Assignment 3
Question 4.

~~Answers from part (a) and (b) are identical.~~

4- The trusses in a residential roof are placed at 600mm o.c. and built using metal plate connectors and 2"x?? D-Fir No. 2 visually graded lumber. The calculated force acting on the bottom chord (in tension) is 45kN. Propose a commercially available size for this component.

Solution

$K_D = 1.0$ standard load (Table 4.3.2.2, page 17, CSA 086-01)
$K_H = 1.1$ load sharing (Table 5.4.4, page 30, CSA 086-01) → Tension // to grain.
$K_{St} = 1.0$ dry service (Table 5.4.2, page 30, CSA 086-01)
$K_T = 1.0$ treatment factor (Table 5.4.3, page 30, CSA 086-01)

using the Wood Design Manual; the Tension Member Selection Tables, Page 162
38 x 140 (2" x 6")
$T_r = (36.1)(1.0)(1.1)(1.0)(1.0) = 39.71$ kN < 45 kN

38 x 184 (2" x 8")
$T_r = (43.8)(1.0)(1.1)(1.0)(1.0) = 48.18$ kN > 45 kN

Use 38 x184 mm No.2 grade D-Fir.

Assignment 4
Question 3.

140 x 140

3 - Find the axial load capacity of a 2.5m long 6" x 6" square sawn column, Dfir-L #1, wet conditions, incised for treatment, no load sharing. Assume pinned and restrained from sway at both ends.

a) Use the formulas given to you in CSA O86.
b) If the column were not incised and used in a dry state, what would its capacity be? Use the Table in the Wood Design Manual. State what table you used.

3- solution:
a) Pr = Phi Fc A Kzc Kc
Phi = 0.8
Fc = fc (Kd Kh Ksc Kt)
Fc = 12.2 (1x1x0.91x1) = 11.1 MPa (Kt = 1 because of the size of the section)

$A = 140 \times 140 = 19600\ mm^2$

$Kzc = 6.3\ (140 \times 2500)^{-0.13} = 1.20 < 1.3$

Ke = 1 pinned at both ends but free to rotate

Cc = 2500/140 = 17.86 < 50

$Kc = [1 + (11.1 \times 1.20 \times 17.86^3/(35 \times 6500 \times 1 \times 1)]^{-1} = 0.750$

Pr = 0.9 (11.1) (19600) (1.20) (0.750) = 156643 N = 157 kN

b) From WDM Table on page 122, Pr = 168 kN

Considerably more due to dry service conditions assumed here. Not that service conditions is not linear, and therefore table values cannot be used directly and modified w/ Ks factor

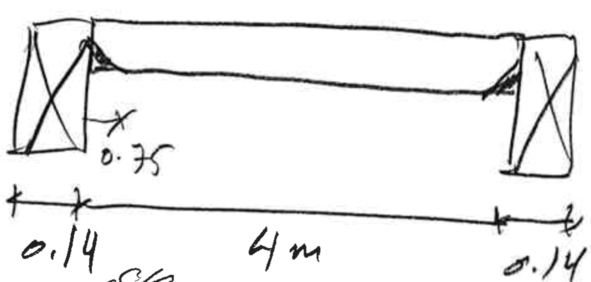
Assignment 5

Question 1 :

No. 2 grade SPF lumber.
insised + treated.
Wet conditions

The wharf is used for loading and unloading only, which seems to indicate that short term loading could be assumed. However, the use of the wharf might change and longer loading would be more appropriate (Both short and normal term calculation will be given)

$W_D = 0.8$ kPa , $W_L = 3.2$ kPa



Decking will provide lateral support.

Spacing = 0.8m.

$w_f = 0.8 \ (1.25(0.8) + 1.5(3.2)) = 4.64$ kN/m. (0.8 = c/c)

$M_f = 4.64 \times 4^2/8 = 9.28$ kNm.

$V_f = 4.64(4)/2 = 9.28$ kN

$Q_f = 9.28$ kN.

