Combined loading - Studwall

| Same or very similar to midterm question. |

Given values and assumptions:

- Stud spacing = 400 mm.
- Stud length = 6.0 m.
- Tributary width = 6.10 m.
- Stud is held in position by roof at top, and foundation at bottom.
- Wall sheathing prevents buckling about weak axis (this is because the sheathing is applied directly on studs. Case 2 is assumed.
- Short term load duration is assumed (wind).
- Use No.1/No.2 grade SPF Lumber.

general:

one should be aware of the fact that even though we are dealing with a combined loading case, the axial load alone case should be checked and could govern design. the reason for that lies in the K_d factor, that is different for axial alone (D+S, or even strictly speaking D alone as well) and for axial combined with wind (D+S+W). In this regard, it is worth mentioning that one <u>cannot</u> add the wind load to the axial load, as they apply in different directions; So, even when the load case is 1.25D + 1.5L+0.4W it does not imply that the actual values should be added together but rather that it is a load case that includes D, L and W acting together on the structural member. Also note, that in the WDM tables, there are two different selection tables, one group for axial load only, and another group for the combined loading.

For this problem we will use the WDM selection tables.

Loads:

- specified dead load, $D = 0.85$ kPa.
- specified snow load, $S = 1.6$ kPa (for strength).

- specified wind load,

KN/m for wind are typical units. → $W = 0.39$ kN/m (strength).

$W = 0.30$ kN/m (serviceability)

→ specified gravity loads on the wall:

Dead: $P_D = 0.85 \times 6.1 \times 0.4$

KN for dead and snow are typical units. → $= 2.07$ kN.

Snow.

$$P_s = 1.6 \times 6.1 \times 0.4$$
$$= 3.90\ KN.$$

Load cases : Strength (ignoring 1.4D will NOT govern)

<u>Case 1</u> : $1.25D + 1.5S.$ $(K_D = 1.0)$.

$$P_f = 1.25 \times 2.07 + 1.5 \times 3.9 = \underline{8.44\ KN}.$$

<u>Case 2</u> : $1.25D + 1.5S + 0.4W$ $(K_D = 1.15)$

$$P_f = 1.25 \times 2.07 + 1.5 \times 3.90 = \underline{8.44\ KN}$$
$$W_f = 0.4 \times 0.39 = \underline{0.16\ KN/m}.$$

<u>Case 3</u> : $1.25D + 0.5S + 1.4W$ $(K_D = 1.15)$.

$$P_f = 1.25 \times 2.07 + 0.5 \times 3.90 = \underline{4.54\ KN}$$
$$W_f = 1.4 \times 0.39 = \underline{0.55\ KN/m}.$$

1) From the stud wall selection tables (compression loading, WDM section 3): Select a size that satisfies the factored compression force for load case 1:

→ Check list satisfied

Try 38 x 140 mm.

$P_r' = 10.1$ KN > 8.44 KN. <u>ok</u>

2) From the stud selection tables (combined loading, section 5.2) select a size that satisfies the interaction equation for load case 2. Try 38 x 184 mm.

$P_r' = 10.2$ KN > 8.44.

$W_r' = 0.470$ kN/m > 0.16 kN/m.

<u>Note</u>: that there could be multiple selections that would satisfy both the axial load and bending load.

3.) Load case 3 :

Try 38 x 184 mm.

$P_r' = 7.66 > 4.54$ KN.

$W_r' = 0.582$ KN/m > 0.55 KN/m.

Shear :

$$V_f = w_f L/2.$$
$$= 0.55 \times 6/2.$$
$$= 1.65 \text{ KN}$$

$V_r = 12.2$ KN > 1.65 KN ok

(Vr can be found at the bottom of the selection table in the grey box).

deflection :

$$\text{max deflection} = \frac{5 W L^4}{384 E_s I}$$

Note : this is wind for SLS

$$= \frac{5 \times 0.30 \times 6000^4}{384 \times \underbrace{187 \times 10^9}} = 27.1 \text{ mm.}$$

