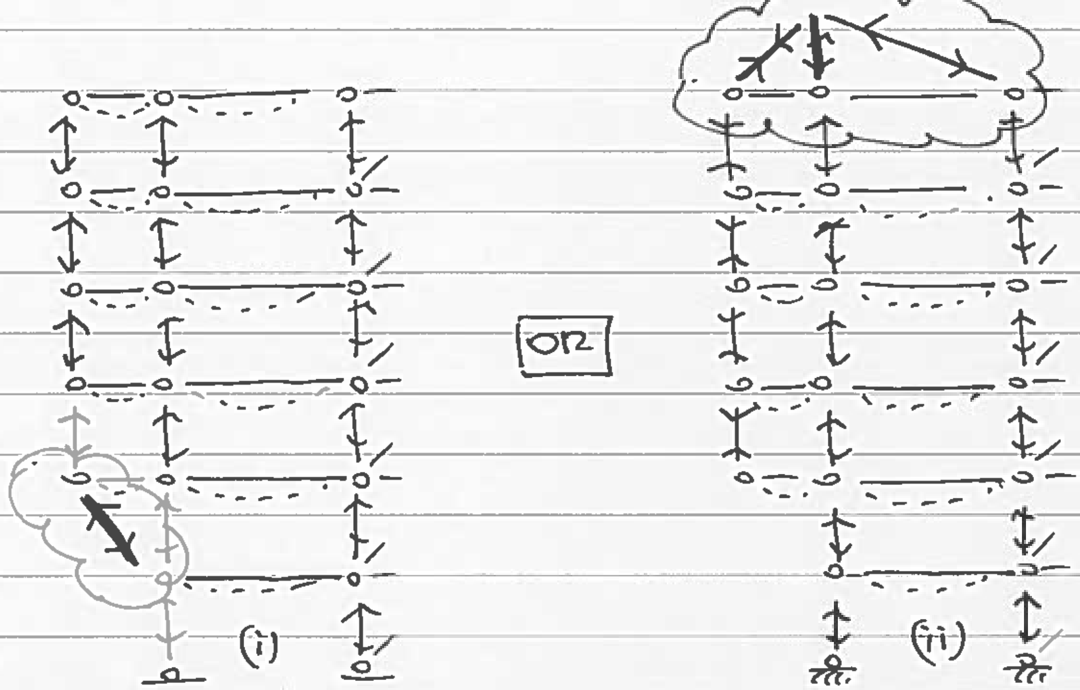
1(a) THERE ARE SEVERAL POSSIBLE SOLUTIONS FOR SUPPORTING THE OVERHANGS, SOME ARE MORE EFFICIENT / VIABLE THAN OTHERS. FOR EXAMPLE: A 5m CANTILEVERED BEAM AT EACH LEVEL IS VERY INEFFICIENT AND A DIAGONAL TIE AT EACH LEVEL WOULD DISRUPT THE INTERNAL FLOOR SPACE. THE TWO MOST VIABLE OPTIONS ARE:



NOTE: PARTIAL MARKS AWARDED FOR STABLE, BUT INEFFICIENT SOLUTIONS.

LOAD PATHS:

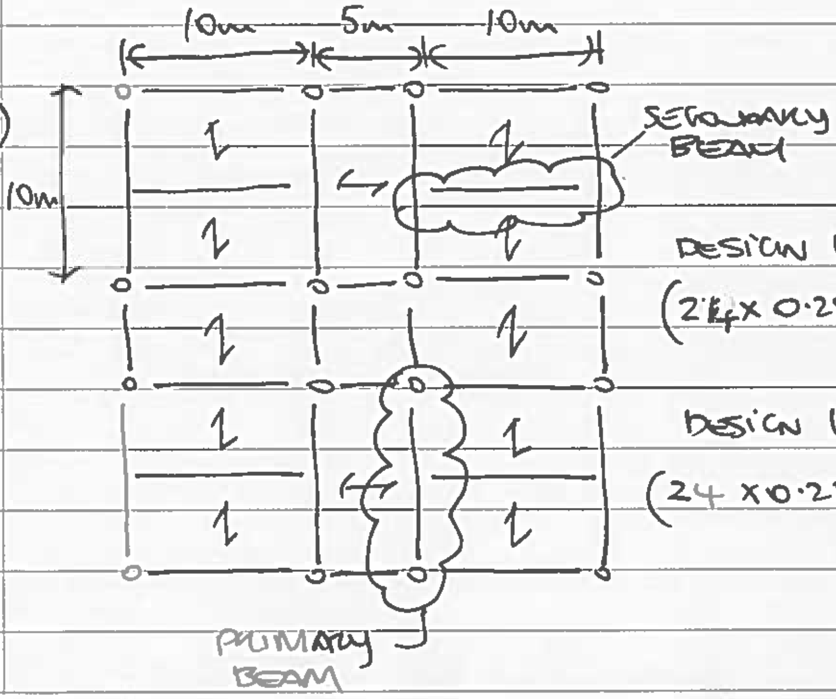
VERTICAL LOADS DUE TO IMPOSED LOADS AND SLAB SELF-WEIGHT ARE TRANSFERED IN FLEXURE IN THE SLABS → SUPPORTED BY THE SECONDARY STEEL BEAMS → IN TURN SUPPORTED IN THE PRIMARY STEEL BEAMS → VERTICAL LOADS IN COLUMNS AT EACH LEVEL.

IN ADDITION FOR OVERHANG SOLUTION (i): SIMILAR TO ABOVE EXCEPT THAT DIAGONAL STRUT TRANSMITS VERTICAL LOAD TO COLUMNS BELOW.

FOR SOLUTION (ii): SIMILAR TO ABOVE, BUT EXTERNAL COLUMN IS IN TENSION AND TRANSMITS LOAD TO TRUSS AT ROOF LEVEL

HORIZONTAL LOADS ARISING FROM WIND PRESSURE ARE TRANSMITTED IN FLEXURE BY CLADDING → THE HORIZONTAL FLOOR PLATES THAT ACT AS STIFF DIAPHRAGMS TO TRANSFER LOADS → BRACED CORE THAT ACTS AS STIFF CANTILEVER ANCHORED AT GROUND LEVEL BY STRUT / TIE ACTION IN ITS MEMBERS AND TRANSFERS AXIAL AND HORIZONTAL SHEAR FORCES TO → FOUNDATIONS.

