single shear, S-P-F lumber



Wood member				Fastener		Factored lateral resistance, P'_r (kN)									
						No. of fasteners in a row									
						2					3				
Thickness mm		Depth mm			Row spacing, S_C mm	Bolt spacing in a row taken as the minimum of the loaded end distances[3], a_{L1}, a_{L2}, and the spacing between bolts in a row, S_R (mm)									
Side, t_1	Main, t_2	Side, d_1	Main, d_2	No. of rows		51	76	102	127	152	51	76	102	127	152
38	38	89	89	1	N/A	3.16[R]	4.74[R]	5.66[f]	5.66[f]	5.66[f]	4.74[R]	7.11[R]	8.49[f]	8.49[f]	8.49[f]
		140	140	2	38	6.32[R]	8.17[G]	9.75[G]	11.3[f]	11.3[f]	8.17[G]	10.5[G]	12.9[G]	15.3[G]	17.0[f]
		140	140		64	6.32[R]	9.49[R]	11.3[f]	11.3[f]	11.3[f]	9.49[R]	14.2[R]	16.6[G]	17.0[f]	17.0[f]
		184	184		89	6.32[R]	9.49[R]	11.3[f]	11.3[f]	11.3[f]	9.49[R]	14.2[R]	17.0[f]	17.0[f]	17.0[f]
		184	184	3	38	9.49[R]	11.6[G]	13.2[G]	14.8[G]	16.3[G]	11.6[G]	14.0[G]	16.3[G]	18.7[G]	21.1[G]
		235	235		64	9.49[R]	14.2[R]	17.0[f]	17.0[f]	17.0[f]	14.2[R]	21.3[R]	23.8[G]	25.5[f]	25.5[f]
		286	286		89	9.49[R]	14.2[R]	17.0[f]	17.0[f]	17.0[f]	14.2[R]	21.3[R]	25.5[f]	25.5[f]	25.5[f]
38	89	89	89	1	N/A	3.16[R]	4.74[R]	6.32[R]	7.90[R]	8.45[d]	4.74[R]	7.11[R]	9.49[R]	11.9[R]	12.7[d]
		140	140	2	38	6.32[R]	8.17[G]	9.75[G]	11.3[G]	12.9[G]	8.17[G]	10.5[G]	12.9[G]	15.3[G]	17.7[G]
		140	140		64	6.32[R]	9.49[R]	12.6[R]	15.0[G]	16.6[G]	9.49[R]	14.2[R]	16.6[G]	19.0[G]	21.4[G]
		184	184		89	6.32[R]	9.49[R]	12.6[R]	15.8[R]	16.9[d]	9.49[R]	14.2[R]	19.0[R]	22.7[G]	25.1[G]
		184	184	3	38	9.49[R]	11.6[G]	13.2[G]	14.8[G]	16.3[G]	11.6[G]	14.0[G]	16.3[G]	18.7[G]	21.1[G]
		235	235		64	9.49[R]	14.2[R]	19.0[R]	22.2[G]	23.8[G]	14.2[R]	21.3[R]	23.8[G]	26.1[G]	28.5[G]
		286	286		89	9.49[R]	14.2[R]	19.0[R]	23.7[R]	25.3[d]	14.2[R]	21.3[R]	28.5[R]	33.6[G]	35.9[G]
38	140	140	140	1	N/A	3.16[R]	4.74[R]	6.32[R]	7.90[R]	8.45[d]	4.74[R]	7.11[R]	9.49[R]	11.9[R]	12.7[d]
		140	140	2	38	6.32[R]	8.17[G]	9.75[G]	11.3[G]	12.9[G]	8.17[G]	10.5[G]	12.9[G]	15.3[G]	17.7[G]
		140	140		64	6.32[R]	9.49[R]	12.6[R]	15.0[G]	16.6[G]	9.49[R]	14.2[R]	16.6[G]	19.0[G]	21.4[G]
		184	191		89	6.32[R]	9.49[R]	12.6[R]	15.8[R]	16.9[d]	9.49[R]	14.2[R]	19.0[R]	22.7[G]	25.1[G]
		184	191	3	38	9.49[R]	11.6[G]	13.2[G]	14.8[G]	16.3[G]	11.6[G]	14.0[G]	16.3[G]	18.7[G]	21.1[G]
		235	241		64	9.49[R]	14.2[R]	19.0[R]	22.2[G]	23.8[G]	14.2[R]	21.3[R]	23.8[G]	26.1[G]	28.5[G]
		286	292		89	9.49[R]	14.2[R]	19.0[R]	23.7[R]	25.3[d]	14.2[R]	21.3[R]	28.5[R]	33.6[G]	35.9[G]
89	89	89	89	1	N/A	7.41[R]	11.1[R]	11.2[g]	11.2[g]	11.2[g]	11.1[R]	16.7[R]	16.8[g]	16.8[g]	16.8[g]
		140	140	2	38	14.8[R]	19.1[G]	22.5[g]	22.5[g]	22.5[g]	19.1[G]	24.7[G]	30.2[G]	33.7[g]	33.7[g]
		140	140		64	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	184		89	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	184	3	38	22.2[R]	27.1[G]	30.8[G]	33.7[g]	33.7[g]	27.1[G]	32.7[G]	38.3[G]	43.8[G]	49.4[G]
		235	235		64	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	50.0[R]	50.5[g]	50.5[g]	50.5[g]
		286	286		89	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	50.0[R]	50.5[g]	50.5[g]	50.5[g]
89	140	140	140	1	N/A	7.41[R]	11.1[R]	11.2[g]	11.2[g]	11.2[g]	11.1[R]	16.7[R]	16.8[g]	16.8[g]	16.8[g]
		140	140	2	38	14.8[R]	19.1[G]	22.5[g]	22.5[g]	22.5[g]	19.1[G]	24.7[G]	30.2[G]	33.7[g]	33.7[g]
		140	140		64	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	191		89	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	191	3	38	21.2[G]	25.9[G]	30.6[G]	33.7[g]	33.7[g]	25.9[G]	32.7[G]	38.3[G]	43.8[G]	49.4[G]
		235	241		64	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	43.0[G]	50.0[G]	50.5[g]	50.5[g]
		286	292		89	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	50.0[R]	50.5[g]	50.5[g]	50.5[g]
89	191	140	140	1	N/A	7.41[R]	11.1[R]	11.2[g]	11.2[g]	11.2[g]	11.1[R]	16.7[R]	16.8[g]	16.8[g]	16.8[g]
		140	140	2	38	14.8[R]	19.1[G]	22.5[g]	22.5[g]	22.5[g]	19.1[G]	24.7[G]	30.2[G]	33.7[g]	33.7[g]
		140	140		64	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	191		89	14.8[R]	22.2[R]	22.5[g]	22.5[g]	22.5[g]	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]
		184	191	3	38	22.2[R]	27.1[G]	30.8[G]	33.7[g]	33.7[g]	27.1[G]	32.7[G]	38.3[G]	43.8[G]	49.4[G]
		235	241		64	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	50.0[R]	50.5[g]	50.5[g]	50.5[g]
		286	292		89	22.2[R]	33.3[R]	33.7[g]	33.7[g]	33.7[g]	33.3[R]	50.0[R]	50.5[g]	50.5[g]	50.5[g]

Notes: see page 428.

