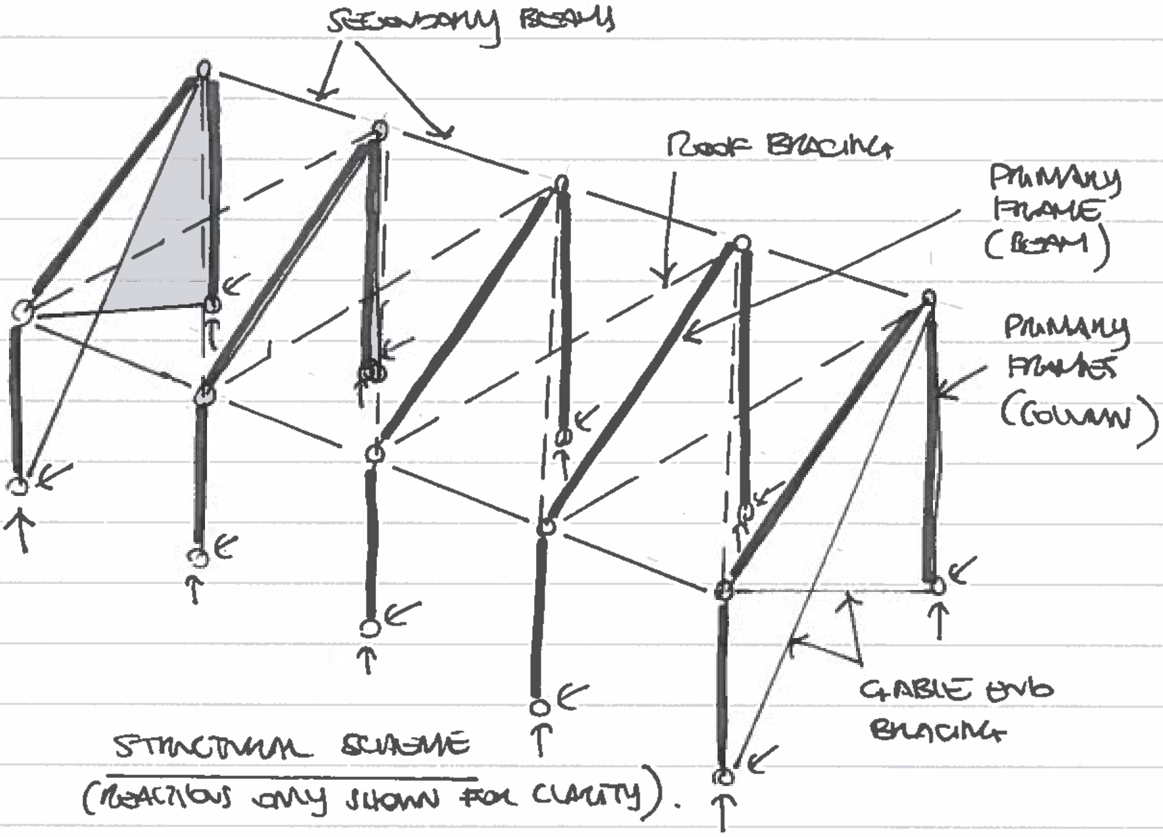
1a)



1

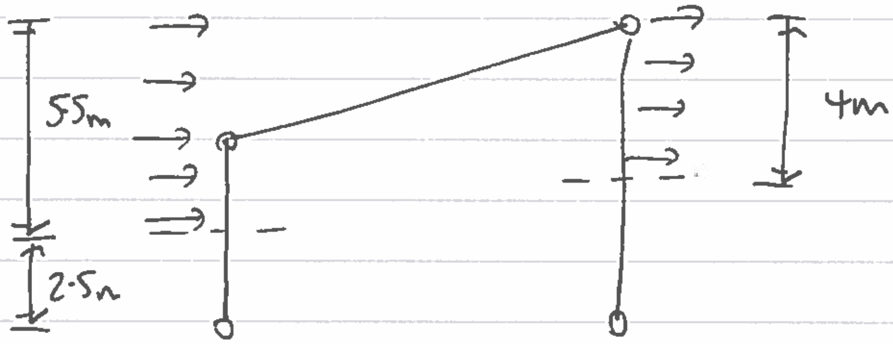
STRUCTURAL SCHEME
(REACTIONS ONLY SHOWN FOR CLARITY).

LOAD PATHS:

VERTICAL LOADS ARISING FROM SELF-WEIGHT OF CLADDING AND SNOW LOADS ARE CARRIED IN FLEXURE OF THE METAL DECK WHICH SPANS BETWEEN THE PRIMARY FRAME BEAMS, WHICH ARE IN TURN SIMPLY SUPPORTED BY THE PRIMARY FRAME COLUMNS, THAT TRANSMIT THE AXIAL LOAD INTO THE
2 FOUNDATIONS.

HORIZONTAL LOADS ARISING FROM WIND LOAD ARE CARRIED IN FLEXURE OF THE METAL CLADDING THAT SPANS BETWEEN THE PRIMARY BEAMS AND COLUMNS. THIS WOULD CAUSE THE PIN-JOINTED MAIN FRAMES TO SWAY. THIS IS PREVENTED BY THE BRACING IN THE ROOF PLANE WHICH FORMS A TRUSS SPANNING THE WHOLE LENGTH OF THE BUILDING, ~~BETWEEN~~
2 THE GABLE ENDS OF THE BUILDING PROVIDE A STIFF VERTICAL DIAPHRAGM TO SUPPORT THE ENDS OF THE ROOF TRUSS AND TRANSMIT THE FORCES INTO THE FOUNDATIONS.

