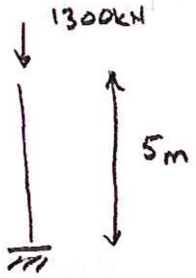
Question 1

1.a)



for a cantilever $L_{ef} = 2L = 2 \times 5\,m$
$= 10\,m$

choose a 305 × 305 section
S355 steel, $\gamma_m = 1.05$

$\lambda_o = \pi \left(\frac{E}{\sigma_y}\right)^{1/2} = \pi \left(\frac{210000}{355/1.05}\right)^{1/2} = 78.3$

in 305 × 305 series r_{yy} varies between 7.69 & 8.27 cm
∴ initially select $r_{yy} = 7.69$ cm (lowest value)

$\lambda = \frac{L_{ef}}{r} = \frac{10000}{76.9} = 130$

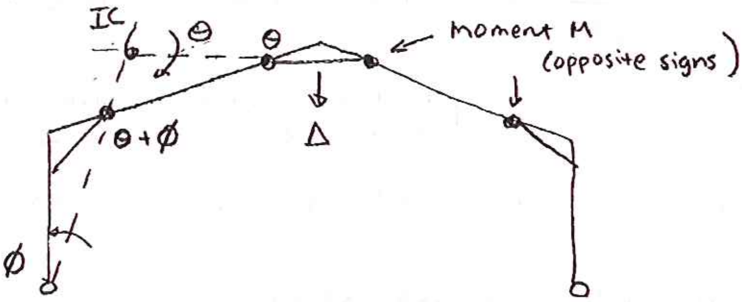
$\bar{\lambda} = \frac{\lambda}{\lambda_o} = \frac{130}{78.3} = 1.66 \rightarrow \chi \sim 0.3$

so $A_{req} \approx \frac{1300000}{(355/1.05) \times 0.3} = 12817\,mm^2$

select 305 × 305 × 118 UC

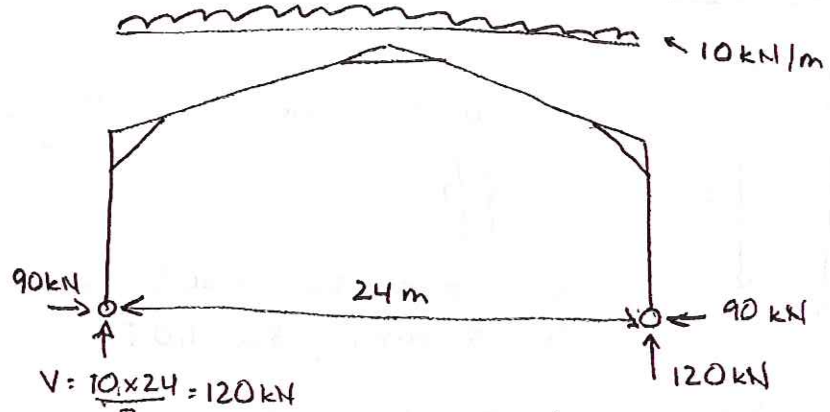
$\left\{ \begin{array}{l} A = 150\,cm^2,\ r_{yy} = 7.77\,cm \\ \bar{\lambda} = 1.64,\ \chi \approx 0.29 \\ P_{provided} = 0.29 \times \frac{355}{1.05} \times 15000 \\ \quad = 1471\,kN \end{array} \right.$

b i) consider mechanism

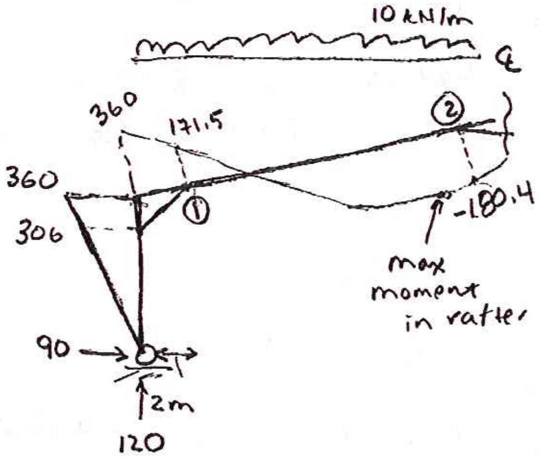


- postulate hinge locations
- find instantaneous centre
- consider a downwards displacement at A & find compatible rotations θ & ϕ
- use virtual work where the work done by the applied loads = energy dissipated $(M(\theta + \theta + \phi))$
- find M

b)ii)



