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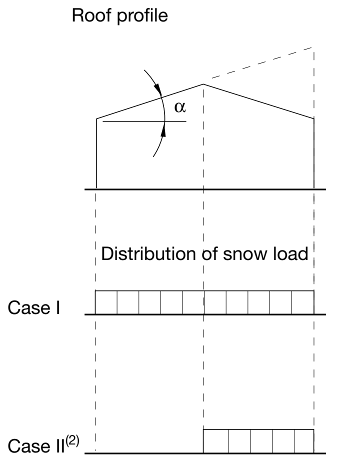
Full and Partial Loading

All roofs must be designed for the full snow load over the entire roof area. In addition, roofs must be designed for combinations of full and partial loading. Flat, shed or gable roofs with slopes less than 15° and arched or curved roofs with rise to run ratios of 1/10 or less must be designed for a combination of full and partial loading. The roof must be designed for full load, with $C_a = 1.0$ on a portion of the roof and half this load on the remainder of the roof. Roof slopes other than these must be designed for partial loadings as described in NBCC-SC. Figure 3.2 describes one of these situations.

Any roof which is able to retain water must be designed for the ponding load that results from a 24 hour rainfall on the horizontal projected area of the roof. This requirement applies whether or not the surface is provided with drainage such as rainwater leaders.

FIGURE 3.2

Snow distributions and loading factors for gable, flat, and shed roofs



Load case	Roof slope, α	Factors C_w	C_s	C_a
I	0° to 90°	1.0[3]	f(α)[1]	1.0
II	15° to 20°	1.0	f(α)[1]	0.25 + α/20
	20° to 90°	1.0	f(α)[1]	1.25

Notes:

1. Varies as a function of slope α as defined in the NBCC.
2. Case II loading does not apply to gable roofs with slopes of 15° or less or to single-sloped (shed) roofs or to flat roofs.
3. For Low and Normal Importance category buildings, C_w may be reduced to 0.75 or, in exposed areas north of the treeline, 0.5.

WOOD I-JOISTS

Wood I-joists are made by gluing solid sawn lumber (usually MSR) or LVL flanges to a plywood or oriented strandboard web using a waterproof adhesive to produce a dimensionally stable lightweight member with known engineering properties (Figures 6.16 and 6.17). Because high strength material that can be spliced into long lengths is used, wood I-joists are capable of spanning further than conventional sawn wood joists. They are also dimensionally stable since the materials are dried prior to manufacture.

Wood I-joists are available in a number of standard sizes (Table 6.12) and in lengths up to 20 m. The I shape of these products gives a high strength to weight ratio. For example, wood I-joists 241 mm deep and 8 m long weigh between 23 and 32 kg (depending on the flange size) which means that they can be installed manually. Most suppliers also stock standard joist hangers and other prefabricated connection hardware specially designed for use with wood I-joists.

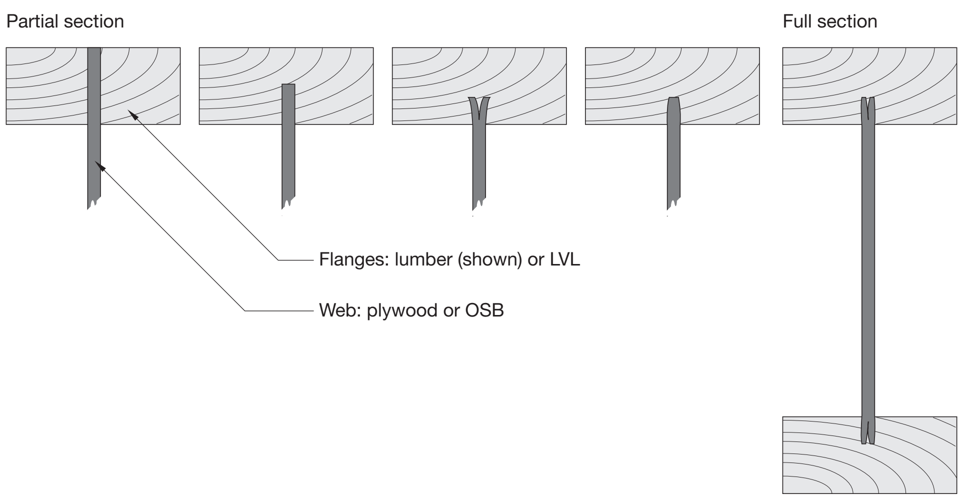
Wood I-joists are proprietary products. Each manufacturer uses a different combination of web and flange materials and a different connection between the web and the flanges.