1b)



DESIGN LOAD @ ULS:

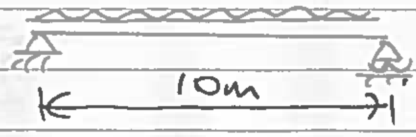
$(24 \times 0.25 \times 1.4) + (2.5 \times 1.6) = 12.4\ kN/m^2$

DESIGN LOAD @ SLS:

$(24 \times 0.25) + (2.5) = 8.5\ kN/m^2$

SECONDARY BEAM

(*) γ_s NOT GIVEN ∴ ASSUME = 1.0



@ ULS: $Z_p \geq M/\sigma_y$

WHERE $M = wL^2/8$

$\therefore Z_p \geq \dfrac{(12.4 \times 10^3 \times 5) \times 10^2}{8 \times 355 \times 10^6 (*)}$

$\geq 2.183 \times 10^{-3}\ m^3$

$\geq 2183\ cm^3$

SELECT UB 457×191×98 (2232 cm^3)

∴ USE UB 533×210×92

@ SLS: $\delta_{max} \leq 5wL^4/384EI$

$\therefore I \geq 5wL^4/384E\delta_{max}$

$I \geq \dfrac{5 \times (8.5 \times 10^3 \times 5) \times 10^4}{384 \times 210 \times 10^9 \times 0.05}$

$\geq 5.27 \times 10^{-4}\ m^4$

$\geq 52700\ cm^4$

SELECT UB 533×210×92 (55230 cm^4)

NOTE: SELF WT ≈ 0.9 kN ≈ 2% w

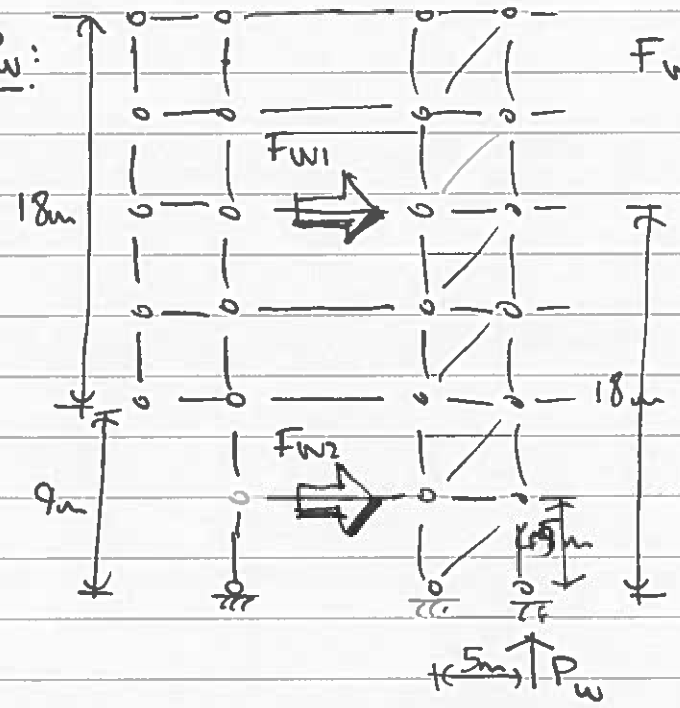
1c) AXIAL FORCE IN COLUMN (P_{TOT}) = AXIAL FORCE FROM VERTICAL (P_V)
+
AXIAL FORCE FROM WIND (P_W)

$$P_V = \left\{ \left[8.5\,kN/m^2 \times (7.5^2 - 2.5^2) \right] \times \underbrace{1.02}_{\text{2 UB SELF. WT.}} \right\} \times 1.2$$

$= 520.2\,kN$ PER FLOOR $\times$ 6 FLOORS

$= \underline{3121.2\,kN}$

P_W:



$F_{W1} = 2.5\,kN/m^2 \times 18m \times 17.5 \times 1.2$
$= 945\,kN$

$F_{W2} = 2.5\,kN/m^2 \times 9m \times 12.5 \times 1.2$
$= 337.5\,kN$

$\therefore P_W = \dfrac{(1080 \times 18) + (202.5 \times 4.5)}{5m}$

$= \underline{3706\,kN}$

$\therefore P_{TOT} = 3121.2 + 3706 = \underline{6827\,kN}$

TRY UC $356 \times 368 \times 202$

$\lambda = 4500 / 96 = 46.9$

$\lambda_0 = \pi \sqrt{210 \times 10^9 / 355} = 76.4$

$\bar{\lambda} = 46.9 / 76.4 = 0.61$

$r/y = 96 / \frac{374.6}{2} = 0.51 \Rightarrow$ CURVE B

$\therefore \chi \approx 0.82$ (FROM CHART)

$\therefore P_{max} = 355 \times 257 \times 10^2\,mm^2 \times 0.82 \leftarrow$ ASSUME $\gamma_s = 1.0$.

