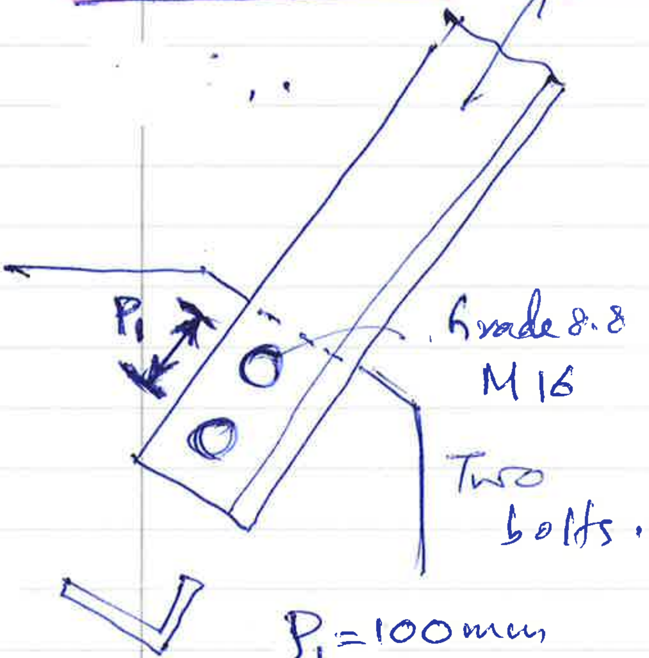
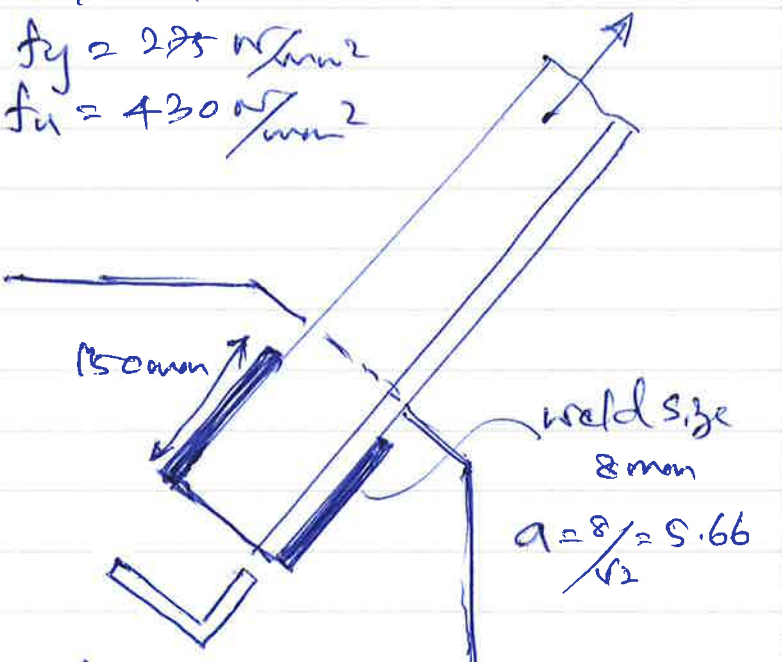
Question (1)

$N_{Ed} = 110\,kN$



S275 Steel
$f_y = 275\,N/mm^2$
$f_u = 430\,N/mm^2$

$N_{Ed} = 110\,kN$



$a = 8/\sqrt{2} = 5.66$

$P_1 = 100\,mm$
$d = 16\,mm$
$d_0 = 18\,mm$

L 100x50x8
$A = 1150\,mm^2$

L100x50x8

$L = 150 - 2a = 138.68\,mm$

(a)(i).

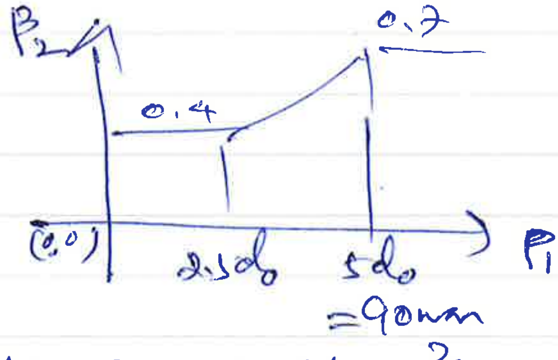
Check for c/s yielding.

$$N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{1150 \times 275}{1.05} = 301.2\,kN > N_{Ed} = 110\,kN$$

No cross sectional yielding.