Calculation

T_f = 16 kN

Try four 3/8" diameter lag screws.

$P_{rw} = P'_{rw}\ L_{ta}\ n_F\ K'\ J_E$

P'_{rw} = 64.0 N/mm from Table 7.15

$n_F = 4$

$P_{rw} = 64.0 \times L_{ta} \times 4 \times 1.15 \times 1.0$

$= 294\ L_t$ (N)

Required $L_{ta} = \dfrac{16 \times 10^3}{294}$ = 54.3 mm < 86 mm (maximum penetration length in Table 7.14) *Acceptable*

Determine the required lag screw length from Table 7.16:

$L_{REQ'D} = (54.3 + 6.4 - 12.7) \times 2$

$=$ 96 mm (3.78")

Therefore, try four 3/8" x 4" lag screws.

Check geometry of connection from Figure 7.10:

loaded edge distance > 38 mm *Acceptable*

spacing = 102 mm > 29 mm *Acceptable*

end distance > 50 mm *Acceptable*

Use four 3/8" x 4" lag screws.

Table 7.21
Effective number of fasteners per row n_{Fe} for connections with wood side plates

Area ratio A_m/A_s	The lesser of A_m and A_s	Actual number of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
0.5	< 8000	2.00	2.76	3.36	3.8	4.08	4.27	4.40	4.41	4.30	4.18	4.08
	8001 to 12000	2.00	2.85	3.52	4.10	4.50	4.76	4.96	5.13	5.20	5.28	5.16
	12001 to 18000	2.00	2.91	3.72	4.40	4.92	5.39	5.68	6.03	6.30	6.49	6.60
	18001 to 26000	2.00	2.94	3.84	4.60	5.22	5.81	6.32	6.75	7.10	7.59	7.92
	26001 to 42000	2.00	3.00	3.88	4.70	5.40	6.02	6.64	7.11	7.60	8.14	8.64
	> 42000	2.00	3.00	3.92	4.75	5.46	6.16	6.80	7.38	8.00	8.58	9.12
1.0	< 8000	2.00	2.91	3.68	4.25	4.68	4.97	5.20	5.31	5.40	5.39	5.28
	8001 to 12000	2.00	2.94	3.76	4.45	5.04	5.46	5.76	5.94	6.10	6.16	6.12
	12001 to 18000	2.00	3.00	3.88	4.65	5.34	5.95	6.40	6.84	7.20	7.48	7.68
	18001 to 26000	2.00	3.00	3.96	4.80	5.52	6.23	6.8	7.47	8.00	8.58	9.00
	26001 to 42000	2.00	3.00	4.00	4.85	5.64	6.37	7.04	7.65	8.40	9.02	9.60
	> 42000	2.00	3.00	4.00	4.95	5.76	6.51	7.28	7.92	8.70	9.46	10.2

Notes:

1. A_m = gross cross-sectional area of main member, mm^2.
   A_s = sum of gross cross-sectional areas of wood side plates, mm^2.
   Area ratio = the lesser of A_m/A_s and A_s/A_m
2. For perpendicular to grain loading, the area of the main member may be taken as the product of the member thickness and the width of the fastener group. For a single row of shear plates or split rings, use the minimum parallel to grain spacing for J_C = 1.0 (shown in Figure 7.16) as the width of the fastener group. For a single row of lag screws, use the minimum parallel to grain spacing as the width of the fastener group.
3. For area ratios between 0.5 and 1.0 interpolate between tabulated values.
   For area ratios less than 0.5 extrapolate from tabulated values.
4. n_{Fe} applies to shear plates, split rings and lag screws.

Splice No.	Location x (mm)	Specified moment (kN·m)	Specified force (kN)	Number of nails used per side of splice	Specified force per nail (kN)	Δ_c for tension splice (mm)	Δ_c for compression splice (mm)	$\Sigma\Delta_c x$ (mm²)
1	4286	133	8.92	20	0.446	0.375	0.375	3212
2	8571	221	14.9	34	0.438	0.362	0.362	6201
3	12857	266	17.8	40	0.445	0.373	0.373	9591
4	12857	266	17.8	40	0.445	0.373	0.373	9591
5	8571	221	14.9	34	0.438	0.362	0.362	6201
6	4286	133	8.92	20	0.446	0.375	0.375	3212
Total								38009

For the panel-nail slip, $e_n = (0.013\ v \cdot s/d_F{}^2)^2 = (0.013 \times 2.41 \times 100/3.25^2)^2 = 0.088$ mm. The lateral deflection at the diaphragm mid-span is:

$$\Delta_d = \frac{5vL^3}{96EAL_D} + \frac{vL}{4B_v} + 0.00061\ Le_n + \frac{\Sigma(\Delta_c x)}{2L_D}$$

$$= \frac{5 \times 2.41 \times 30000^3}{96 \times 9500 \times 10640 \times 15000} + \frac{2.41 \times 30000}{4 \times 5700} + 0.00061 \times 30000 \times 0.088 + \frac{38009}{2 \times 15000}$$

$= 8.3$ mm

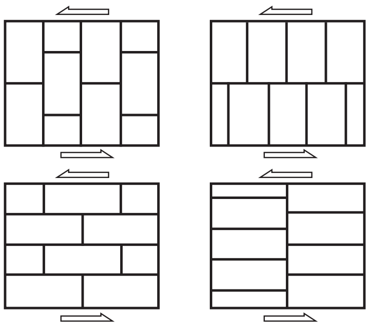
Use 12.5 mm CSP plywood with 2-1/2" nails spaced at 100 mm at blocked diaphragm boundaries and panel edges. The chords are two 38 x 140 mm No.1 or No.2 grade S-P-F members with splices at at 4.29 m interval with 3" nails. The maximum deflection of the diaphragm at the mid-span is 8.3 mm.

