1a) PRELIMINARY SIZING:

- SIMPLY SUPPORTED SLAB $d = \frac{SPAN}{20} = \frac{4000}{20} = 200mm$

$$COVER\ (2hr) = 35mm$$
$$\therefore HEIGHT\ h = 200 + 35 + \phi/2$$
$$h \simeq 250mm$$

- SIMPLY SUPPORTED BEAM $d = \frac{SPAN}{20} = \frac{8000}{20} = 400mm$

$$COVER\ (2hr) = 50mm$$
$$\therefore HEIGHT\ h = 400 + 50 + \phi_{SHEAR} + \frac{\phi_{LONG}}{2} \simeq 500mm$$
$$WIDTH\ b \simeq 500/2 = 250mm\ (> 200mm)$$
$$\therefore BEAM = 250(b) \times 500(h)$$

4 - COLUMN: $b \times h = 300 \times 300mm$

LOADS:

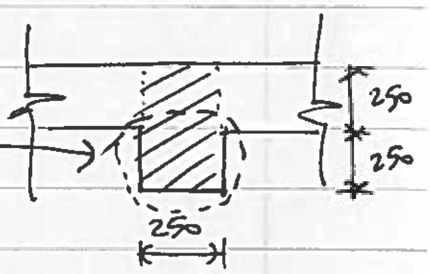
- VERTICAL ULS LOAD ON SLAB:

$$w_s = (3 \times 1.6) + (0.25 \times 24 \times 1.4)$$
$$= 12.6\ kN/m^2$$

1

- VERTICAL ULS LOAD ON 8m BEAM:

$$w_b = (12.6 \times 4) + (0.25^2 \times 24 \times 1.4)$$
$$= 52.5\ kN/m$$



1

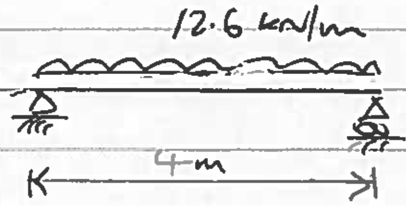
- VERTICAL ULS LOAD ON CENTRAL COLUMN:

$$P_c = (52.5 \times 4 \times 2) + (0.3^2 \times 4 \times 24 \times 1.4)$$
$$= 432\ kN$$

1

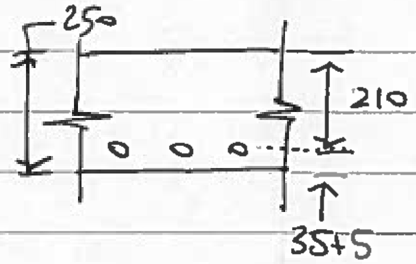
1bi) <u>SLAB REINFORCEMENT</u>:

$$M_{MAX} = \frac{12.6 \times 4^2}{8} = 25.2 \text{kNm}$$



$$M_u = 0.225 f_{cu} bd^2/\gamma_c$$
$$= 0.225 \times 50 \times 1000 \times 210^2/1.5$$
$$= 330.75 \text{kNm} > 25.2 \text{kNm}$$

∴ NO COMPRESSION STEEL REQUIRED



$A_s \rightarrow$ TAKE $z \simeq 0.8d$

$$\therefore A_s = \frac{M_{MAX} \gamma_s}{f_y \; 0.8d} = \frac{25.2 \times 10^6 \times 1.15}{460 \times 0.8 \times 210} = \underline{\underline{375 \text{mm}^2/\text{m}}}$$

3

∴ PROVIDE T10 @ 200mm (393 mm²/m)

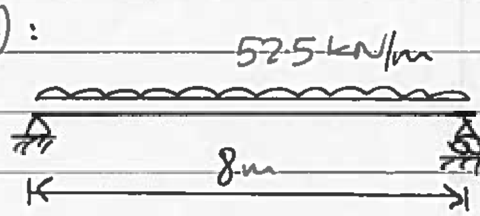
NOTE: CHECK ASSUMPTION FOR Z:

$$\frac{x}{d} = \frac{A_s f_y}{\gamma_s} \Big/ \left(0.6 f_{cu} bd/\gamma_c\right)$$
$$= \frac{393 \times 460}{1.15} \Big/ \frac{0.6 \times 50 \times 1000 \times 210}{1.5}$$
$$= 0.039 \Rightarrow x = 0.98d > 0.8d$$

