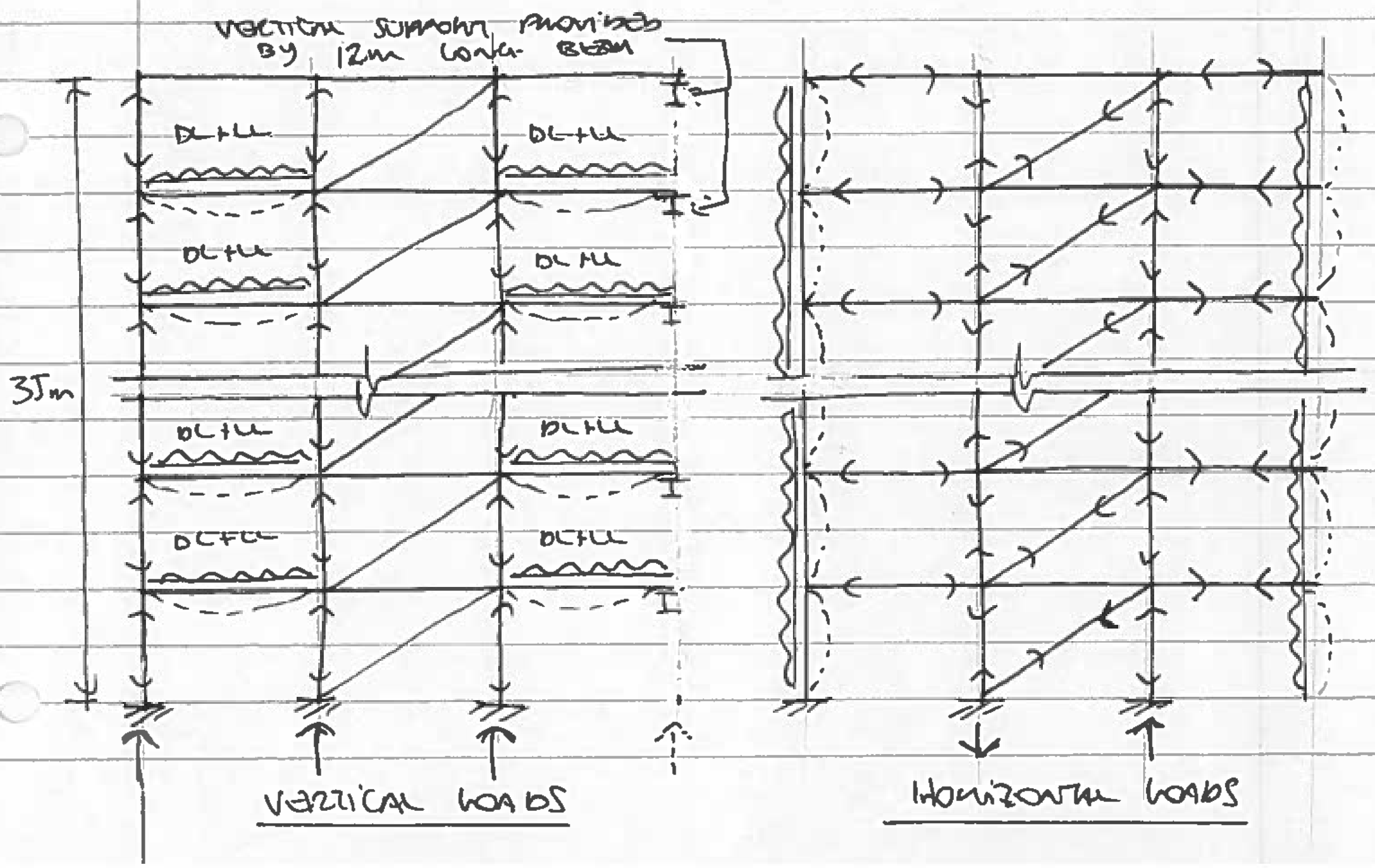
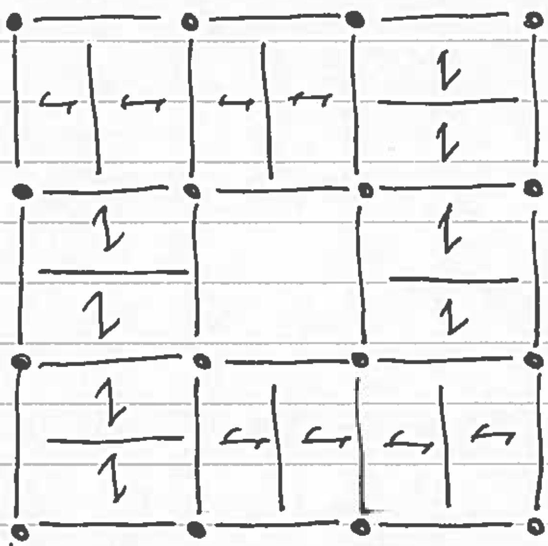
1 a) VERTICAL LOADS DUE TO SLAB SELF-WEIGHT AND IMPOSED LOADS ARE TRANSFERRED IN FLEXURE IN THE SLABS → THE SLABS ARE SUPPORTED BY THE SECONDARY STEEL BEAMS → IN TURN SUPPORTED BY THE PRIMARY STEEL BEAMS → WHICH TRANSFER THE VERTICAL LOADS TO THE COLUMNS AT EACH FLOOR → DOWN TO THE FOUNDATIONS.