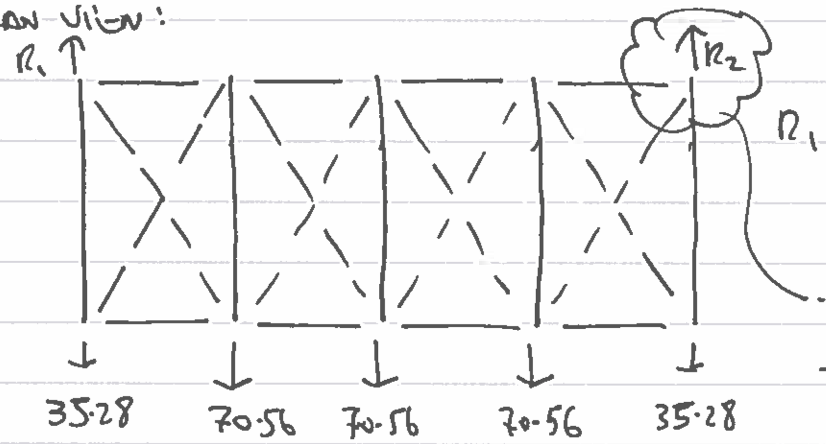
1b i) WIND LOADS ON ROOF TRUSS:



LATERAL LOAD P PER FRAME $= \left[\left(1.2\,kN/m^2\right) \times 5.5m \times 6m + 0.8\,kN/m^2 \times 4m \times 6m\right] \times 1.2$

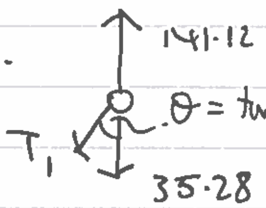
$= 70.56\,kN$

PLAN VIEW:



$R_1 = R_2 = \dfrac{282.24\,kN}{2}$

$= \underline{141.12\,kN}$



$\theta = \tan^{-1} 6/4$

$= 33.7°$

$\therefore$ TENSION IN ROOF BRACING $T_1 = (141.12 - 35.28) / \cos 33.7$

$= 127.2\,kN$

STRENGTH CHECK:

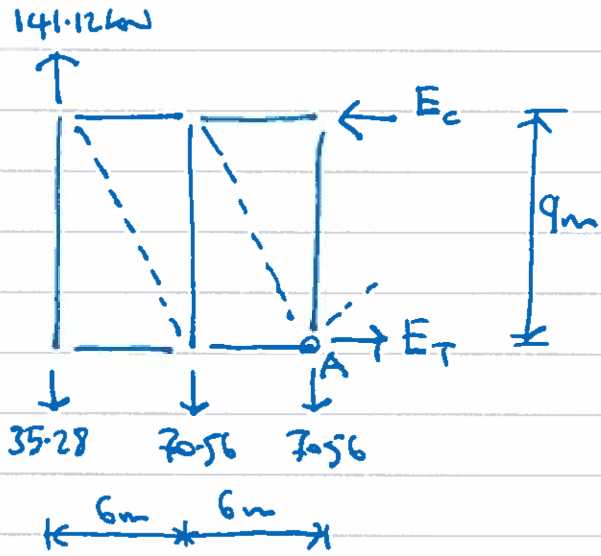
$P = A\sigma_y / \gamma_m \quad \therefore \quad A = \dfrac{P\gamma_m}{\sigma_y}$

$= 127.2 \times 10^3 \times 1.1 / 355$

$= 394.1\,mm^2$

$\therefore$ USE CH 76x38 $(A = 856\,mm^2)$

COMPRESSION IN EAVES MEMBER.



A

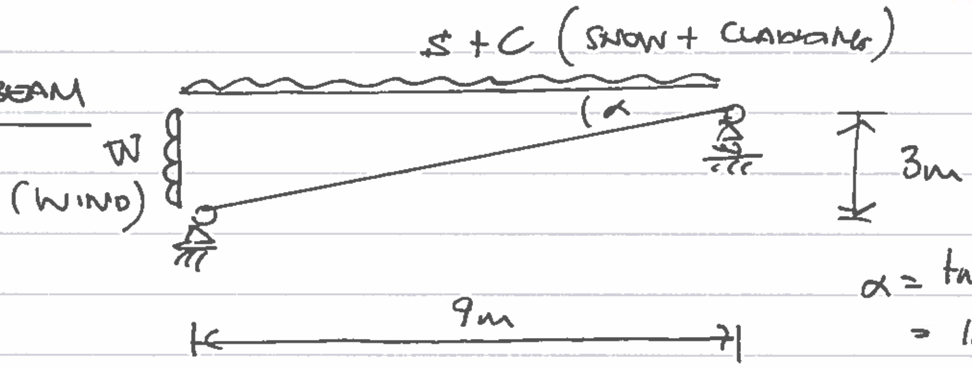
$$E_c = \frac{1}{9}\Big[(141.12 - 35.28)12 - (70.56 \times 6)\Big]$$

$$= 94.08\text{ kN}$$

$|$COMPRESSION IN EAVES MEMBER $E_c| < |$TENSION IN BRACING $T_1|$ $\therefore$ IF EAVES MEMBER IS RESTRAINED AGAINST BUCKLING NO FURTHER CHECKS ARE NECESSARY (USE CH 78 × 38 THROUGHOUT).