∴ ASSUMPTION IS SAFE, BUT COULD RECALCULATE WITH $z = 0.98d$

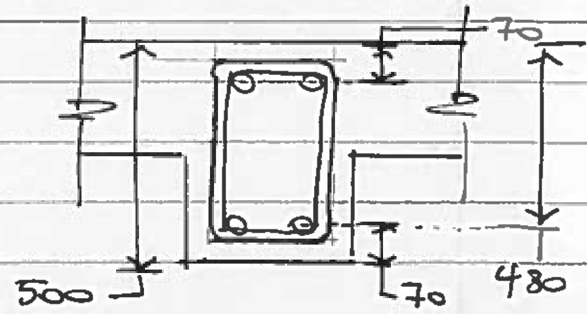
1bii) <u>BEAM REINFORCEMENT (LONGITUDINAL)</u>:

$$M_{MAX} = 52.5 \times 8^2/8 = 420 \text{kNm}$$



$$M_u = 0.225 f_{cu} bd^2/\gamma_c$$
$$= 0.225 \times 50 \times 250 \times 430^2/1.5$$
$$= 347 \text{kNm} < 430 \text{kNm}$$

∴ COMPRESSION STEEL REQUIRED



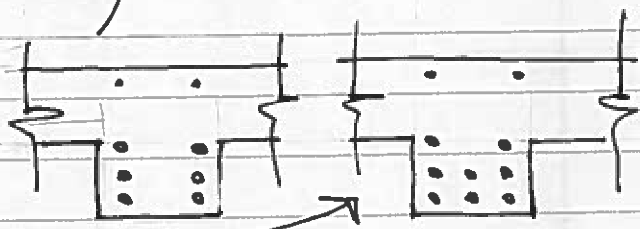
$$A'_s = (M_{MAX} - M_u)\gamma_s / [f_y(d-d')]$$
$$= 73\times10^6 \times 1.15 / [460(430-70)]$$
$$= 507\ mm^2$$

∴ PROVIDE 2T20 (628mm²)

$$A_s = \frac{\gamma_s}{f_y}\left(\frac{0.6 f_{cu} bd}{2\gamma_c}\right) + A'_s$$
$$= \frac{1.15}{460}\left(\frac{0.6\times50\times250\times430}{2\times1.5}\right) + 628$$
$$= 3316\ mm^2$$

3 ∴ PROVIDE 6T32 →

OR 8T25

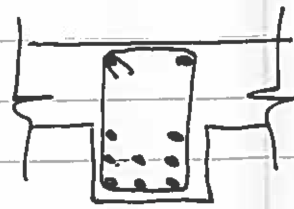


NOTE: SHOULD RECALCULATE WITH REDUCED d, TO SHOW STEEL PROVIDED IS SATISFACTORY.

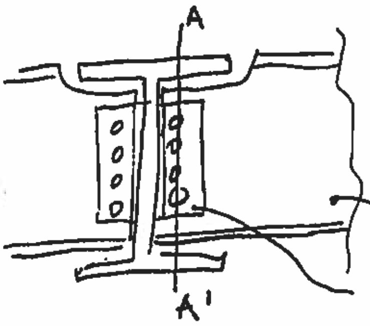
BEAM REINFORCEMENT (SHEAR):

$$V_{Max} = 52.5\times 8/2 = 210\ kN$$
$$A_{sv} = V_{max}\, s\, \gamma_s / f_y\, 0.9 d \cot\theta$$
$$= \frac{210\times10^3\times275\times1.15}{460\times0.9\times430\times2.5}$$
$$= 149\ mm^2$$

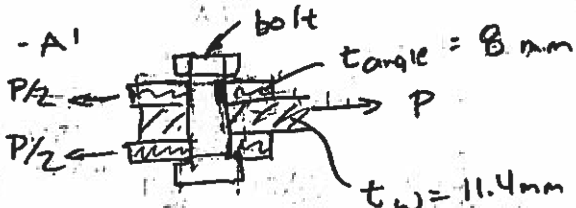
2 ∴ PROVIDE T10 @ 275 (157mm²) →



2(a)



20mm Ø bolts in 22mm Ø holes
ULS shear = 375 MPa
ULS bearing = 550 MPa
457×191×98 UB $t_w = 11.4$mm
90×90×8 angle



- Check shear - section at A - A'
  2 shear planes

$$V_{max} = \pi \frac{20^2}{4} \times \underset{\text{shear planes}}{2} \times \underset{\text{bolts at A-A'}}{4} \times 375 = 942\text{kN}$$

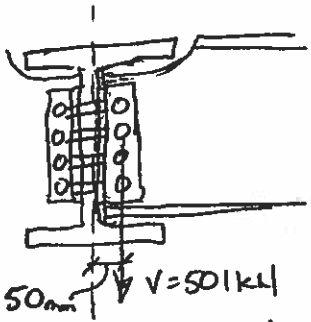
- check bearing - web of secondary beam most critical

$$V_{max} = \underset{\text{bolt } \phi}{20} \times \underset{t_w}{11.4} \times \underset{\text{bolts}}{4} \times 550 = 501 \text{ kN}$$