As a result, each manufacturer produces a joist with unique strength and stiffness characteristics. Design values for I-joists are derived using the procedures in Clause 14 of CSA O86. I-joist manufacturers register their products with the Canadian Centre for Materials in Construction (CCMC). Sample EI values are shown in Table 6.13.

Manufacturers' literature contains allowable load tables and span tables similar to those found in the NBCC for lumber joists. Their literature also contains recommended installation procedures that may differ from solid sawn joists. Chapter 2 of the Wood Design Manual contains further information on installation of I-joists. Most suppliers will provide layout drawings for a particular job showing the required size and location of joists and installation details.

FIGURE 6.16

Wood I-joist configurations



Tongue and groove (T&G) plywood is often used for floor sheathing. It has a factory machined tongue along one of the long edges and a groove along the other, which when interlocked, eliminates the need for blocking the edges from below.

All structural plywood products are marked with a grade stamp that indicates the CSA Standard to which it is produced (Figure 6.20).

Plywood

Principal Applications	Roof Sheathing, Floor Sheathing, Wall Sheathing, Sheathing in Preserved Wood Foundations
Applicable Manufacturing Standards	CSA O121 *Douglas Fir Plywood* CSA O151 *Canadian Softwood Plywood*
Applicable Service Conditions	Dry or Wet
Treatability With Wood Preservatives	Treatable
Applicable Fastenings	Nails, screws

Oriented Strandboard (OSB) and Waferboard

OSB and waferboard are structural panels made from poplar wafers that are laminated together with a waterproof phenolic adhesive (Figure 6.21).

The wafers from which waferboard is manufactured are randomly oriented making the strength properties along both the width and length identical.

Oriented strandboard is similar to waferboard, but the wafers are narrower and oriented in the long direction of the panel in the outer layers. This gives the panel added strength and stiffness in the long direction.

Oriented Strandboard (OSB)

Principal Applications	Roof Sheathing, Floor Sheathing, Wall Sheathing
Applicable Manufacturing Standards	CSA O437 *OSB and Waferboard* CAN/CSA O325 *Construction Sheathing*
Applicable Service Conditions	Dry
Treatability With Wood Preservatives	Not Recommended
Applicable Fastenings	Nails, screws

OSB is by far the most common type of structural panel made from wafers. The manufacture of waferboard (panels with randomly distributed wafers) is on the decline.

The general product standard is CSA O437, *OSB and Waferboard*. The product standard contains three designations O-1 and O-2 indicate an oriented panel (OSB) (Figure 6.22), while R-1 indicates a random panel (waferboard).

Another standard that applies to OSB and waferboard, as well as to plywood, is a performance standard: CSA O325, *Construction Sheathing*. This standard sets performance ratings for specific end uses such as floor, roof and wall sheathing in light-frame construction. For example, a panel marked with a 1R24 indicates roof sheathing on supports spaced 24 inches on centre and without support on the long edges of the panel. A typical example of panel marking on construction sheathing is shown in Figure 6.23.

Sheathing conforming to CSA O325 is referenced in Part 9 of the NBCC. In addition, design values for construction sheathing OSB conforming to CSA O325 are listed in CSA O86 allowing engineering design of roof sheathing, wall sheathing and floor sheathing using CSA O325 rated OSB.

8.1 Compression Members

INTRODUCTION

The load carrying capacity of an axially loaded wood compression member depends on both the compression strength of the wood and on stability. Column stability in turn depends on stiffness which is affected by the slenderness of the member.