IF EAVES MEMBER NOT RESTRAINED AGAINST BUCKLING DEFINE EAVES MEMBER AS PIN-JOINTED STRUT:

1b ii) PRIMARY BEAM



$\alpha = \tan^{-1} 3/9$
$= 18.4^\circ$

2 UDL ⊥ to BEAM $= W \sin\alpha + S{+}C(\cos\alpha)$
$= 0.32W + 0.95(S + C)$

$W = 1.2\,kN/m^2 \times 3m \times 6m = 21.6\,kN$
$C + S = 0.95\,kN/m^2 \times 9m \times 6m = 51.3\,kN$

$\therefore$ UDL @ SLS $= (0.32 \times 21.6) + (0.95 \times 51.3) = 55.65\,kN$
UDL @ ULS $= (0.32 \times 21.6 \times 1.2) + (0.95 \times 51.3 \times 1.2) = 66.78\,kN$

BENDING CAPACITY (ULS):
$M_{max} = 66.78 \times 9/8 = 75.12\,kNm$
$\therefore Z_p \geq \left(75.12 \times 10^6 \times 1.1/355\right) \times 10^{-3} = 233\,cm^3$

2 $\therefore$ SELECT UB 254x102x22 ($Z_p = 259\,cm^3$).

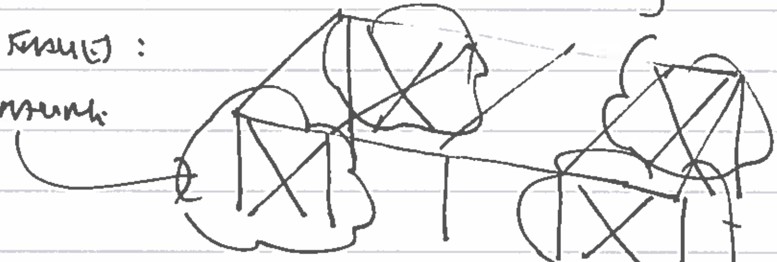
DEFLECTION CHECK (SLS)

$\sqrt{9^2 + 3^2}$

1 $\delta = \dfrac{5 \times 55.65 \times 10^3 \times 9^3}{384 \times 210 \times 10^9 \times 2841 \times 10^{-8}} = 0.0885\,m = \dfrac{SPAN}{101}$

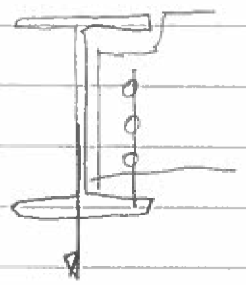
$\therefore$ UB254x102x22 IS ADEQUATE.

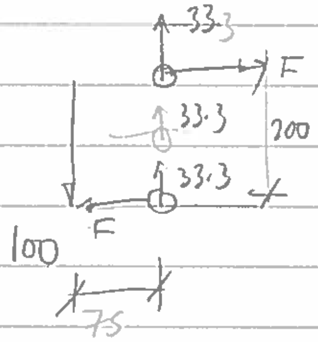
1c) CROSS BRACING SHOWN IN PART (a) CATERS FOR +VE AND −VE ~~DIRECTIONS~~ WIND DIRECTION PARALLEL TO THE PRIMARY FRAMES.
2 FOR WIND ⊥ TO FRAMES:
INTRODUCE CROSS BRACING.



3D3. 2019 FAM/1

2 a)





Moment equilib:

$$(100\,kN)(75_{mm}) = F(200mm)$$

$$\Rightarrow F = 37.5 kN$$

∴ At bolt A:

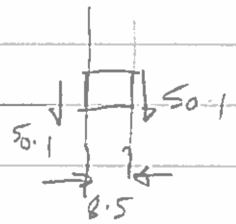
2

$$R = \sqrt{(33.3)^2 + (37.5)^2}$$

$$= 50.1 \text{ kN}$$

on shear plane of bolt

ii) Web bearing stress

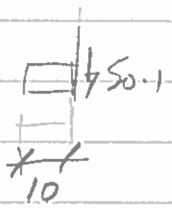


$$\sigma_b = \frac{2(50.1 \times 10^3 N)}{20(8.5)}$$

2

$$= 589 \text{ MPa}$$

iii) Cleat bearing stress



2

$$\sigma_b = \frac{50.1 \times 10^3}{20(10)}$$

$$= 250 \text{ MPa}$$

Allowable load, i) Bolt $\Rightarrow \frac{39.4}{50.1} \times 200kN = (0.79)(200) = 157 kN$

ii), iii) Bearing $\Rightarrow \frac{1.5(275)}{589} \times 200 = 0.70(200) = 140 kN$

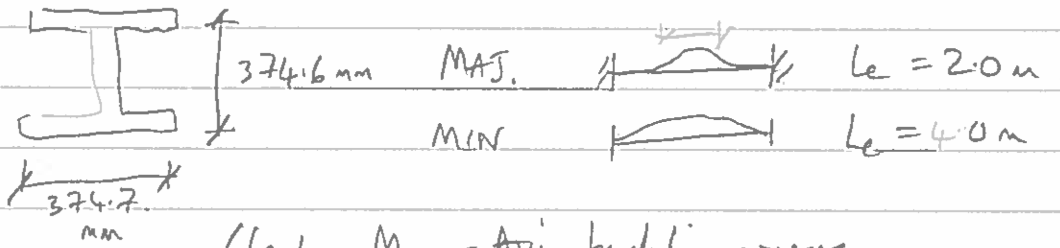
2 So bearing governs, and peak load = 140 kN

b) UC 356 × 368 × 202 in S275 steel.