(3)

∴ bearing controls, maximum force = **501 kN**

2)b)i) find design forces on bolts through primary web

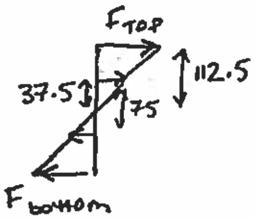


consider one side of beam

ULS tension = 500 MPa

M along line of primary bolts = 501 × 50mm = 25.1 kNm , $V_{applied}$ = 501kN

· consider linear distribution of forces



$$F_i = C \times \frac{d_i}{\Sigma d_i^2}$$

$$F_{TOP} = 25.1\text{ kNm} \times 1000 \times \frac{112.5}{2(112.5)^2 + 2(37.5)^2}$$

$$= \underline{\underline{100.4\text{ kN}}}$$

maximum tension in top bolts (1 either side)

$\therefore \sigma_{top\ applied} = \dfrac{100.4\text{ kN} \times 10^3}{2} / \pi 20^2/4 = \underline{\underline{159.8\text{ MPa}}}$

(2: bolt on either side)

shear in each bolt = 501kN / 8 = $\underline{\underline{62.6\text{ kN}}}$

$\therefore \tau_{applied} = 62.6\text{kN} \times 10^3 / \pi 20^2/4 = \underline{\underline{199.3\text{ MPa}}}$

(ii) check interaction

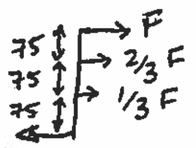
$$\left(\frac{\sigma}{\sigma_{max}}\right) + \left(\frac{\tau}{\tau_{max}}\right) \leq 1$$

$$\left(\frac{159.8}{500}\right) + \left(\frac{199.3}{375}\right) = 0.32 + 0.53 = \underline{0.85 \leq 1}$$

<u>connection is adequate</u>

Note - could also assume different force distribution.

e.g.



would give $F \times 75\left(3 + \tfrac{2}{3} \times 2 + \tfrac{1}{3} \times 1\right) = 25.1 \times 1000$

$\therefore F = 71.7\text{ kN}$

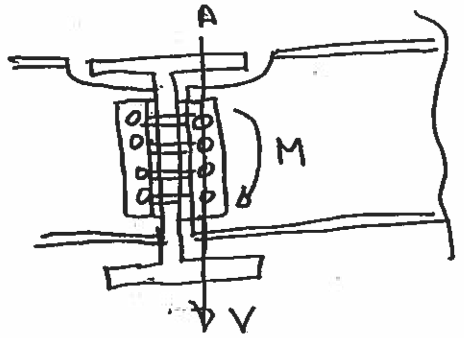
which leads to $\sigma_{top} = \dfrac{71.7 \times 10^3}{2} / \pi \dfrac{20^2}{4} = 114.1\text{ MPa}$

$$\frac{114.1}{500} + \frac{199.3}{375} = 0.76 \leq 1$$

(iii) other checks - need to check angle OK for shear, bending, prying, also check edge & end distances

(2)

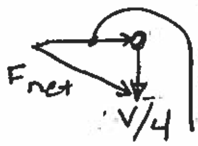
2) c) i)



V = 350 kN

M = ?

forces in secondary beam at top bolt level



reaction from moment = F_{top}

assuming linear distribution of forces

$$\therefore F_{TOP} = M \times \frac{112.5}{2(112.5)^2 + 2(37.5)^2} = M\,(0.004)$$

$$F_{net} = \sqrt{(V/4)^2 + (F_{TOP})^2} = \sqrt{\left(\frac{350}{4}\right)^2 + (M \times 0.004)^2}$$

as bearing controls (see pt b (i))

$$F_{allow} = 20 \times 11.4 \times 550 = 125.4 \text{ kN} = F_{NET}$$

(7)

$$\therefore (125.4)^2 = (87.5)^2 + (M \times 0.004)^2 \Rightarrow \underline{\underline{M = 22.5 \text{ kNm}}}$$

(ii) design forces in primary

$$M = 22.5 + 350 \times 0.05 = 40 \text{ kNm}$$

new tension in top bolt $= \frac{100.8}{2} \times \frac{40}{25.1} = \underline{\underline{80 \text{ kN}}}$

$$\left(\text{or } \frac{71.7}{2} \times \frac{40}{25.1} = 57.1 \text{ kN}\right)$$

3

$$\text{Shear} = \frac{V}{8} = \frac{350}{8} = \underline{\underline{43.8 \text{ kN}}}$$