Check for Local fracture near the holes.

$$N_{u,Rd} = \frac{\beta_2 A_{net} f_u}{\gamma_{M2}}$$



$P_1 = 100\,mm > 5d_0 = 90\,mm$

$\therefore \beta_2 = 0.7$

$A_{net} = A - d_0 t = 1150 - 18 \times 8 = 1006\,mm^2$

$N_{u,Rd} = \frac{0.7 \times 1006 \times 430}{1.25} = 242.25 \text{ kN} > N_{Ed} = 110 \text{ kN}$

No local fracture near the holes.

∴ L 100×50×8 can withstand the design loading //

(ii). In welded end connections for unequal angles connected by its larger leg, Fig (1)[b] the eccentricity is neglected (Negligible effect due to eccentricity). Therefore

A_{net} = Gross area A.

$N_{pl,Rd}$ is same as case a(i).

$$N_{u,Rd} = 0.9 \frac{A f_u}{\gamma_{M_2}} > \frac{0.7 A_{net} f_u}{\gamma_{M_2}}$$

↑ $N_{u,Rd}$ in Section a(i).

(Fracture resistance) → Increase ↑

The member can carry the loads without failure. //

(b)(i). bolt Class 8.8 , $f_{ub} = 800\ N/mm^2$

M16 2 bolts. $f_{yb} = 640\ N/mm^2$

bolts are subjected to shear.

$$F_{v,Ed} = \frac{110\,kN}{2} = 55\,kN$$

(Shear force per bolt)

M16 bolts, shear planes pass thro threaded area.

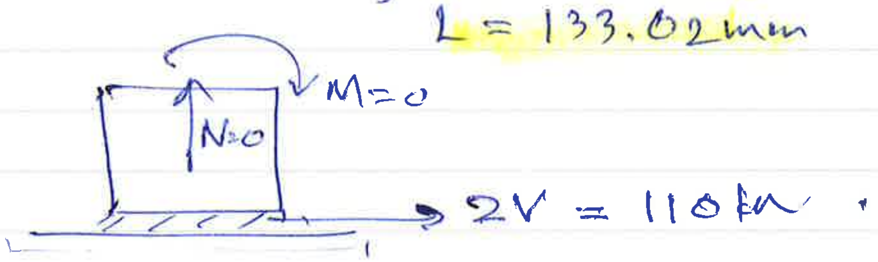
$$\therefore F_{v,Rd} = \frac{0.6\, f_{ub}\, A_s}{\gamma_{M2}} = F^{*}_{d,v} = 60.3\ kN$$

$\therefore F_{v,Ed} = 55\,kN < F_{v,Rd} = 60.3\,kN$ ok//

Bolts are suitable.//

(ii). fillet weld ; $a = 5.66\ mm$

$L = 133.02\,mm$



$$\tau_{\perp} = \sigma_{\perp} = 0 ; \quad \tau_{\parallel} = \frac{V}{La} = \frac{55\times10^3}{138.68\times5.66}$$

$$= 70.1\ N/mm^2$$

Design Criterion;

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} = \sqrt{3}\,\tau_{\parallel} = \sqrt{3}\times70.1 = 121.36\ N/mm^2$$

$$f_u/\beta_w\gamma_{M2} = \frac{430}{1.25\times0.85} = 405\ N/mm^2$$

$$\therefore \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} = 121.36\ N/mm^2 < f_u/\beta_w\gamma_{M2} = 405\ N/mm^2$$ ok//

Size of the weld is suitable//

Question (2)

(a).(i).