$V = \frac{10 \times 24}{2} = 120\,kN$



RAFTER

M_{haunch} ① $= 90 \times 4.35 - 120 \times 2m + 10 \times 2^2/2$

$= 171.5$ kN.m

M_{haunch} ② $= 90 \times 5.94 - 120 \times 11 + 10 \times 11^2/2$

$= -180.4$ kN m

Choose stanchion with $M_p = 306$ kNm

$$Z_p = \frac{M_p}{\sigma_y} = \frac{306 \times 10^6}{275/1.05} = 1.168 \times 10^6\,mm^3 = 1168\,cm^3$$

say 356 x 171 x 67 (1211 cm^3, some allowance for axial force 120 kN)

also need to check axial interaction with bending

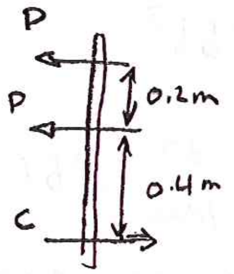
iii) Need a stiffener across stanchion at A and lateral restraint at corner A (3 compression forces meeting at a point). This could be achieved by tieing to longitudinal eaves member. Compression flange restraint required e.g. using purlins or roof cladding. Need to check overall frame is stable by providing bracing.

b) iii) continued

3 layers of bolts, each layer takes say $\frac{120}{3} = 40\,kN$ in Shear

<u>but</u> also a moment applied of 360kNm

assume top bolts all yielded : neglect bottom bolts and that the compression force acts at say 100mm above A



$P \times 0.6 + P \times 0.4 = 360\,kNm$

$P \approx 360\,kN$

one bolt on either side ∴

$P_{bolt} = 360/2 = 180\,kN$ → quite high, may need more bolts

Question 2

2 (a) $\Delta = \frac{5}{384} \cdot \frac{wL^4}{EI}$ $\qquad I = \frac{bh^3}{12}$

at buckling $\quad 0.88 \frac{wL^2}{8} = \frac{\pi E b^3 h}{24 L}$

$$\therefore \quad wL^3 = \frac{\pi}{2.64} \cdot E b^3 h$$

at deflection limit. $\quad \frac{L}{200} = \frac{5}{384} \cdot \frac{wL^4}{E b h^3} \cdot 12$

$$\therefore \quad wL^3 = \frac{32}{1000} \cdot E b h^3$$

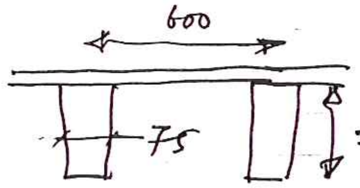
for buckling first $\quad \frac{\pi}{2.64} E b^3 h \leq \frac{32}{1000} E b h^3$

$$\therefore \quad \frac{b^2}{h^2} \leq \frac{0.032 \times 2.64}{\pi}$$

$$\frac{b}{h} \leq 0.164$$

[30%]

2 (b) (i)



$k_{mod} = 0.7$ from Data sheet

$$f_{m,d} = k_{mod}\, k_h\, k_{crit}\, k_{ls}\, f_{m,k} / \gamma_m$$

$$= 0.7 \times 1.0 \times k_{crit} \times 1.0 \times 24 / 1.3 = 12.9\, k_{crit}$$

Load on beam $= 3.5 \times 0.6 = 2.1$ kN/m

$$\sigma_{max} = \frac{wL^2}{8} \cdot \frac{6}{bh^2} = \frac{3 \times 2.1}{4 \times 75 \times 220^2} \cdot L^2 = 4.34 \times 10^{-7} L^2$$

MPa if L in mm

if $k_{crit} = 1$, $\quad f_{m,d} = 12.9$

when equal to σ_{max}, $\quad L = \sqrt{\frac{12.9}{4.34 \times 10^{-7}}} = 5,453$ m.