HORIZONTAL LOADS ARISING FROM WIND PRESSURE ARE TRANSMITTED IN FLEXURE BY THE CLADDING INTO THE HORIZONTAL FLOOR PLATES → THE FLOOR PLATES ACT AS STIFF DIAPHRAGM TO TRANSFER THESE LOADS TO THE BRACED CORE → THE CORE ACTS AS A VERTICAL CANTILEVER ANCHORED AT GROUND LEVEL TO RESIST THESE HORIZONTAL LOADS BY MEANS OF AXIAL FORCES (TENSION / COMPRESSION) IN THE MEMBERS OF THE BRACED CORE → THE AXIAL FORCES AND HORIZONTAL SHEAR ARE TRANSMITTED TO THE FOUNDATIONS



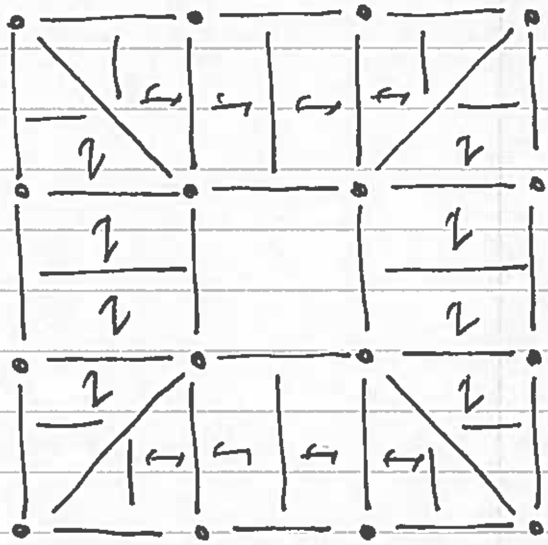
TWO POSSIBLE LAYOUTS:

1b)



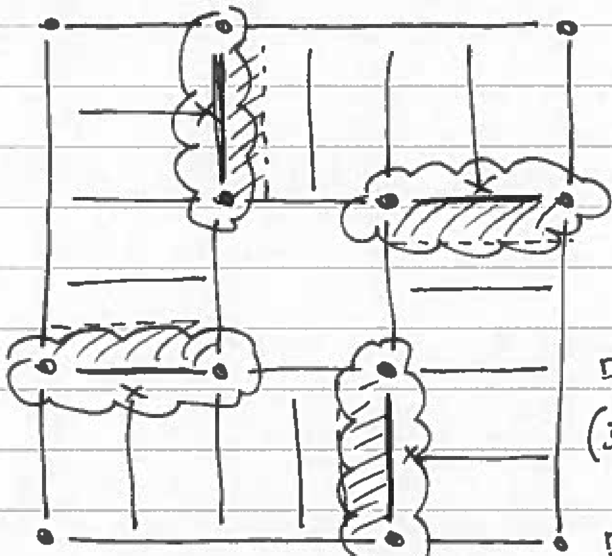
(1) COLUMNS AT EVERY 6×6m GRID POSITION.

BEAMS AS SHOWN OR SIMILAR.



(2) RADIATING BEAMS.

1c)



HEAVIEST LOADED 6m BEAMS SHOWN

UDL + POINT LOAD @ MID-SPAN.

DESIGN LOAD @ ULS:

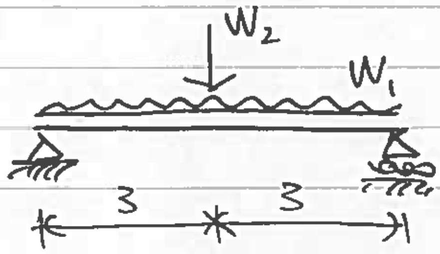
$(3.6 \times 1.4) + (2.5 \times 1.6) = 9.04\,kN/m^2$

DESIGN LOAD @ SLS:

$(3.6 \times 1.0) + (2.5 \times 1.0) = 6.1\,kN/m^2$

STRENGTH CHECK: $Z_p > M/\sigma_y$ $\left(W_1 L/8 + W_1 L/4\right)$

DEFLECTION CHECK: $\frac{l}{200} > \delta_{max}$ $\left(\frac{5 W_1 L^3}{384 EI} + \frac{W_2 L^3}{48 EI}\right)$



	W_1	W_2
ULS	81.36 kN	81.36 kN
SLS	54.9 kN	54.9 kN.

@ ULS

$M_{max} = 81.36 \times 6 / 8 + 81.36 \times 6 / 4 = 183 \text{ kNm}$

$Z_p > \left(183 \times 10^6 / 355\right) \times 10^{-3} = 515.5 \text{ cm}^3$

$\therefore$ SELECT UB 254 x 146 x 43 $\left(Z_p = 566 \text{ cm}^3\right)$

CHECK FOR SELF WT. OF BEAM = 46 kg/m ≐ 0.45 kN/m

$\therefore W_1 = 81.36 + 3.78 = 85.14 \text{ kN}$

$W_2 = 81.36 + 1.89 = 83.25 \text{ kN}$

2

$\therefore$ UB 254 x 146 x 43 is SUFFICIENT.

@ SLS

$W_1 = 54.9 + 2.7 = 57.6 \text{ kN}$

$W_2 = 54.9 + 1.35 = 56.25 \text{ kN}$

$$\therefore \delta_{max} = \frac{5 \times 57.6 \times 10^3 \times 6000^3}{384 \times 210 \times 10^3 \times 6544 \times 10^4} + \frac{56.25 \times 10^3 \times 6000^3}{48 \times 210 \times 10^3 \times 6544 \times 10^4}$$

[N; mm] [N; mm]

2

$= 30.2 \text{ mm} \simeq l/200$

$\therefore$ UB 254 x 146 x 43 is SATISFACTORY

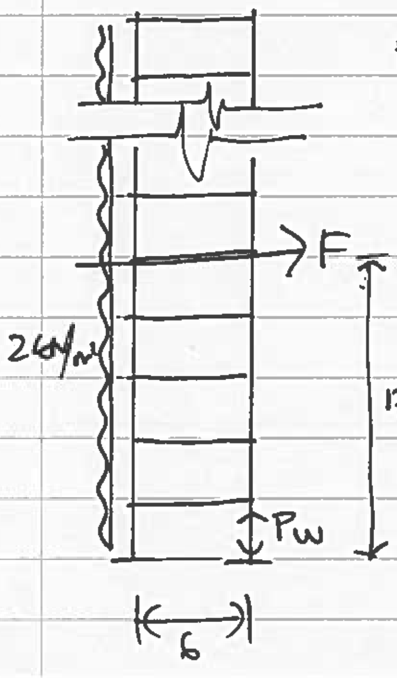
1d) AXIAL FORCE IN COLUMN (P_{TOT}) = AXIAL FORCE DUE TO VERTICAL LOAD (P_v)
+ AXIAL FORCE DUE TO WIND LOAD (P_w)