$= 7481\,kN > P_{TOT}$

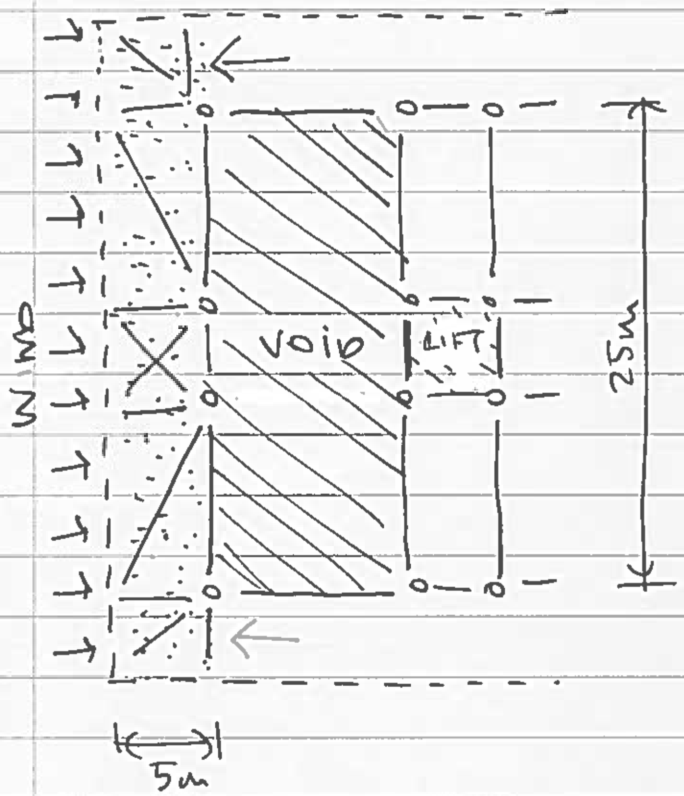
$\therefore$ $\underline{\underline{\text{UC } 356 \times 368 \times 202}}$ IS SATISFACTORY.

1 d) <u>EFFECT ON VERTICAL LOAD PATH:</u>

OPTION 1(i) IS NO LONGER VIABLE BECAUSE BEAM CONNECTED TO BOTTOM OF PROPOSED DIAGONAL STRUT IS NO LONGER POSSIBLE AND HORIZONTAL COMPONENT OF FORCE CAN NO LONGER BE TRANSMITTED.

THEREFORE OPTION 1(ii) IS THE VIABLE ONE.

<u>EFFECT ON HORIZONTAL LOAD PATH:</u>

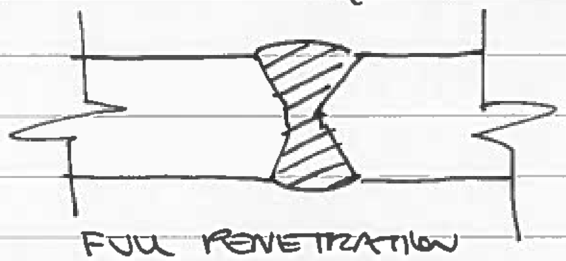


THE STRIP OF SLAB SHOWN DOTTED MUST TRANSMIT HORIZONTAL LOADS ACROSS THE VOID AT EACH LEVEL ∴ DESIGN AS 5m DEEP BEAM (OR TRUSS) SPANNING 25m.

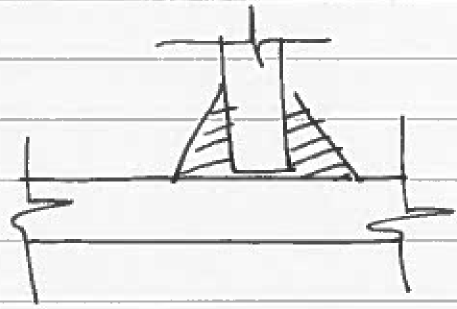
(TRUSS OPTION SHOWN).

2a) WELDING : • FUSING OF PARENT METAL AND ELECTRODE BY MEANS OF ELECTRIC ARC.

- WELDS CAN BE WITHIN THICKNESS OF PARENT METAL (BUTT WELD) OR EXTERNAL TO THE PARENT METAL (FILLET WELD)



FULL PENETRATION BUTT WELD

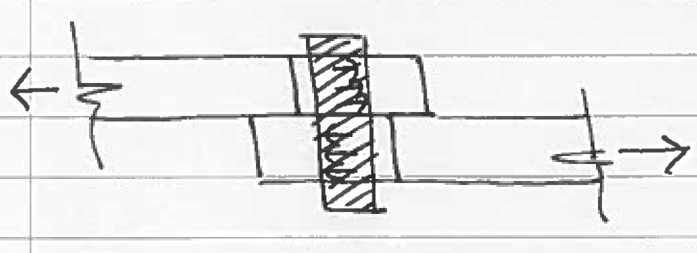


FILLET WELD

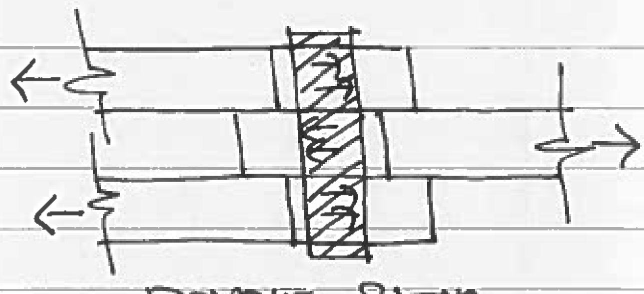
- WELDS CAN PROVIDE FULL-STRENGTH JOINTS
- LOW MATERIAL COST BUT SKILLED LABOUR
- MAY CONTAIN DEFECTS LEADING TO FATIGUE
- PREFERABLY USED IN FABRICATION SHOP - SITE WELDING TO BE USED WITH CAUTION.

BOLTING

- HOLES TYPICALLY DRILLED IN FABRICATION SHOP, JOINT ASSEMBLED ON-SITE.
- BOLTS TRANSFER LOADS IN SHEAR / BEARING / DIRECT TENSION AND SHOULD BE DESIGNED ACCORDINGLY.
- SINGLE OR DOUBLE SHEAR



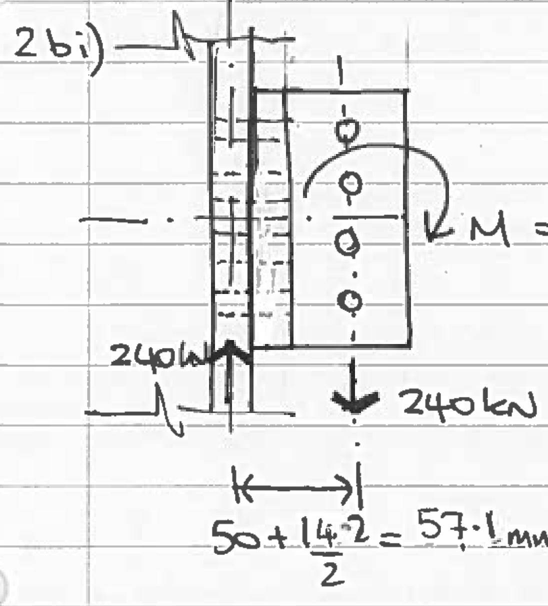
SINGLE SHEAR



DOUBLE SHEAR

- FEWER QUALITY CONCERNS THAN WELDED
- FULL STRENGTH BOLTED JOINTS MORE CUMBERSOME THAN WELDED.