But we must check for LT buckling.

$$\sigma_{m.crit} = \frac{M_{crit} \cdot y}{I} = \frac{\pi}{L} \cdot \frac{Eb^3h}{24} \cdot \frac{1}{0.88} \cdot \frac{6}{\cancel{b}h^2}$$

$$= \frac{\pi}{L} \cdot \frac{Eb^2}{3.52\,h} = \frac{\pi \times 7.4 \times 10^3 \times 75^2}{3.52 \times 220 \cdot L}$$ (MPa if L in mm)

$$= \frac{169{,}000}{L}$$

Guess $L = 5{,}400$: $\sigma_{m,cr} = 31.27$

$$\lambda_{rel,m} = \sqrt{\frac{24}{31.27}} = 0.88$$

$$k_{cr} = 1.56 - 0.75 \times 0.88 = 0.902$$

$f_{m,d} = 11.64$ MPa $\quad L_{max} = \underline{5{,}178 \text{ mm}}$

guess $L = 5{,}200$ $\quad \sigma_{m,cr} = 32.5$ $\quad \lambda_{rel,m} = 0.859$

$$k_{cr} = 1.56 - 0.75 \times 0.859 = 0.916$$

[50%] $\quad f_{m,d} = 11.81$ MPa $\quad L_{max} = \underline{5{,}217 \text{ mm}}$

take $\underline{\underline{5.2 m}}$

(ii) <u>Bookwork</u> on $E_{0.05}$, k_h etc.

On deflection limit: $\frac{b}{h}$ is in fact $\frac{75}{220} = 0.34$ which is well above the limit calculated in (a). So Δ limit will occur first. But it still might be OK – much will depend on γ_f at ULS. Assume 2, deflection under 1.05 kN/m

$$\frac{5}{384} \cdot \frac{1{,}050 \cdot (5{,}200)^4 \times 12}{7.4 \times 10^3 \quad 75 \times 220^3} = 20.1 \text{ mm deflection}$$

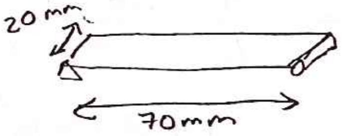
$$\frac{L}{200} = \frac{5200}{200} = 26 \text{ mm deflection. So OK.}$$

But if $\gamma_f \approx 1.5$, <u>not</u> OK.

[20%]

Question 4

4 a)



0.125 mm GFRP plies
balanced, symmetric

i) E_x required = 29000 MPa, 33% plies in ±45° directions

From E-glass/epoxy chart for Young's Modulus
x-axis co-ordinate = 33%
y-axis co-ordinate = 29 GPa
→ need 50% 0° plies
for a total of 100% this leaves 17% 90° plies

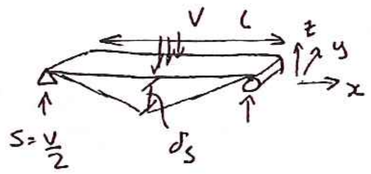
33% ± 45°, 50% 0°, 17% 90°

ii) from E-glass/epoxy chart for shear Modulus
x-axis co-ordinate = 33%
y-axis co-ordinate (from graph) = 7.4 GPa
→ G_{xy} = 7400 MPa

from E-glass/epoxy chart for Poisson's ratio
x-axis co-ordinate : 33%
follow curve for 50% 0° plies to find
y-axis co-ordinate = 0.25
→ ν_{xy} = 0.25

from E-glass/epoxy chart for Young's Modulus
x-axis co-ordinate = 33%
follow curve for 17% 0° plies (for E_y direction)
y-axis co-ordinate = 17 GPa
→ E_y = 17000 MPa

4 b)



at neutral axis $\tau = \frac{3}{2}\frac{S}{bh}$

constant shear force in shear span

$\tau = G\gamma$

$S = V/2$

$\gamma = \frac{\delta_s}{L/2}$

$\tau = \frac{3}{2}\frac{S}{bh}$

combine equations to give

$$\delta_s = \frac{\tau}{G} \cdot \frac{L}{2} = \frac{1}{G}\left(\frac{3}{2} \frac{V}{2 \cdot bh}\right)\frac{L}{2}$$

$$= \frac{3VL}{8bhG}$$

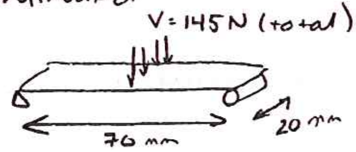
Need to consider G_{yz}, there are no vertical fibers in this direction so the stiffness will be dominated by the properties of the matrix which tends to have a low shear stiffness

∴ shear deflections may be an issue

need to consider ways of designing the system to increase resistance to transverse shear.