$I_{MAJ} = 66260 \times 10^{-8}\ m^4$

$I_{MIN} = 23690 \times 10^{-8}\ m^4$

$A = 257 \times 10^{-4}\ m^2$

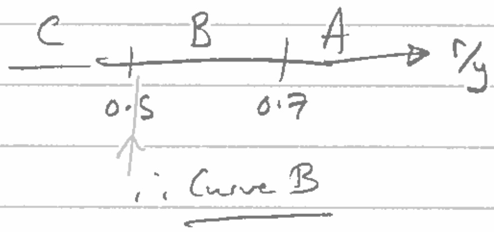


Clearly Minor Axis buckling governs.

$$r = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{23690\ cm^4}{257\ cm^2}} = 9.6\ cm = \underline{\underline{96\ mm}}$$

$$y_{EXT\ FIBRE} = \frac{374.7}{2} = 187.4\ mm$$

$$r/y = \frac{96}{187.4} = \underline{0.51}$$



$$\lambda = \frac{L}{r} = \frac{4000}{96} = \underline{41.7}$$

$$\lambda_0 = \pi\sqrt{\frac{E}{\sigma_y}} = \pi\sqrt{\frac{210 \times 10^3}{275}} = 86.8$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_0} = \frac{41.7}{86.8} = \underline{0.48}$$

Graph $\bar{\lambda} = 0.48$, Curve B $\rightarrow \chi \approx 0.9$.

$$N_{pl} = A_{st}\,\sigma_y = 257 \times 10^2\ mm^2\ (275\ N/mm^2)$$
$$= 7067\ kN$$

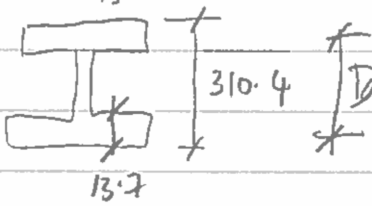
6

$$N_{Design} = 0.9\,(7067) = \underline{6361\ kN}$$

c) UB 305 × 165 × 54   ~ $\underline{S275}$.

Uniform moment ⇒ no shear.

$$I_{MAJ} = 11700 \times 10^{-8}\ m^4$$
$$I_{MIN} = 1063 \times 10^{-8}\ m^4$$
$$J = 34.8 \times 10^{-8}\ m^4$$
$$Z_{MAJ} = 846 \times 10^{-6}\ m^3$$



$D = 310.7 - 13.7 = \underline{\underline{296.7\,mm}}$

$$C_w = \frac{D^2 I_{yy}}{4} = \frac{(0.2967)^2(1063)}{4} \times 10^{-8}\ m^6.$$
$$= 23.4 \times 10^{-8}$$

$J = 34.8 \times 10^{-8}$

$$M_c = \frac{\pi}{6}\left\{(210\times10^9)(1063\times10^{-8})\left[\frac{210\times10^9}{2.6}\times + \frac{\pi^2}{36}(210\times10^9)\,23.4\times10^{-8}\right]\right\}^{1/2}$$

$$= \frac{\pi}{6}(210\times10^9)\,10^{-8}\left[1063\left(\frac{34.8}{2.6} + \frac{\pi^2}{36}\,23.4\right)\right]^{1/2}$$

$$= \frac{\pi}{6}(2100)\,145$$

$$= \underline{\underline{159.5\ kNm}}$$

$$M_p = (275 \times 10^3)(846\times10^{-6}) = \underline{\underline{233\ kNm}}$$

$$\lambda = \sqrt{\frac{M_p}{M_c}} = \sqrt{\frac{233}{159.5}} = 1.21 \rightarrow \chi_{LT} = 0.5$$

