applied load. In addition, the lumber members must be covered with minimum 9.5 mm panel sheathing, or 17 mm lumber sheathing that is overlaid with panel sheathing or wood finish flooring. In all cases the sheathing must be fastened to the joists to provide a minimum stiffness equivalent to that provided by 2" common nails at 150 mm centres at the edges of sheathing panels and 300 mm centres elsewhere.

Design

Sawn lumber joists must be designed so that the following criteria are satisfied:

1. Factored moment resistance $M_r \geq$ maximum factored moment M_f.
2. Factored shear resistance $V_r \geq$ maximum factored shear force V_f. In the calculation of maximum shear force, the loads within a distance from the support equal to the depth of the member may be neglected.
3. Factored notch shear resistance for members with end notches on the tension side at supports $F_r \geq$ maximum factored reaction force Q_f.
4. Factored bearing resistance $Q_r \geq$ maximum factored reaction Q_f (refer to Chapter 6).
5. Maximum deflection under specified loads $\leq$ deflection criteria (actual $E_SI \geq$ required E_SI).
6. For joisted floor systems where vibration design is required, vibration-controlled span $\ell_v \geq$ joist span L.

The Joist Selection Tables provide M_r, V_r and E_SI values. In addition, the Serviceability Table (Deflection) provides the required E_SI value for a maximum deflection of L/360 under uniform loading.

For joisted floor systems where a vibration serviceability check is required (vibration from normal walking action of occupants), the Joist Selection Tables provide vibration-controlled spans for sawn lumber joists supporting wood subfloors, with or without a concrete topping. The tables are developed based on the calculation method in O86 A.5.4.5.1.

NBC A-9.23.4.2.(2) provides an alternative vibration design method for floor joists within the scope of Part 9 buildings. Further information on the NBC vibration criterion, including design examples using that procedure, is given in Chapter 11.

MAY 2023 Errata

Wall Panel Selection Tables

E1 35 mm plies

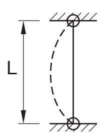
CLT Wall

Factored compressive resistance P_r (kN/m)

Grade E1	Major strength direction				Minor strength direction			
L	3-Ply	5-Ply	7-Ply	9-Ply	3-Ply	5-Ply	7-Ply	9-Ply
m	105 mm	175 mm	245 mm	315 mm	105 mm	175 mm	245 mm	315 mm
2.0	1050	1790	2380	2930		511	844	1120
2.5	851	1650	2270	2810		425	784	1070
3.0	669	1490	2140	2710		343	719	1010
3.5	518	1330	2020	2600		271	652	961
4.0	399	1170	1880	2490		213	582	905
4.5	309	1020	1740	2370		168	515	846
5.0	241	881	1600	2250		132	451	786
5.5		756	1460	2120			393	726
6.0		647	1330	2000			340	666
6.5		553	1200	1870			294	608
7.0		473	1080	1740			254	552
7.5		406	966	1610			220	500
8.0		350	865	1490			191	452
8.5		302	773	1380			166	408
9.0			691	1270				367
Q_r (kN/m)	445	742	1040	1340	445	742	1040	1340

Notes:

1. For unsupported length L ≤ 2.0 m, use P_r for L = 2.0 m.
2. Where P_r values are not given, the slenderness ratio exceeds 43 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term loading (dead plus snow or live due to occupancy) ($K_D = 1.0$)
   - dry service conditions (Wet service conditions not permitted for CLT) ($K_S = 1.0$)
   - no fire-retardant treatment (untreated lumber) ($K_T = 1.0$)
   - effectively held in position at each end ($K_e = 1.0$)
   - concentrically loaded (e = 0).
4. L = unsupported length.



$K_e = 1.0$

MAY 2023 Errata

Wall Panel Selection Tables

E2

35 mm plies

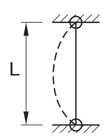
CLT Wall

Factored compressive resistance P_r (kN/m)

Grade E2	Major strength direction				Minor strength direction			
L m	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm
2.0	971	1670	2230	2740		430	691	909
2.5	780	1530	2120	2640		366	647	870
3.0	609	1380	2000	2530		302	600	832
3.5	468	1230	1880	2430		245	551	793
4.0	359	1080	1750	2320		196	499	753
4.5	277	932	1610	2210		157	448	710
5.0	215	801	1480	2090		125	398	666
5.5		684	1340	1970			351	621
6.0		583	1220	1840			308	575
6.5		497	1090	1720			270	531
7.0		424	980	1600			236	487
7.5		363	876	1480			206	446
8.0		312	782	1360			180	407
8.5		269	697	1260			158	370
9.0			622	1150				337
Q_r (kN/m)	588	980	1370	1760	588	980	1370	1760

Notes:

1. For unsupported length L ≤ 2.0 m, use P_r for L = 2.0 m.
2. Where P_r values are not given, the slenderness ratio exceeds 43 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term loading (dead plus snow or live due to occupancy) (K_D = 1.0)
   - dry service conditions (Wet service conditions not permitted for CLT) (K_S = 1.0)
   - no fire-retardant treatment (untreated lumber) (K_T = 1.0)
   - effectively held in position at each end (K_e = 1.0)
   - concentrically loaded (e = 0).
4. L = unsupported length.



K_e = 1.0

Wall Panel Selection Tables

E3

35 mm plies

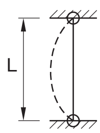
CLT Wall

Factored compressive resistance P_r (kN/m)

Grade E3	Major strength direction				Minor strength direction			
L m	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm
2.0	804	1390	1860	2290		307	493	648
2.5	643	1270	1760	2200		262	462	620
3.0	499	1150	1660	2110		217	429	594
3.5	382	1020	1560	2020		176	394	566
4.0	292	888	1450	1930		141	357	537
4.5	225	767	1340	1830		113	321	507
5.0	175	657	1220	1730		90.7	286	476
5.5		560	1110	1630			253	444
6.0		476	1000	1530			222	412
6.5		405	898	1420			195	381
7.0		345	803	1320			170	350
7.5		295	717	1220			149	321
8.0		253	639	1120			130	293
8.5		218	569	1030			114	267
9.0			507	945				243
Q_r (kN/m)	294	490	686	882	294	490	686	882

Notes:

1. For unsupported length L ≤ 2.0 m, use P_r for L = 2.0 m.
2. Where P_r values are not given, the slenderness ratio exceeds 43 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term loading (dead plus snow or live due to occupancy) (K_D = 1.0)
   - dry service conditions (Wet service conditions not permitted for CLT) (K_S = 1.0)
   - no fire-retardant treatment (untreated lumber) (K_T = 1.0)
   - effectively held in position at each end (K_e = 1.0)
   - concentrically loaded (e = 0).
4. L = unsupported length.