$P_v = \left[1.2 \times 6.1 \text{ kN/m} \times \left(6^2 - 3^2\right)\right] + \left[0.45 \text{ kN/m} \times 1.2 \times 3\text{m} \times 5.5\right]$

$= 206.55 \text{ kN} \times 10$ FLOORS

$= 2065.5 \text{ kN}$

1



$$F = \left(2\,kN/m^2 \times 35m \times \frac{18m}{2}\right) \times 1.2 = 756\,kN$$

$$\therefore P_W = 756 \times 17.5/6 = 2205\,kN$$

$$P_{TOT} = 2205 + 2066 = 4271\,kN$$

Try UC 254×254×132

$$\lambda = 3500/66.9 = 52.3$$

$$\lambda_1 = \pi\sqrt{(210\times10^3/355)} = 76.4$$

$$\bar{\lambda} = 52.3/76.4 = 0.68$$

$$r/y = 66.9/(278.3/2) = 0.48 \text{ (curve c)}$$

3 $\chi = 0.72$ (FROM CHART)

$$\therefore P_{max} = 355 \times 168\times10^2 \times 0.72 = 4294\,kN > P_{TOT}$$

CHECK UPDATED P_{TOT} TO INCLUDE COLUMN SELF WT.

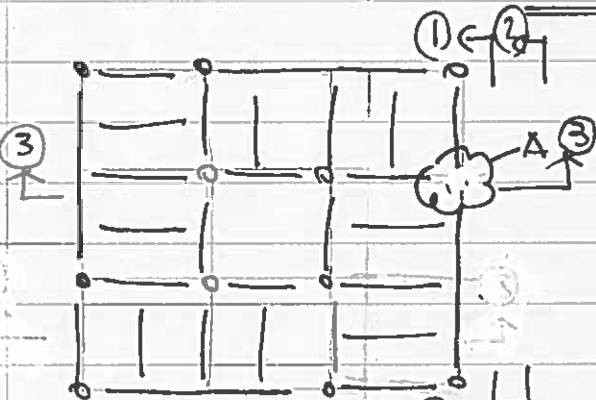
$$132\,kg/m \times 9.81 \times 35 \times 1.2 = 54.4\,kN$$

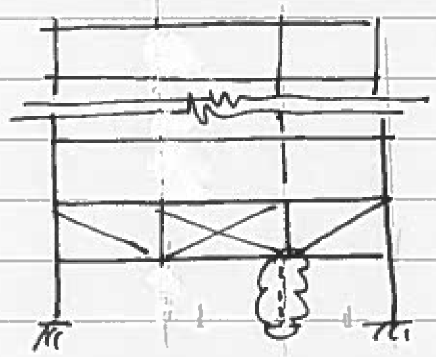
$$\therefore P_{TOT} = 4271 + 54.4 = 4325\,kN$$

THIS IS ONLY MARGINALLY LARGER THAN P_{MAX}

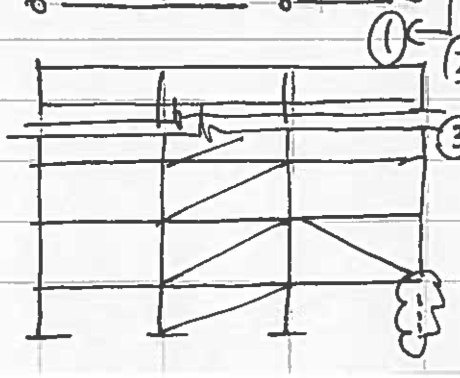
∴ UC 254×254×132 is SATISFACTORY

1(e)

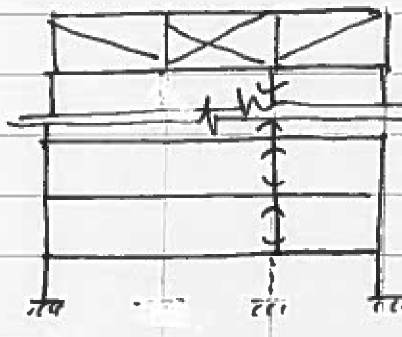




(1) TRUSS IN FACADE AT FIRST FLOOR LEVEL



(3) INTERNAL TRUSS @ 1ST FLOOR

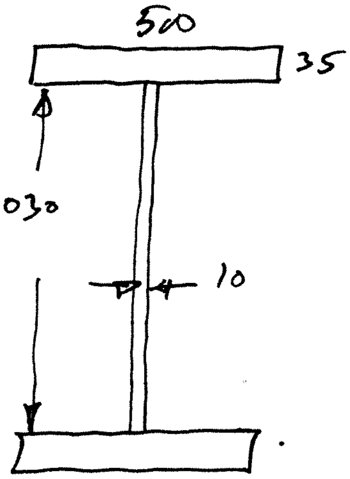


(2) TRUSS AT/ABOVE 10TH FLOOR AND HANGER TO 1ST FLOOR LEVEL

2

Q2

(a)



$$Z_p = 515 \times 10 \times 515 + 500 \times 35 \times 1065 = 21,290 \ \text{cm}^3$$

$$M_{resist} = \frac{355}{1.1} \times 21,290 \times 10^{-3} \ \text{kNm} = 6,870 \ \text{kNm}$$

[20%] $\therefore$ allowable $W = \frac{6,870 \times 8}{26} = \underline{2,113 \ \text{kN}}$