BEAM / CLEAT BOLTS

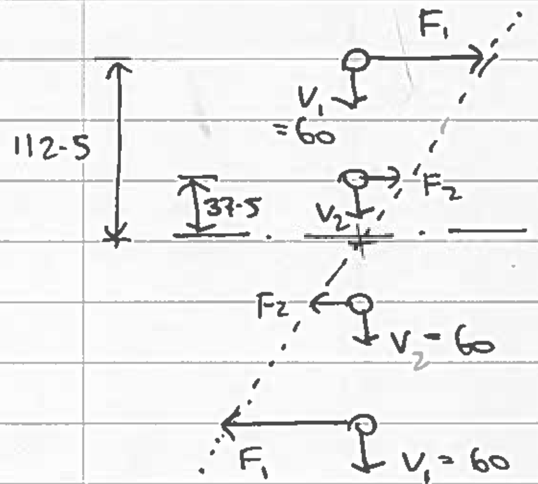


$M = 240 \times (57.1 \times 10^{-3}) = 13.7\ kNm$

VERTICAL SHEAR $= 240/4$

$V_1 = V_2 = \underline{\underline{60\ kN}}$ / BOLT.

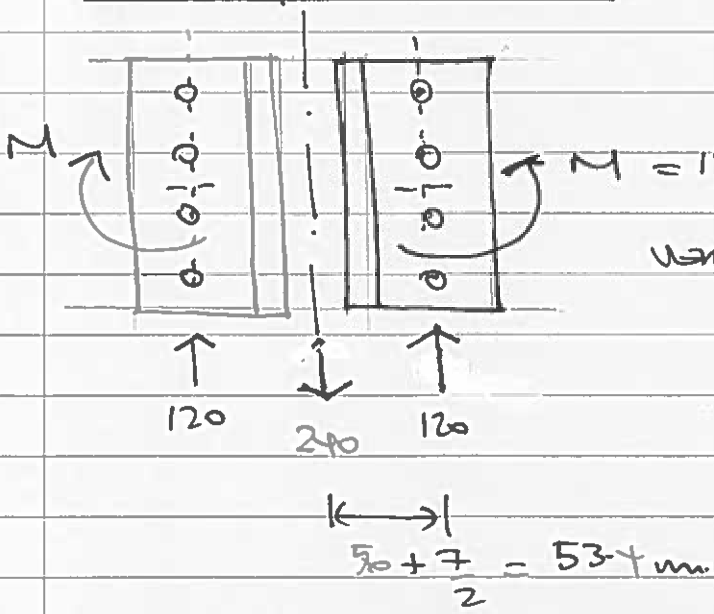
(IN DOUBLE-SHEAR)



$F_1 = \dfrac{13.7 \times 10^3 \times 112.5}{[2(112.5)^2 + 2(37.5)^2]} = \underline{\underline{54.8\ kN}}$ DOUBLE SHEAR

$F_2 = \dfrac{13.7 \times 10^3 \times 37.5}{[2(112.5)^2 + 2(37.5)^2]} = \underline{\underline{18.3\ kN}}$ DOUBLE SHEAR

COLUMN / CLEAT BOLTS.

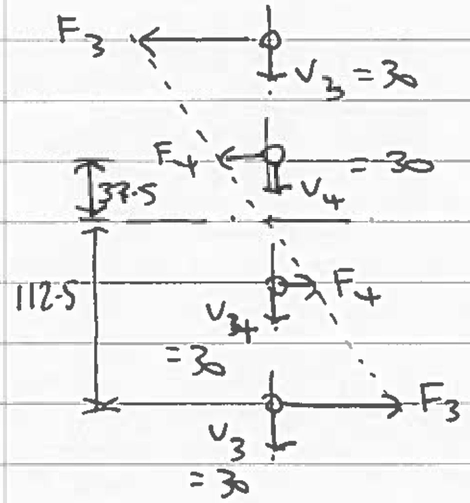


$M = 120 \times (53.4 \times 10^{-3}) = 6.41\ kNm$

VERTICAL SHEAR $= 120/4$

$V_3 = V_4 = \underline{\underline{30\ kN}}$

(IN SINGLE SHEAR)



$$F_3 = \frac{6.41\times10^3 \times 112.5}{\left[2(112.5)^2 + 2(37.5^2)\right)} = \underline{\underline{25.64\text{kN}}}$$ SINGLE SHEAR

$F_4 = \underline{\underline{8.55\text{kN}}}$ IN SINGLE SHEAR

2bii) BOLTS IN BEAM / CLEAT ARE MARGINALLY MORE LOADED :

$$S'_{max} = \sqrt{(54.8^2 + 60^2)} \Big/ 2$$

$\leftarrow$ SHEAR PLANES (DOUBLE SHEAR)

$$= 40.6 \text{ kN}$$

$\therefore$ SHEAR STRESS IN BOLT $= \dfrac{40.6\times10^3}{4\pi r^2}$

$$= \underline{\underline{32.3 \text{ N/mm}^2}} < (0.6\times460) \therefore \text{ ok.}$$

- CLEAT CAPACITY $= 355 \times 20 \times 7$
  $= \underline{\underline{49.7 \text{ kN} > 32.3 \text{ kN} \therefore \text{ ok.}}}$

- BEAM WEB CAPACITY $= 355 \times 20 \times 7$
  $= \underline{49.7\text{kN} < (32.3\times2) \therefore \text{ FAILS}}$
  INCREASE WEB THICKNESS TO > 9.1mm

- COLUMN FLANGE CAPACITY $= 355 \times 20 \times 14.2$
  $= \underline{\underline{100.8 \text{ kN} \therefore \text{ ok.}}}$