K_e = 1.0

MAY 2023 Errata

Wall Panel Selection Tables

V1

35 mm plies

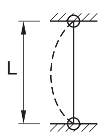
CLT Wall

Factored compressive resistance P_r (kN/m)

Grade V1	Major strength direction				Minor strength direction			
L m	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm
2.0	763	1300	1730	2120		430	691	909
2.5	619	1190	1640	2040		366	647	870
3.0	487	1080	1560	1960		302	600	832
3.5	377	968	1460	1880		245	551	793
4.0	291	852	1370	1800		196	499	753
4.5	225	742	1270	1720		157	448	710
5.0	176	641	1170	1630		125	398	666
5.5		550	1060	1540			351	621
6.0		471	965	1450			308	575
6.5		403	872	1360			270	531
7.0		345	784	1260			236	487
7.5		296	703	1170			206	446
8.0		255	629	1090			180	407
8.5		220	563	1000			158	370
9.0			503	922				337
Q_r (kN/m)	588	980	1370	1760	588	980	1370	1760

Notes:

1. For unsupported length L ≤ 2.0 m, use P_r for L = 2.0 m.
2. Where P_r values are not given, the slenderness ratio exceeds 43 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term loading (dead plus snow or live due to occupancy) (K_D = 1.0)
   - dry service conditions (Wet service conditions not permitted for CLT) (K_S = 1.0)
   - no fire-retardant treatment (untreated lumber) (K_T = 1.0)
   - effectively held in position at each end (K_e = 1.0)
   - concentrically loaded (e = 0).
4. L = unsupported length.



K_e = 1.0

Wall Panel Selection Tables

V2
35 mm plies

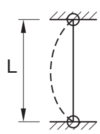
CLT Wall

Factored compressive resistance P_r (kN/m)

Grade V2	Major strength direction				Minor strength direction			
L m	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm	3-Ply 105 mm	5-Ply 175 mm	7-Ply 245 mm	9-Ply 315 mm
2.0	643	1070	1420	1750		511	844	1120
2.5	529	994	1360	1680		425	784	1070
3.0	423	908	1290	1620		343	719	1010
3.5	332	818	1220	1560		271	652	961
4.0	259	728	1140	1500		213	582	905
4.5	202	639	1070	1430		168	515	846
5.0	159	557	987	1360		132	451	786
5.5		482	907	1290			393	726
6.0		416	829	1220			340	666
6.5		358	753	1150			294	608
7.0		308	682	1080			254	552
7.5		266	615	1000			220	500
8.0		230	554	934			191	452
8.5		199	498	867			166	408
9.0			447	802				367
Q_r (kN/m)	445	742	1040	1340	445	742	1040	1340

Notes:

1. For unsupported length L ≤ 2.0 m, use P_r for L = 2.0 m.
2. Where P_r values are not given, the slenderness ratio exceeds 43 (maximum permitted).
3. Tabulated values are valid for the following conditions:
   - standard term loading (dead plus snow or live due to occupancy) (K_D = 1.0)
   - dry service conditions (Wet service conditions not permitted for CLT) (K_S = 1.0)
   - no fire-retardant treatment (untreated lumber) (K_T = 1.0)
   - effectively held in position at each end (K_e = 1.0)
   - concentrically loaded (e = 0).
4. L = unsupported length.



K_e = 1.0

Example 4: Glulam Bottom Chord

Design the bottom chord of a heavy timber truss that has a maximum factored tensile force of 100 kN combined with a factored bending moment of 8 kN•m. The bottom chord length is 8.0 m. The loading is due to a combination of dead plus snow loads. The service conditions are dry. Use 20f-EX grade Spruce-Pine glulam.

Calculation

A review of the checklist for the Tension Member and Beam Selection Tables indicates that the tabulated tension resistances T_{rG} may be used without modification and the bending moment resistances M'_r are required to be multiplied by the lesser of the size factor K_{Zbg} and the lateral stability factor K_L. Therefore select a series of sections and check the interaction equation as follows:

Section (b × d) mm	T_{rN} kN	M'_r kN•m	Lesser of K_L and K_{Zbg}	$M_r = M'_r \times$ (Lesser of K_L and K_{Zbg}) kN•m	$T_f/T_r + M_f/M_r$	Acceptable
80 × 152	139	7.10	1.0	7.10	1.85	No
80 × 190	174	11.1	1.0	11.1	1.30	No
80 × 228	208	16.0	1.0	16.0	0.981	Yes

Assuming the bottom chord is laterally supported at both ends to prevent lateral displacement and rotation, the lateral stability factor K_L = 1.0 for all three trial sections as the member depth-to-width ratio d/b < 4 (O86 Clauses 7.5.6.3.2 and 6.5.3.2.3).

Use 80 × 228 mm 20f-EX grade Spruce-Pine glulam.

5.4 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 $(P_f/P_r) + (M_f/M_{r,f})P\Delta = 1.0$, where $P\Delta$ represents the amplification factor accounting for the P-delta effect as mentioned in Section 5.1. 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 compressive resistance without considering wind load and the resistance to dead plus wind loads should also be verified. Deflection of the wall under specified loads, the shear resistance of the wall along with the bearing resistance of the supporting member should also be checked. The Wall Panel Selection Tables (Combined Loads) include stiffness and resistance values $(EI)_{eff,f,0}$, $(GA)_{eff,f,0}$, $V_{r,f,0}$. In addition, the selection tables also provides the bearing resistance Q_r of the supporting CLT floor panel of the same grade.

5

Combined Loads

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 = 1.15) for the combined load condition?
2. Is the load duration “standard term” (K_D = 1.0) for the bearing of the supporting floor panel?
3. Is the service condition “dry” (K_S = 1.0)? (CLT not permissible in wet service conditions).
4. Is the material free of strength-reducing chemicals (K_T = 1.0)?
5. Is the wall loaded eccentrically less than or equal to half of the wall thickness ($e \leq d/2$)?
6. Is the wall effectively pinned at each end (K_e = 1.0)?
7. 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, 3, 4 or 5 is no, detemine the bending moment and compressive resistances in accordance with Sections 2.9 and 3.5, and recheck the interaction equation. If the answer to questions 6 or 7 is no, calculate the capacity of the panel following the provisions in the CSA O86.

Notes:

1. The tabulated values are taken as the minimum of the yielding resistance, splitting resistance of the side member, and row shear, group tear-out and net tension resistances of the main member.
2. The tabulated values are applicable when the main member is in tension. For main member in compression, the values are conservative when the failure mode is indicated as row shear (for connections with one fastener per row), group tear-out or net tension.
3. The governing failure mode for each connection configuration is denoted by one of the following superscripts:

   (a)-(g), indicating the specific yielding failure mode as per O86 Clause 12.4.4.3.2, or

   "R", "G", "T" or "S", indicating failure due to row shear, group tear-out, net tension or splitting, respectively.