(b) Firstly, shear stress on web plate near supports

$$\tau \approx \frac{V}{dt} = \frac{1050 \times 10^3}{10 \times 1030} = 102 \ \text{MPa}$$

from Data Sheet, plastic $q_w = 0.6 \times 355 = 213$ MPa. So no danger of yield

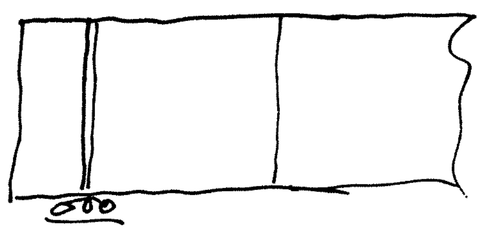
But buckling? If no stiffeners, $a/d \to \infty$, Data Sheet

$$q_{cr} \approx 0.75 \times \left(\frac{1000 \times 10}{1030}\right)^2 = 71 \ \text{MPa} < \tau$$

so stiffeners required. Try $a/d = 1$, $q_{cr} \approx 165$ MPa,

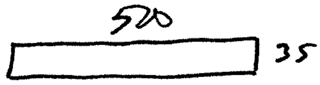
probably ok even allowing for imperfections, interaction etc.

Second bearing: force 1050 kN, at yield requires 300 mm length of 10 mm plate, even without crippling or buckling.
So probably need one stiffener at support :—



Midspan — check local buckling, b/t ratios for plastic

[30%] behaviour; welds take longitudinal shear, then at supports.

(c) 

$$P_{cr} = \frac{\pi^2 EI}{l^2} \quad \text{etc.}$$

For full yield of compression member, Data Sheet, $\bar{\lambda} \le 0.2$

$$\lambda_0 = \pi\sqrt{E/\sigma_y} = 76.4 \qquad \therefore \lambda = 0.2 \times 76.4 = 15.3$$

$$r = \sqrt{\frac{I}{A}} = \frac{d}{\sqrt{12}} = 144 \text{ mm} \quad (\text{for lateral buckling})$$

$$\therefore \text{ max. } l = 15.3 \times 144 = \underline{\underline{2.2 \text{ m}}}$$

This is quite close together! Could be awkward to provide, unless a deck or floor slab helps. [30%]

(d) Could prevent lateral-torsional buckling — but web plates even thinner than before so would need more stiffening near supports. And how do you practically get such stiffeners inside the box?

C_w negligible: so $M_{cr} = \frac{\pi}{L}\sqrt{EI_{yy} . GJ}$ for uniform moment

$$I_{yy} = \frac{7 \times 50^3}{12} + 2 \times 103 \times 0.5 \times 24.7^2 = 136,000 \text{ cm}^4$$

$$\therefore M_{cr} = \frac{\pi}{26}\sqrt{210 \times 10^9 \times 136,000 \times 10^{-8} \times 81 \times 10^9 \times 245,000 \times 10^{-8}} = 28,800 \text{ kNm}$$

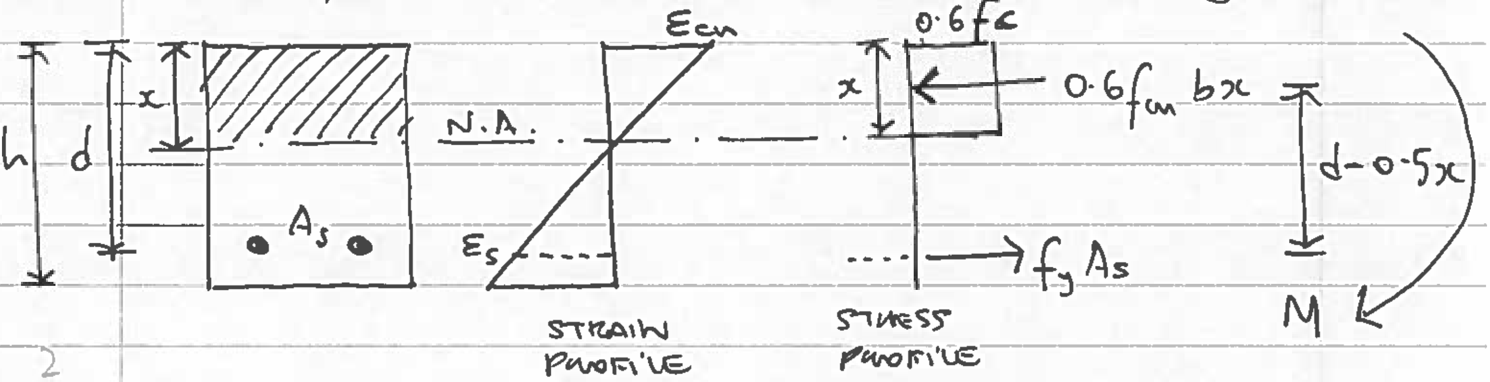
so max. M at midspan say $\frac{1}{0.88} \times 28,800 = 32,800$ kNm

$$\therefore \bar{\lambda} = \sqrt{\frac{6,870}{32,800}} = 0.46$$

[Detailed calculation not really needed]

[20%] From charts not much reduction in strength at midspan due to LT buckling — but problems at supports!

3a) BENDING MOMENT CAPACITY: IF THE BOND BETWEEN STEEL REINFORCEMENT AND CONCRETE IS GOOD, CURVATURE IN THE BEAM WILL RESULT IN A LINEAR VARIATION OF STRAIN THROUGH THE SECTION (PLANE SECTIONS REMAIN PLANE). CONCRETE HAS NEGLIGIBLE TENSILE CAPACITY, THEREFORE IN A SINGLY REINFORCED BEAM, THE STEEL PROVIDES THE LONGITUDINAL TENSILE RESISTANCE, WHILE THE CONCRETE
2 PROVIDES THE LONGITUDINAL COMPRESSIVE RESISTANCE. THE LEVER ARM BETWEEN THESE TWO RESISTANCES (FORCES) IS THE BENDING MOMENT CAPACITY.