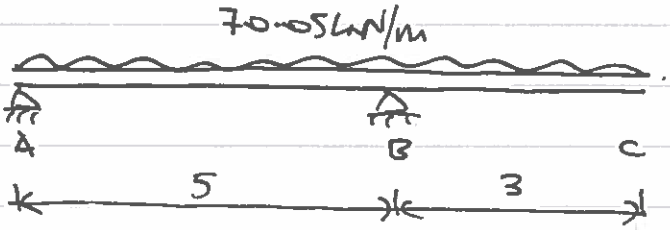
6

$$M_{DESIGN} = 0.5(233) = \underline{\underline{116.5\ kNm}}$$

3a) DESIGN LIVE LOAD = 40 kN/m × 1·6 = 64 kN/m

DESIGN DEAD LOAD = 24 kN/m^3 × 0·6 × 0·3 × 1·4 = 6·05 kN/m

TOTAL DESIGN LOAD = 70·05 kN/m



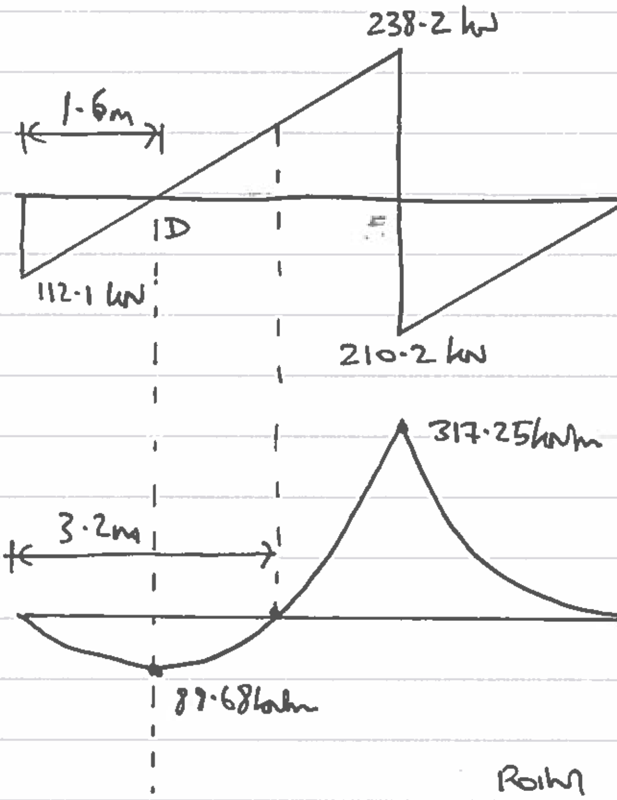
B↻ : $5R_A = 70·05 \times 8 \times 1$

∴ $R_A = 112·1$ kN

$R_B = 448·3$ kN

3 SFD

3 BMD



D↻ : $M_D = -70·5 \times 1·6 \times \frac{1·6}{2} + 112·1 \times 1·6 = 89·68$ kNm

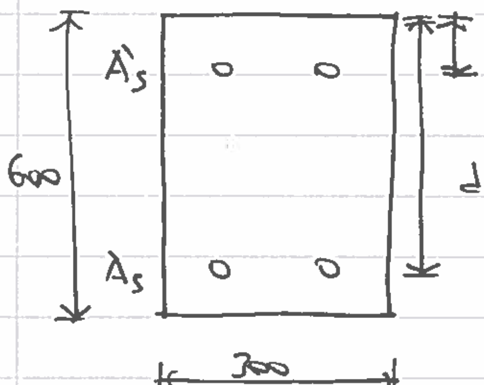
or

$M = \int_0^{1·6} S\,dx = 112·1 \times 1·6/2 = 89·68$ kN

B↻ $M_B = -70·5 \times 3 \times 1·5 = -317·25$

POINT OF CONTRAFLEXURE $l_1 = 1·6 \times 2 = 3·2$m

3 b) CONSIDER CROSS-SECTION AT B: ($M = 317·25$ kNm ; $V = 238·2$ kN)



$d' = 40 + 12·5 + 6 = 58·5$mm

$d = 600 - 58·5 = 541·5$mm

LONGITUDINAL REINFORCEMENT:

IF $A'_s = 0$; $M_u = 0·225 f_{cu} b d^2 / \gamma_c$

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$$M_u = 0.225 \times 50 \times 300 \times 541.5^2 / 1.5$$
$$= 659.8 \text{ kNm} > 317.25 \text{ kNm}$$

$\therefore$ NO COMPRESSION STEEL REQUIRED.

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

$$= \frac{1.5}{1.15} \cdot \frac{460\times10^6}{0.6\times50\times10^6} \cdot \frac{1}{0.3\times0.5415} \cdot A_s$$

$$\therefore x/d = 123 A_s \qquad - (1)$$

ALSO $M_u = A_s f_y \dfrac{d}{\gamma_s}\left(1 - \dfrac{x}{2d}\right) \qquad - (2)$

$$\therefore M_u = A_s \cdot 460\times10^6 \cdot \frac{0.5415}{1.15}\left(1 - \frac{123 A_s}{2}\right) \leftarrow$$ SUBSTITUTE (1) INTO (2)

$$M_u = 216.6\times10^6 A_s (1 - 61.5 A_s) \qquad - (3)$$

AT CROSS-SECTION B, $M_u = 317.25$ kNm

$\therefore$ (3) BECOMES : $61.5 A_s^2 - A_s + 1.46\times10^{-3} = 0$

SOLVING QUADRATIC FOR LOWER ROOT :

$$A_s = \frac{+1 \pm 0.8}{123} = 1.626\times10^{-3} \text{ m}^2$$

$$= 1626 \text{ mm}^2$$

4

1 IN NO. 25mm ϕ BAR = 491 mm^2

$\therefore$ PROVIDE 4T25 (1964 mm^2).