4 a) continued

iii)



$e_T = 0.3\%$, $e_c = 0.7\%$ } e_T is more critical

$$M = \frac{V.L}{4}$$

$$N_x = \frac{M y t}{I} = \frac{VL}{4} \cdot \frac{t/2 \cdot t}{b t^3} \cdot 12 = \frac{3}{2}\frac{VL}{bt}$$

check strain in x-direction

$$\varepsilon_x = \frac{1}{E_x t}\left(N_x - \cancel{N_y}^{\,0} \nu_{xy}\right)$$

at failure $\varepsilon_x = e_T = 0.3\%$

$$e_T = \frac{1}{E_x t}\left(\frac{3}{2}\frac{VL}{bt}\right) \quad \therefore \quad t = \sqrt{\frac{3VL}{2E_x b e_T}}$$

$$t_{min} = \sqrt{\frac{3 \times 145 \times 70}{2 \times 29000 \times 20 \times 0.3/100}} = 2.96\ mm$$

check strain in y-direction

$$\varepsilon_y = \frac{1}{E_y t}\left(\cancel{N_y}^{\,0} - N_x \nu_{xy}\right)$$

$$e_T = \frac{1}{E_y t}\left(\frac{3}{2}\frac{VL}{bt}\right)\nu_{xy} \quad \therefore \quad t = \sqrt{\frac{3VL\nu_{xy}}{2E_y b e_T}}$$

$$t_{min} = \sqrt{\frac{3 \times 145 \times 70 \times 0.25}{2 \times 17000 \times 20 \times 0.3/100}} = 1.93\ mm$$

$\therefore$ ε_x controls, need a minimum thickness of 3 mm = 24 × 0.125 mm plies

lay up should be symmetric & balanced with

50% 0° → 12 plies

17% 90° → 4 plies

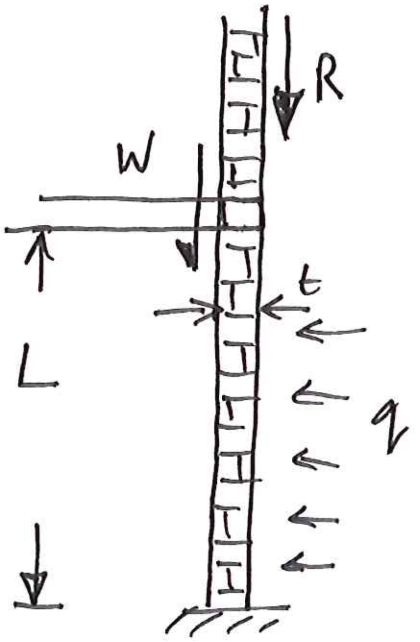
33% ±45° → 8 plies

QUESTION 3

Masonry Question.

Three cases to consider. Less concerned with the equations they derive than with the logic of understanding and explaining the load paths.

(a) The external wall is carrying an axial load and also carrying the wind load.



Wall must be designed to carry the total axial load

$P = W + R$

Plus the lateral wind pressure q.

The likely modes of failure will be either strength, so they will need a combination of a stress P/A from the axial load, plus a bending stress which will be of the form

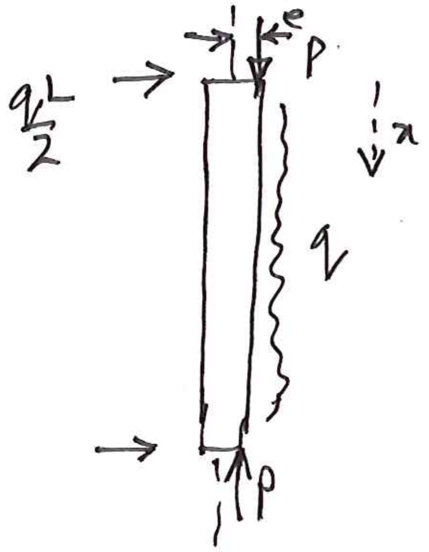
$M = \frac{wL^2}{8}$ $\quad \sigma = \frac{My}{I} = \frac{M \cdot 6}{t^2}$ (per unit width).