2

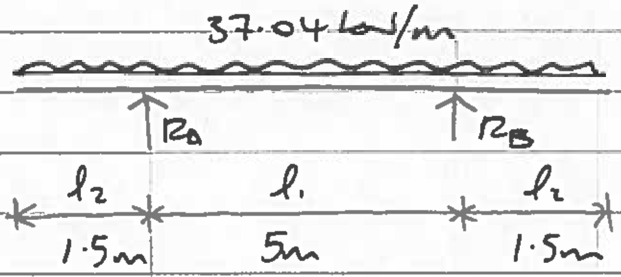
$$\therefore\ M = \underbrace{A_s f_y (d - 0.5x)/\gamma_s}_{\text{GOVERNED BY STEEL; MOMENTS ABOUT STEEL.}} \ \not> \ \underbrace{0.225 f_{cu} b d^2/\gamma_c}_{\text{GOVERNED BY CONCRETE COMPRESSIVE FAILURE; MOMENTS ABOUT CONCRETE } (x = d/2)}$$

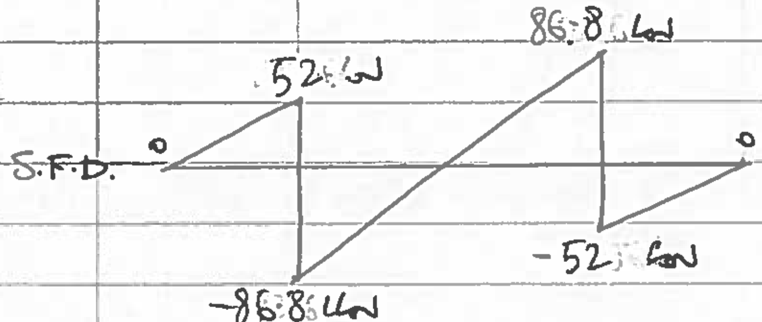
b i) DESIGN LIVE LOAD = 20 kN/m × 1.6 = 32 kN/m

DESIGN DEAD LOAD = 24 kN/m³ × 0.4 × 0.2 × 1.4 = 2.69 kN/m

34.69 kN/m

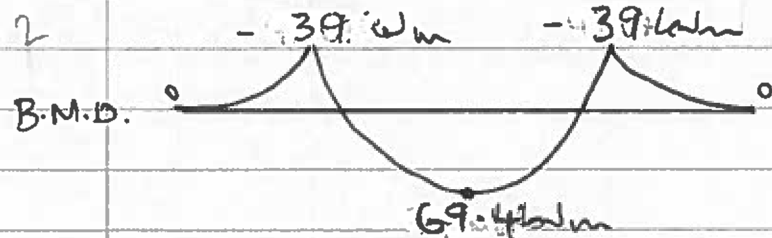
3D3/2017/3/2





$R_A = R_B = 34.69 \times 8 / 2$

$= 138.8\,kN$



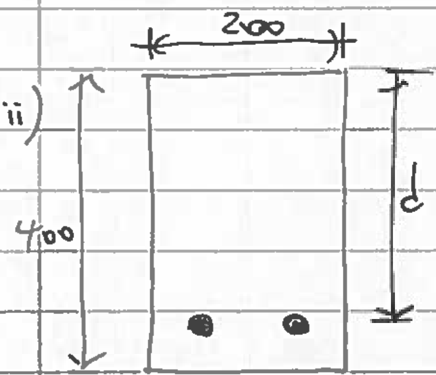
$M_A = M_B = -34.69 \times 1.5^2/2$

$= -39\,kNm$

$M_{max} = \dfrac{34.69 \times 5^2}{8} - 39$

$= 69.4\,kNm$

bii)



$f_{cu} = 40\,MPa$; $f_y = 460\,MPa$

$\gamma_c = 1.5$; $\gamma_s = 1.15$

Cover $= 40mm$; $\phi_l = 25mm$; $\phi_s = 12mm$

$\therefore d = 400 - (40 + 12 + \frac{25}{2}) = \underline{335.5mm}$

$M_u = 0.225 f_{cu} b d^2 / \gamma_c$

$= 0.225 \times 40 \times 200 \times 335.5^2 / 1.5$

$= 135\,kNm > M_{max}$

$\therefore$ NO COMPRESSION STEEL REQUIRED.

CALCULATE A_s :

$M_u = \dfrac{A_s f_y}{\gamma_s}(d - 0.5x)$ — (1)

$x/d = f_y A_s \gamma_c / 0.6 f_{cu} b d \gamma_s$

$\therefore x = 460 A_s \times 1.5 / 0.6 \times 40 \times 200 \times 1.15 = 0.125 A_s$

SUBSTITUTE INTO (1):

$$M_u = \frac{460 A_s}{1.15}\left(335.5 - \frac{0.125 A_s}{2}\right)$$

$$\therefore M_u = 400 A_s (335.5 - 0.0625 A_s) \quad - (2)$$

FOR MAX. SAGGING MOMENT $M_u = M_{max} = 69.4$ kNm

$$\therefore (2) \rightarrow 25 A_s^2 - 134200 A_s + 69.4 \times 10^6 = 0$$

$$\therefore A_s = \frac{134200 \pm \sqrt{134200^2 - (4 \times 25 \times 69.4 \times 10^6)}}{2 \times 25}$$

$$= 579.8 \text{ mm}^2$$

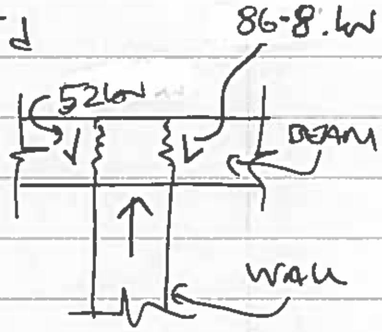
1 No 25mm ϕ BAR = 491 mm²

3 ∴ PROVIDE 2T25 (982 mm²) → [CHECK $\frac{A_s}{bd}$ = 1.46% ∴ OK]

NOTE: SINCE T25 IS MINIMUM BAR DIAMETER PROVIDE 2T25 IN HOGGING (TOP) STEEL OVER SUPPORTS.