For wood columns, which are usually rectangular in shape, the slenderness ratio shall be calculated for both axes. The measure of slenderness, C_c, is the ratio of the effective length to the associated dimension. The effective length is used to account for conditions of end restraint considering the possible buckled shape for a given column.

As slenderness increases, load carrying capacity decreases (as a cubic function of slenderness) so that a short column can support more load than a long column of the same cross section, grade and species. The size effect on material strength also affects columns and is taken into account separately from slenderness by using the factor K_{Zc}.

SAWN LUMBER COLUMNS

For sawn lumber, the slenderness ratio, C_c, is restricted to a maximum value of 50 (at which sawn lumber columns may have only about 10 to 25% of the capacity they have at a slenderness ratio of about 5).

The slenderness factor, K_C, is used to relate slenderness to load capacity for lumber columns. The formulation used for K_C in CSA O86 is one based on a cubic Rankine-Gordon expression. The reliability assessments leading to selecting the performance factor, ϕ, for lumber columns incorporated a nominal load eccentricity of 5% of the member width, on average.

To design a sawn lumber column to support a given factored load, determine the effective length, select a member size, grade and species group and calculate the factored compressive resistance parallel to grain for both areas. If the resistance is less than the load, select another size and recalculate.

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

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

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

where

ϕ = 0.8

$F_c = f_c\,(K_D\, K_H\, K_{SC}\, K_T)$

f_c = specified strength in compression parallel to grain, MPa (CSA O86 Tables 5.3.1A, 5.3.2B, 5.3.1C, 5.3.1D, 5.3.2 and 5.3.3)

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

K_{Zc} = size factor

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

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

K_C = slenderness factor

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

for buckling in direction of d

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

for buckling in direction of b

Where

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

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

K_e = effective length factor given in CSA O86 Table A5.5.6.1

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

= as specified in Tables 5.3.1A to 5.3.1D for visually graded lumber

= 0.82 E for MSR lumber

= 0.75 E for MEL lumber

38 × 184 mm sloping rafters are untreated and belong to the Spruce-Pine-Fir group. They will be seasoned and the service conditions will be dry.

Design the heel joint between the 38 × 184 mm and two 38 × 235 mm members using nails.

Heel Connection

Try 3 in (76 mm) long common round wire nails

diameter = 3.66 mm

Required penetration in the head side member is 3 d_F.

$$3 \times 3.66 = 11 \text{ mm} < 38 \text{ mm}$$

Required penetration in the point side member is 5 d_F.

$$5 \times 3.66 = 18 \text{ mm} < 38 \text{ mm}$$

Design for lateral strength resistance

Factored tensile force, N_f

$$N_f = 2500 \text{ N}$$

Factored resistance, N_r

$$N_r = \phi N_u n_F n_s J_F$$

where

ϕ = 0.8

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

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

n_F = number of fasteners in the connection

n_s = 1 (nails are in single shear)

$J_F = J_E J_A J_B J_D$

J_E = 1.0 (nails are driven into side grain)

J_A = 1.0 (nails are not toe nailed)

J_B = 1.0 (nails are not clinched)

J_D = 1.0 (not used in diaphragm construction)

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

$= 1.0$

Unit Lateral Strength Resistance

The penetration depth of the nail is 76 mm (3 inch), therefore this is a two member wood to wood connection.

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

where

t_1 = 38 mm

d_F = 3.66 mm

$f_2 = 50G(1 - 0.01d_F)$

$= 50 \times 0.42 \times (1 - 0.01 \times 3.66)$

$= 20$ MPa

G = 0.42 (CSA O86, Table A. 10.1)

t_2 = 38 mm

$f_3 = 110G^{1.8}(1 - 0.01d_F)$

$= 110 \times 0.42^{1.8} \times (1 - 0.01 \times 3.66)$

$= 22$ MPa

$f_y = 50(16 - d_F)$

$= 50 \times (16 - 3.66)$

$= 617$ MPa

For lumber side plates:

$f_1 = 50G(1 - 0.01d_F)$

$= 20$ MPa

(a) $f_1 d_F t_1$

$= 20 \times 3.66 \times 38$

$= 2814$ N

(b) $f_2 d_F t_2$

$= 20 \times 3.66 \times 38$

$= 2814$ N

(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1 + f_3)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F}\right)$

$= 20 \times 3.66^2 \times$

$$\left(\sqrt{\frac{1}{6} \times \frac{22}{(20 + 22)} \times \frac{617}{20}} + \frac{1}{5} \times \frac{38}{3.66}\right)$$

$= 996$ N

(e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}}+\frac{1}{5}\frac{t_2}{d_F}\right)$

$$= 20 \times 3.66^2 \times$$

$$\left(\sqrt{\frac{1}{6} \times \frac{22}{(20+22)} \times \frac{617}{20}} + \frac{1}{5} \times \frac{38}{3.66}\right)$$

$$= 996 \text{ N}$$

(f) $f_1 d_F^2 \frac{1}{5}\left(\frac{t_1}{d_F}+\frac{f_2}{f_1}\frac{t_2}{d_F}\right)$

$$= \frac{1}{5} \times 20 \times 3.66^2 \times$$

$$\left(\frac{38}{3.66} + \frac{20}{20} \times \frac{38}{3.66}\right)$$

$$= 1113 \text{ N}$$

(g) $f_1 d_F^2 \sqrt{\frac{2}{3}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}}$

$$= 20 \times 3.66^2 \times$$

$$\sqrt{\frac{2}{3} \times \frac{22}{(20+22)} \times \frac{617}{20}}$$

$$= 879 \text{ N} \qquad \textit{(Governs)}$$

Therefore, based on the above unit yield equations, n_u = 879 N.

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

$$= 879 \times (1.0 \text{ x } 1.0 \text{ x } 1.0)$$

$$= 879 \text{ N}$$

$$N_r = \phi N_u n_F n_s J_F$$

$$= 0.8 \text{ x } 879 \text{ x } n_F \text{ x } 1\ 1.0$$

$$= 703\ n_F \text{ (N)}$$

$$N_r \geq N_f$$

$$703\ n_F \geq 2500 \text{ N}$$

$$n_F \geq 3.6$$

Use four 3 inch nails.

Joint Configuration

Spacing parallel to grain

$$16\ d_F = 16\ (3.66)$$

$$= 59 \text{ mm}$$

Spacing perpendicular to grain

$$8\ d_F = 30 \text{ mm}$$

*Loaded and unloaded end distance

$$15\ d_F = 55 \text{ mm}$$

Edge distance

$$4\ d_F = 15 \text{ mm}$$

*Although CSA O86 allows shorter end distances in some cases, it is recommended where possible to use end distances of $15d_F$ or more, to prevent splitting.

Therefore, use two 3 inch nails from each side (Figure 11.29).

FIGURE 11.29

Heel connection

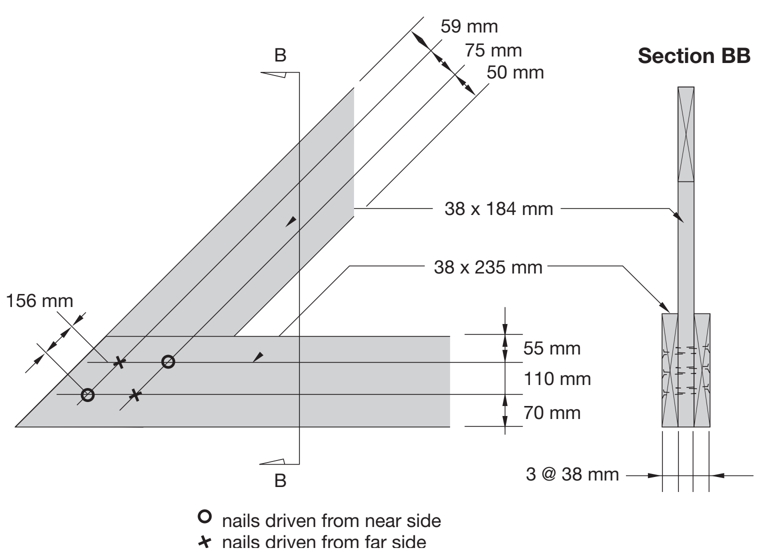


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TABLE 11.8

Minimum end distances for timber rivet joints

Number of rivet rows, n_R	Minimum end distance, a (mm) Load parallel to grain	Load perpendicular to grain
1, 2	75	50
3 to 8	75	75
9, 10	100	80
11, 12	125	100
13, 14	150	120
15, 16	175	140
17 and greater	200	160