For K_S and K_T we need a member size, therefore use beam tables to select a preliminary section, then show all calculations

Given the applied moment and shear an 89 x 286 member will be selected

From table 5.3.1.A:

$F_B = 11.8$ MPa, $F_{cp} = 5.3$ MPa, $E = 9500$ MPa

Table 5.4.2:

$K_{Sb} = 0.84$, $K_{SV} = 0.96$ $K_{Scp} = 0.67$ $K_{SE} = 0.94$.

Table 5.4.3:

$K_T = 0.85$, $K_{TE} = 0.95$

$K_H = 1.0$ No system

$K_D = 1.15$ or 1.0

$K_{ZB} = 1.1$ $K_{ZV} = 1.1$ (Table 5.4.5)

$K_L = 1.0$ Lateral support provided.

Also $d/b = \frac{286}{89} = 3.2 < 4.0$.

<u>Moment:</u>

$M_r = \varphi F_B S K_{Zb} K_L$

$F_B = 11.8(1.15 \times 1.0 \times 0.84 \times 0.85) = 9.69$ MPa.

$M_r = 0.9(9.69) \frac{89 \; 286^2}{6}(1.1) \; 1.0 \times 10^{-6} = 11.6$ kNm $> M_F$ ∴ ok

Normal duration $M_r = 10.1$ kNm ⇒ ok

Shear:

$V_r = \phi F_v \tfrac{2}{3} A_n K_{ZV}$.

$F_v = 1.5(1.15 \times 1. \times 0.96 \times 0.85) = 1.4076$. MPa

$V_r = 0.9(1.41) \tfrac{2}{3} (89\ 286) \times 1.1 \times 10^{-3}$.

$= 23.7 kN > V_f = 9.28$ kN $\therefore$ ok.

Normal load duration : $V_r = 20.61$ kN. $\therefore$ ok

Bearing:

$K_B = 1.0$ support at member ends.

$K_{Zcp} = 1.0$ (Table 5.5.7.5)

$Q_r = \phi F_{cp} A_B K_B K_{Zcp}$.

$F_{cp} = 5.3(1.15 \times 0.67 \times 0.85) = 3.47$ MPa

$Q_r = 0.8(3.47)\ 75 \times 89 \times 1 \times 1 \times 10^{-3}$

$= 18.5$ kN $> Q_f$ $\therefore$ ok

Normal duration : $Q_r = 16.1$ ⇒ still ok.

Deflection:

$L/180 = 22.2$ mm. for total specified load.

$L/360 = 11.1$ mm. for live specified load.

$E_s = 9500(0.94 \times 0.95) = 8484$ MPa.

$W_L = 0.8(3.2) = 2.56$ kN/m

$W_{tot} = 0.8(0.8 + 3.2) = 3.2$ kN/m.

$$\Delta_{D+L} = \frac{5wL^4}{384 E_s I} = \frac{5(3.2)\,4000^4}{384(8484)\frac{89 \times 286^3}{12}}$$

$$= 7.2 \text{ mm.} < L/180 \quad \therefore \text{ ok}$$

$$\Delta_L = \frac{5 \times 2.56 \times 4000^4}{384(8484)\frac{89 \times 286^3}{12}} = 5.8 \text{ mm} < L/360. \quad \therefore \text{ ok.}$$

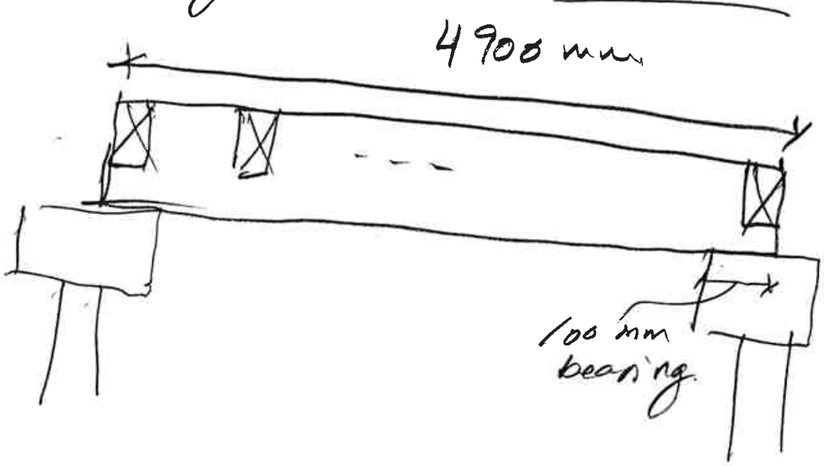
A 89 × 286 SPF No.2 lumber member may be used for the cross beams of the wharf.