SHEAR REINFORCEMENT:

TAKE STRUT ANGLE = 45°, s = 0.75d



$$V_{Rd,c} = A_{sv} f_y (0.9d)(\cot\theta) / s\,\gamma_s$$

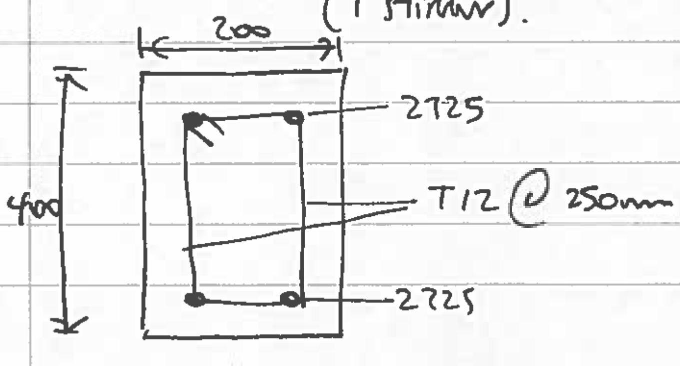
1

$$\therefore A_{sv} = \frac{86.8 \times 10^3 \times 0.75 \times 1.15}{460 \times 0.9 \times \cot 45°}$$

$$= 180.8 \text{ mm}^2$$

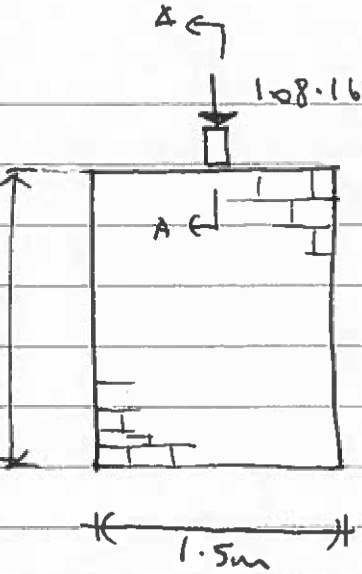
[NOTE $V_{Rd,max} >> V_{Rd,s}$ (CONCRETE STRUT) (STEEL STIRRUP)]

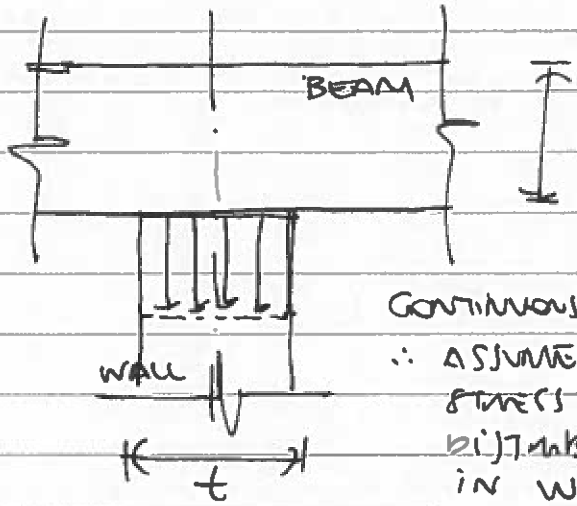
4 ∴ PROVIDE 2T12 LEG (216 mm²) (1 STIRRUP).



2 NO SIGNIFICANT OPPORTUNITY FOR CURTAILMENT ALONG LENGTH ∴ U/C THIS THROUGHOUT 8m LENGTH.

biii)





CONTINUOUS BEAM
∴ ASSUME UNIFORM STRESS DISTRIBUTION IN WALL
$(e = 0.05t)$

1 2 POSSIBLE FAILURE MODES: (1) CRUSHING UNDER BEAM
(2) BUCKLING OF WALL

CHECK CRUSHING:

1 $P = f_k bt / \gamma_m$

$\therefore t = P\gamma_m / f_k b = 138.8 \times 10^3 \times 3.5 / 8 \times 200 = 303.6\text{mm}$

CHECK BUCKLING:

1 $P = \beta f_k l t / \gamma_m \quad : \quad h_{ef} / t_{ef} = \dfrac{3000}{303.6} = 9.88$

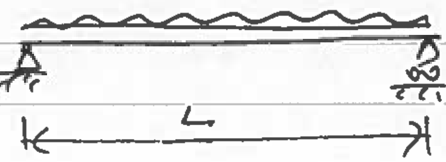
$\therefore \beta \approx 0.87$ (FROM CHART)

$\therefore P = 0.87 \times 8 \times 1500 \times 236.6 / 3.5$

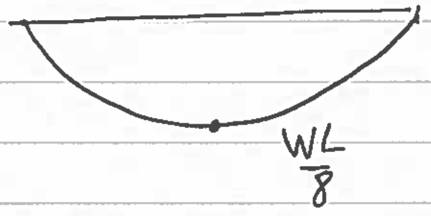
$= 705.7\text{kN} > 138.8\text{kN}$

$\therefore$ CRUSHING GOVERNS $\Longrightarrow$ $t = 236.6\text{mm}$

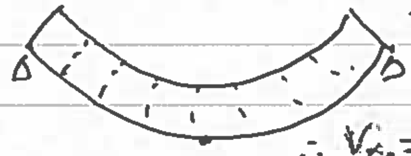
4a)



BMD



→



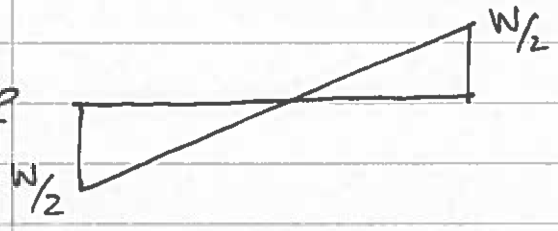
DEFLECTIONS

$$v_f = \frac{5WL^3}{384EI}$$

$$\therefore v_f = \frac{5WL^3}{32Ebh^3}$$

2

SFD



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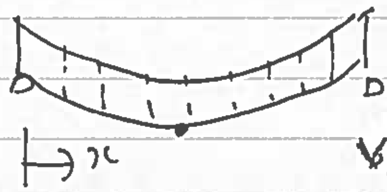
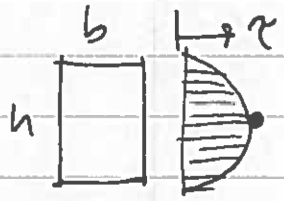