Table 8.5
Adjustment factor for unblocked shearwalls, J_{us}

Fastener spacing at supported edges, mm	Fastener spacing at intermediate studs, mm	Stud spacing, mm 300	400	500	600
150	150	1.0	0.8	0.6	0.5
150	300	0.8	0.6	0.5	0.4

Notes:

1. The adjustment factor shall be applicable only to wood structural panels and the stated fastener spacings.
2. The shear resistance of an unblocked shearwall shall be determined by the smaller of a) multiplying the adjustment factor for unblocked shear shearwalls, J_{us}, by the lateral resistance of sheathing-to-framing connection of a blocked shearwall with fasteners spaced at 150 mm on centre along panel edges and 300 mm on centre along intermediate framing members, or b) the panel buckling resistance.
3. Panels are installed either horizontally or vertically as shown below.



Shearwalls with Multiple Layers

Shearwalls may be constructed with:

- wood structural panels on both sides;
- gypsum panels on both sides; or
- wood structural panels on one side and gypsum panels on the opposite side.

The factored shear resistance of panels on both sides of the same shearwall may be added together. Where a wood structural panel is applied over 12.7 mm or 15.9 mm thick gypsum wallboard, the shear resistance may be calculated by assuming that the second layer is applied directly to framing and using the actual fastener penetration into the framing member, provided minimum nail penetration requirement is met. In all other cases with multiple layers of panels on the same side of the shearwall, only the shear resistance of the panel closest to the studs is considered in design.

Shearwalls with Multiple Wall Segments

Where shearwalls have openings for windows or doors, only the full height segments between openings and at the ends of the shearwalls are considered in shearwall design. In the case of blocked shearwalls where the height of the shearwall, H_s, measured from the bottom of the bottom plate to the top of the top plate, is greater than 3.5 times the length of the segment, L_s, the segment is not permitted to provide lateral resistance. For unblocked shearwalls, where H_s is greater than $2L_s$, the segment is not permitted to be included in the lateral resistance calculation.

Steps to calculate forces for shearwall segments without hold-downs, assuming shear forces are distributed to segments based on their relative resistance:

Calculate J_{hd} for all segments of the upper storey shearwall before proceeding to the next lower storey.

1. Divide the shearwall into segments ($L_{s,1}$, $L_{s,2}$).
2. Calculate the dead load to resist overturning at the top of the segments ($P_{t,ij}$), and the bottom of the segments (P_{ij}).
3. Calculate $J_{hd,ij}$ for each of the segments.
4. Calculate the shear resistance of each segment ($V_{rs,ij}$).
5. Distribute the storey shear force ($V_{fs,i}$) to the segments based on their relative resistance.
6. Calculate the uplift force at the end of each segment (R_{ij}).
7. Repeat steps 2 to 6 for lateral loads acting in the opposite direction.
8. Repeat steps 1 to 7 for each lower storey shearwall.

Figure 8.6
Calculation of $P_{t,ij}$, P_{ij} and R_{ij} for a two-segment shearwall in a two-storey building

$$P_{t,21} = q_2 \frac{L_{s,1}}{2} + q_2 \frac{L_{s,3}}{2}$$

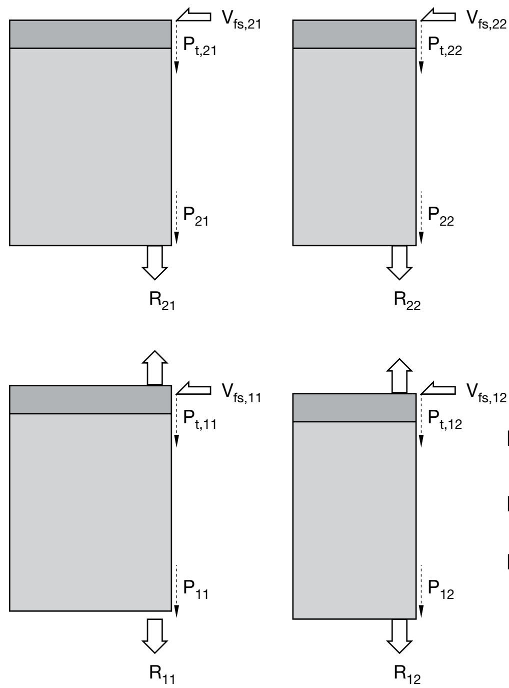
$$P_{21} = P_{t,21} + q_{w,21} \frac{L_{s,1}}{2}$$

$$R_{21} = V_{fs,21} \frac{H_{s,2} + d_2}{L_{s,1}} - P_{21}$$

$$P_{t,22} = q_2 \frac{L_{s,2}}{2}$$

$$P_{22} = P_{t,22} + q_{w,22} \frac{L_{s,2}}{2}$$

$$R_{22} = V_{fs,22} \frac{H_{s,2} + d_2}{L_{s,2}} - P_{22}$$



$$P_{t,11} = q_1 \frac{L_{s,1}}{2} + q_1 \frac{L_{s,3}}{2} - R_{21}$$

$$P_{11} = P_{t,11} + q_{w,11} \frac{L_{s,1}}{2}$$

$$R_{11} = V_{fs,11} \frac{H_{s,1} + d_1}{L_{s,1}} - P_{11}$$

$$P_{t,12} = q_1 \frac{L_{s,2}}{2} - R_{22}$$

$$P_{12} = P_{t,12} + q_{w,12} \frac{L_{s,2}}{2}$$

$$R_{12} = V_{fs,12} \frac{H_{s,1} + d_1}{L_{s,2}} - P_{12}$$

Resistance to Overturning

Resistance to overturning is provided by chord members at the ends of the shearwall segments. The chord members must be designed so that the factored compression and tensile resistance is greater than or equal to maximum factored axial force. The axial chord forces need to include the chord forces from upper storeys.