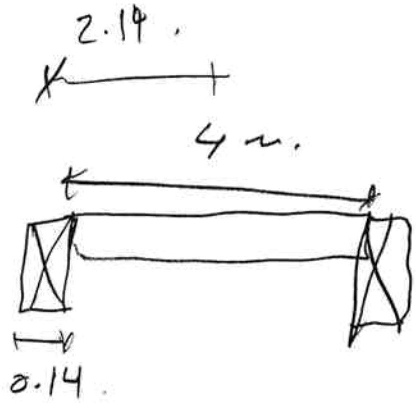
B). Hem-Fir 140 x 393 No.1 grade (Timber)
incised + treated.
Wed conditions

DL = 0.8 KPa , LL = 3.2 KPa.

you can either consider the beam
to be loaded with a series of
point loads or uniformly.
Will use uniform approach for simplicity.

Tributary width = 2.14 m.





$$w_f = 2.14(1.25(0.8) + 1.5(3.2))$$
$$= 12.4 \text{ KN/m}.$$

Will use full length 4.9 m.
to be conservative

$M_f = 12.4 \times 4.9^2/8 = 37.2$ kN m.

$V_f = 12.4 \times 4.9/2 = 30.4$ kN.

$Q_f = 30.4$ kN.

Table 5.3.1C.

$F_B = 11.7$ MPa, $F_v = 1.2$, $F_{cp} = 4.6$

$E = 10\,000$ MPa.

Table 5.4.2. (cover 89 mm).

$K_{SB} = 1.0$ $K_{SV} = 1.0$ $K_{Scp} = 0.67$ $K_{SE} = 1.0$.

$K_T = 1.0$ $K_{TE} = 1.0$ Wet, but size > 89 mm.

$K_H = 1.0$ No system

$K_D = 1.15$ ($K_D = 1.0$ will be checked)

$K_{zb} = 0.9$ $K_{zv} = 0.9$ (table 5.4.5)

$K_L = 1.0$ ← lateral load provided at 0.8 m.
Also $d/b = 393/140 = 2.8 < 4$.

<u>Moment:</u>

$M_r = \phi F_b S K_{zb} K_L$.

$F_B = 11.7(1.15 \times 1 \times 1 \times 1 \times 1) = 13.45$ MPa

$M_r = 0.9(13.45)\dfrac{140 \times 393^2}{6} 0.9 \times 1 \times 10^{-6}$

$= 39.3$ kN m ∴ ok.

Normal duration: $M_r = 34.3$ kN m ⇒ Not adequate.

Shear:

$$V_r = \varphi F_v \ 2/3 \ A_n \ K_{zv}$$

$$F_v = 1.2(1.15 \times 1 \times 1 \times 1) = 1.38 \text{ MPa}$$

$$V_r = 0.9(1.38) \ 2/3 \ (140 \times 393) \times 0.9 \times 10^{-3}$$
$$= 41.0 \text{ kN} > V_f = 30.4 \quad \text{ok}$$

Normal duration ~ 35.65 kN ok

Bearing:

$$K_B = 1.0$$

$$K_{zcp} = 1.0 \quad [T. 5.5.7.5]$$

$$Q_r = \varphi F_{cp} A_B K_B K_{zcp}$$

$$F_{cp} = 4.6(1.15 \times 0.67 \times 1.0) = 3.54 \text{ MPa}$$

$$Q_r = 0.8(3.54)(100 \times 140) \ 1.0 \times 1.0 \times 10^{-3}$$
$$= 39.6 \text{ kN} > Q_F \quad \text{ok}$$

Normal load duration:

$$Q_r = 34.5 \text{ kN} > Q_F \quad \text{ok}$$

Deflection:

$L/180 = 27.2$ mm. Total load.

$L/360 = 13.6$ Live load.

$E_s = 10000\ (1.0 \times 1.0) = 10000$ MPa.