$v_s = ?$

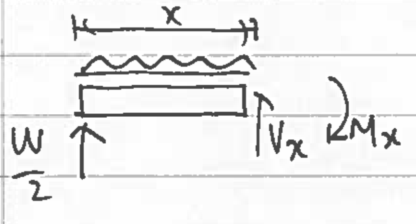
TOTAL DEFLECTION $v_{TOT} = v_f + v_s$

3 $$\tau_{NA} = \frac{VA\bar{y}}{Ib} = \frac{V \times h/2 \times b \times h/4}{\frac{bh^3}{12} \cdot b} = \frac{3V}{2bh}$$



HOW DOES THIS VARY ALONG SPAN L?

→ CONSIDER FREE BODY DIAGRAM.

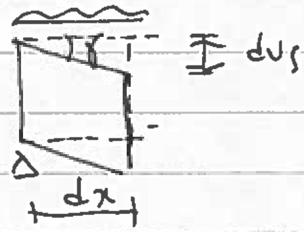


$$\Rightarrow V_x = \left(-\frac{W}{2} + \frac{Wx}{L}\right)$$

2 $$\therefore \tau_{NA}(x) = \frac{3}{2bh}\left(-\frac{W}{2} + \frac{Wx}{L}\right) = \gamma_{NA} G \qquad -(1)$$

ALSO

$$dv_s = \gamma_{NA}\, dx \qquad -(2)$$



2 $\therefore$ SUBSTITUTING FOR γ_{NA} INTO (1) AND (2):

$$dv_s = \frac{3}{2Gbh}\left(-\frac{W}{2} + \frac{Wx}{L}\right) dx$$

$$V_s = \int \frac{3}{2Gbh}\left(-\frac{W}{2} + \frac{Wx}{L}\right) dx$$

$$= \frac{3}{2Gbh}\left(-\frac{Wx}{2} + \frac{Wx^2}{2L}\right) + A$$

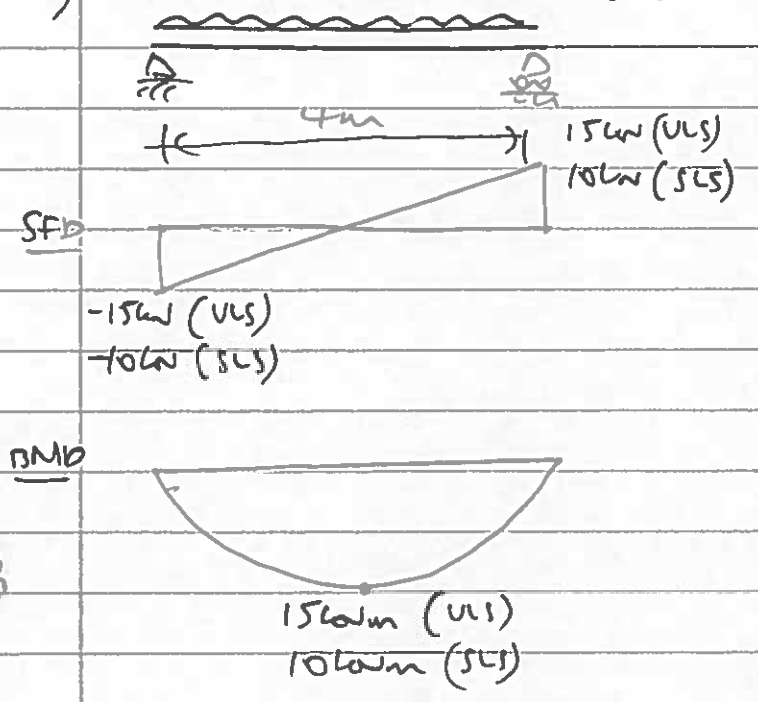
At $x = 0$; $V_s = 0$ $\therefore$ $A = 0$

$$V_s = \frac{3W}{4Gbh}\left(-x + \frac{x^2}{L}\right)$$

At $x = L/2$ $\rightarrow$ $V_s = \frac{3WL}{16\,Gbh}$

2 $\therefore$ $V_{Tot} = \frac{5WL^3}{32\,Ebh^3}\left[1 + \frac{6Eh^2}{5GL^2}\right]$

b.i) $W = 30kN$ (ULS) $= 20kN$ (SLS)



C24: $f_{mk} = 24MPa$

$f_{vk} = 2.5MPa$

$E_{0.05} = 7.4\,GPa$

$G_{mean} = 0.69\,GPa$

$\rho = 420kg/m^3$

$\delta_{max} = \frac{4000}{200} = 20mm$

FLEXURAL STRENGTH: $f_{md} = f_{mk}/\gamma_M = \frac{24}{1.3} = 18.4\,MPa$

$$f_{md} \geq \frac{M}{Z} = \frac{6M}{bh^2} \;\therefore\; h \geq \sqrt{\frac{6M}{bf_{m,d}}} = \sqrt{\frac{6\times 15\times 10^6}{200\times 18.4}} = \underline{156\,mm}$$

DEFLECTION : $V_{TOT} = \frac{5WL^3}{32Ebh^3}\left(1 + \frac{6Eh^2}{5GL^2}\right)$

$\left(V_{TOT} = \frac{4m}{20} = \right) \rightarrow 0.02 = \frac{1.35 \times 10^{-4}}{h^3}\left(1 + 1.19h^2\right)$