	Segment 1	Segment 2
L_s (m)	4	3
v_{rs} (kN/m)	8.59	8.59
V_{rs} (kN)	34.3	25.8

Determine if the shear resistance of the shearwall is adequate:

$$\Sigma V_{rs} = (34.3 + 25.8) = 60.1 \text{ kN} > 45.4 \qquad \textit{Acceptable}$$

The over capacity coefficient for the first storey:

$$C_1 = \frac{V_{r1}}{V_{f1}} = \frac{60.1}{45.4} = 1.32$$

Check the ratio of second storey to first storey over-capacity coefficients:

$$0.9 < \frac{C_2}{C_1} = \frac{1.28}{1.32} = 0.970 < 1.2 \qquad \textit{Acceptable}$$

For an earthquake from the North, along Segment 1 on the first storey, the dead loads are calculated as follows:

Note the floor dead load and the wall self-weight of full height segments are assumed to contribute to resisting overturning forces.

$$P_D(\text{for } P_f) = 0.5 \text{ kN/m} \times \left(\frac{4 \text{ m}}{2} + \frac{3 \text{ m}}{2}\right) + 0.99 \text{ kN/m} \times \frac{4 \text{ m}}{2} + 5.71 \text{ kN} = 9.44 \text{ kN}$$

$$P_D(\text{for } T_f) = 0.5 \text{ kN/m} \times \left(\frac{4 \text{ m}}{2}\right) + 0.99 \text{ kN/m} \times \frac{4 \text{ m}}{2} + 4.96 \text{ kN} = 7.94 \text{ kN}$$

The table below summarizes the axial chord design forces P_f and T_f for Segments 1 and 2 under an earthquake from the North or South direction.

4. Determine the required depth at the midpoint of the rafter.

   Since the rafter section is not curved $K_X = K_M = 1.0$ and $M'_r = M_{rb}$.
   Try d = 342:

Section	M'_r kN•m	C_c	F_{cb} MPa	F_{cb}/E' $\times 10^{-6}$	P_r kN	$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_{rb}}\left[\frac{1}{1-\frac{P_f}{P_E}}\right]$
130 x 342	69.8	25.2	24.2	77.5	407	0.96

Therefore, select d = 342 mm.

5. Determine the required depth at the peak.
   From the Beam Selection tables in Section 2.5, interpolate between depths of 228 and 266 mm to obtain the shear resistance for d = 247 mm:

   $V_r = \frac{41.5 + 35.6}{2} = 38.6$ kN > 23.3 kN *Acceptable*

6. Verify that the depth at the peak is adequate for a factored vertical reaction of 23.3 kN. Use the connection shown in Detail 7.32 in Section 7.12.
   Try two pairs of 2-5/8" diameter shear plates back to back.

   $\theta = 90 - 38.66 = 51.3°$

   $P_r = \phi\, p_u\, n_{Fe}\, n_R\, K'\, J'$

   $Q_r = \phi\, q_u\, n_{Fe}\, n_R\, K'\, J'$

   $\phi\, p_u$ = 16.2 kN (Table 7.19 in Section 7.8)

   $\phi\, q_u$ = 13.8 kN (Table 7.19)

   $n_{Fe} = 2.0$

   $n_R = 1$

   $K' = 1.0$

   J_O = 0.81 (linear interpolation between J_O = 1.0 at 0° and J_O = 0.67 at 90°)

   $J' = 0.81\, J_C$

   $P_r = 16.2 \times 2 \times 0.81 \times J_C = 26.2\, J_C$ (kN)

   $Q_r = 13.8 \times 2 \times 0.81 \times J_C = 22.4\, J_C$ (kN)

   Using Table 7.1 in Section 7.1:

   $\frac{Q_r}{P_r} = \frac{22.4}{26.2} = 0.85$

   X = 0.90

   $N_r = 26.2 \times 0.90\, J_C = 23.6\, J_C$ kN > 23.3 kN *Acceptable*

b) Using d = 813 mm calculate the lesser of M_{rt} and compare to maximum positive moment:

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

$$M_{rt} = \phi F_{tp} \frac{2}{3} A R_C K_{Ztp}$$

$$\phi F_{tp} = 0.859 \text{ MPa (Table 9.3 in Section 9.2)}$$

$$S = \frac{130 \times 813^2}{6} = 14.3 \times 10^6 \text{ mm}^3$$

$$A = 130 \times 813 = 106 \times 10^3 \text{ mm}^2$$

$$R_C = 2800 + \left[\frac{813}{2}\right] = 3206 \text{ mm}$$

$$K_{Ztp} = \frac{35}{(A R_c \beta)^{0.2}} = \frac{35}{\left[130 \times 813 \times 3206 \times \frac{51.3^\circ \times \pi}{180}\right]} = 0.70$$

$$K_R = \left[0.12 + 0.06\left[\frac{813}{3206}\right] + 0.12\left[\frac{813}{3206}\right]^2\right]^{-1} = 7.00$$

$$M_{rt} = 0.859 \times 14.3 \times 0.70 \times 7.00$$

$$= 60.2 \text{ kN•m} > 21.0 \text{ kN•m} \qquad \textit{Acceptable}$$

$$M_{rt} = 0.859 \times \frac{2}{3} \times 106 \times 3206 \times 0.70 \times 10^{-3}$$

$$= 136 \text{ kN•m} > 21.0 \text{ kN•m} \qquad \textit{Acceptable}$$

8. Calculate trial arch dimensions.

In the calculation of arch dimensions the depth at the base and peak are limited by the following rules established through experience:

- The slope between a tangent point and peak or base should not exceed 3.5°.
- The depth at peak or base should be at least 0.63 of the tangent depth to limit deflection.
- The taper from upper tangent point to peak should be uniform.

Using these rules the trial arch dimensions are as follows:

Location	Joint	Depth mm	Comment
Peak	0	285	Must be ≥ 0.63 of depth at pt. 2
Mid point of rafter	1	371	For uniform taper
Upper tangent point	2	456	Same as pt. 4 for ease of fabrication
Haunch	3	818	From geometry
Lower tangent point	4	456	As per design
Base	5	342	Slope from pt. 4 to pt. 5 limited to 3.5°

Figure 9.2c
Step 3: Temporary bracing of top chord plane

Truss top chords are very susceptible to lateral buckling before they are braced or sheathed. Continuous lateral bracing should be installed within 150 mm of the ridge line or centre line and at approximately 2.4 to 3 m intervals between the ridge line of sloped trusses or centre line of flat trusses and the eaves. Diagonals, set at 45 between the lateral bracing, form the required stability of the top chord. On longer span trusses, lateral bracing and diagonals may require closer spacing. If possible the continuous lateral bracing should be located on the underside of the top chord so that it does not have to be removed as sheathing is applied. This will ensure that the trusses are held securely during installation of the decking. Bracing lumber should be no less than 38 × 89 mm by 3.05 m long.