$w_L = 2.14\ (3.2) = 6.85$ kN/m

$W_{tot} = 2.14\ (0.8 \times 3.2) = 8.56$ kN/m.

$$\Delta_{D+L} = \frac{5 \times 8.56 \times 4900^4}{384\ (10000)\ \frac{140 \times 393^3}{12}} = 9.0 \text{ mm} < \frac{L}{180} \therefore ok$$

$\Delta_L = 7.2$ mm $< L/360 \therefore$ ok.

Connections (also assignment 6, Prob. 3)

Uplift load / stud = 1.45 KN

a) • Top plate connected to stud anchor with lag screws.

Two 3/8" dia. x 3" long lag scoews are suggested

$P_{rw} = \varphi \, Y_w \, L_t \, n_F \, J_E$

$\varphi = 0.6$

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

y_w can be found in table 10.6.5.1 for Doug. Fir. / 3/8 ⇒ 120 N/mm

L_t : length of paneteration of threaded portion

$L_t = L/2 + 12.7 - tip$
$= 38.1 + 12.7 - 6.4$
$= 44.4$

$n_F = 2$, $K_D = 1.15$, $J_E = 1.0$ all other cases than end grain.

$P_{rw} = 7.35$ KN > 1.45

1. Stud anchor connected to stud with single bolt loaded in double shear. parallel to grain

- Suggested one 1/2" bolt

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

where: $\phi = 0.7$

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

Formula can also be written as

$$P_r = P_r' n_s n_F K' J'$$ (WPM selection tables)

$K_D = 1.15 = K'$

$J_L = 1.0$ @ 10 dia

$= 0.75$ @ 7 dia

$= 0.81$ @ 7.71 dia $= J'$

Assumed smallest stud size → For 38 mm thick member, double shear, steel side plate

$P_r' n_s = 7.28$

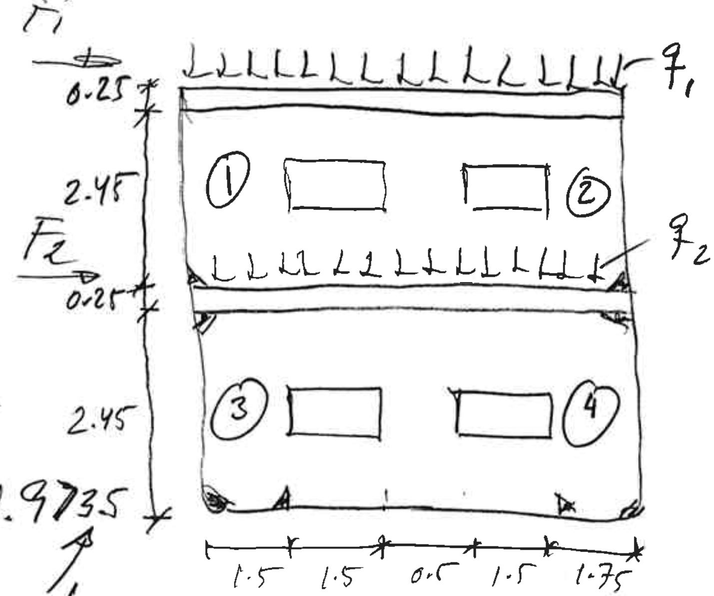
$P_r = 7.28 \times 1 \times 1.15 \times 0.81$

$= 6.78$ kN > 1.45 kN <u>ok</u>

Assignment 6

1) Shear wall

$q_1 = 0.5$ kN/m
$q_2 = 0.6$ kN/m
$q_w = 0.3$ KPa
} specified.



Note

Aspect ratio for wall segment between windows is $2.45/0.5 = 4.9 > 3.5$ Limit

∴ Cannot be used as a shear wall.

Stated:
- wind loading.
- SPF studs at 500 mm.
- 12.5 mm CSP sheathing / 12.5 mm Gypsum.
- Panel edges supported / panels in vertical position.
- 100 mm c/c panel edge / 300 mm c/c panel field nail spacing
- 150 mm c/c Gyp. panel edge / 300 mm c/c gyp. panel field nail spacing

→ Solve for max F_1 and F_2.

2½" Nail $d = 3.25$ mm $L = 63.5$ mm.