SHEAR REINFORCEMENT

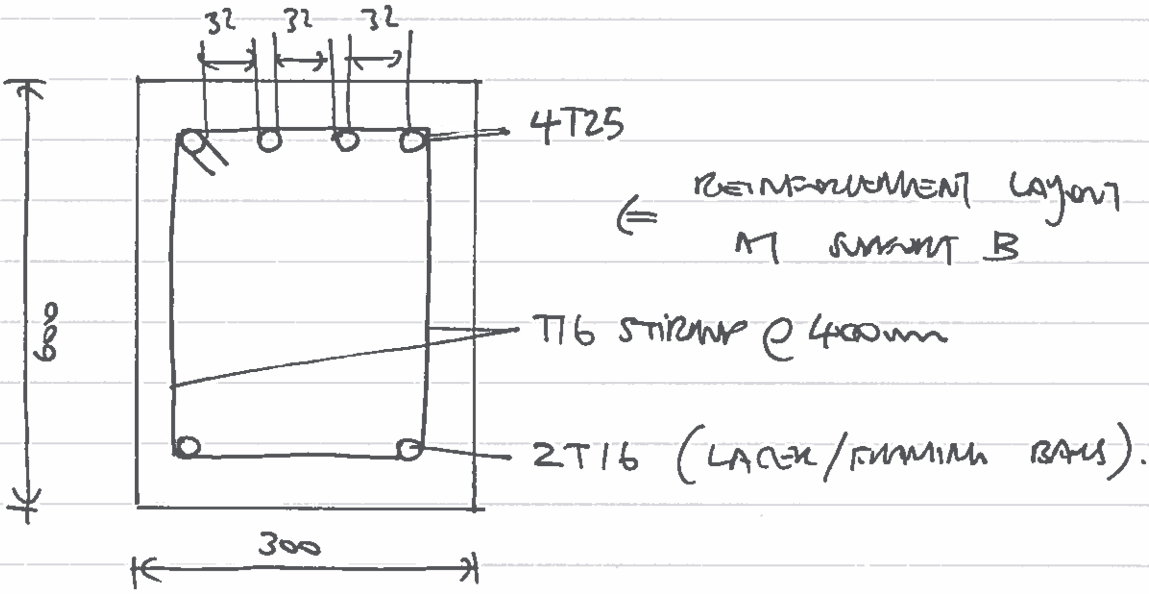
ASSUME $\cot\theta = 2.5$; $s = 0.75 \times 541.5 \rightarrow$ SAY 400mm

$$\therefore A_{sw} = V_{Rds}\, s\, \gamma_s / f_y \, 0.9 d \cot\theta \qquad - (4)$$

$$= \frac{238.2\times10^3 \times 400 \times 1.15}{460(10\ 0.9\times541.5)\ 2.5} = 147.8 \text{ mm}^2$$

2 ∴ PROVIDE 1 STIRRUP (2 LEGS) OF T12 ($226 mm^2$)

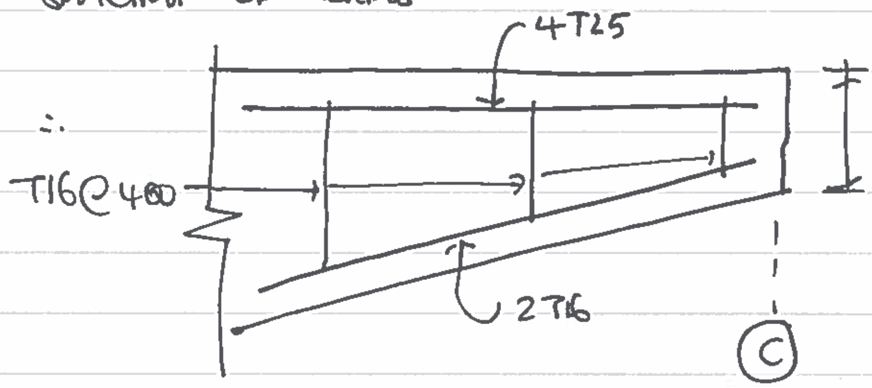
2



3c) $M_u \simeq 0.8 f_y d A_s$ AND $V_{Rds}, V_{Rdc} \propto 1/d$

∴ BY INSPECTION OF BMD $h_c = 0$, BUT THIS WOULD NOT PROVIDE MINIMUM COVER OF 40mm AND SPACING OF BARS

2 ∴

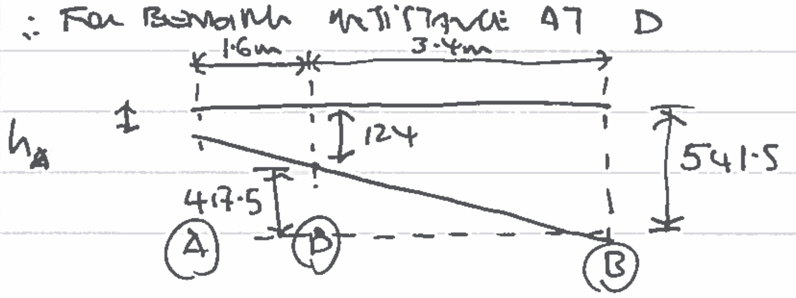


$h_c =$
$40 + 25 + 30 + 16 + 40$
$h_c = 151 mm$

h_A is limited by shear resistance minimums at A and by minimum depth of beam for bending at D.