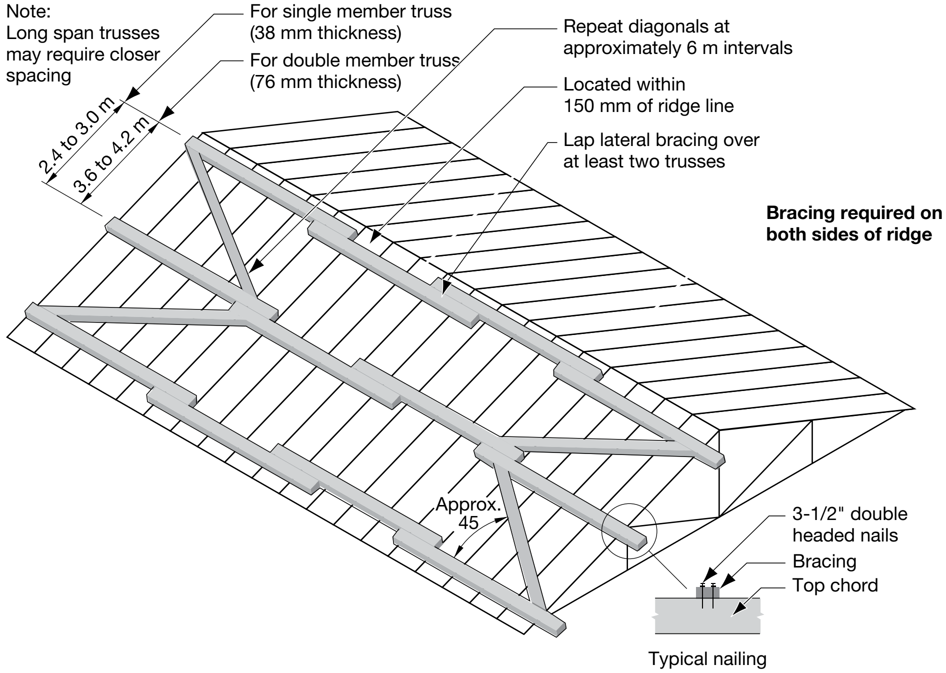
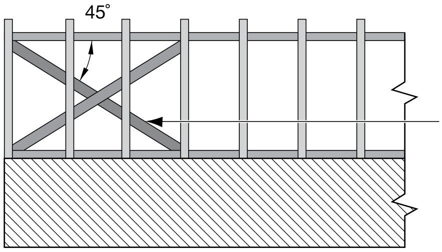


Figure 9.2d
Step 4: Temporary bracing of web member plane

Temporary bracing of the web member plane is usually installed at the same location specified on the engineering plan for permanent bracing. Permanent lateral web bracing should be called out on the truss design to reduce the buckling length of the individual web members. The bracing can form part of the temporary and permanent web bracing system. Sets of diagonal bracing should not be spaced more than 6 m apart (clear space between end of one set of braces and start of another set).



Note: Where lateral bracing is shown on the truss design drawings, it must be installed so that truss will support design loads.

Figure 9.2f
Step 6: Permanent bracing of top chord plane (large buildings)

If plywood floor or roof sheathing is properly applied with staggered joints and adequate nailing, a continuous diaphragm action is developed to resist lateral movement at the top chord, and additional bracing in the plane is generally not required. Some metal roofing materials may act as a diaphragm when properly lapped and nailed, but selection and use of these materials is at the discretion of the building designer. If purlins are used, spaced not to exceed the buckling length at the top chord, diagonal bracing should be applied to the underside of the top chord to prevent lateral shifting of the purlins. The diagonal bracing should be installed on both sides of the ridge line in all end bays. If the building exceeds 18 m in length, this bracing should be repeated at intervals not exceeding 6 m.

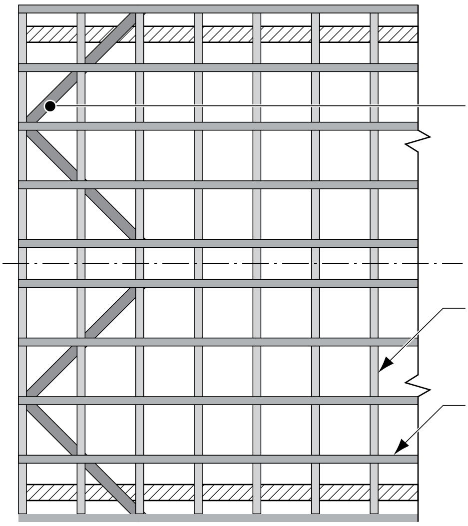
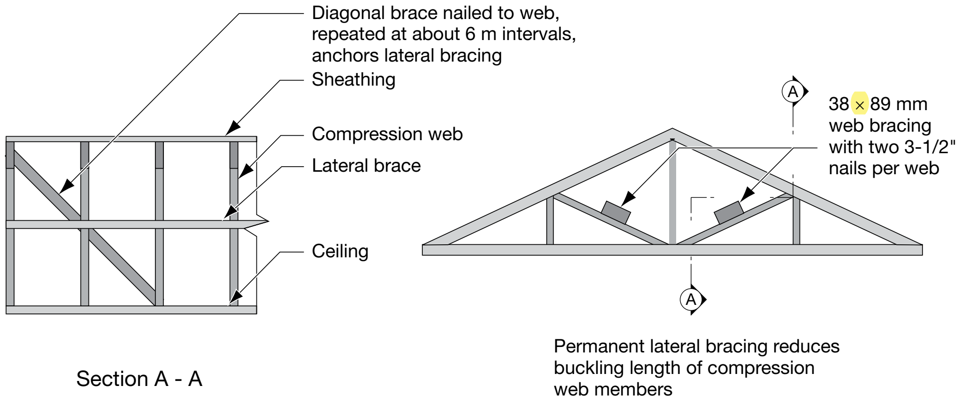


Figure 9.2g
Step 7: Permanent lateral bracing to web member or bottom chord plane (all buildings)

Permanent bracing in web and bottom chord planes is usually applied as temporary bracing (Steps 4 and 5). Lateral bracing of compression web members is a typical method to prevent buckling. The method used to anchor the lateral bracing must be specified by the designer. Bottom chord bracing helps to maintain truss spacing, also can resist buckling caused by stress reversal. Multiple-bearing or cantilevered trusses can result in compressive forces in bottom chords.