1 b iii) <u>COLUMN REINFORCEMENT:</u>

CENTRAL COLUMN IS SYMMETRICALLY AND AXIALLY LOADED

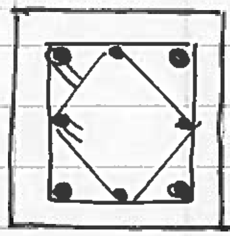
$$\therefore N = \frac{0.6 f_{cu} bh}{\gamma_c} + \frac{A_s f_y}{\gamma_s}$$

$$\therefore A_s = \left(N - \frac{0.6 f_{cu} bh}{\gamma_c}\right)\frac{\gamma_s}{f_y}$$

$$= \left(432 \times 10^3 \times 8 - \frac{0.6 \times 50 \times 300^2}{1.5}\right)\frac{1.15}{460}$$

$$= 4140 \text{ mm}^2$$

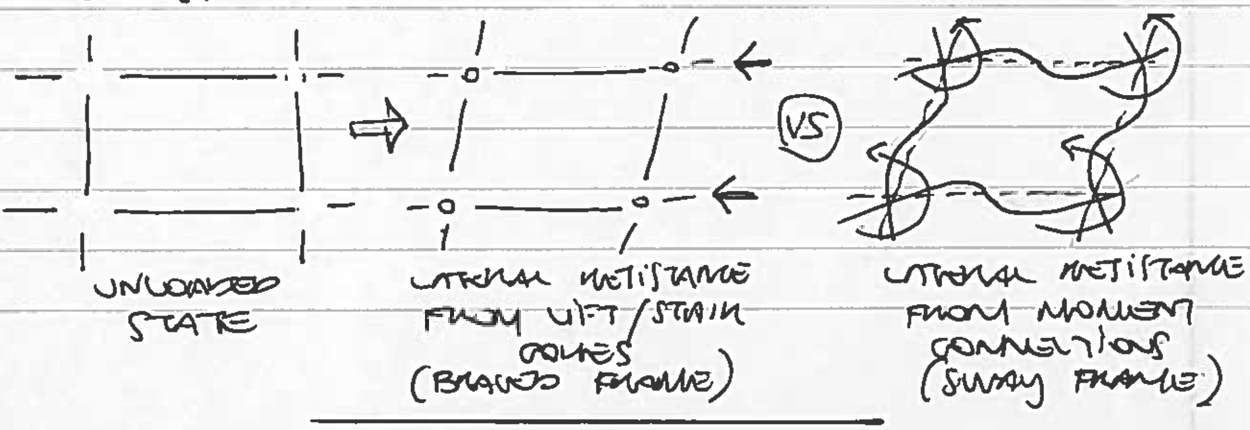
1 $\therefore$ PROVIDE 4T32 + 4T25 (5176 mm²)



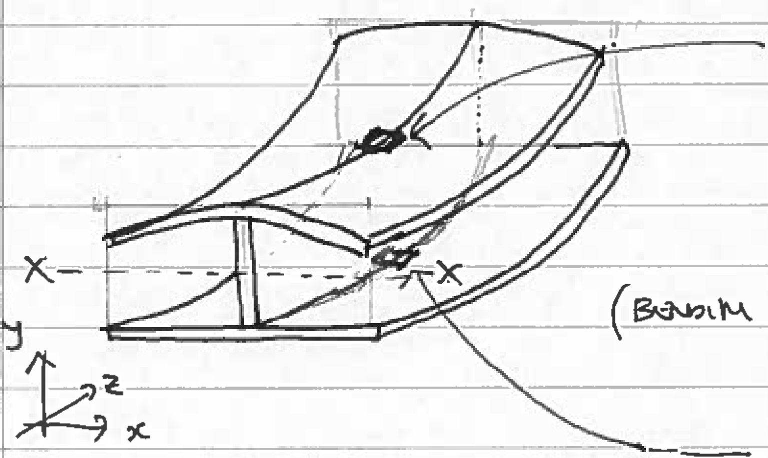
1c) <u>ALTERNATIVE DESIGN</u> (RIGID CONNECTIONS):

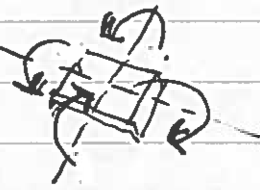
- THE VERTICAL LOAD PATH WOULD REMAIN LARGELY UNCHANGED, EXCEPT FOR THE ADDITIONAL COLUMN THAT WOULD BE REQUIRED TO REPLACE THE LIFT / STAIR CORES. 2

- THE HORIZONTAL LOAD PATHS (LATERAL STABILITY) WOULD BE PROVIDED BY THE BENDING STIFFNESS OF THE COLUMN / BEAM CONNECTIONS. (AS SHOWN BELOW) THIS WOULD REQUIRE STEEL REINFORCEMENT ACROSS THE JOINTS. 2



3 (a) THE TWO LOCATIONS OF INTEREST IN A TYPICAL GLASS I-BEAM ARE AS SHOWN:





TOP OF FLANGE

$M_x + M_z$

(BENDING OF FLANGE & LONGITUDINAL BENDING).