   Cells where row shear, group tear-out or splitting governs are shaded.
4. When the shear resistance calculated using the net section of the side member (O86 Clause 6.5.4.3) is lower than the tabulated value, the shear resistance is displayed as a superscript after the failure mode.
5. The minimum loaded end distance, a_L, is the maximum of 50 mm and 5 times the fastener diameter (O86 Clause 12.4.3.1).
6. For the calculation of splitting resistance of the side member, the minimum unloaded edge distance $e_p = 1.5\ d_F$ is assumed.
7. For the calculation of group tear-out and net tension resistances of the main member, the fastener hole diameter is assumed to be 2 mm larger than the fastener diameter.
8. The lateral resistance P'_r is required to be adjusted to end-use conditions using modification factors depending on the governing failure mode as follows:

   $P_r = P'_r\ (K_D\ K_{SF}\ K_T)$ for yielding failure mode (a), (b) or (f),

   $P_r = P'_r\ (K_{Dy}\ K_{SF}\ K_T)$ for yielding failure mode (d), (e) or (g),

   $P_r = P'_r\ (K_D\ K_{Sv}\ K_T)$ for row shear,

   $P_r = P'_r\ (K_D\ K_{St}\ K_T)$ for group tear-out or net tension,

   $P_r = P'_r\ (K_D\ K_{SF}\ K_T)$ for splitting.

   Modification factors are provided in the Checklist for the Selection Tables.
9. Tabulated values are based on No.1/ No.2 grade for dimension lumber (38 to 102 mm in smaller dimension) and No.2 grade for timbers (114 or more in smaller dimension).
10. Tabulated values are based on ASTM A307 grade steel bolts or dowels.
11. N/A = not applicable.
12. Additional bolt and dowel selection tables are available at www.cwc.ca.

Notes:

1. The tabulated values are taken as the minimum of the yielding resistance, splitting resistance of the side members, and row shear, group tear-out and net tension resistances of the main member.
2. The tabulated values are applicable when the main member is in tension. For main member in compression, the values are conservative when the failure mode is indicated as row shear (for connections with one fastener per row), group tear-out or net tension.
3. The governing failure mode for each connection configuration is denoted by one of the following superscripts:

   (a)-(g), indicating the specific yielding failure mode as per O86 Clause 12.4.4.3.2, or

   "R", "G", "T" or "S", indicating failure due to row shear, group tear-out, net tension or splitting, respectively.

   Cells where row shear, group tear-out or splitting governs are shaded.
4. When the shear resistance calculated as the summation of the side members using net section (O86 Clause 6.5.4.3) is lower than the tabulated value, the shear resistance is displayed as a superscript after the failure mode.
5. The minimum loaded end distance, a_L, is the maximum of 50 mm and 5 times the fastener diameter (O86 Clause 12.4.3.1).
6. For the calculation of splitting resistance of the side members, the minimum unloaded edge distance $e_p = 1.5\ d_F$ is assumed.
7. For the calculation of group tear-out and net tension resistances of the main member, the fastener hole diameter is assumed to be 2 mm larger than the fastener diameter.
8. The lateral resistance P'_r is required to be adjusted to end-use conditions using modification factors depending on the governing failure mode as follows:

   $P_r = P'r\ (K_D\ K_{SF}\ K_T)$ for yielding failure mode (a) or (c),

   $P_r = P'_r\ (K_{Dy}\ K_{SF}\ K_T)$ for yielding failure mode (d) or (g),

   $P_r = P'_r\ (K_D\ K_{Sv}\ K_T)$ for row shear,

   $P_r = P'_r\ (K_D\ K_{St}\ K_T)$ for group tear-out or net tension,

   $P_r = P'_r\ (K_D\ K_{SF}\ K_T)$ for splitting.

   Modification factors are provided in the Checklist for the Selection Tables.
9. Tabulated values are based on No.1/ No.2 grade for dimension lumber (38 to 102 mm in smaller dimension) and No.2 grade for timbers (114 or more in smaller dimension).
10. Tabulated values are based on ASTM A307 grade steel bolts or dowels.
11. N/A = not applicable.
12. Additional bolt and dowel selection tables are available at www.cwc.ca.

Establishment of Design Values for Joist Hangers

Since joist hangers are proprietary, CSA O86 provides a method for determining design values but does not provide the design values in the Standard. Instead it requires that joist hangers be tested in accordance with ASTM Standard D7147, *Standard Specification for Testing and Establishing Allowable Loads of Joist Hangers*. Test results are listed in the *Manual of Evaluation Reports* published by the *Canadian Construction Materials Centre* (CCMC). The evaluation report also includes resistance values derived from the test results.

Design

Design values for joist hangers are published by the manufacturer. CSA O86 gives design procedures and formulae for joist hangers and requires the effect of the factored loads to be less than or equal to the factored resistance of the hanger.

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

$$N_r = \phi\, n_u K'$$

where:

ϕ = 0.6

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

K' is a composite modification factor given below.

Modification Factors

The modification factors adjust the factored resistances for the actual conditions of use. The following factors apply to the design of joist hangers:

$$K' = K_D\, K_{SF}\, K_T$$

where:

K_D = duration of load factor. Note that no increase for short term duration is permitted where the ultimate resistance is governed by the strength of the steel. Check with the manufacturer for appropriate short term K_D values.

K_{SF} = service condition factor. For hangers connected with nails see section 7.2. For hangers connected with bolts see section 7.4.

K_T = treatment factor

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

K_T = treatment factor. The tabulated lateral resistances of diaphragms assume no treatment (K_T = 1.0).

s = fastener spacing evenly distributed along boundary frame members, but not less than 65 mm.

n_L = number of sheathing layers; it is incorporated into the Diaphragm Selection Tables.

= 1 for single-layer of diagonal lumber sheathing

= 2 for double-layer of diagonal lumber sheathing

J_D = factor for diaphragm and shearwall construction; it is incorporated into the Diaphragm Selection Tables.

= 1.3

The tabulated shear resistances for diaphragms sheathed with diagonal lumber sheathing are based on a thickness of 19 mm for each layer and a 65 mm edge nail spacing.