Note: Where lateral bracing is shown on the truss design drawings, it must be installed so that truss will support design loads.

Table 9.16
Treatable lumber species

Species combinations	Treatable species	Grade stamp identification
D.Fir-L	Coast Douglas fir	D Fir (N)
Hem-Fir	Western hemlock	W Hem (N) or Hem-Fir (N)
	Amabilis fir	Am Fir (N) or Hem-Fir (N)
S-P-F	Lodgepole pine	L Pine (N)
	Jack Pine	J Pine (N)
	Alpine Fir	Alpine Fir (N)
	Balsam Fir	B Fir (N)
Northern	Red pine	R Pine (N)
	Western white pine	W.W.Pine
	Eastern white pine	East White Pine or (EW Pine) (N)
	Grand fir	G fir (N)
	Ponderosa pine	P Pine
	Eastern hemlock	East Hemlock (N) or (E Hem) (N)

Notes:
1. Douglas fir lumber is restricted to Pacific coast Douglas fir.
2. Hem-Fir is the only species combination where all species are suitable for preservative treatment.

Figure 9.4
Example of qualification mark

Designers should be aware that the appearance of other treated wood products is similar to PWF material. However, treated wood products not intended for PWFs do not have the same degree of treatment and do not normally conform to the more stringent requirements for PWFs.

Fasteners and Connectors

Nails used in PWFs must be in accordance with CSA B111. Hot dipped galvanized or stainless steel nails are required for all treated material.

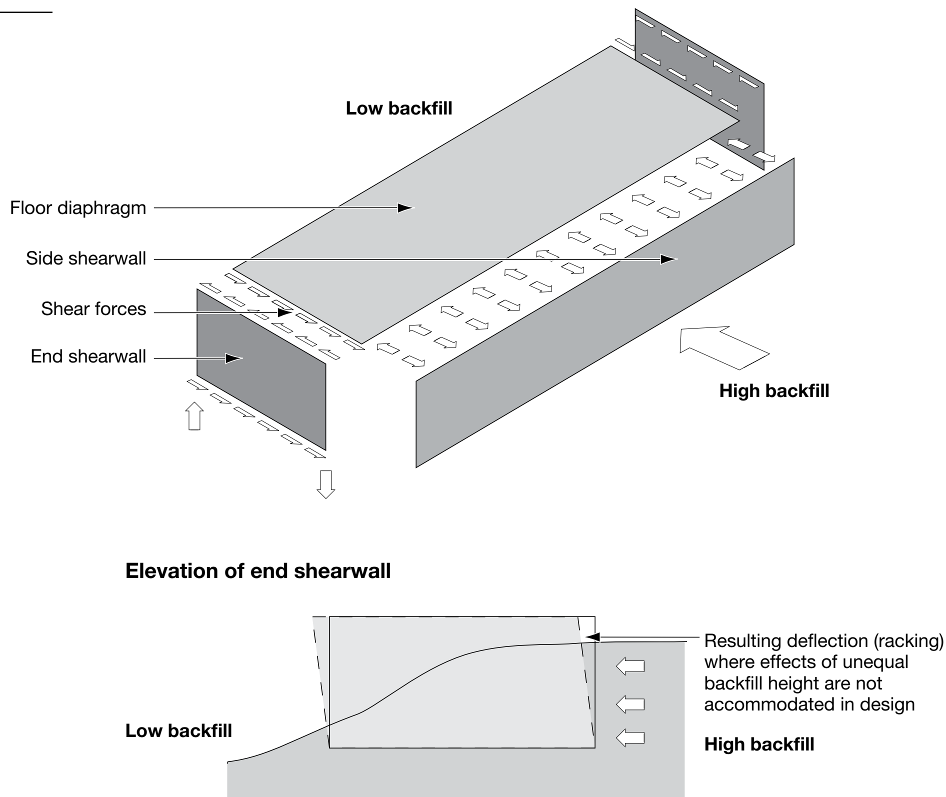
Framing anchors and straps should be used wherever the magnitude of loads is such that it is impractical to transfer the loads through nailing alone.

CSA S406 gives details on construction methods and fastening techniques that are applicable for use in typical PWFs.

Racking Resistance

Foundation walls may be subjected to racking loads, for example, loads acting in a direction parallel to the wall to create shearing forces in the plane of the wall (see Figure 9.6).

Figure 9.6
The effect of unequal back-fill heights



Note: Decreasing nail spacing around perimeter of end shearwall will help to prevent racking.

For rectangular shaped foundations, the net differential force F acting normal to the top of the wall having the greater depth of backfill is determined as follows:

$$F = \frac{e_f L_t}{6L}(H_1^3 - H_2^3)$$

where:

H_1 = height of backfill on the side of the structure with deeper backfill (m)

H_2 = height of backfill on the opposite side of the structure (m)

L_t = length of the structure having greatest depth of backfill (m)

L = length of the studs in the PWF (m)

e_f = equivalent factored fluid weight of soil (kN/m^3)

3. Specified design pressure governed by deflection

$$w_\Delta = \frac{\Delta_{max} \times 10^3}{\left[\frac{0.0064\,L_2^4}{EI}\right] + \left[\frac{C\,L_2^2}{GA}\right]}$$

where:

w_Δ = specified design pressure uniformly distributed over three or more equal spans and governed by bending and shear deflections (kN/m²)

Δ_{max} = maximum deflection (mm)

L_2 = clear span (mm)

EI = specified bending stiffness from Table 9.19 (N•mm²/mm)

C = shear deflection constant from Table 9.20

GA = planar shear rigidity from Table 9.19 (N/mm)

Tables 9.21 and 9.22 provide the allowable design pressures for deflection limits of L/270 and L/360. Table 9.21 is for DFP with face/back grain perpendicular to supports and Table 9.22 for DFP with face/back grain parallel to supports.