TABLE 11.9

Nail materials, finishes and coatings

Type		Abbr.	Remarks
Material	Aluminum	A	For improved appearance and long life; increased stain and corrosion resistance.
	Steel-mild	S	For general construction.
	Steel-high-carbon hardened	Sc	For special driving conditions; improved impact resistance.
	Stainless steel, copper and silicon-bronze	E	For superior corrosion resistance; more expensive than hot-dip galvanizing.
Finish & Coatings	Bright	B	For general construction; normal finish; not recommended for exposure to weather.
	Blued	Bl	For increased holding power in hardwood; thin oxide finish produced by heat treatment.
	Heat treated	Ht	For increased stiffness and holding power; black oxide finish.
	Phoscoated	Pt	For increased holding power; not corrosion resistant.
	Electrogalvanized	Ge	For limited corrosion resistance; thin zinc plating; smooth surface; for interior use.
	Hot-dip galvanized	Ghd	For improved corrosion resistance; thick zinc coating; rough surface; for exterior use.

TABLE 11.10

Diameter and minimum yield strength of wood screws

	Gauge number 6	8	10	12
Diameter, mm*	3.50	4.16	4.82	5.48
Minimum yield strength, MPa†	690	620	550	550

* For wood screw diameters greater than gauge 12, design in accordance with lag screw requirements.
† Linear interpolation for yield strength may be used.

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FIGURE 12.5

Shearwall and diaphragm action

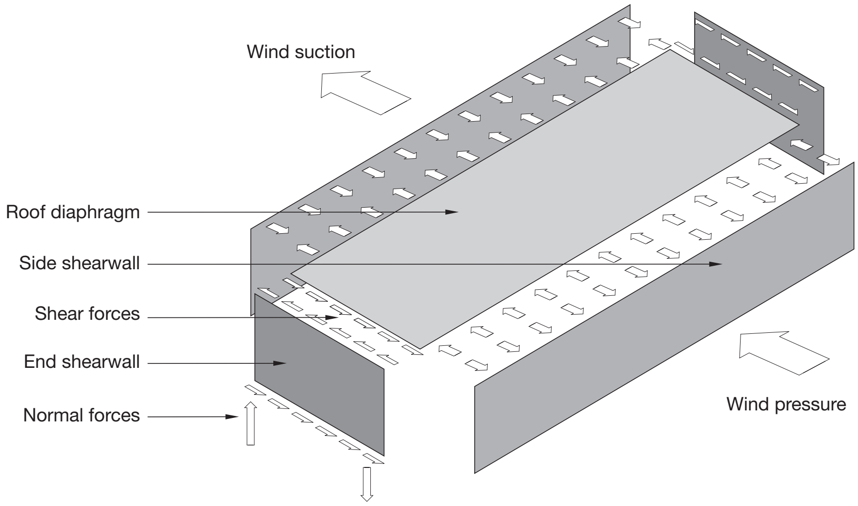
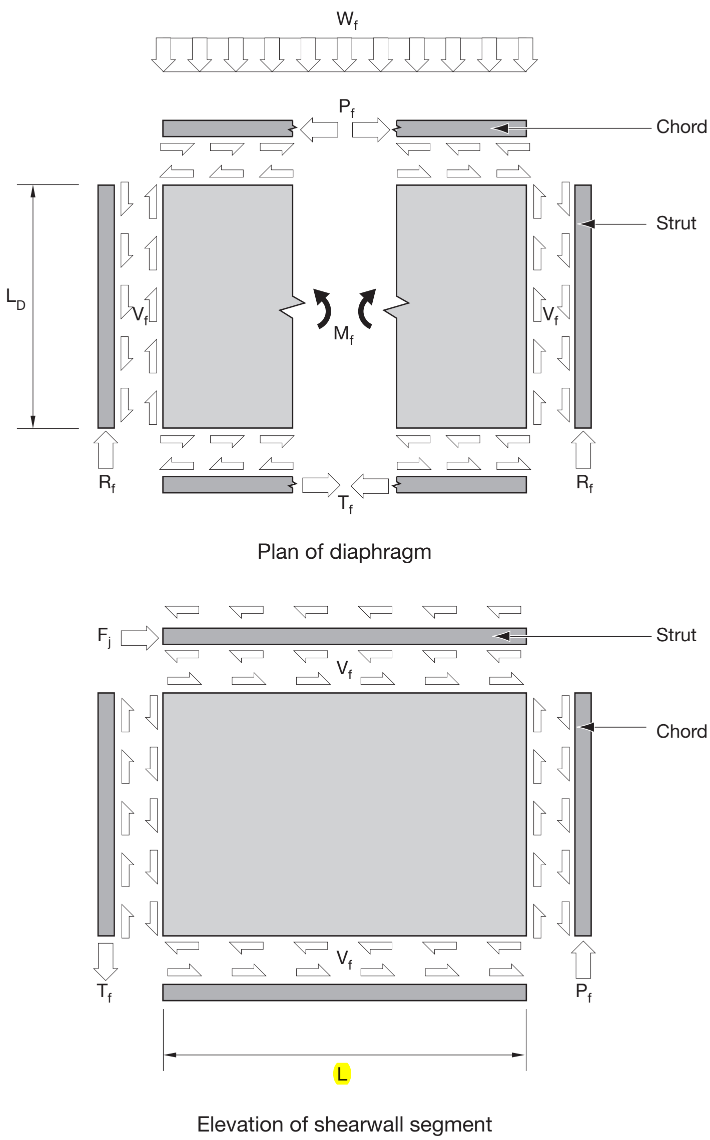