Table 8.2
Fastener row factor, J_f^*, for diaphragms

Number of rows	Minimum thickness of framing member, mm	J_f^*
1	38	1.00
	64[2]	1.12
2	64[2]	2.00
	89[3]	2.25
3	89[3]	3.00

Notes:
1. The fastener row factor, J_f^*, is taken as the values from O86 Table 11.1 divided by 0.89. J_f^* is unity for diaphragms with one row of fasteners and 38 mm thick framing members.
2. Or two 38 mm thick members adequately connected to transfer the factored shear force.
3. Or three 38 mm thick members adequately connected to transfer the factored shear force.

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Checklist: **Diaphragms**

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

1. Is load duration “short term” (K_D = 1.15)?
2. Is the service condition “dry” and are the framing members seasoned prior to fabrication (moisture content ≤ 19%) (K_{SF} and K_S equal 1.0)?
3. Is the material free of strength-reducing chemicals (K_T = 1.0)?
4. Is there one row of fasteners and the thickness of framing member is at least 38 mm (J_f^*)?
5. Is the diaphragm blocked (J_{ud} = 1.0)?

If the answer to any of these questions is no, refer to the design procedure section and adjust the tabulated values. Otherwise, the Diaphragm Selection Tables can be used directly.

Table 8.3
Strength adjustment factor, J_{ud}, for unblocked diaphragms

Configuration case	J_{ud}
1	0.89
2, 3, 4, 5 and 6	0.67

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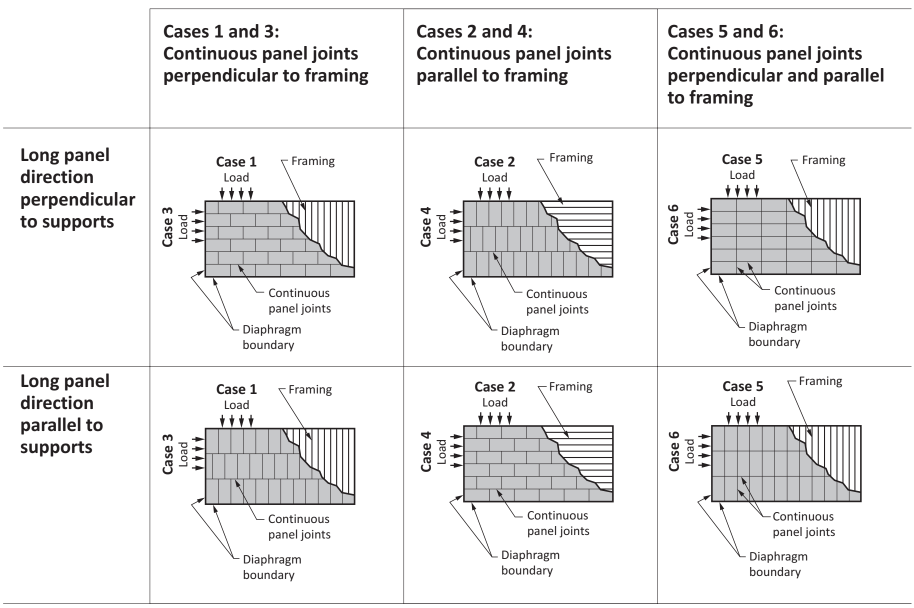
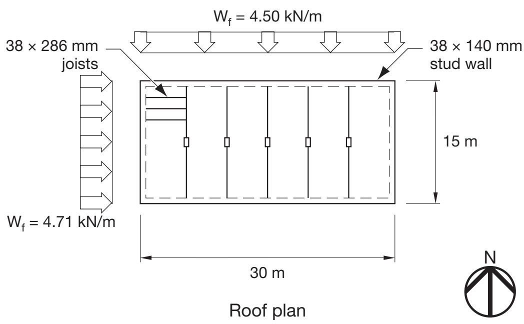


Courtesy of APA – The Engineered Wood Association

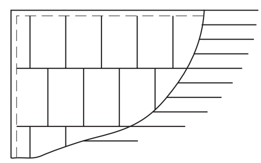
Example 1: Roof Diaphragm

Design the roof diaphragm shown below for the following conditions:

- one storey warehouse framed with S-P-F lumber
- building height = 4 m
- CSP plywood thickness (based on gravity loads) = 12.5 mm
- wind in two directions (the factored wind load is 4.50 kN/m in North-South direction, and 4.71 kN/m in East-West direction)
- dry service conditions
- flexible diaphragm
- the wall double top plate is spliced by 20 gauge steel plates.



Roof plan



Plywood layout

Calculation

Shear force along East and West walls:

$$v_f = \frac{w_f L}{2L_D} = \frac{4.5 \times 30.0}{2 \times 15.0} = 4.50 \text{ kN/m}$$

Shear force along North and South walls:

$$v_f = \frac{w_f L}{2L_D} = \frac{4.71 \times 15.0}{2 \times 30} = 1.18 \text{ kN/m}$$

1. Design of sheathing

From the Diaphragm Selection Tables, try 12.5 mm CSP plywood with 3.25 mm diameter (2-1/2" long) common wire nails, and 38 mm wide S-P-F framing member. Using a blocked diaphragm with one row of nails spaced at 100 mm at the panel edges, the factored shear resistance for both directions is:

v_r = 7.17 kN/m *Acceptable*

Note that 2-1/2" nails spaced at 300 mm on centre along intermediate framing members are also required.

2. Design of chords

The double top plate of the stud wall, made of two 38 x 140 mm lumber, will be used as chord members. By inspection, the bending moment in the North-South direction will govern.

$$M_f = \frac{w_f L^2}{8} = \frac{4.50 \times 30.0^2}{8} = 506 \text{ kN•m}$$

h = 15.0 – 0.140 = 14.9 m

Therefore,

$$T_f = P_f = \frac{M_f}{h} = \frac{506}{14.9} = 34.0\ \text{kN}$$

The tension and compression force is resisted by the two lumber plates. From the Tension Member Selection Tables check 38 × 140 mm No.1/No.2 grade S-P-F sawn lumber for short term loading (K_D = 1.15):

T_r = 34.2 x 1.15 x 2 = 78.6 kN > 34.0 kN *Acceptable*

If splice plates are not used, the joints should be staggered, one top plate should be designed to resist the full induced axial force (tension and compression), and the other top plate should be designed as splice plate.

Since the chord is fully laterally braced along its length by wall studs and sheathing, and by roof joists, rim joists, or blocking, buckling of the chord due to compression is not possible and tension controls the design.

3. Design of splice in tension chord

There are six splice locations spaced evenly at 4.29 m along the 30 m diaphragm span. The chords are spliced with a nailed 20 gauge steel side plate on each side of the double top plate using 3" (3.66 mm in diameter) nails driven from opposite sides of the chords.

From the Nail and Spike Selection Tables, $N'_r\ n_s$ is 0.722 kN.