Table 9.19
Design capacities for CSA O121 certified exterior plywood applied as concrete formwork

Design capacities per 1 mm width

Face grain oriented **perpendicular** to supports

Type of plywood	Nominal panel thickness mm	Number of plies	Bending moment M N•mm/mm	Planar shear V N/mm	Bending stiffness EI × 10^6 N•mm²/mm	Planar shear rigidity GA N/mm
Good-one-side Douglas Fir Plywood (DFP)	17	5,6,7	600	8.54	2.51	503
	19	5,6,7	705	9.82	3.26	588
	21	7	1080	10.2	5.35	785
	24	7	1370	12.5	7.84	753
	27	9	1870	14.5	11.8	951
	30	9	2090	16.3	14.7	953
Sheathing grade Douglas Fir Plywood (DFP)	15.5	4,5	677	6.69	2.62	531
	18.5	5,6,7	890	9.39	4.08	648
	20.5	5,6,7	1020	10.3	5.49	658
	22.5	6,7,9	1420	11.4	7.98	854
	25.5	7,9	1720	13.2	10.5	842
	28.5	9	2290	15.4	15.5	1060
	31.5	9,11	2570	16.7	19.3	1050

Notes:

1. All capacities adjusted for wet service conditions. Bending moment and planar shear capacities adjusted for short term duration of loading.
2. All capacities are based on the lowest section properties for construction of any number of plies in the table.

Thermal Effects

The coefficient of linear thermal expansion in the longitudinal direction of wood is from one-tenth to one-third of the values for metals, concrete, and glass. While the values in the transverse directions are larger than in the longitudinal direction, they are still usually less than those of other structural materials.

The approximate coefficients of linear thermal expansion for Douglas fir are:

Longitudinal	Tangential	Radial
4×10^{-6} per °C	40×10^{-6} per °C	27×10^{-6} per °C

It is usually unnecessary to consider thermal expansion in wood structures, since dimensional changes caused by thermal effects will be insignificant compared to those caused by variations in moisture content.

Table 11.21 **Timber shrinkage across the grain**

Species	Shrinkage coefficient Maximum[1]	Average[2]
Generic lumber		0.0020
Douglas fir	0.0031	0.0026
Western hemlock	0.0036	0.0029
Western red cedar	0.0019	0.0013
Eastern and western spruce	0.0033	0.0022
Eastern hemlock	0.0029	0.0022
Red pine	0.0024	0.0020

Notes:

1. The maximum shrinkage coefficient refers to the coefficient in the tangential direction.
2. The average shrinkage coefficient is the average of the coefficients in the tangential direction and radial direction.
3. The shrinkage coefficient in the longitudinal direction is estimated as 0.00005.
4. The shrinkage or swelling of a wood member is estimated as:

   $S = D \times (M_i - M_f) \times c$

   where:
   S = shrinkage or swelling in the dimension being considered (mm)
   D = actual dimension (mm)
   M_i = lesser of the initial moisture content or the fibre saturation point (28%)
   M_f = final moisture content
   c = shrinkage coefficient as given in this table.
5. Use engineering judgement for shrinkage of structural composite lumber (SCL). SCL material typically has much lower initial moisture content than sawn lumber.

Significance of Checking

General

Checking of wood is due to differential shrinkage of the fibres in the inner and outer portions of a member and is associated with rapid lowering of the moisture content at and near the surface of the member. Differential shrinkage causes tensile stresses perpendicular to the grain in the outer portions of the member, which may exceed the fibre strength and result in checking.

If wood is dried slowly enough to permit a very gradual transmission of moisture from the inner core to the outer surfaces, the tensile stress perpendicular to grain in the outer portion will be less than the corresponding fibre strength and no checking should occur. However, some seasoning checks may occur when timbers are installed under conditions of low relative humidity that require rapid adjustment to a much lower equilibrium moisture content.

Minor checks confined to the surface areas of a wood member very rarely have any effect on the strength of the member. Deep checks could be significant if they occur at a point of high shear stress. Checks in columns are not of structural importance, unless the check develops into a through split that will increase the slenderness ratio (C_c) of the column.

Glulam

One of the major advantages of glulam is that the members are free from major checking due to the exclusive use of kiln dried lumber in its fabrication. Glulam members glued within the range of moisture contents set out in CSA O122 (7 to 15%) approximate the moisture content in normal use, thereby minimizing checking.

Rapid changes in moisture content after gluing will result in shrinkage or swelling that might develop stresses in both the glued joint and the wood. In many instances good design minimizes these changes; for example, the outlet registers of a forced hot air heating system should not direct hot air onto members. The use of sealers, paints, etc., retard, but do not prevent moisture changes. Care is required during transit, storage and throughout all stages of building construction to avoid rapid changes in the moisture content of glulam members. Such changes could occur as a result of the sudden application of heat to buildings under construction in cold weather.

Differences in the shrinkage rate of the individual glued laminations tend to concentrate shrinkage stresses at or near the glue lines. For this reason, when checking does occur, it is usually adjacent to glue lines. The presence of wood fibre separation in the laminations of checked members indicates there is adequate glue bond to resist delamination.

Site conditions often require that laminated members be stored on the construction site. They should be stored on edge and protected from rain and sunshine by a loosely fitting cover such as tarpaulin.

If small surface checks are considered to mar the appearance, it is customary to fill them with a suitable wood filler and refinish the surface as required, but only after the inner portion of the member has reached the same moisture content as that of the outer surfaces. Otherwise, the inner portion may be subjected to excessive tensile stresses perpendicular to grain, which could result in further hidden checking inside. It is therefore recommended to delay any filling of surface checks for not less than a year after checking has been observed.

If reasonable care to prevent rapid drying or adequate sealers are not applied, large checks or even splits could occur, which may require structural repair. Such repairs can be undertaken with the guidance of a qualified timber engineer, perhaps by epoxy injection or by mechanical means.

Prior to the 2014 edition of the standard, the connection service-condition factors, K_{SF}, were applied to all connections. In the 2014 edition, brittle failure modes were introduced for bolt and dowel connections. An experimental study conducted by Legras et al. (2010) demonstrated that K_{Sv} and K_{St} for lumber are more appropriate than K_{SF} to determine the resistance of row shear and group tear-out failures in bolted or dowel connections.

The use of tight-fit dowels with large steel plates in service conditions with significant moisture variations should be avoided. Where this condition cannot be avoided, the K_{SF} values for drift pins and lag screws should apply.

Clause 12.2.1.7.3 — Treatment factor, K_T

Some chemical fire retardants have been shown to cause wood strength loss, thus affecting the ductility and strength of connections in wood-based materials. CSA O86 requires that tests be carried out on the effects of fire retardants, or any potentially strength-reducing chemicals used with wood-based materials to determine an appropriate treatment factor. Additional information on the effects of fire-retardant treatment can be found in Clauses 5.3.4, 6.4.3 and 7.4.3 of the Standard and the corresponding Commentary sections.

Clause 12.2.2.3 — Group of fasteners

The total resistance of a connection with a small group of fasteners has been assumed traditionally to be the product of the resistance of a single fastener times the number of fasteners. This assumption of equal loads on each fastener was the basis of early design standards.