∴ Effective depth $d_D \simeq M / 0.8 f_y A_s = 89.68\times10^6 / 0.8\times460\times1964$
$= 124 mm$

∴ For bending resistance at D



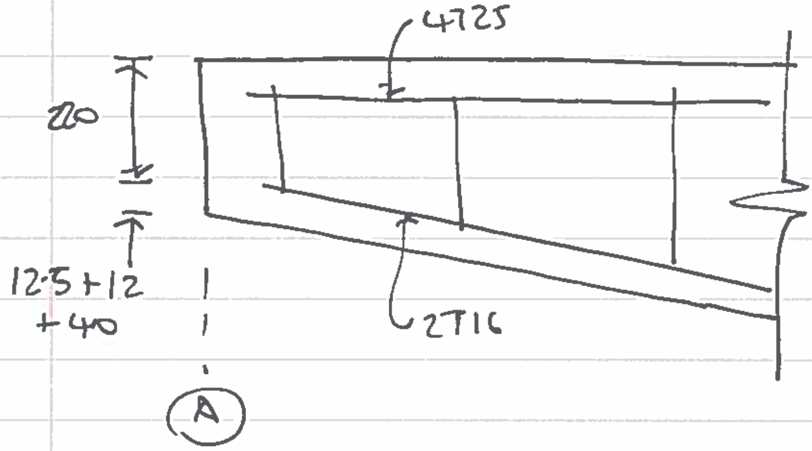
$h_A = 541.5 - \left(\frac{5}{3.4} \times 417.5\right)$
$= -ve$
∴ NOT CRITICAL

4 $\therefore h_A$ is limited by rebar capacity required at A.

Re-arranging Eq (4): $d = V_{Rds}\, s\, \gamma_s / f_y A_s\, 0.9 \cot\theta$

$$= \frac{112.1 \times 10^3 \times 400 \times 1.15}{460 \times 226 \times 0.9 \times 2.5}$$

$$= 220\text{mm}$$



$\therefore h_A = 220 + 12.5 + 12 + 40$
$= 284.5\text{mm}$

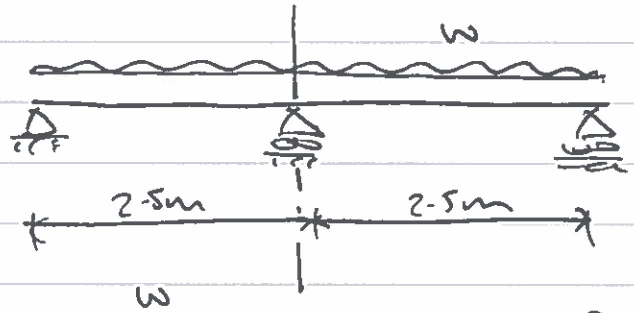
say $\underline{\underline{285\text{mm}}}$.

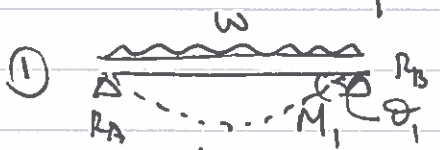
4a) LOADS @ ULS:

LONG TERM : $3.5\ \text{kN/m} \times 1.5 = 5.25\ \text{kN/m} = w_L$

SHORT TERM : $(3.5 + 1.2) \times 1.5 = 7.05\ \text{kN/m} = w_S$

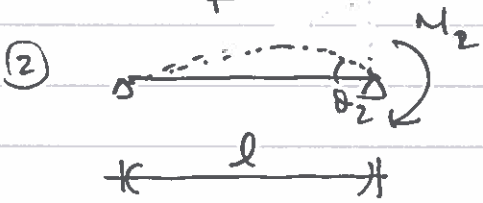
CONSIDER TIMBER BEAM:





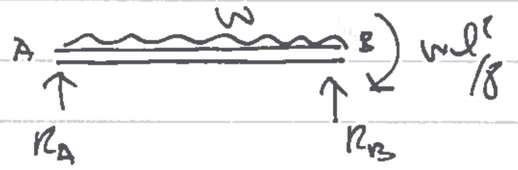
$M_1 = \frac{wl^2}{8}$; $\theta_1 = \frac{wl^3}{24EI}$

+



$\theta_2 = \frac{M_2 l}{3EI} = \theta_1$ DUE TO SYMMETRY

$\therefore M_2 = wl^2/8 = \underline{0.78w}$

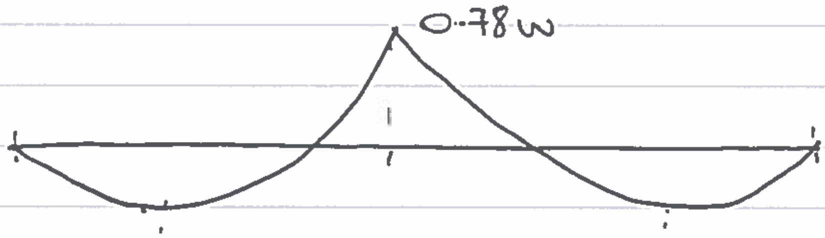
2 

$\overset{\frown}{B}: R_A l - wl^2/2 + wl^2/8 = 0$

$\therefore R_A = 3wl/8 = \underline{0.94w}$

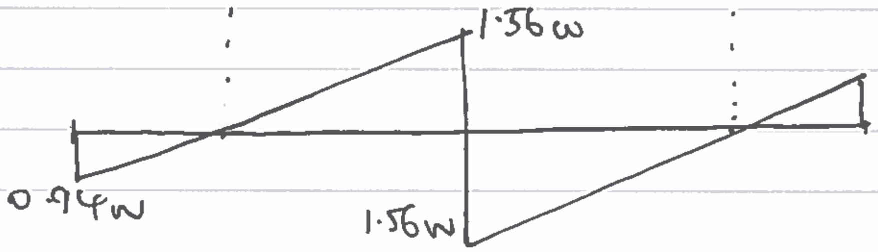
$R_B = 5wl/8 = \underline{1.56w}$

BENDING MOMENT DIAGRAM (TIMBER BEAM):