K_D = 1.15 (short term load duration)

J_D = 1.0 (nails used in chord member, not sheathing)

$K' = K_D\ K_{SF}\ K_T = 1.15$

$J_F = J_E\ J_A\ J_B\ J_D = 1.0$

Resistance per nail is 0.722 x 1.15 = 0.83 kN

The forces in the splices will vary based on the distance from the supports:

Distance from support (m)	Moment at the splice location (kN·m)	Chord Force (kN)	Number of nails required at the splice location
4.29	248	16.6	20 per side of splice; 40 total
8.57	413	27.7	34 per side of splice; 68 total
12.8	496	33.3	40 per side of splice; 80 total

Nails are to be placed symmetrically on each side of the splice and are to be spaced in accordance with Figure 7.4.

4. Shearwall and diaphragm connections

The details shown in Figure 8.4 may be used to transfer shear and lateral forces between the diaphragm and the walls. The spacing of nails or framing anchors must be calculated so that the resistance of the connection exceeds the applied forces.

5. Deflection calculation

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

The specified shear force in the north-south direction governs.

$v = \frac{4.5}{1.4} \times 0.75 = 2.41 \text{kN/m}$ (specified force for serviceability calculation)

$E = 9500$ MPa

$A = 38 \times 140 \times 2 = 10640$ mm^2

$L = 30.0$ m

$L_D = 15.0$ m

$B_v = 5700$ N/mm

The chord splices are located at every 4.29 m along the 30 m diaphragm span. For chord-splice slip, Δ_c is the sum of the nail slip in the wood on both sides of the joint. Assuming the gap at the butt joints in the compression chord exceeds the splice slip, there is identical amount of splice slip in the compression chords as that in the tension chords.

The chord splice slip (Δ_c) is determined using the e_n equation as per O86 Clause A.11.7 and replacing (v×s) with the specified force per nail at the splice and multiplying by 2 to account for slip in nailed splices on each side of the joint.

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)	$\Sigma\Delta_c x$ (mm^2)
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.

MAY 2023 Errata

Stud species: S-P-F

Factored Shear Resistance v_{rd}[1,2] (kN/m) (cont'd)

Common nail size: Length (in.)	Common nail size: Diameter (mm)	Panel type	Panel thickness (mm)	Factored lateral resistance of sheathing-to-framing connection[3,4,5] (kN/m), Nail spacing at panel edges (mm): 150	125	100	75	50**	Panel buckling resistance[6,7] (kN/m)
2-1/2	3.33++	DFP or CSP	9.5	4.50	5.39	6.68	8.50	11.0	8.42
			12.5	4.98	5.97	7.39	9.41	12.2	22.9
			15.5	5.46	6.55	8.11	10.3	13.4	35.9
			18.5	5.94	7.12	8.82	11.2	14.6	52.7
		OSB	9.5	4.50	5.39	6.68	8.50	11.0	10.3
			11	4.74	5.68	7.03	8.96	11.6	15.1
			12	4.90	5.87	7.27	9.26	12.0	18.5
			15	5.38	6.45	7.99	10.2	13.2	29.1
			18	5.86	7.03	8.71	11.1	14.4	50.8
3	3.66*	DFP or CSP	9.5	5.09	6.11	7.57	9.63	12.5	8.42
			12.5	5.60	6.71	8.31	10.6	13.7	22.9
			15.5	6.10	7.32	9.06	11.5	15.0	35.9
			18.5	6.61	7.92	9.81	12.5	16.2	52.7
		OSB	9.5	5.09	6.11	7.57	9.63	12.5	10.3
			11	5.35	6.41	7.94	10.1	13.1	15.1
			12	5.51	6.61	8.19	10.4	13.5	18.5
			15	6.02	7.22	8.94	11.4	14.8	29.1
			18	6.52	7.82	9.69	12.3	16.0	50.8
3	3.76*	DFP or CSP	9.5	5.28	6.33	7.84	9.98	13.0	8.42
			12.5	5.79	6.94	8.60	10.9	14.2	22.9
			15.5	6.30	7.55	9.35	11.9	15.5	35.9
			18.5	6.81	8.16	10.1	12.9	16.7	52.7
		OSB	9.5	5.28	6.33	7.84	9.98	13.0	10.3
			11	5.53	6.64	8.22	10.5	13.6	15.1
			12	5.70	6.84	8.47	10.8	14.0	18.5
			15	6.21	7.45	9.23	11.7	15.2	29.1
			18	6.72	8.06	9.98	12.7	16.5	50.8

Notes: see pages 606-607.

Stud species: Northern

Factored Shear Resistance v_{rd}[1,2] (kN/m) (cont'd)

Common nail size		Panel type	Panel thickness (mm)	Factored lateral resistance of sheathing-to-framing connection[3,4,5] (kN/m) Nail spacing at panel edges (mm)					Panel buckling resistance[6,7] (kN/m)
Length (in.)	Diameter (mm)			150	125	100	75	50**	
2-1/2	3.33++	DFP or CSP	9.5	4.20	5.04	6.24	7.94	10.3	8.42
			12.5	4.68	5.61	6.95	8.85	11.5	22.9
			15.5	5.16	6.19	7.67	9.76	12.7	35.9
			18.5	5.34	6.41	7.93	10.1	13.1	52.7
		OSB	9.5	4.20	5.04	6.24	7.94	10.3	10.3
			11	4.44	5.32	6.59	8.40	10.9	15.1
			12	4.60	5.52	6.83	8.70	11.3	18.5
			15	5.08	6.10	7.55	9.61	12.5	29.1
			18	5.34	6.41	7.93	10.1	13.1	50.8
3	3.66*	DFP or CSP	9.5	4.75	5.70	7.05	8.98	11.7	8.42
			12.5	5.25	6.30	7.80	9.94	12.9	22.9
			15.5	5.76	6.91	8.55	10.9	14.1	35.9
			18.5	6.26	7.51	9.30	11.8	15.4	52.7
		OSB	9.5	4.75	5.70	7.05	8.98	11.7	10.3
			11	5.00	6.00	7.43	9.46	12.3	15.1
			12	5.17	6.20	7.68	9.78	12.7	18.5
			15	5.67	6.81	8.43	10.7	13.9	29.1
			18	6.18	7.41	9.18	11.7	15.2	50.8
3	3.76*	DFP or CSP	9.5	4.92	5.90	7.31	9.30	12.1	8.42
			12.5	5.43	6.51	8.06	10.3	13.3	22.9
			15.5	5.94	7.12	8.82	11.2	14.6	35.9
			18.5	6.45	7.73	9.58	12.2	15.8	52.7
		OSB	9.5	4.92	5.90	7.31	9.30	12.1	10.3
			11	5.17	6.21	7.68	9.79	12.7	15.1
			12	5.34	6.41	7.94	10.1	13.1	18.5
			15	5.85	7.02	8.69	11.1	14.4	29.1
			18	6.36	7.63	9.45	12.0	15.6	50.8

Notes: see page 606-607.