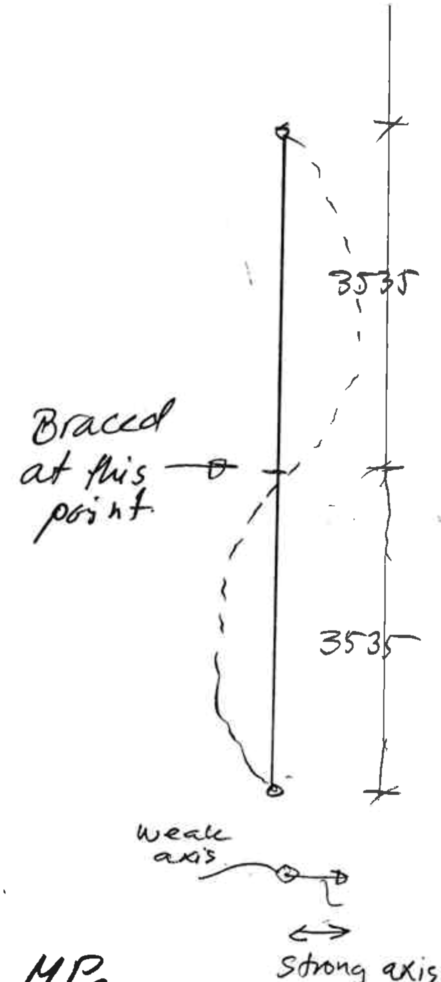
can be found in the selection table grey box as well.

Combined loading.

Beam - Column :

Northern No. 1 140 × 191

Note that it is important to make clear what the bracing assumption are.



From table 5.3.1D. we get

$F_C = 6.7$ MPa , $F_B = 9.0$ MPa.

$E = 7000$ MPa , $E_{05} = 5000$ MPa

assume

$K_D = 1$, $K_H = 1$, $K_{SC} = 1$, $K_{SB} = 1$, $K_{SE} = 1$, $K_T = 1$

$K_{TE} = 1$.

A computer analysis for a specific loading scenario showed that the maximum bending moment is 0.5 kNm. and it will occur at the mid point. The axial force in the member is found to be 91 kN.

No need for this example to determine shear and bearing.

Amplify moment using direct method:

moment should be amplified and this is an easy way to do it without calculating the deflection.

$$M_F^* = M_F' \left(\frac{1}{1 - \frac{P_f}{P_E}} \right).$$

$$P_E = \frac{\pi^2 E_s I}{(K_E L)^2}$$

$$= \frac{\pi^2\ 7000 \times 1 \times 1 \times \frac{140 \times 191^3}{12} \times 10^{-3}}{(1 \times 3.535)^2}$$

$$= \underline{449.4\ KN}.$$

$$M_F^* = 0.5 \frac{1}{\left(1 - \frac{91}{449.4}\right)} = 0.627\ KNm.$$

Moment resistance:

$$M_r = \phi F_B S K_{ZB} K_L$$

$$F_B = 9.0(1\times1\times1\times1) = 9.0 \text{ MPa.}$$

$$C_B = \left(\frac{1.92\,(3535)\,191}{140^2}\right)^{1/2} = 8.1 < 10$$

$$\therefore K_L = 1.0.$$

$$K_{Zb} = 1.3.$$

$$M_r = 0.9\,(9.0)\,\frac{140\times191^2}{6}\,1.3\times1\times10^{-6.}$$
$$= \underline{8.96 \text{ kNm.}}$$

Compression:

P_r weak axis will govern

$$C_{cy} = \frac{1.0\times3.535}{140} = 25.25 < 50 \quad \therefore \text{ok.}$$

$$F_C = 6.7(1\times1\times1\times1) = 6.7.$$

$$K_{ZC} = 6.3(140\times3535)^{-0.13} = 1.146 < 1.3 \quad \therefore \text{ok.}$$

$$K_C = \left(1.0 + \frac{6.7(1.146)\,25.25^3}{35\,(5000\times1\times1)}\right)^{-1} = 0.586.$$

$$P_r = \phi F_C A K_{ZC} K_C.$$

$$= 0.8\,(6.7)\,140\times191\,(1.146)\,0.586\times10^{-3.}$$

$$= 96.25 \text{ kN}$$

Interaction equation:

$$\frac{P_F}{P_r} + \frac{M_F^*}{M_r}$$

$$= \frac{91}{96.25} + \frac{0.627}{8.96} = \underline{\underline{1.02}}$$

use engineering judgment to accept or not, only 2% over.