3

BOTTOM OF FLANGE

M_z

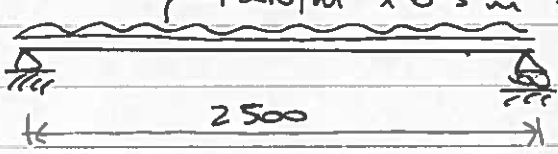
(LONGITUDINAL BENDING)

∴ PRINCIPAL STRESS AT TOP OF FLANGE $= \sigma_x + \nu\sigma_z$

" " AT BOTTOM " " $= \sigma_z$

LOADS FOR BENDING IN Z :

$4\,kN/m^2 \times 0.5\,m = 2\,kN/m$ (PER I-BEAM) + SELF WT.



SELF WT. OF I-BEAM (say $t = 12\,mm$)

$= \left[2(500 \times 12) \times 10^{-6} + (100 \times 12) \times 10^{-6}\right] \times 1\,m \times 25\,kN/m^3$

$= 0.33\,kN/m$

∴ ULS LONG TERM $= 0.33 \times 1.4 = 0.462\,kN/m$

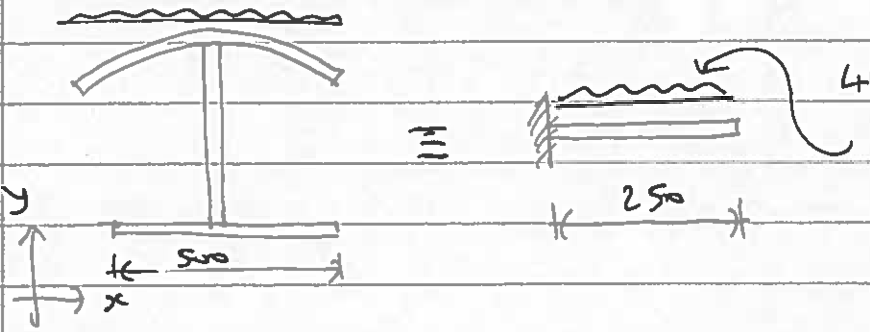
ULS SHORT TERM $= 0.462 + (2\,kN/m \times 1.6) = 3.662\,kN/m$

NOTE: $0.3 \times$ SHORT TERM > LONG TERM

∴ SHORT TERM GOVERNS.

∴ $M_{z\,max} = w\ell^2/8 = 3.662 \times 2.5^2/8 = \underline{2.86\,kNm}$

LOADS FOR BENDING IN X:



$4\,kN/m^2 \times 1m \times 1.6$
$= 6.4\,kN/m$ + SELF WT.

ULS LONG TERM $= 25\,kN/m^3 \times 0.012 \times 1.4 = 0.42\,kN/m$

ULS SHORT TERM $= 0.42 + 6.4 = 6.82\,kN/m$

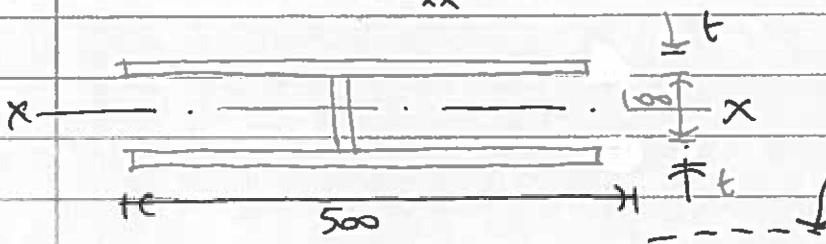
NOTE: $0.3 \times$ SHORT TERM > LONG TERM

$\therefore$ SHORT TERM GOVERNS.

$$\therefore M_{x\,max} = \frac{wl^2}{2} = \frac{6.82 \times 0.25^2}{2} = \underline{0.213\,kNm}$$

STRESS σ_z IN BOTTOM FLANGE:

$$\sigma_z = \frac{M_z y}{I_{xx}}$$



THIS TERM CONSTITUTES > 98% OF I_{xx}

$$I_{xx} = 2\left[\frac{500t^3}{12} + 500t(50+t)^2\right] + \frac{t\,100^3}{12}$$

$$\therefore I_{xx} \simeq 1000t(50+t)^2$$

$$\sigma_z = \frac{2.86\times10^6\,Nmm \times (50+t)}{1000t\,(50+t)^2} = \underline{\underline{\frac{2.86\times10^3}{t(50+t)}\,N/mm^2}}$$