Pen. $63.5 - 12.5 = 51$ mm > 38 mm (Table 9.5.1A)

$V_d = 10.3$ kN/m (Table 9.5.1.A) wood sheathing

$K_d = 1.15$ wind loading short duration.

$K_{SF} = 1.0$ Dry service.

$J_{UB} = 1.0$ ALL panel edges supported.

$J_n = 1.0$ Standard walls.

$J_{sp} = 0.8$, SPF framing

$J_{HD} = ?$ } depends on
$L_w = ?$ } segment.

$V_{rs} = \varphi V_d K_D K_{SF} J_{UB} J_{sp} J_{hd} J_n L_w$.

$= 0.7(10.3) \times 1.15 \times 1 \times 1 \times 0.8 \times 1 = 6.63$ kN/m

without J_{HO} & L_w.

$V_d = 1.7$ kN/m (Table 9.5.1 B) Gypsum Panels.

$J_{HD} = ?$ } depends on.
$L_w = ?$ } segment.

$V_{rg} = \varphi V_d J_{HD} L_w$.

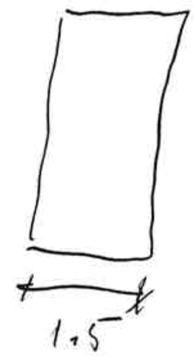
$= 0.7 \times 1.7 = 1.19$ kN/m.

without J_{HD} & L_w.

Segment ①

$L_w = 1.5\,m$ Trib. wall $= 1.5/2 = 0.75\,m.$

$H_s = 2.45\,m.$



$V_{HD} = \underbrace{7.82}_{\text{Contribution from wood panel and gypsum panel.}}(1.5) = 11.7\,kN.$

Contribution from wood panel and gypsum panel.

$P_{top} = \underbrace{0.9}_{\alpha_D \text{ for dead load}}\ \overset{q/1}{(0.5)}\ \overset{trib.}{0.75} = 0.33\,kN.$

α_D for dead load acting to counteract the overturning.

$q_w = 0.9 \times (0.3)\ 2.45 = 0.66\,kN/m.$

$P = P_{top} + q_w \frac{l}{2} = 0.33 + 0.66\,\frac{1.5}{2}$

$= 0.79\,kN.$

Holddown on uplift side → $J_{HD} = 1.0.$

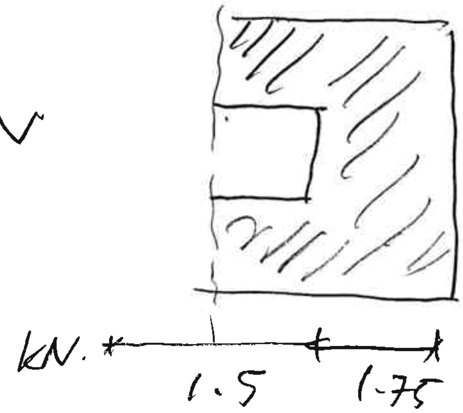
→ $\underline{\underline{V_{rs} = 11.7}}$

Segment (2):

$L_w = 1.75\ m$

Trib wall $= \frac{1.75}{2} + \frac{1.5}{2}$

$= 1.625\ m$



$H_s = 2.45\ m$

$V_{HD} = 7.82\,(1.75) = 13.7\ kN$

$P_{top} = 0.9\,(0.5)\ 1.625 = 0.73\ kN.$

$q_w = 0.66\ kN/m$

$P = 0.73 + 0.66 \times \frac{1.75}{2} = 1.31\ kN.$

No holddown on uplift side:

$$J_{Hd} = \sqrt{1 + 2 \times \frac{1.31}{13.7} + \left(\frac{2.45}{1.75}\right)^2} - \frac{2.45}{1.75}$$

$= 0.375$

$V_{rs} = 13.7\,(0.375) = \underline{5.14\ kN}.$

Upper story capacity $= 11.7 + 5.14 = 16.8\ kN.$

Assume the applied load to be $16.8\ kN.$

Uplift forces:

Seg 1.: $F = 16.8 \times \frac{11.7}{16.8} = 11.7$ kN

Seg. 2: $F = 16.8 \times \frac{5.10}{16.8} = 5.10$ kN.