Need to check Shear, bearing, deflection, etc.

Midterm Q.3.

A) Wet conditions – due to swimming pool.

Same bracing length in strong and weak axis $\Rightarrow$ use bottom part of Column for design.

$$P_f = 95 + 53 = 148\,KN.$$

$K_{SC} = 0.75 \qquad K_{SE} = 0.9$

$$K_D = K_H = K_T = 1.0.$$

From Column selection table for 5.2 m:

For $L = 5\,m \rightarrow P_{ry} = 0.75(212) = 159\,kN.$

$P_{rx} = 0.75(246) = 184.5\,kN.$

For $L = 5.5 \rightarrow P_{ry} = 0.75(175) = 131.3\,kN.$

$P_{rx} = 0.75(206) = 154.5\,kN.$

$$Y\text{ axis}: \quad \frac{5.5 - 5}{131.3 - 159} = \frac{5.5 - 5.2}{131.3 - P_{ry}}$$

$\Rightarrow P_{ry} = 147.9\,KN \sim 148.$

$X\text{ axis} = \quad \Rightarrow \quad P_{rx} = 172.5\,kN$

Based on the column selection table the selected section has adequate strength.

→ How the tables were used, however, is not appropriate → see question C).

B) $F_c = 25.2 \text{ MPa}, \quad E = 9700 \text{ MPa}$

$F_c = 25.2(1 \times 1 \times 0.75 \times 1) = 18.9 \text{ MPa}$

$E_{05} = 0.87(9700) = 8439 \text{ MPa}$

$P_r = \varphi F_c A K_{ZCG} K_c$

use full Length.

$K_{ZCG} = 0.68(0.175 \times 0.190 \times 8.2)^{-0.13}$

$= 0.805 \leq 1.0 \therefore ok.$

$C_{cy} = \dfrac{1.0(5200)}{175} = 29.7 < 50 \quad \therefore ok.$

$C_{cx} = \dfrac{1.0(5200)}{190} = 27.4 < 50 \quad \therefore ok.$

$K_{cy} = \left(1 + \dfrac{18.9(0.805)\,29.7^3}{35(8439)\,0.9(1.0)}\right)^{-1} = 0.400.$

$K_{cx} = 0.459.$

$P_{ry} = \underline{161.9 \text{ kN.}}$

$P_{rx} = \underline{185.8 \text{ kN.}}$

c) The calculated values do not match those obtained from the tables for the following reasons:

1) interpolation: P_r is <u>not</u> linearly related to column length.

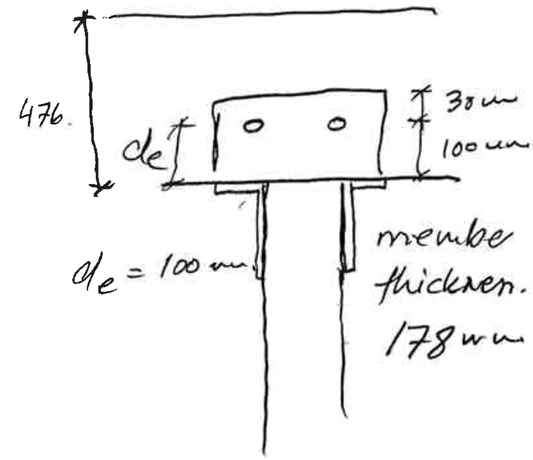
2) Service conditions → also <u>not</u> linearly related to P_r.

3) Size factor → k_{ZCq} is based on full height of column. Tables cannot be used when braced intermedially.

Connections: Bolt.

Design the bolted connection to resist wind uplift acting on the SCL header at a glulam column

D. Fir-L values are used.



Net uplift is 8.54 KPa. → $Q_f = 37.2$ KN.
$V_f = 18.6$ kN.

$K_D = 1.15$ (wind), $K_{SF} = 1.0$, $K_T = 1.0$.

Try two rows of 3/4" bolts.