FIGURE 12.6

Free body diagram of a shearwall and diaphragm



13.4 Tongue and Groove Decking in Garden Centre

GIVEN VALUES AND ASSUMPTIONS

Design the tongue and groove decking in the garden centre.

- Support spacing = 2.03 m
- Controlled random pattern in east-west direction
- 3/12 roof slope = 14°
- Decking width = 133 mm
- Use Select grade spruce for appearance
- Deflection limit is L/240 for snow load, and L/180 for total specified load

CALCULATION

Specified dead load = 0.35 kPa (includes self-weight plus roofing)
= 0.35 / cos14°
= 0.36 kPa (on a horizontal projection)

Specified snow load for strength calculations = 1.60 kPa

Specified snow load for serviceability calculations = 1.44 kPa

Factored loading w_f = (1.250 × 0.36) + (1.5 × 1.60) = 2.85 kPa
Specified live load w_L = 1.44 kPa
Total specified loading w = 0.36 + 1.44 = 1.80 kPa
Checklist satisfied (*WDM* Section 2.2). From Decking Selection Tables try 38 mm thickness

W_{FR} = 9.88 kPa > 2.85 kPa *(Acceptable)*

$W_{\Delta R}$ = 2.12 kPa > 1.44 kPa for L/240 deflection *(Acceptable)*

$W_{\Delta T}$ = 2.12 × 1.33 = 2.82 kPa > 1.80 kPa for L/180 deflection *(Acceptable)*

Use 133 × 38 mm Select grade spruce decking.