NOTE: σ AT BOTTOM OF WEB $= \dfrac{2.86\times10^6 \times (50)}{1000t\,(50+t)^2} = \dfrac{143\times10^3}{t\,(50+t)^2}$

STRESSES σ_z AND σ_x IN TOP OF FLANGE:

$$\sigma_z = -\frac{2.86\times10^3}{t(50+t)} \ N/mm^2$$

2 $$\sigma_x = \frac{0.213\times10^6 \times t/2}{1000t^3/12} = \frac{1278}{t^2} \ N/mm^2$$

GLASS TYPE AND THICKNESS:

SHORT TERM GOVERNS $\therefore$ $f_{gd} = \frac{45\times1\times1}{1.8} = 25MPa$ (ANNEALED GLASS)

FLANGE THICKNESS:

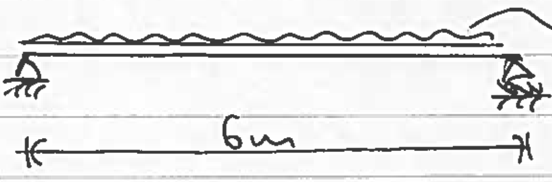
$$\sigma_x - \nu\sigma_z \leq 25 \ MPa$$

$$\frac{1278}{t^2} + \frac{0.22\times2.86\times10^3}{t(50+t)} \leq 25$$

$$25t^3 + 1250t^2 - 1907t - 63900 \leq 0$$

3 $\therefore$ $t \geq 7.37mm$ ANNEALED GLASS

SAY $t = 8mm$ ANNEALED GLASS.

3b) 

$4kN/m^2 \times 2.5m/2 = 5kN/m$

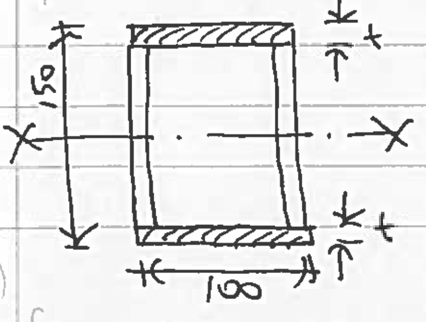
ULS SHORT TERM $= 5\times1.6 = 8kN/m$

$M_{z\,max} = 8\times6^2/8 = 36kNm$

NOTE: THE SELF WEIGHT OF THE GLASS BEAM $= 0.33kN/m \times 2\times2.5\times\frac{8}{12}$ $= 0.55kN/m$ SHOULD BE ADDED TO THIS

$\sigma_z = M_z y/I_{xx}$

$I_{xx} \simeq 2(100t \times 75^2)$
$= 1.125\times10^6 t$



$\therefore \sigma_z = 36\times10^6 \times 75 / 1.125\times10^6 t \leq 410$

3 $\therefore t \geq 5.95$, say 6mm.

(+3)

8.

3c)

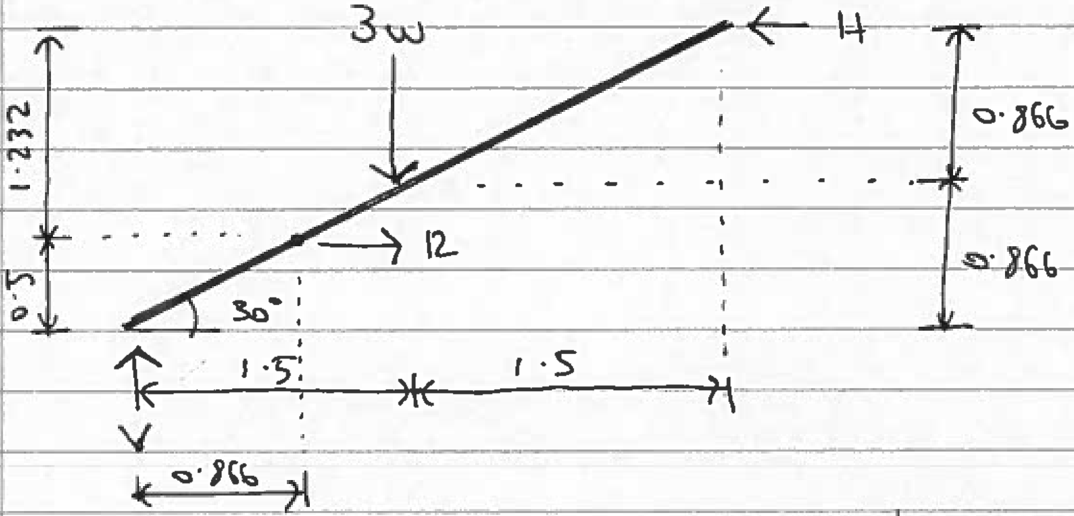
- CHECK DEFLECTION OF GLASS I-BEAM, PARTICULARLY EDGE OF GLASS FLANGE AT MID-SPAN. (SLS)
- CHECK DEFLECTION OF FRP PROFILE AT MID-SPAN (SLS), SHOULD ALSO CONSIDER SHEAR DEFORMATIONS.
- FULL COMPOSITE ACTION BETWEEN GLASS FLANGES AND GLASS WEB — HOW IS THIS ACHIEVED IN REAL-WORLD?
- USE OF MONOLITHIC ANNEALED GLASS FOR WALKING SURFACE / STRUCTURAL ELEMENTS IS NOT ADVISABLE DUE TO POOR POST-FRACTURE PERFORMANCE / LOW ROBUSTNESS / RISK OF INJURY ∴ USE LAMINATED GLASS TO PROVIDE RESIDUAL STRENGTH AFTER ONE GLASS LAYER FRACTURES.