$0.02h^3 - 1.61 \times 10^{-4} h^2 - 1.35 \times 10^{-4} = 0$

SOLVING CUBIC $\Rightarrow h = 0.192m$

$= 192mm$

DEFLECTION : $V_{max} = 4m/200 = 0.02m$

$$V_{TOT} = \frac{5WL^3}{32Ebh^3}\left(1 + \frac{6Eh^2}{5GL^2}\right)$$

$$0.02 = \frac{1.35 \times 10^{-4}}{h^3}\left(1 + 1.19h^2\right)$$

$$0.02h^3 - 1.61 \times 10^{-4} h^2 - 1.35 \times 10^{-4} = 0$$

SOLVING CUBIC $\Rightarrow h = 0.192m$

$\therefore \underline{h = 192mm}$

$\Rightarrow$ **DEFLECTION GOVERNS $h = 192mm$**

SHEAR STRENGTH : $f_{vd} = f_{vk}/\gamma_m = \dfrac{2.5}{1.3} = 1.92\ \text{MPa}$

$f_{v,d} \geq \dfrac{3V}{2bh} \quad \therefore \ h \geq \dfrac{3V}{2bf_{v,d}} = \dfrac{3 \times 15 \times 10^3}{2 \times 200 \times 1.92} = \underline{58.5\ \text{mm}}$

DEFLECTION : $v_{TOT} = \dfrac{5WL^3}{32Ebh^3}\left(1 + \dfrac{6Eh^2}{5GL^2}\right)$

RE-ARRANGE INTO CUBIC FOR h AND SOLVE

OR CHECK V_{TOT} FROM $h = 256\ \text{mm}$

NOTE SLS

$V_{TOT} = \dfrac{5 \times (20 \times 10^3)^3}{32 \times 7.4 \times 10^3 \times 256^3}\left(1 + \dfrac{6 \times 7.4 \times 10^3 \times 256^2}{5 \times 0.69 \times 10^3 \times 4000^2}\right)$

3

$= 10.07\ (1 + 0.05)$

$= 10.57\ \text{mm} < 20\ \text{mm} \quad \therefore$ 256mm is SUFFICIENT

→ FIRE GOVERNS $h = 256\ \text{mm}$

b ii) ASSUMPTIONS:

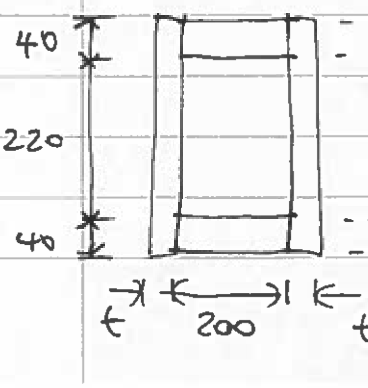
(1) TIMBER IS LOCATED IN FLANGES ∴ SHEAR DEFORMATIONS ARE NEGLIGIBLE

(2) ASSUME FULL COMPOSITE ACTION i.e. NO SHEAR DEFORMATIONS IN ADHESIVE BONDING TIMBER TO GLASS

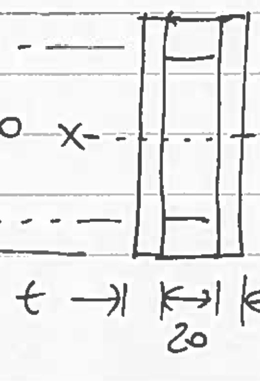
2

(3) DESIGN GLASS ON THE BASIS OF MAXIMUM TENSILE STRESS AT MID-SPAN i.e. DISREGARD STRESS VARIATION ALONG SPAN AND ASSUME $k_A = 1.0$

(4) ASSUME EDGE STRENGTH OF GLASS IS SAME AS SURFACE STRENGTH.



$\dfrac{E_{TIMBER}}{E_{GLASS}} = \dfrac{7.4\ \text{GPa}}{74\ \text{GPa}} = 10$



GLASS TRANSFORMED SECTION

2 $I_{xx} = (20 + 2t)\,300^3/_{12} - 20 \times 220^3/_{12}$

$= \underbrace{4.5\times10^6 t}_{\text{WEBS}} + \underbrace{27.25\times10^6}_{\text{FLANGES}}$ — (1)

BENDING CHECK

'AN' GLASS : $f_{gd} = \dfrac{1.0 \times 1.0 \times 45}{1.8} = 25\text{MPa}$

'FT' GLASS : $f_{gd} = 25\text{MPa} + \dfrac{90}{1.2} = 100\text{MPa}$

$f_{gd} \geqslant My/_I \quad \therefore \quad I \geqslant My/_{f_{gd}}$

$\therefore \quad I_{AN} \geqslant 30\times10^6 \times 150/_{25} = 100\times10^6\ \text{mm}^4$

$I_{FT} \geqslant 30\times10^6 \times 150/_{100} = 45\times10^6\ \text{mm}^4$

SUBSTITUTE INTO (1):

$t_{AN} = (100\times10^6 - 27.25\times10^6)/4.5\times10^6 = \underline{16.16\text{mm}}$

$t_{FT} = (45\times10^6 - 27.25\times10^6)/4.5\times10^6 = \underline{3.94\text{mm}}$

DEFLECTION CHECK: (NO SHEAR DEFORMATIONS)

FOR BOTH 'AN' AND 'FT' GLASS:

$\delta_{max} = 20\text{mm} \geqslant 5WL^3/_{384EI}$ NOTE SLS

$\therefore \quad I \geqslant \dfrac{5 \times 20\times10^3 \times 4000^3}{384 \times 74\times10^3 \times 20} = 11.26\times10^6\ \text{mm}^4$

CONSIDER (1) : $I_{required} < 27.25\times10^6$ i.e. FLANGES ARE SUFFICIENT AND WEBS ARE THEORETICALLY NOT REQUIRED TO SATISFY DEFLECTION LIMIT

2 $\therefore$ BENDING GOVERNS $\Rightarrow$ $t_{AN} \geqslant \underline{\underline{16.16\text{ mm (SAY 19mm)}}}$

$t_{FT} \geqslant \underline{\underline{3.94\text{mm (SAY 4mm)}}}$