Stud is fully restrained in the narrow direction by sheathing

$K_e = 1.0$

L_e = actual length (no lateral bracing specified)

$= 3000$ mm

Slenderness ratio

$C_c = L_e/d$

$= 3000/140$

$= 21.4$

Column capacity

$P_r = \phi F_c A K_{Zc} K_c$

$\phi = 0.8$

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

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

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

For this design

$K_D = 1.0$

$K_H = 1.10$ (CSA O86 Table 5.4.4 Case 1)

$K_{Sc} = 1.0$

$K_T = 1.0$

Calculations

$F_c = 9.0\ (1.0 \times 1.1 \times 1.0 \times 1.0)$

$= 9.9$ MPa

$K_{Zc} = 6.3(140 \times 3000)^{-0.13}$

$= 1.17$

$$K_C = \left[1.0 + \frac{9.9 \times 1.17 \times 21.4^3}{35 \times 5500 \times 1 \times 1}\right]^{-1}$$

$= 0.629$

$P_r = 0.8 \times 9.9\ (38 \times 140) \times 1.17 \times 0.629$

$= 31.0$ kN *(Acceptable)*

Use 38 × 140 mm Stud Grade S-P-F studs at 400 mm centres.

Note: the designer may also wish to check stud bearing on the bottom plate. See Stud Tables in the Wood Design Manual.

EXAMPLE 8.3: GLULAM COLUMN DESIGN

Design a glulam column for the conditions given in Example 8.1 using 12c-E grade Spruce-Pine.

Load Effects and Combinations

Total factored load

$P_f = 145$ kN

Calculations

Specified strength for 12c-E grade Spruce-Pine glulam from CSA O86 Table 6.3

$f_c = 25.2$ MPa

$E = 9700$ MPa

$E_{05} = 0.87 \times 9700$

$= 8440$ MPa

Because fixity is difficult to attain and to maintain, the degree of end restraint will hold the ends in position but will not prevent rotation. Thus:

$K_e = 1.0$

Effective length

L_e = actual length (no lateral bracing specified)

$= 3000$ mm

38 × 184 mm sloping rafters are untreated and belong to the Spruce-Pine-Fir group. They will be seasoned and the service conditions will be dry.

Design the heel joint between the 38 × 184 mm and two 38 × 235 mm members using nails.

Heel Connection

Try 3in (76 mm) long common round wire nails

diameter = 3.66 mm

Required penetration in the head side member is $3d_F$.

$$3 \times 3.66 = 11 \text{ mm} < 38 \text{ mm}$$

Required penetration in the point side member is $5d_F$.

$$5 \times 3.66 = 18 \text{ mm} < 38 \text{ mm}$$

Design for lateral strength resistance

Factored tensile force, N_f

$$N_f = 2500 \text{ N}$$

Factored resistance, N_r

$$N_r = \phi N_u n_F n_s J_F$$

where

ϕ = 0.8

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

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

n_F = number of fasteners in the connection

n_s = 1 (nails are in single shear)

$J_F = J_E J_A J_B J_D$

J_E = 1.0 (nails are driven into side grain)

J_A = 1.0 (nails are not toe nailed)

J_B = 1.0 (nails are not clinched)

J_D = 1.0 (not used in diaphragm construction)

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

$= 1.0$

Unit Lateral Strength Resistance

The penetration depth of the nail is 76 mm (3 inch), therefore this is a two member wood to wood connection.

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

where

t_1 = 38 mm

d_F = 3.66 mm

$f_2 = 50G(1 - 0.01d_F)$

$= 50 \times 0.42 \times (1 - 0.01 \times 3.66)$

$= 20$ MPa

G = 0.42 (CSA O86, Table A. 10.1)

t_2 = 38 mm

$f_3 = 110G^{1.8}(1 - 0.01d_F)$

$= 110 \times 0.42^{1.8} \times (1 - 0.01 \times 3.66)$

$= 22$ MPa

$f_y = 50(16 - d_F)$

$= 50 \times (16 - 3.66)$

$= 617$ MPa

For lumber side plates:

$f_1 = 50G(1 - 0.01d_F)$

$= 20$ MPa

(a) $f_1 d_F t_1$

$= 20 \times 3.66 \times 38$

$= 2814$ N

(b) $f_2 d_F t_2$

$= 20 \times 3.66 \times 38$

$= 2814$ N

(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

$= 20 \times 3.66^2 \times$

$\left(\sqrt{\frac{1}{6} \times \frac{22}{(20 + 22)} \times \frac{617}{20}} + \frac{1}{5} \times \frac{38}{3.66} \right)$