4

4 a) **LOADS @ ULS**

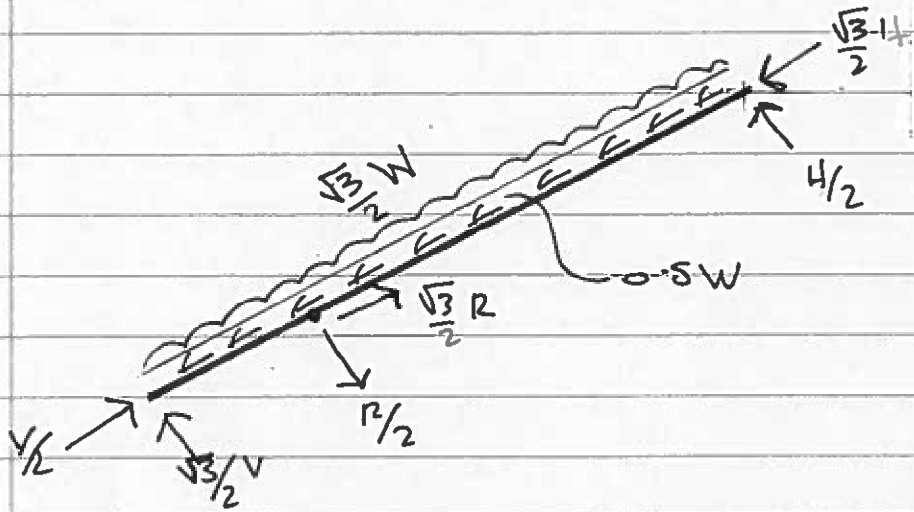
LONG TERM : $2.5 kN/m \times 2m \times 1.5 = 7.5 kN/m$

SHORT TERM : $(2.5 + 0.6) \times 2m \times 1.5 = 9.3 kN/m$



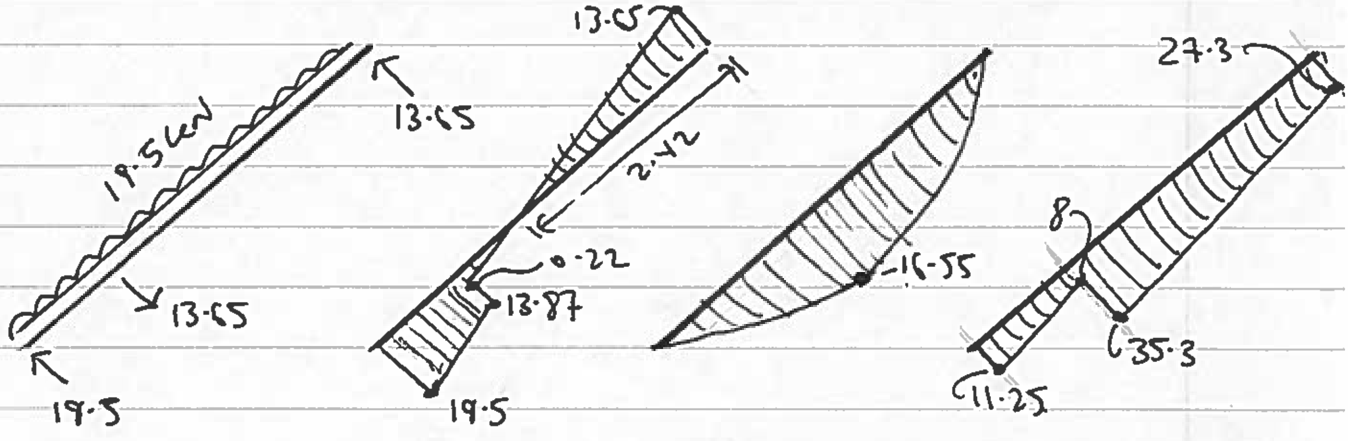
	LONG TERM	SHORT TERM
• $V = 3w \longrightarrow$	22.5 kN	27.9 kN
• $H = [0.866V + (1.5 - 0.866)3w] / (1.732 - 0.5)$	27.3 kN	33.9 kN
• $R = H \longrightarrow$	27.3 kN	33.9 kN