MAY 2023 Errata

Diaphragm Selection Tables

Diagonal Lumber Sheathing

Factored Shear Resistance v_{rd}[1,2,3] (kN/m)

Stud species	Common nail size Length (in.)	Common nail size Diameter (mm)	Side Member Thickness (mm)	Number of diagonal layers	Side Member Species D.Fir-L	Hem-Fir	S-P-F	Northern
D.Fir-L	2.5	3.25	19	1	5.97	5.75	5.46	4.92
	2.5	3.33	19	1	6.19	5.96	5.66	5.10
	2.5	3.25	19	2	11.9	11.5	10.9	9.83
	2.5	3.33	19	2	12.4	11.9	11.3	10.2
Hem-Fir	2.5	3.25	19	1	5.88	5.67	5.38	4.85
	2.5	3.33	19	1	6.09	5.88	5.58	5.03
	2.5	3.25	19	2	11.8	11.3	10.8	9.70
	2.5	3.33	19	2	12.2	11.8	11.2	10.1
S-P-F	2.5	3.25	19	1	5.75	5.54	5.26	4.75
	2.5	3.33	19	1	5.96	5.74	5.46	4.93
	2.5	3.25	19	2	11.5	11.1	10.5	9.50
	2.5	3.33	19	2	11.9	11.5	10.9	9.85
Northern	2.5	3.25	19	1	5.47	5.28	5.01	4.53
	2.5	3.33	19	1	5.66	5.47	5.19	4.70
	2.5	3.25	19	2	10.9	10.6	10.0	9.06
	2.5	3.33	19	2	11.3	10.9	10.4	9.40

Notes:

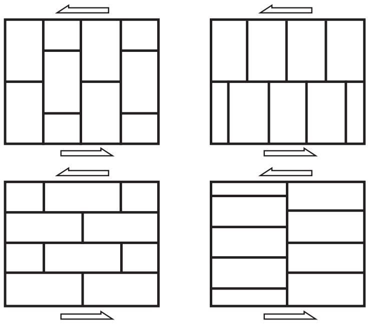
1. Factored lateral shear resistance determined based on boundary framing nail spacing of 65 mm.
2. The tabulated lateral resistances can be used for wind and seismic design. For seismic design, all lateral resistances are governed by sheathing-to-framing connections yielding in ductile mode (d) as defined in O86 Clause 12.9.3.2.
3. Single-layer diagonal lumber consists of 19 mm boards laid at an angle of 45° to the supports. Boards fastened to each intermediate member with two common nails for 19 x 140 mm boards and three common nails for 19 x 184 mm or wider boards. One additional nail is required in each board at the boundary framing member.
4. End joints in adjacent rows of boards are to be staggered by at least one joist space. Joints on the same support are to be separated by at least two rows of boards.
5. Double-layer diagonal sheathing should meet the requirements of notes 2 and 3 and consist of two 19 mm thick layers of diagonal lumber at 90° to each other and on the same face of the supporting members. The nails used for fastening the outer layer must penetrate the framing member by a minimum of $8d_F$ and nails in the inner and outer layers must be staggered along the framing members.
6. For diaphragms constructed with a single layer of 38 mm thick diagonal sheathing using common nails (d=4.06 mm), the design shear resistance for 19 mm boards fastened with common nails (d_F = 3.25 mm or d_F = 3.33 mm) may be used if there are no splices in adjacent boards on the same support and the supports are not less than 89 mm in depth or 64 mm in thickness.

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 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.

A shearwall is designed to resist the sum of factored shear forces from upper storey shearwalls and diaphragms. Where materials or construction details vary between shearwall segments, the resistance of the shearwall may be conservatively estimated based on the minimum of all segments capacity. Alternatively, the total factored shear force on the shearwall, V_{fs}, may be distributed to each of the segments. Different methods are used to distribute the shear force on a shearline to each of the shearwall segments. The distribution of lateral forces to the shearwall segments may be based on the relative resistance or relative stiffness of the segments. See the information on Lateral Force Distribution to Shearwall Segments later in this section.

The shearwall sheathing and the sheathing-to-framing connections must be designed so that the factored shear resistance of the shearwall segment per unit length, v_{rs}, is equal to or greater than the maximum shear force in the shearwall segment per unit length, v_{fs}. The maximum shear force is assumed to be uniformly distributed to the full length of the shearwall segment parallel to the applied load, L_s, and $v_{fs} = \frac{V_{fs}}{L_s}$.

Resistance of shearwalls may vary with load direction and shearwalls must be designed for lateral loads acting in opposite load directions. An example of this situation is provided in Example 1 in this section. The resistance of the shearwall in each direction must be greater than the corresponding lateral force.

Shearwall Segments without Hold-downs

Traditionally, shearwalls have been designed using chords and hold-downs at the ends of all shearwall segments. Hold-downs are designed to transfer the chord segment overturning force, T_f, to the shearwall or foundation below. In addition, CSA O86 includes provisions for design of shearwall segments without hold-downs.

Without hold-downs, the overturning tension force is transferred from the top wall plate to the bottom wall plate through the shearwall sheathing. Since a portion of the shearwall sheathing and the sheathing-to-framing connections are used to resist the overturning force while resisting lateral force, the shear resistance of the sheathing is reduced. Even though hold-down anchors are not used, anchorage is still required to transfer the uplift force from the wall bottom plate to the foundation or shearwall below (see Figure 8.8 and Clause 11.4.4 of the O86 Commentary).

In some cases, hold-downs may be placed at one end of a shearwall segment. Where the lateral force is acting at the end of the segment that contains the hold-down, overturning uplift force will be resisted by the hold-down. When the lateral force is acting at the direction not equipped with hold-down, the sheathing resists overturning uplift force. The shear resistance of the segment is multiplied by the hold-down-effect factor, J_{hd}, which is required to be calculated for loads acting in opposite directions.