Seg 1: $R^* = 11.7 \frac{2.45 + 0.25}{1.5} - 0.79 = 20.3$ kN.

Seg 2: $R^* = 5.10 \frac{2.45 + 0.25}{1.75} - 1.31 = 6.56$ kN.

Seg. 3:

Has holddown top & bottom. ∴ $J_{HD} = 1.0$

$L_w = 1.5$ m Trib. wall $= 0.75$ m.

$H_s = 2.45$ $V_{HD} = 7.82(1.5) = 11.7$ kN.

$P_{top} = q_z \times$ Trib. wall $- R$

$= 0.9(0.6)\,0.75 - 20.3 = -19.9$ kN

$P = -19.9 + 0.66 \times \frac{1.5}{2} = -19.4$ kN.

$V_{rs} = 11.7$ kN ($J_{HD} = 1$).

Segment (4):

Has holddown at bottom only.

$$J_{HD} = \frac{V_{HD} + P_T}{V_{HD}}$$

$L_w = 1.75\ m$ Trib. wall $= 1.625\ m$

$H_s = 2.45\ m$

$V_{HD} = 7.82(1.75) = 13.7\ kN$

$P_{top} = 0.9(0.6)\ 1.625 - 6.56$

$= -5.68$

$P = -5.68 + 0.66\ \frac{1.75}{2} = -5.11\ kN.$

$$J_{HD} = \frac{13.7 - 5.7}{13.7} = 0.584$$

$V_{RS} = 0.584 \times 13.7 = \underline{8\ kN}$

Lower storey capacity $11.7 + 8 = 19.7\ kN$.

Assume the applied load to be 19.7 kN.

(16.8 kN from upper story and 2.9 from lower

Uplift forces:

Segment 3 : $F = 19.7 \times \frac{11.7}{19.7} = 11.7$ kN.

Segment 4 : $F = 19.7 \times \frac{8}{19.7} = 8$ kN.

Seg. 3 : $R^{*} = 11.7 \times \frac{2.45 + 0.25}{1.5} - (-19.4)$

$= 40.5$ kN.

(−19.4 → P for segment:)

Seg. 4 : $R^{*} = 8 \times \frac{2.45 + 0.25}{1.75} - (-5.11)$

$= 17.45$ kN

(−5.11 → P for seg. 4)

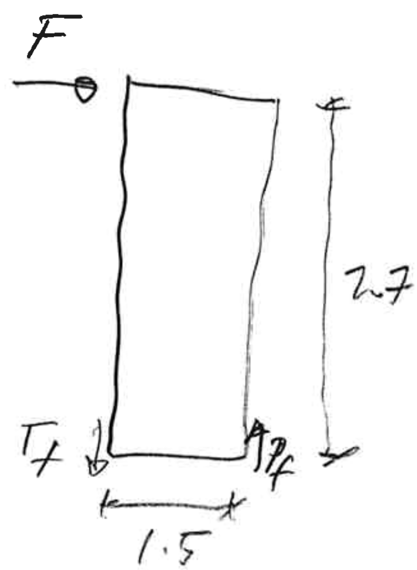
Chord forces:

Seg. 1 : $F = 11.7$ kN.

$P_f = T_f$

$\Sigma M = 0$: $-P_f(1.5 - 0.15) + 11.7(2.7)$ (0.15 → reduction.)

$P_f = T_f = 23.4$



Seg. 2

$F = 5.1$ No holddown ∴ No T_f.

$\Sigma M = 0 \Rightarrow -P_f(1.75 - 0.15) + 5.1(2.7)$.

$P_f = \underline{8.6 \text{ kN}}$.

Seg. 3:

$F = 11.7 \text{ kN}$ $P_f = T_f$

$\Sigma M = 0 \Rightarrow -P_f(1.5 - 0.15) + 11.7(2.7)$.

$P_f = T_f = \underline{23.4 \text{ kN}}$.

Seg. 4:

$F = 8.0$

$\Sigma m = 0 \Rightarrow -P_f(1.75 - 0.15) + 8.0(2.7)$

$P_f = T_f = \underline{13.5 \text{ kN}}$