$Q_r' n_s = 18.4$ kN (linearly interpolation for 178 mm from bolt selection tables WDM Section 7.3).

$n_F = 2$, $K' = 1.15$, $J_R = 1.0$ (WDM Section 7.3) 1 bolt pr row.

$Q_r = 18.4 \times 2 \times 1.15 \times 1.0$
$= 42.3 > 37.2$

Shear:

From beam selection table

$$V_r = 190 \text{ KN}.$$

Modifying for effective depth and load duration:

$$V_r = \frac{100}{476} \times 190 \times 1.15$$

$$= 45.9 \text{ KN} > 18.6 \text{ KN}.$$

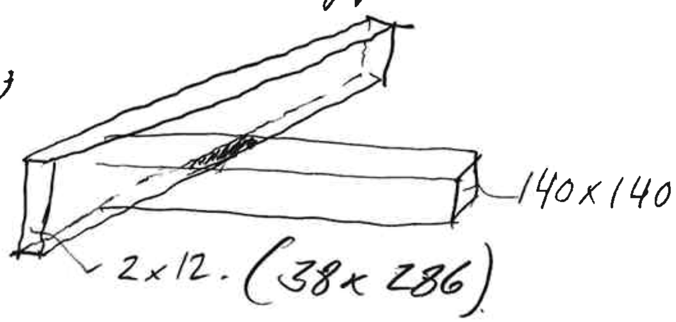
Bearing:

This deals with compression $\perp$ to grain.

The formula is:

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

ALL are familiar with calculating ϕF_{cp} and K_{zcp}. Typically, the bearing area is unknown, so it is either calculated or assumed and then verified. The bearing area is simply the area on which the member is supported. In the following case:



the area is:

$$38 \times 140 = 5320\ mm^2.$$

It is typical that the member's dimension decides the bearing area, which has to be checked.

K_B is the bearing factor. Fibres that under compression force, if the area is small enough can become in tention and can act to help resist the compression force. It is however conservative to assume $K_B = 1$ because any value higher than 1.0 will increase the resistance. K_B is also 1.0 in area of high bending stress, or at end of member (support).

In the WDM the formula is showed as a function of L_b: bearing length; but this is essentially the same as the above formula (where $q_r = \phi F_{cp} b K_B K_{zcp}$.

Example 1: Shear walls. WDM p. 480

Two storey office building, SPF @ 400.
Dry service.

$F_2 = 20\,kN$.

$F_1 = 60\,kN\,(40 + 20)\,kN$.

$q_2 = 0\,kN/m$ (roof).

Floor dead 0.5 kN/m (0.85 factor should be 0.9).

$q_w = 0.35\,kPa$.

depth of roof and floor framing = 0.3 m.

Second storey:

$V_r = 4.19\,kN/m$.

$V_{rg} = 0.98\,kN/m$.

Check for the need for holddown.

$H_s < 3.6\,m$

$V_r + V_{rg} \leq 8.3\,kN/m$.

V_r based on nail diameter $\leq 3.25\,mm$.

and edge panel nail spacing $\geq 100\,mm$

$$V_{hd} = \Sigma(V_r J_n J_{ub} + V_{rg})\,L_w$$

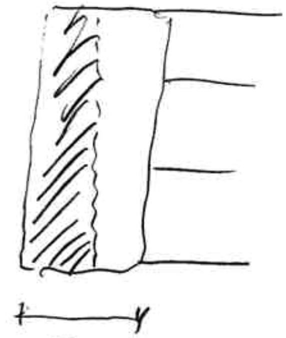
$$= (4.19 \times 1 \times 1 + 0.98) \times L_w$$

$$= 5.17\,L_w.$$

$P_t = 0$ does not support roof.
✓ should be 0.9.

$q_w = 3\,m \times 0.35\,kPa * 0.85$
$= 0.89\,kN/m$.

$$P_j = P_t + q_w \frac{L_w}{2}$$


Seg 1.

$J_{HD} = 1$ if holddown.

$$= \sqrt{1 + 2\frac{P_j}{V_{hd}} + \left(\frac{H_s}{L_w}\right)^2} - \frac{H_s}{L_w} \le 1.0$$

$$V_{rs} = V_{hd}\, J_{hd}$$

<u>Segment 1:</u> wind from North.