2 b iii) SHEAR FORCE IN CLEAT = 120 kN

- $t_{SHEAR} = 120 \times 10^3 / 355 \times 0.6 \times (290 - 4 \times 22)$

$= \underline{2.79\,mm}$

- $t_{BENDING} = 12 M_y / \sigma d^3 \leftarrow \left[\sigma = \frac{M_y}{I} ; \quad I = \frac{tb^3}{12} \right]$

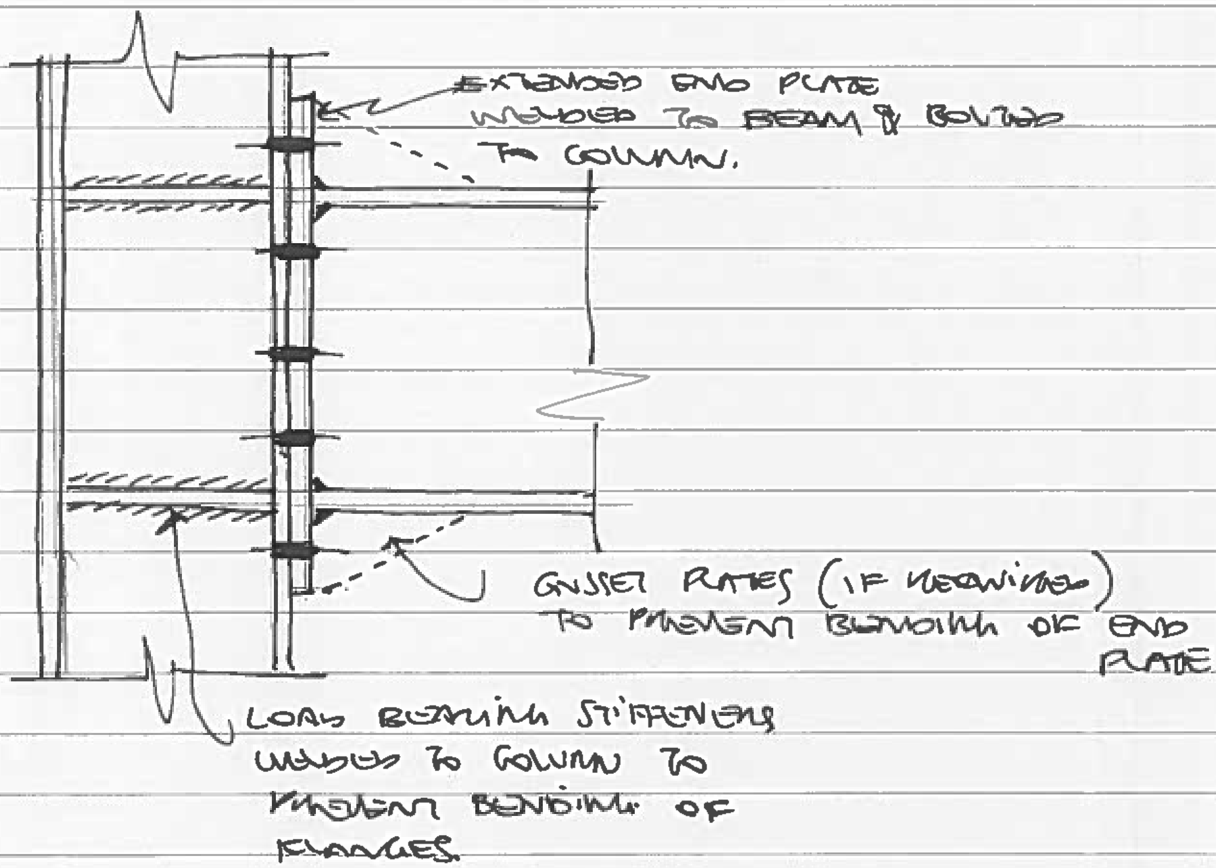
$= 12 \times \frac{13.7 \times 10^6}{2} \times 145 / 355 \times 290^3$

$= \underline{1.38\,mm}$

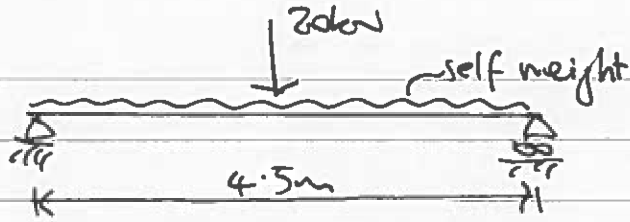
- $t_{BEARING} = \underline{4.5\,mm}$ (FROM PART 2bii)

$\therefore$ CLEAT OF 7mm IS SATISFACTORY.

2 c)



3a)



ULS L.L. $= 20 \times 1.6 = 32.0$ kN

ULS D.L. $= 24 \times 0.2 \times 0.4 \times 1.4 = 2.7$ kN/m

- SLS LIMIT (FROM DATASHEETS) $= l/d \leq 14 \therefore d \geq 285$ mm

- ULS LIMIT

– MAXIMUM BENDING MOMENT IS AT MID-SPAN:

$$M_{max} = 32 \times 4.5/4 + 2.7 \times 4.5^2/8 = \underline{42.83 \text{ kNm}}$$

– MAXIMUM SHEAR FORCE IS AT MID-SPAN $V = \underline{32 \text{ kN}}$

DESIGN LONGITUDINAL BARS (A_s)

- TRY $d = 400 - 40 - 12 - 12.5 = 335.5$ mm $> 285 \therefore$ OK AT SLS
- CONCRETE CRUSHING $M_c = 0.225 f_{cu} bd^2/\gamma_c$

$$= 0.225 \times 40 \times 200 \times 335.5^2/1.5$$
$$= 135 \text{ kNm} > 42.83 \text{ kNm}$$

$\therefore$ NO COMPRESSION STEEL REQUIRED.