$= 996$ N

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Deflection of Diaphragms

In accordance with Clause 9.7.2 of CSA O86-09, the following formula may be used to estimate the lateral static deflection at mid-span, Δ_d, mm, of a simply supported blocked diaphragm:

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

Where:

Δ_d = lateral deflection at mid-span (mm)

v = maximum shear force per unit length due to specified lateral loads (N/mm)

L = diaphragm span perpendicular to the direction of the load (mm)

A = cross-sectional area of chord members (mm^2)

E = modulus of elasticity of chords (MPa)

L_D = depth of diaphragm parallel to the direction of the load (mm)

B_V = shear-through-thickness rigidity (N/mm) (refer to Tables 7.3A, 7.3B and 7.3C of CSA O86)

e_n = nail deformation for a particular load per nail (mm) (Table 12.1)

$\sum(\Delta_c x)$ = sum of the individual chord-splice slip values, Δ_c, on both sides of the diaphragm, each multiplied by its distance x to the nearest support

Openings in the Sheathing

Most diaphragms have openings for pipes, ductwork, elevators, stairwells and skylights. Openings in high shear areas may have to be reinforced with additional framing members and connections to transfer the forces around the opening (Figure 12.12A).

TABLE 12.1

Deformation e_n for shearwall and diaphragm calculations[2]

Load per nail[1] N		Deformation, e_n mm		
	Nail length (in.)	2	2-1/2	3
	Gauge	11-3/4	10	9
	Diameter (mm)	2.84	3.25	3.66
300		0.46	0.29	0.23
400		0.76	0.46	0.35
500		1.20	0.64	0.49
600		1.89	0.88	0.66
700		2.97	1.21	0.86
800			1.70	1.13
900			2.33	1.48
1000				1.95

Notes:

1. Load per nail = v_s s
   where:
   v_s = maximum specified shear force per unit length along the diaphragm boundary or the top of shearwall
   s = nail spacing along the boundary of an interior panel
2. Values divided by 2 for seasoned lumber
3. Adapted from the Uniform Building Code (International Conference of Building Officials, Whittier, California).

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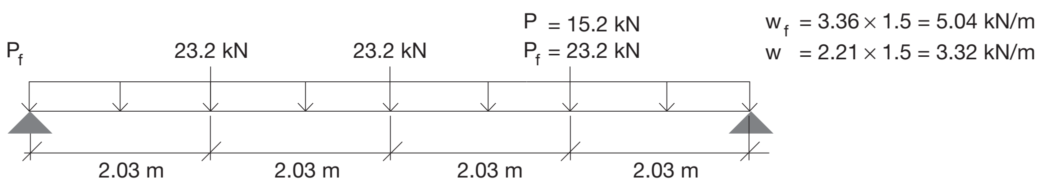
13.6 Glulam Header Beam in Garden Centre

GIVEN VALUES AND ASSUMPTIONS

Design the glulam header beam B3 in the garden centre.

- Beam span = 8.13 m
- Use 20f-E D.Fir-L glulam.
- w_f and w based on 1.5 m tributary width.
- P_f and P from B2.



CALCULATION

$$M_f = \frac{w_f L^2}{8} + \frac{P_f L}{2}$$

$$= \frac{5.04 \times 8.13^2}{8} + \frac{23.2 \times 8.13}{2}$$

$$= 136 \text{ kN•m}$$

$$V_f = \frac{w_f L}{2} + \frac{3P_f}{2}$$

$$= \frac{5.04 \times 8.13}{8} + \frac{3 \times 23.2}{2}$$

$$= 55.2 \text{ kN}$$

For L/180, deflection is based on total specified load

$$E_S I_{REQ'D} = 180 \times \frac{5wL^3 + 19PL^2}{384}$$

$$= 180 \times \frac{5 \times 3.32 \times 8130^3 + 19 \times 15\,200 \times 8130^2}{384}$$

$$= 13\,200 \times 10^9 \text{ N•mm}^2$$

Checklist satisfied (*WDM* Section 2.5)