$L_w = 4\,m$.

$V_{HD} = 5.17 \times L_w = 5.17 \times 4$
$= 20.68\,kN$.

$P_t = 0$

$P_j = 0 + 0.89 \times \frac{4}{2} = 1.78\,kN$.

There is a holddown $J_{HD} = 1.0$.

$V_{rs} = 20.7 \times 1 = \underline{20.7\,kN}$

Wind from South:

$P_t = 0$

$P_j = 1.78$

No hold down $\rightarrow$ Calculate J_{hd}

$$J_{hd} = \sqrt{1 + 2\frac{1.78}{20.7} + \left(\frac{3}{4}\right)^2} - \frac{3}{4}$$

$$= 0.567$$

$$V_{rs} = 20.7 \times 0.567$$

$$= \underline{11.7 \text{ kN}}$$

$$F_j = F \times \frac{V_{rs}}{\Sigma V_{rs}}$$

$$R_j = \frac{F_j (H_s + d)}{L_w} - P_j$$

Segment 1, wind from North:

$$F_j = 20 \times \frac{20.7}{40.8} = 10.1$$

$$R_j = \frac{10.1(3 + 0.3)}{4} - 1.78$$

$$= 6.6 \text{ kN}$$

First Storey:

$$V_r = 6.12 \text{ kN/m}.$$

$$V_{ry} = 0.98 \text{ kN/m}.$$

$$V_{HD} = (6.12 \times 1 \times 1 + 0.98)\, L_w.$$
$$= 7.10\, L_w.$$

$$P_t = 0.5 \text{ kN/m} \times \text{Trib Wall} - R_j \text{ from 2}^{nd} \text{ story}$$

$$q_w = 0.89 \text{ kN/m}.$$

$$P_j = P_t + q_w \frac{L_w}{2}.$$

$J_{HD} = 1$ if holddown at bottom and top.

$= 1$ where P_t = positive

$= \dfrac{V_{hd} + P_t}{V_{hd}}$. only holddown at bottom.

$$V_{rs} = V_{hd} \times J_{hd}.$$

Seg. 1 wind from North.

$L_w = 4.$

$V_{hd} = 28.4.$

R_j from 2nd storey $= 6.6$ kN.

Trib. wall width. 2 m.

$P_t = 0.5 \times 2 - (6.6)$

$= -5.6$ kN.

$P_j = P_t + f_w \frac{L_w}{2}.$

$= -5.6 + 0.89 \times \frac{4}{2}$

$= -3.8.$

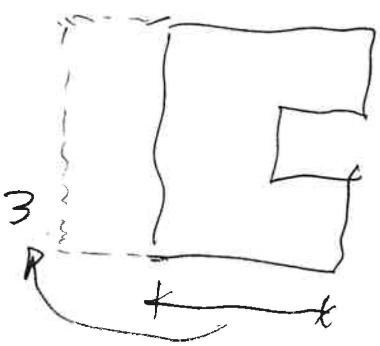
Hold down top & bottom $\rightarrow J_{HD} = 1.0.$

$V_{rs} = 28.4 \times 1 = 28.4.$

Wind from south:

$R_j = 3.24.$

Trib. wall length $= \frac{4}{2} + \frac{2}{2} = 3$



$P_t = 0.5 \times 3 - 3.24$

$= -1.74.$

$$P_j = -1.74 + 0.89 \times \frac{4}{2}$$
$$= 0.04$$

Holddown only bottom.

$$J_{HD} = \frac{28.4 + (-1.74)}{28.4}$$
$$= 0.939$$

$$V_{rs} = 28.4 \times 0.939$$
$$= \underline{\underline{26.6 \text{ kN}}}$$

<u>Chord members:</u>

$$P_f = T_f = F_j \frac{(H_s + d)}{h}$$

Segment 1 wind from North.

$$F_j = 10.1$$
$$h = 4 - 0.15 = 3.85$$
$$T_f = P_f = 10.1 \left(\frac{3 + 0.3}{3.85}\right)$$
$$= 8.7$$