However, tests on large, heavily loaded connections have shown that each connection can have a total load capacity significantly lower than that of one fastener times the number of fasteners. A theoretical analysis by Lantos (1969), based on assumed elasticity of timbers end-connected by wood or steel side members under tension, showed that the end fasteners in each row could carry above-average loads whereas the middle fasteners could carry below-average loads. This was confirmed by researchers such as Cramer (1968), and empirically by comparison of calculated values with test results.

Tables 12.2 and 12.3 list factors to be used for determining the resistance of connections made with two or more shear plates, split rings or lag screws. The group effects for other fasteners such as bolts and timber rivets are incorporated in the design equations. Relatively ductile connections made with truss plates, nails or spikes are influenced less by this effect. For these fasteners, the group action factor is not applied.

Clause 12.2.2.4 — Washers

Clause 12.2.2.4 requires washers between wood and the head (or nut) of a bolt, or lag screw, unless a steel strap or plate is used. Standard cut washers are sufficient with bolts and lag screws used alone for lateral loads, but when

G = shear modulus (N/m^2)

A = cross-sectional area of the member (m^2)

ℓ = joist span (m)

For I-joists, most manufacturers provide design value for the shear factor K, where K = 8GA/f_s. Substituting K into the above equation, one obtains the formula
for the I-joist apparent bending stiffness as follows:

$$\frac{I}{EI_{joist}} = \frac{I}{EI_{true}} + \frac{96}{K\ell^2} \qquad \textit{(Eq. 3)}$$

For information on calculating $(EI)_{joist}$ for wood trusses, refer to FPInnovations's Mid-rise Wood-Frame Construction Handbook (Hu and Chui, 2015).

References

Chui, Y. H. 2002. Application of ribbed-plate theory to predict vibrational serviceability of timber floor systems. The Proc. of the 7th World Conf. on Timber Engineering. Shah Alam, Malaysia.

CWC. 1997. Development of design procedures for vibration controlled spans using engineered wood members. Concluding report for Canadian Construction Materials Centre and the Consortium of manufactures of engineered wood products used in repetitive member floor systems. Canadian Wood Council, Ottawa.

Hu, L. J. 1998. Effects of Solutions for Sound Isolation and Fire Resistance on Serviceability of Engineered Wood Floors. Proceedings of the Fifth World Conference on Timber Engineering, Vol. 2, August 17-20, Montreux.

Hu, L. J. 2000. Serviceability Design Criteria for Commercial and Multi-Family Floors. Final Report No. 4 for Canadian Forestry Service, FPInnovations, Sainte-Foy.

Hu, L. J. 2005. Serviceability and Buildability of Engineered Wood Floors. Final Report to Canadian Forest Service Project No. 3632, FPInnovations, Sainte-Foy.

Hu, L. J., Smith, I. and Chui, Y. H. 1994. Vibration analysis of ribbed plates with a rigid intermediate line support. Journal of Sound and Vibration. 178(2), 163-175.

Hu, L. J. and Tardif, Y. 2000. Effectiveness of strongback/wood I-blocking for improving vibration performance of engineered wood-frame floors. Wood Design Focus, Vol. 11, No. 3.

Hu, L. J. and Chui, Y. H. 2006. Performance Benefits Expected with the Use of Continuous Span Joists in Wood-Based Floors. Technote 06-07E. FPInnovations, Sainte-Foy.

Hu, L. J. and Chui, Y. H. 2015. Chapter 4: Floor vibration control. In Midrise Wood-Frame Construction Handbook. Edited by Chun, N. and Popovski, M. First edition, FPInnovations Speical Publication SP-57E, FPInnovataions, Point-Claire, QC.

Hu, L. J., Chui, Y. H. and Cuerrier-Auclair, S. 2018. Chapter 7 – Vibration: Vibration Controlled Designs for Mass Timber Floors and Tall Wood Buildings in CLT Handbook, Second Canadian edition, FPInnovatons, Point Claire, QC.

Khokhar, A. 2004. Influence of lateral element stiffness on performance of wooden floors. MScFE Thesis. University of New Brunswick, Fredericton, N.B.

buildings. For floor vibration induced by rhythmic activities, guidance can be found in the NBC Structural Commentary D, *Deflection and Vibration Criteria for Serviceability and Fatigue Limit States*.

Underpinning the design equation in Clause A.8.5.3 is the findings from Hu (2000) and others that first natural frequency of a floor and static deflection of the floor system under a concentrated load at the centre provide a good indication of the vibration performance of the floor. The performance criterion developed by Hu (2000) based on measured responses in the field is given in Equation (39).

$$\frac{f_1}{d^{0.39}} \geq 15.3 \qquad \text{(Eq. 39)}$$

where:

f_1 = measured first natural frequency of the floor (Hz)

d = measured deflection of the floor at the centre under a point load of 1 kN (mm)

As a general rule, an increase in natural frequency or a decrease in static deflection under a concentrated load leads to an improvement in floor vibration performance. The first natural frequency is a function of the floor system stiffness and mass, while static deflection is a function of floor system stiffness. This explains the inclusion of the two calculation variables of effective bending stiffness, $(EI)_{eff}$, and linear mass, m, in the design equation.

The design equation is based on a structural model of a single-span beam with simply-supported boundary conditions. Given that CLT can be produced in long spans, it is common to design CLT panels to span continuously over more than two supports. Moreover, Hernandez and Chui (2014) has found that the use of extensive fasteners or clamping force applied by the walls above the floor ends, such as that caused by the perimeter walls, internal load-bearing or partition walls, leads to additional end fixity (i.e., the ends are restrained from free rotation). As the applied load over support and number of fasteners increase, the static deflection is decreased and natural frequency is increased. Therefore in CLT buildings, especially multi-storey buildings, the actual support conditions are most likely more favorable than the simply-supported boundary conditions, leading to better floor vibration performance than the idealized simply-supported conditions tested in the laboratory environment.

Flexible support can be caused by inadequate stiffness of any supporting beam. For this reason, to ensure adequate beam stiffness, it is recommended that the mid-span deflection of a floor-supporting beam under a 1 kN load be limited to 0.2 mm. This recommendation is based on limited research conducted by FPInnovations, and will be refined when results from future research is available.

In addition to performance improvement expected from in-service CLT panel support conditions, floor vibration performance can be improved by the presence of non-structural components. Performance-enhancing