RESOLVING ⊥ AND // TO RAFTER:



NOTE: $W = 3w$

LONG TERM LOADS, SHEAR FORCES, BENDING MOMENTS & AXIAL FORCES:



(4)

NOTE: SHORT TERM ARE SIMILAR TO ABOVE EXCEPT THAT ALL VALUES ARE MULTIPLIED BY $\left(9.3/7.5\right)$.

4b) DEPTH OF RAFTER h :

LONG TERM: $M_{max} = 16.55\ kNm$

$V_{max} = 19.5\ kN$

$f_{md} = 0.7 \times 1 \times 24/1.3 = 12.92\ MPa$

$f_{vd} = 0.7 \times 1 \times 2.5/1.3 = 1.346\ MPa$

BENDING: $d \geq \sqrt{\dfrac{6M}{\sigma b}} = \sqrt{\dfrac{6 \times 16.55 \times 10^6}{12.92 \times 150}} = 226\ mm$

2

SHEAR: $d \geq \dfrac{V}{\tau b} = \dfrac{19.5 \times 10^3}{1.346 \times 150} = 97\ mm$

SHORT TERM: $M_{max} = 20.52\ kNm$

$V_{max} = 24.18\ kN$

$f_{md} = 0.9 \times 1 \times 24/1.3 = 16.6\ MPa$

$f_{vd} = 0.9 \times 1 \times 2.5/1.3 = 1.73\ MPa$

BENDING : $d \geq \sqrt{\dfrac{6 \times 20.52 \times 10^6}{16.6 \times 150}} = 222$ mm.

2

SHEAR : $d \geq \dfrac{24.18 \times 10^3}{1.73 \times 150} = 93$ mm

$\therefore$ BENDING IN LONG TERM GOVERNS $d > 226$ mm,

SAY $d = 230$ mm.

DIA. OF STEEL ROD :

$\dfrac{\pi D^2}{4} \cdot \dfrac{\sigma_y}{\gamma_m} \geq 33.9$ kN

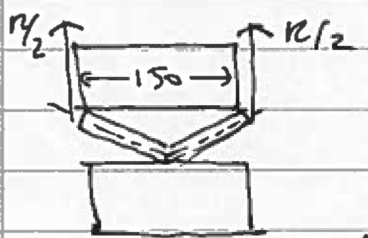
2

$\therefore \quad D \geq \sqrt{\dfrac{33.9 \times 10^3 \times 1.15 \times 4}{355 \times \pi}} = 11.82$ mm

SAY $D = 12$ mm.

4c) TWO POSSIBLE FAILURE MODES :

① PLASTIC HINGE IN BOLT



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

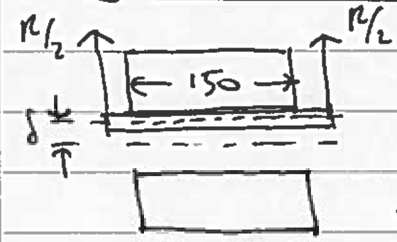
3+3

$\dfrac{150 R}{8} = \dfrac{0.8 \times 355 \times d^2}{6 \times 1.15}$

$\therefore d = \sqrt{\dfrac{33.9 \times 10^3 \times 150}{8} \times \dfrac{6 \times 1.15}{0.8 \times 355}}$

$= 24.9$ mm.

② BEARING FAILURE IN TIMBER



$R\,\delta = 150 f_{h1d}\, d\, \delta$

$\therefore d = R / 150 f_{h1d}$

$f_{h1d} = k_{mod} f_{h0k} / (k_{90} \sin^2 30 + \cos^2 30)\gamma_m$

$f_{h0k} = 0.082(1 - 0.01 \times 25)\, 350$
$= 21.5$ MPa

$\therefore f_{h1d} = k_{mod}\, 21.5 / (1.1625 \times 1.3) = 14.22 k_{mod}$

$\therefore$ SHORT TERM $d = 33.9 \times 10^3 / 150 \times 14.22 \times 0.9$
$= 17.7$ mm

LONG TERM $d = 27.3 \times 10^3 / 150 \times 14.22 \times 0.7$
$= 18.3$ mm

$\therefore \quad d = 25$ mm