Hold-downs are not required for shearwall segments where the basic resistance is adjusted by the J_{hd} factor, and the following conditions are met:

- the sum of the specified basic shear resistances on both sides of the shearwall is less than 10.3 kN/m;

e_n = sheathing-to-framing connection deformation (mm) (see e_n equation in CSA O86 A.11.7)

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

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

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

Note: This equation is an approximation.

where:

d_F = fastener diameter (mm)

K_m = service-creep factor (Table A.23 of CSA O86)

P_{ij} = specified uplift restraint force for storey i at the bottom of the end stud of segment j (N)

s_n = sheathing fastener spacing along panel edge (mm)

Deflection of Multi-Storey Stacked Shearwalls

Since the 2014 edition of CSA O86 it is specified that for the calculation of deflection of multi-storey buildings the multi-storey effects shall be considered. Clause A.11.7.1 of CSA O86 gives one methodology of calculating the deflection of multi-storey shearwalls. It is a mechanics-based approach which is appropriate for light-frame shearwalls with wood-based panels, assumed to be cantilevered from its base and stacked for the full height of the building. This methodology expands on the concept of deflection of single storey shearwall, including contributions from bending, panel shear, nail slip and vertical elongation of the wall anchorage system. The multi-storey equation takes into account the cumulative rotational effects from the storeys below.

Design Procedure

The factored shear resistance, V_r, of shearwalls sheathed with wood structural panels, gypsum wallboard, or diagonal lumber sheathing, shall be taken as follows:

$$V_r = \Sigma V_{rs} = \Sigma(v_{rs} \bullet L_s)$$

where:

V_{rs} = factored shear resistance for each segment along the length of the shearwall (kN)

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

Shearwall Selection Tables

Diagonal Lumber Sheathing

Factored Shear Resistance v_{rs} (kN/m)

Stud species	Common nail size		Side Member Thickness (mm)	Number of diagonal layers	Side Member Species			
	Length (in.)	Diameter (mm)			**D.Fir-L**	**Hem-Fir**	**S-P-F**	**Northern**
D.Fir-L	2.5	3.25	19	1	5.97	5.75	5.46	4.92
	2.5	3.33	19	1	6.19	5.96	5.66	5.10
	2.5	3.25	19	2	11.9	11.5	10.9	9.83
	2.5	3.33	19	2	12.4	11.9	11.3	10.2
Hem-Fir	2.5	3.25	19	1	5.88	5.67	5.38	4.85
	2.5	3.33	19	1	6.09	5.88	5.58	5.03
	2.5	3.25	19	2	11.8	11.3	10.8	9.70
	2.5	3.33	19	2	12.2	11.8	11.2	10.1
S-P-F	2.5	3.25	19	1	5.75	5.54	5.26	4.75
	2.5	3.33	19	1	5.96	5.74	5.46	4.93
	2.5	3.25	19	2	11.5	11.1	10.5	9.50
	2.5	3.33	19	2	11.9	11.5	10.9	9.85
Northern	2.5	3.25	19	1	5.47	5.28	5.01	4.53
	2.5	3.33	19	1	5.66	5.47	5.19	4.70
	2.5	3.25	19	2	10.9	10.6	10.0	9.06
	2.5	3.33	19	2	11.3	10.9	10.4	9.40

Notes:

1. Factored lateral shear resistance determined based on boundary framing nail spacing of 65 mm.
2. The tabulated lateral resistances can be used for wind and seismic design. For seismic design, all of the sheathing-to-framing connection resistances are governed by ductile yield mode (d) as defined in O86 Clause 12.9.3.2.
3. Single-layer diagonal lumber consists of 19 mm boards laid at an angle of 45° to the supports. Boards fastened to each intermediate member with two common nails for 19 x 140 mm boards and three common nails for 19 x 184 mm or wider boards. One additional nail is required in each board at the boundary framing member.
4. End joints in adjacent rows of boards are to be staggered by at least one joist space. Joints on the same support are to be separated by at least two rows of boards.
5. Double-layer diagonal sheathing are to meet the requirements of notes 2 and 3 and consist of two 19 mm thick layers of diagonal lumber at 90° to each other and on the same face of the supporting members. The nails used for fastening the outer layer must penetrate the framing member by a minimum of $8d_F$ and nails in the inner and outer layers must be staggered along the framing members.
6. For shearwalls constructed with a single layer of 38 mm thick diagonal sheathing using common nails (d=4.06 mm), the design shear resistance for 19 mm boards fastened with common nails (d_F = 3.25 mm or d_F = 3.33 mm) may be used if there are no splices in adjacent boards on the same support and the supports are not less than 89 mm in depth or 64 mm in thickness.

Clause 10.6 — Stressed skin panels

Clause 10.6.1 — General

Floor and roof constructions consisting of longitudinal ribs of lumber with plywood or OSB glued to the top and bottom of the lumber are referred to as stressed skin panels. These panels are usually 1.22 m (4') wide, identical to the width of the typical plywood and OSB panel. Ribs commonly are of standard lumber sizes and can be spaced to suit the requirements of the design. Ribs and flanges form an integral part of the cross section.

The engineering theory of bending typically assumes that plane sections remain plane after bending. This is not the case for stressed skin panels. When subjected to bending moment, the panel between the ribs will deform. As a result, at some distance away from the ribs, the panel will be stressed less than the longitudinal ribs. This phenomenon is known as shear lag and was investigated in detail by Foschi (1969b). At first, shear lag was considered in design by calculating an effective width as a function of the panel thickness. Foschi (1970) showed that the ratio of rib height to rib spacing, and rib height to panel length, had a far greater effect on shear lag than did plywood thickness. The 1976 edition of CSA O86 therefore introduced a geometry modification factor for bending.

More detailed methods of design of stressed skin panels have also been proposed by Mazur (1968), Kuenzi (1976) and Smith (1979), and reviewed by Booth (1976).

Clause 10.6.2 — Effective stiffness

Effective stiffness is calculated to account for asymmetry in the panel construction.

Clauses 10.6.3.1 to 10.6.3.7 — Bending and shear resistance

To determine the moment carrying capacity of a stressed skin panel, three resistances have to be considered separately: that of the tension flange, the compression flange and the web. The resistance is calculated based on the principles of engineering mechanics and then multiplied by the geometry reduction factor X_G to account for the effect of shear lag.

The compression flange also has to be checked to ensure that no buckling will take place. This phenomenon has been studied by Foschi (1969a). For a uniformly distributed load, Clause A.10.6.3.4 in Annex A includes a buckling equation based on Foschi's work. Since there was no information available regarding a buckling coefficient for OSB, Clause 10.6.3.4 was rewritten to avoid buckling of the compression flange, while the existing clause was moved to the Annex A and restricted to plywood skins.

To determine the shear resistance of a stressed skin panel, three locations have to be checked: the web at the neutral axis of the panel, the attachment of the flange to the web on the compression side and on the tension side. In these calculations, corresponding (matched) values of Q_w, B_a and y must be used. The factored shear resistance based on the shear strength of the lumber is calculated as in Clause 6.