- $$M_u = \frac{A_s f_y}{\gamma_s} d\left(1 - 0.5\frac{x}{d}\right) = \frac{A_s f_y}{\gamma_s}(d - 0.5x) \quad (1)$$

WHERE $$x = \left(f_y A_s / 0.6 f_{cu} b\right)\left(\frac{\gamma_c}{\gamma_s}\right)$$

$$= \left(460 A_s / 0.6 \times 40 \times 200\right)\left(\frac{1.5}{1.15}\right)$$

$$\therefore \quad x = 0.125 A_s$$

SUB INTO (1):

$$42.83 \times 10^6 = \frac{460 A_s}{1.15}\left(335.5 - \frac{0.125 A_s}{2}\right)$$

$$25 A_s^2 - 134200 A_s + 42.83 \times 10^6 = 0$$

$$\therefore A_s = \frac{134200 \pm \sqrt{134200^2 - 4(25)(42.83 \times 10^6)}}{2(25)}$$

$$= 340.7 \text{ mm}^2 \quad (\text{LOWER ROOT})$$

$\therefore$ PROVIDE 2T25 BARS (982 mm^2)

SHEAR REINFORCEMENT (A_{sv})

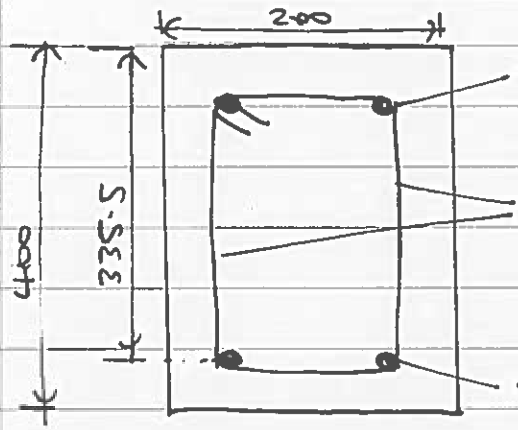
TAKE STRUT ANGLE $= 45°$; $s = 0.75d$

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

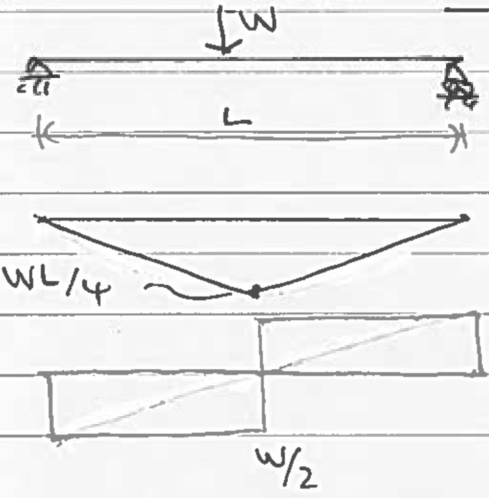
$$\therefore A_{sv} = \frac{32 \times 10^3 \times 0.75 \times 1.15}{460 \times 0.9 \times \cot 45°} = 66.7 \text{ mm}^2$$

$\therefore$ PROVIDE 2T12 LEGS (1 STIRRUP) $\Rightarrow$ (216 mm^2)

[NOTE $V_{Rd,max} >> V_{Ed,s}$]



3b)

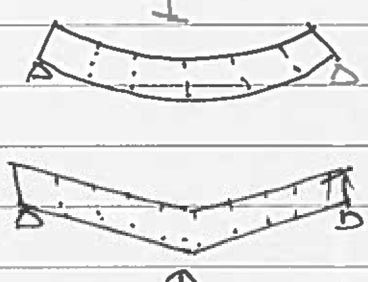


(NOTE: NEGLIGIBLE SELF-WT.)

BMD →

SFD →

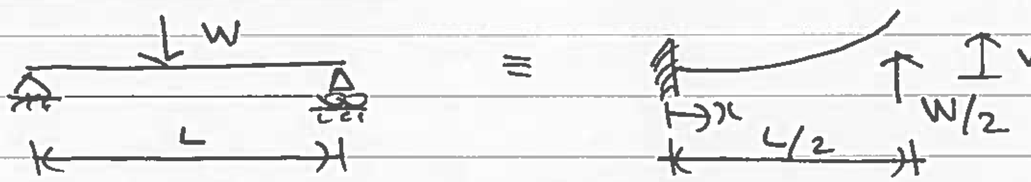
$V_f = WL^3/48EI$



$V_s = ?$

TOTAL DEFLECTION $V_{TOT} = V_f + V_s$

SHEAR DEFLECTION (v_f) :

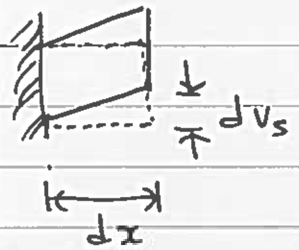


FLEXURAL DEFLECTION $v_f = \frac{W}{2}\left(\frac{L}{2}\right)^3 \Big/ 3EI = \frac{WL^3}{4Eh^3b}$

SHEAR DEFLECTION :

$$\tau_{max} = \frac{SA\bar{y}}{Ib} = \frac{W/2 \cdot bh/2 \cdot h/4}{\frac{bh^3}{12} \cdot b} = \frac{3W}{bh} = \gamma_m G$$

ALSO $dv_s = \gamma_{na} dx$



$$\therefore v_s = \int_0^{L/2} \frac{3W}{4bhG} dx = \frac{3Wx}{4bhG} + C$$

@ $x = 0$, $v_s = 0$ $\therefore$ $C = 0$

$\Rightarrow$ WHEN $x = L/2$ $v_s = 3WL/8bhG$

$$\therefore v_{TOT} = v_f + v_s = \frac{WL^3}{4Eh^3b}\left[1 + \frac{3Eh^2}{2GL^2}\right]$$

BENDING CHECK.