2 

$w = w_L$ OR w_S

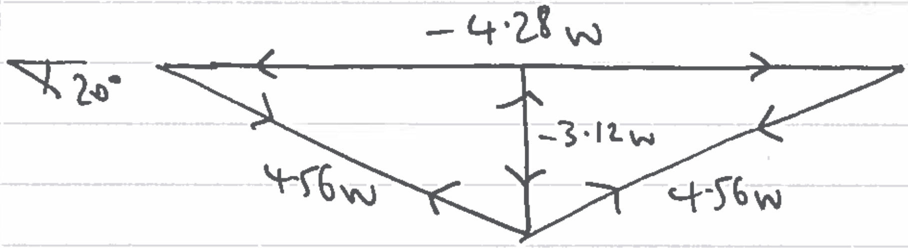
SHEAR FORCE DIAGRAM (TIMBER BEAM):

2 

$w = w_L$ OR w_S

3D3/2017/4/2

AXIAL FORCES :

2 

+ TENSION
− COMPRESSION
$W = W_L$ OR W_S

STRUT $= -1.56W \times 2 = -3.12\,W$
TIE ROD $= 1.56W/\cos 70° = 4.56\,W$
TIMBER $= -4.56\,W \cos 20° = -4.28\,W$

4b) LONG TERM

$M_{max} = 0.78 \times 5.25 = 4.1$ kNm
$V_{max} = 1.56 \times 5.25 = 8.2$ kN
$f_{md} = 0.7 \times 1 \times 16/1.3 = 8.62$ MPa
$f_{vd} = 0.7 \times 1 \times 1.8/1.3 = 0.97$ MPa

BENDING: $\sigma = M/z$ WHERE $z = bd^2/6$ (RECTANGULAR SECTION)

3
3 $\therefore d \geq \sqrt{6M/\sigma b} = \sqrt{\dfrac{6 \times 4.1 \times 10^6}{8.62 \times 125}} = 151$ mm

SHEAR: $d \geq \dfrac{V}{\tau b} = \dfrac{8.2 \times 10^3}{0.97 \times 125} = 68$ mm

SHORT TERM

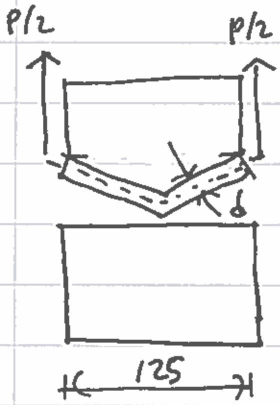
$M_{max} = 5.5$ kNm
$V_{max} = 11$ kN
$f_{md} = 0.9 \times 1 \times 16/1.3 = 11.1$ MPa
$f_{vd} = 0.9 \times 1 \times 1.8/1.3 = 1.25$ MPa

3
BENDING : $d \geq \sqrt{6M/\sigma b} = \sqrt{6 \times 5.5 \times 10^6 / 11.1 \times 125} = 154$ mm
SHEAR : $d \geq V/\tau b = 11 \times 10^3/(1.25 \times 125) = 70.4$ mm

$\therefore$ SHORT TERM BENDING GOVERNS $d \geq 154$ mm

4(c) TWO POSSIBLE FAILURE MODES:

① PLASTIC HINGE IN BOLT



d = dia.

$P = 4.65w$

$= 32.78 kN$

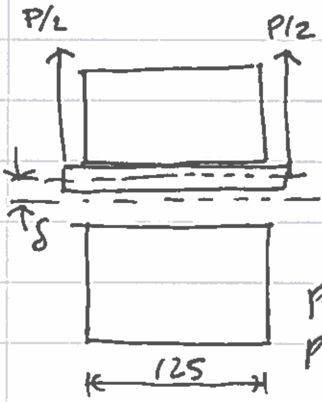
$M_{y} = 0.8 \sigma_y d^3 / 6 \gamma_m$

$$\frac{125P}{8} = \frac{0.8 \times 355 \times d^3}{6 \times 1.15}$$

$$\therefore d = \sqrt[3]{\frac{32.78 \times 10^3 \times 125}{8} \times \frac{6 \times 1.15}{0.8 \times 355}}$$

3 $= \underline{23.1 mm}$

② BEARING FAILURE IN TIMBER



$P\delta = 125 f_{h1d}\, d\, \delta$

$\therefore d = P / 125 f_{h1d}$

$P = 32.78$ (SHORT TERM)

$P = 19.5$ (LONG TERM)

$f_{h1d} = k_{mod} f_{h0k} / (k_{90} \sin^2 20 + \cos^2 20) \gamma_m$

$f_{h0k} = 0.082(1 - 0.01 \times 25) 310$

$= 19$ MPa

$k_{90} = 0.90 + 0.015 d = 1.275$

$\therefore f_{h1d} = k_{mod}\, 19 / (1.275 \times 0.117 + 0.883) 1.3$

$= 14.16\, k_{mod}$ MPa

$\therefore$ SHORT TERM $d = 32.78 \times 10^3 / 125 \times 14.16 \times 0.9$

$= \underline{20.6 mm}$

3 LONG TERM $d = 19.5 \times 10^3 / 125 \times 14.16 \times 0.7$

$= \underline{15.7 mm.}$

$\therefore$ PLASTIC HINGE IN BOLT GOVERNS. (d = 23.1mm)

say $\underline{\underline{d = 25 mm}}$