When calculating the shear resistance of the glued attachment between the web and flanges, both the shear strength of the web and that of both flanges have to be considered together with their appropriate resistance factors. The values of y_t or y_c corresponding to the value of B_a must be used. The Standard assumes that the bond between the web and the flanges is adequate and that failure will occur in either the web or the flange but not in the adhesive holding the two together.

As a result of shear lag, the actual shear at the flange-web interface is less than would be calculated using the engineering theory of bending (Foschi, 1969b). CSA O86 therefore allows an increase in the factored shear resistance.

This increase is a function of the contact area between the web and flanges and the clear spacing of the ribs. In the case of lumber shear strength at the flange-web interface, the shear-modification factor has been set to 2. This last increase is not based on any consideration of joint geometry, but simply reflects the fact that shear stresses published for lumber are greatly reduced from test values to allow for possible checking at the neutral axis of lumber.

The strength modification factors for duration of load K_D, service conditions K_S and treatment K_T are the same as those for plywood, OSB or lumber respectively.

Other applicable modification factors are the stress joint factor X_J, the panel geometry reduction factor X_G and the shear modification factor X_V, discussed in Clause 10.5.

Clause 10.6.3.8 — Deflection

Shear lag in stressed skin panels reduces panel stiffness (Foschi, 1969a and b). The panel geometry reduction factor X_G accounts for this effect, resulting in a small increase in the calculated deflection.

One way to increase the capacity of diaphragms is to increase the number of fasteners per unit length. This can be achieved through the use of multiple rows of fasteners to prevent lumber from splitting. APA – *The Engineered Wood Association* has carried out test program to study the performance of diaphragms with multiple rows of fasteners. The test results in the APA Research Report 138 (Tissell and Elliott, 2000) showed that the lateral load capacity of the diaphragm can be increased if multiple rows of fasteners are used along the sheathing perimeter. Based on the test results, design values of diaphragms with multiple rows of fasteners have been included in the *International Building Code* (IBC) since 2003.

To ensure the performance of diaphragms with multiple rows of fasteners, the fasteners are to be placed along all the edges of each panel not less than 64 mm on centre. The minimum distance is 9 mm between fastener rows and between the outside row and the panel edge. The fasteners between rows are required to be staggered. These requirements, together with the 64 mm minimum width and thickness requirement for framing members in Clause 11.3.2.1, are intended to limit splitting of wood members.

A gap of not less than 2 mm is required to be left between adjoining panel edges to allow for dimensional changes due to moisture in both framing members and the wood structural panels.

Clause 11.3.2.4 – Additional construction requirements

Splitting of the bottom plate of the shearwalls has been observed in tests as well as in structures subjected to earthquakes. Splitting of plates can be caused by the rotation of individual sheathing panels inducing upward forces through the nails at one end of the panel and downward forces at the other end. With the upward forces from the nails and a downward force from the anchor bolt, the bottom plate acts like a cantilever beam and cross-grain bending stresses are introduced. Splitting can be prevented by use of large, sufficiently stiff plate washers.

To prevent stud splitting (particularly where two panels meet), it is required that for nail spacing at 50 mm, or 75 mm with 3.66 mm nails, or 150 mm with panels applied on both faces of a wall, framing members consist of at least 64 mm thick lumber or two 38 mm framing members properly connected to transfer the factored shear force, with staggered fasteners.

Clause 11.3.3 – Shearwalls using gypsum wallboard

While the lateral load resistance of gypsum wallboard was not recognized in early CSA O86 provisions, some building codes have long recognized the use of gypsum wallboard to resist lateral loads induced by earthquake or wind. Where gypsum wallboard is used in combination with wood structural panels in the same shearwall or parallel shearwalls, a ductility-related seismic-force-modification factor, R_d, equal to 2 for the entire building was found to be appropriate for designing light-frame wood residential buildings (Ceccotti and Karacabeyli, 2002).

differentiated between DFP, CSP or OSB sheathing. It is possible for the sheathing in shearwalls and diaphragms to buckle before the sheathing-to-framing connections reach their capacities. The panel buckling resistance is calculated using a model proposed by Dekker et al. (1978) and based on the assumption that the panel is simply supported on four edges and subjected to a uniform shear stress along the panel edges. This simplification ignores the contribution of the intermediate stud support and is therefore conservative.

Clause 11.6.2.3 Shearwall segments with gypsum wallboard sheathing

In Table 11.4 of CSA O86, specified shear strength values are provided for gypsum wallboard shearwalls with nail spacing 200 mm, 150 mm and 100 mm on the panel edges. The use of those design values is restricted to Type X (fire rated) wallboard conforming to ASTM Standard C1396/C1396M, and also to platform frame construction where the height of a storey does not exceed 3.6 m.

Clause 11.6.2.4 — Shearwall segments with diagonal lumber sheathing

Diagonal lumber-sheathed shearwalls represent an older type of construction, with design provisions based originally on monotonic racking tests. These provisions were modified in 2009 based on more recent cyclic shearwall research by Ni and Karacabeyli (2007) at Forintek Canada Corp. (now FPInnovations). The specified shear strength of shearwalls with double-layer diagonal sheathing was reduced from three times to two times the shear strength of single-layer shearwalls, to reflect the results of testing and analysis.

In 2009 edition of the standard, additional guidance was provided on how to address forces at boundary members and at corners of boundary elements in single-layer lumber shearwalls and diaphragms. Loading on single-layer shearwalls and diaphragms creates additional forces that need to be addressed in design. These forces occur in the boundary members parallel to the interior framing members (i.e., studs in shearwalls or joists in diaphragms). The normal load component, acting perpendicular to the boundary member, creates a moment, M_f, that must be accounted for in design of the boundary member. The boundary load component also causes the corners of the shearwall or diaphragm to separate. The boundary frame members in single-layer diagonally sheathed shearwalls and diaphragms shall be designed to resist the separation force P_c.

In the 2017 update to the 2014 edition of the standard, the diagonal lumber provisions were revised to address the mechanics-based approach introduced in 2014. Of greatest significance, the specified shear strengths for shearwalls and diaphragms of 8 kN/m for a single-layer of diagonally sheathed lumber and 16 kN/m for a double-layer were removed, and a new equation for calculating the specified shear strength, v_d, was introduced based on the lateral strength resistance of the sheathing-to-framing connection along the boundary edges. The calculation of the specified shear strength, v_d, was

Symbol	Meaning
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_{90}	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

Symbol	Meaning
K'	Composite Modification Factor
K_B	Length of Bearing Factor
K_C	Slenderness Factor
K_{creep}	Creep Adjustment Factor
K_D	Load Duration Factor
K_{Dy}	Load Duration Factor for Yielding Resistance of Bolts or Dowels
K_e	Effective Length Factor
K_{fi}	Adjustment Factor for Fire Resistance
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