$M_{MAX} = 20 \times 4{\cdot}5 \times 1{\cdot}6/4 = 36 \text{ kNm}$ (SELF WT. NEGLIGIBLE)

$\sigma_{max} = M/Z = 6M/bh^2$

WHERE $\sigma = f_{md} = f_{mk}/1{\cdot}3$

$\therefore f_{mk} = 1{\cdot}3 \times 6 \times 36 \times 10^6 / 200 \times 400^2$

$\therefore f_{mk} = \underline{8{\cdot}78 \text{ MPa}}$

<u>SHEAR CHECK</u> (V_{max} @ MID SPAN)

$V_{max} = 20\,kN \times 1.6$ (SELF WT. NEGLIGIBLE)

$\tau_{max} = V_{max}/bh$ WHERE $\tau_{max} = f_{v,d} = f_{v,k}/1.3$

$\therefore f_{v,k} = 1.6 \times 1.3 \times 20 \times 10^3 / 400 \times 200$

$f_{v,k} = \underline{0.52\,MPa}$ $\longrightarrow$ NOTE WITH $\tau_{max} = \frac{3}{2} V/bh$ $f_{v,k} = \underline{0.39\,MPa}$

<u>DEFLECTION. CHECK</u> (SLS $\therefore \gamma_L = 1.0$)

NOTE: $E/G \simeq 16$ FOR ALL TIMBER GRADES

$$\therefore \nu_{TOT} = \frac{WL^3}{4Eh^3b}\left(1 + \frac{24h^2}{L^2}\right)$$

$$\therefore E = \frac{WL^3}{4\nu_{TOT}h^3b}\left(1 + \frac{24h^2}{L^2}\right)$$

WHERE $\nu_{TOT} = \delta_{max} = \frac{4500}{200} = 22.5\,mm$

$$= \frac{20\times10^3 \times 4500^3}{4 \times 22.5 \times 400^3 \times 200}\left(1 + \frac{24 \times 400^2}{4500^2}\right)$$

$$= 1582\,(1 + 0.19)$$

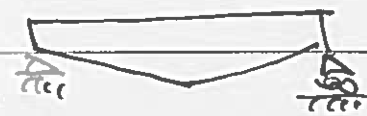
$$= 1883\,N/mm^2$$

$$= \underline{1.88\,GPa}$$

$\therefore$ BENDING GOVERNS, BUT LOWEST GRADE C14 IS SUFFICIENT.

3c) • BOTH THE CONCRETE AND TIMBER BEAMS HAVE EXCESS CAPACITIES, SO POSSIBLE TO REDUCE DEPTH FOR BOTH, BUT CHECK THAT THIS DOES NOT VIOLATE THE SPAN/DEPTH SLS RECOMMENDATION FOR CONCRETE.

• GIVEN THE B.M. AND S.F. DIAGRAMS IT IS ALSO POSSIBLE TO SHAPE THE BEAMS:



4a) LOADS ON GLASS : ASSUME GLASS THICKNESS = $3h_g$ = 3x8mm.

$L.L. = 2.5\,kN/m^2 \times 1\,m^2 = 2.5\,kN/m$

$D.L. = 2500\,kg/m^3 \times 9.81\times10^{-3} \times 0.008\times3 = 0.59\,kN/m$

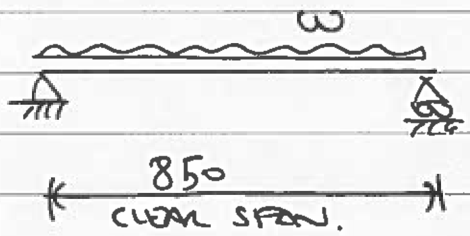
TOTAL LOAD @ SLS = 3.09 kN/m

$L.L. = 2.5\,kN/m^2 \times 1\,m^2 \times 1.6 = 4.0\,kN/m$

$DL = 2500\,kg/m^2 \times 9.81\times10^{-3} \times 0.008 \times 3 \times \cancel{1.4} = 0.83\,kN/m$

TOTAL LOAD @ ULS = 4.83 kN/m

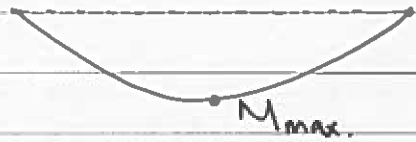
CONSIDER GLASS :



$M_{max} = wl^2/8$

$M_{max,LT} = \frac{0.82 \times 0.85^2}{8} = 0.074\,kNm$

$M_{max,ST} = \frac{4.82 \times 0.85^2}{8} = 0.44\,kNm$



GLASS BENDING STRESS CHECK :

$\sigma = My/I \quad \therefore \quad h_g = \sqrt{6M/\sigma b}$

NOTE 1: GLASS IS LAYERED ∴ BENDING MOMENT IS DISTRIBUTED EQUALLY ONTO 3 LAYERS

NOTE 2: σ = GLASS BENDING STRENGTH f_{gd}

$f_{gd} = k_{mod} k_{sp} f_{g;k} + \frac{f_{gd}}{\gamma_{mv}}$

$= \frac{1\times1\times45}{1.8} + \frac{90}{1.2} = 100\,MPa$ (SHORT TERM)

$= \frac{0.3\times1\times45}{1.8} + \frac{90}{1.2} = 82.5\,MPa$ (LONG TERM)

$\therefore h_{g,ST} = \sqrt{6\times0.44\times10^6 / 100\times1000\times3} = 2.96\,mm$ (ST)

$h_{g,LT} = \sqrt{6\times0.074\times10^6 / 82.5\times1000\times3} = 1.34\,mm$ (LT)

GLASS DEFLECTION CHECK:

$$\delta_{max} = 5wl^4/384EI$$

NOTE 1: GLASS IS LAYERED ∴ LOAD IS SPLIT EVENLY BETWEEN 3 LAYERS.

$$\therefore h_g = \sqrt[3]{60wl^4/(384Eb\delta_{max} \times 3)}$$

$$= \sqrt[3]{\frac{60 \times 3.09 \times 850^4}{384 \times 70 \times 10^3 \times 1000 \times 4.25 \times 3}}$$

$$= \underline{6.56mm}$$

NOTE 2: $\delta_{max} = 850/200 = 4.25m$

∴ GLASS THICKNESS GOVERNED BY DEFLECTION

USE 3 x 8mm THICK LAMINATED GLASS.