The alternative ~~fa~~ failure mode is likely to be

buckling. This will take the form

$$P_{crit} = \frac{\pi^2 EI}{L^2}$$ plus some allowance for eccentricity of the axial load and the effect of the wind load. Lecture notes give $e_a = t\left(\frac{1}{2400}\left(\frac{h}{t}\right)^2 - 0.015\right)$

Good if they quote but I really want a discussion that imperfections are possible



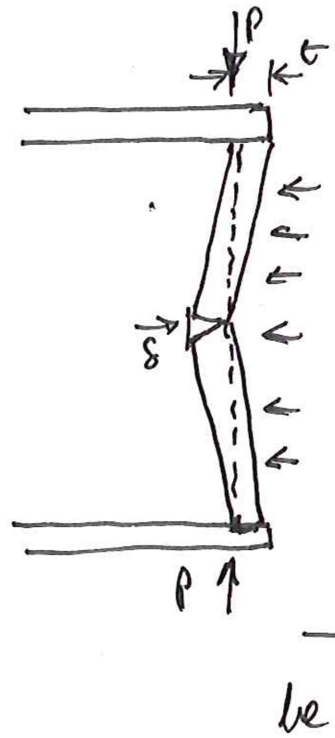
$$\text{Moment} = \frac{qL}{2}\cdot x + Pe + P \cdot v$$

v = lateral displacement

$$= EI\frac{d^2v}{dx^2}$$

$\Rightarrow$ Critical load and stresses..

(b)



Equilibrium for half wall (from lecture notes)

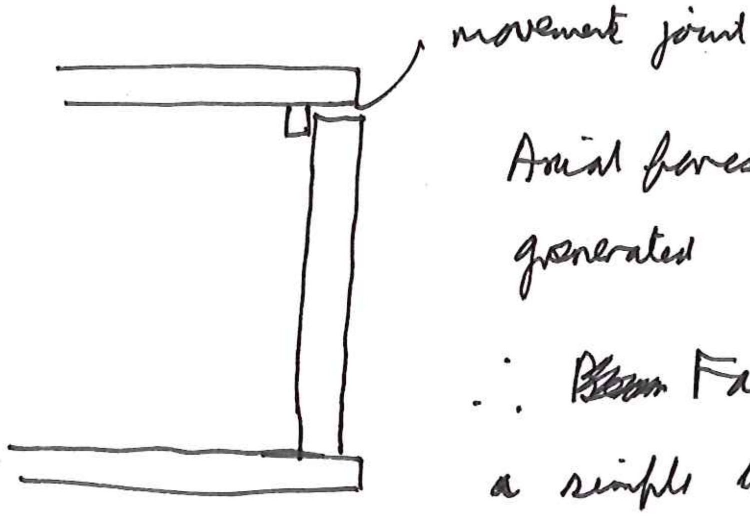
$$P(t-\delta) + q\left(\frac{h}{2}\cdot\frac{h}{4}\right) - \frac{hq}{2}\left(\frac{h}{2}\right) = 0$$

$\delta \ll t$

$$\Rightarrow q = \frac{8Pt}{h^2}$$

This will give P which can then be compared with available strength

(c)



Axial forces can now not be generated

∴ ~~Beam~~ Facade behaves as a simple wall SS at top and bottom.

Need to consider stresses due to weight of wall (only) + bending effect from the wind

Compare with allowable stresses.

Comparison of methods.

(c) is simplest to construct but cannot generate axial forces. Wall will probably need to be thicker to generate enough axial force to counteract bending.

(b) Better than (c) because axial forces ~~can~~ can be generated. Would be built after internal frame. Difficulty of ensuring tightness of fit so axial force is generated

(a) Since wall has to carry all the loads a substantially thicker wall is likely to be needed.

⇒ (b) is probably better.