STEEL BENDING STRESS CHECK.

$$I = \frac{1}{12}\left(150^4 - 134^4\right) = 1.53 \times 10^7\ mm^4$$

$$\sigma_y = My/I = wL^2y/8I \quad \therefore \quad L = \sqrt{\frac{8\sigma_y I}{wy}}$$

$$\therefore L = \sqrt{\frac{8 \times 355 \times 1.33 \times 10^7}{4.82 \times 75 \times 1.15}} = 9.53 \times 10^3\ mm = \underline{9.53m}$$

STEEL DEFLECTION CHECK.

$$\delta_{max} = 5wL^4/384EI \quad \text{WHERE } \delta_{max} = L/200$$

$$\therefore L = \sqrt[3]{(384EI/1000w)}$$

$$= \sqrt[3]{384 \times 210 \times 10^3 \times 1.53 \times 10^7 / 1000 \times 4.82}$$

$$= 6.35 \times 10^3\ mm$$

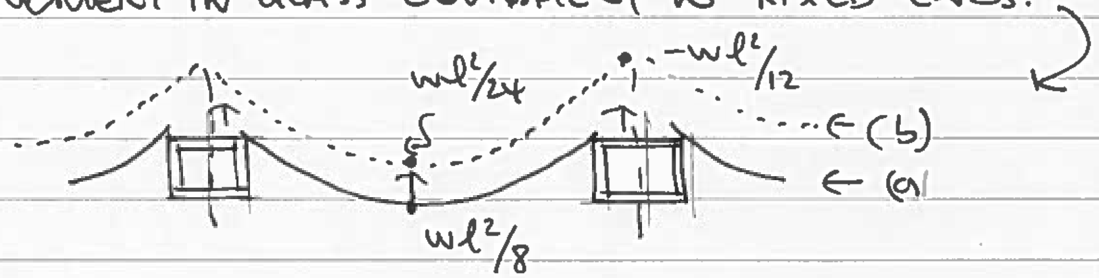
$$= \underline{6.35m}$$

∴ STEEL SPAN GOVERNED BY DEFLECTION $= \underline{\underline{6.35m}}$

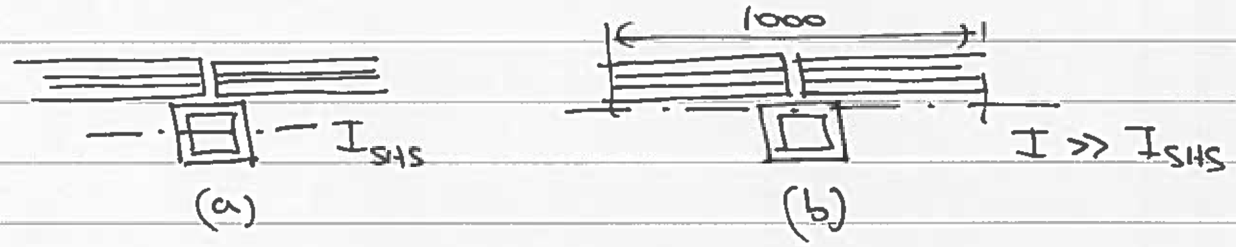
3D3 / 2018/4/3

4b) • GLASS LAYERS NOW ACT COMPOSITELY AS ONE LAYER OF THICKNESS $3h_g$

• ADHESIVE AT ENDS OF GLASS INDUCES HOGGING MOMENT IN GLASS EQUIVALENT TO FIXED ENDS.



• STEEL SHS IS BONDED TO GLASS PLATES AND ACTS COMPOSITELY WITH GLASS. ∴ INCREASE OF 2ND MOMENT OF AREA AND SHIFT OF NEUTRAL AXIS:



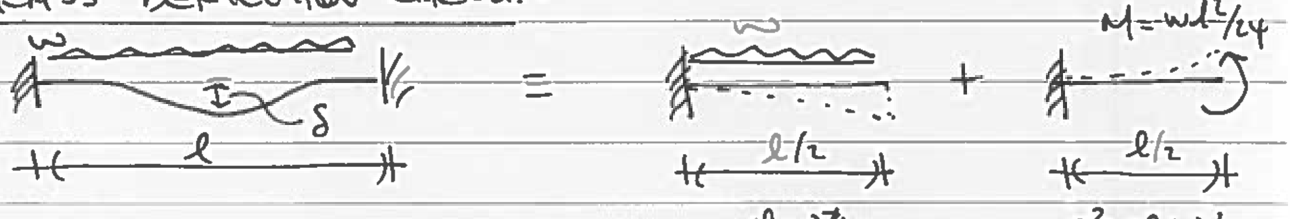
GLASS BENDING STRESS CHECK:

$$M_{max,ST} = wl^2/8 = 4.82 \times 1.0^2/8 = 0.6\,kNm$$

$$3h_{g,ST} = \sqrt{6M/\sigma_b} = \sqrt{6 \times 0.6 \times 10^6 / 1000 \times 100} = 6\,mm$$

$$\therefore h_{g,ST} = 2\,mm.$$

GLASS DEFLECTION CHECK:



$$\delta = \frac{w(l/2)^4}{8EI} - \frac{wl^2}{24} \cdot \frac{(l/2)^2}{2EI}$$

$$= \frac{wl^4}{128EI} - \frac{wl^4}{192EI}$$

$$\therefore \delta = wl^4/384EI$$

$$\therefore 3h_y = \sqrt[3]{12wl^4 / 384 E b \delta_{max}}$$

$$= \sqrt[3]{12 \times 3.09 \times 1000^4 / 384 \times 70 \times 10^3 \times 1000 \times 4.25}$$

$$= 6.87 mm.$$

$$\therefore h_y = 2.29 mm.$$

∴ GLASS THICKNESS GOVERNED BY DEFLECTION

USE 3×3mm THICK LAMINATED GLASS.

4c) OPTION (a) IS SIGNIFICANTLY LESS EFFICIENT THAN OPTION (b) IN TERMS OF USE OF MATERIALS, BUT IT WOULD BE CONSIDERABLY EASIER TO REPLACE ELEMENTS IN OPTION (a) THAN IN OPTION (b), FOR EXAMPLE IN THE CASE OF GLASS FRACTURE.