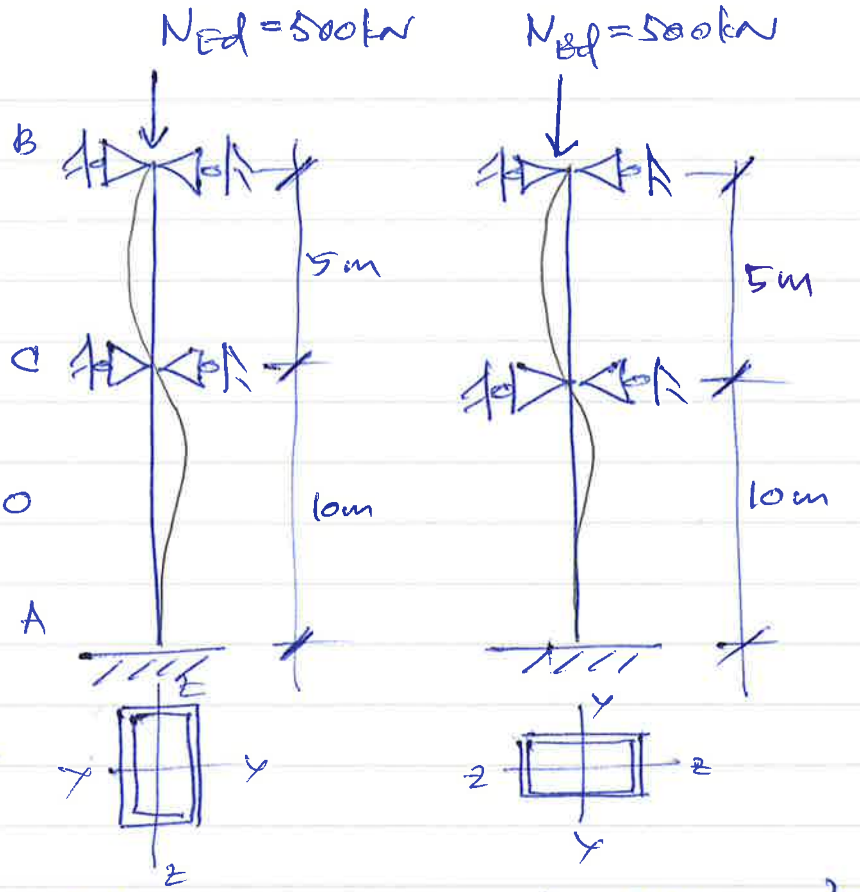
For BC part

$L_{cr,y} = L_{cr,z} = 5\ m$

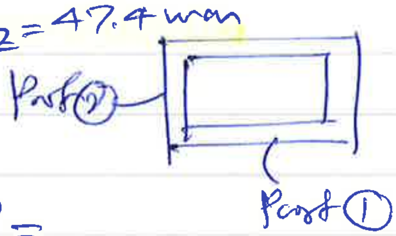
For AC part

$L_{cr,y} = L_{cr,z} = 0.7 \times 10$
$= 7m$

S 355 Steel
$f_y = 355\ N/mm^2$
$\varepsilon = 0.81$



RHS 200x120x10 $A = 5850\ mm^2$
$i_y = 71.3\ mm$ $i_z = 47.4\ mm$



Class Classification

Part (1): Compression

$$c/t\varepsilon = \frac{B - 3t}{t\varepsilon} = \frac{200 - 3\times10}{10\times0.81} = 20.99 < 33$$

class (1)

Part (2) Compression

$$c/t\varepsilon = \frac{H - 3t}{t\varepsilon} = \frac{120 - 3\times10}{10\times0.81} = 11.11 < 33$$

class (1)

c/s is class (1)

Check for c/s yielding

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{5850\times355}{1.05} = 1977.8\ kN$$

$> N_{Ed} = 500\ kN$

ok

Check for overall flexural buckling.

$$N_{b,Rd} = \chi A f_y / \gamma_{M1}$$

$$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \cdot \frac{1}{\lambda_1} = \frac{7000}{71.3} \cdot \frac{1}{93.9 \times 0.81} = 1.291$$

$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \cdot \frac{1}{\lambda_1} = \frac{7000}{47.4} \cdot \frac{1}{93.9 \times 0.81} = 1.942$$

Hot finished hollow cross section curve "a"

$\alpha_y = \alpha_z = 0.21$ as $\bar{\lambda}_y < \bar{\lambda}_z$

Buckling axis is z-z axis. (weak axis)

$$\phi_z = 0.5\left[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2\right]$$

$$= 0.5\left(1 + 0.21(1.942 - 0.2) + 1.942^2\right)$$

$$= 2.569$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^2}} = 0.235$$

$$N_{b,Rd} = \chi_z \frac{A f_y}{\gamma_{M1}} = \frac{0.235 \times 5860 \times 355}{1.05} = 465.3\,kN$$

$N_{Ed} = 500\,kN > N_{b,Rd} = 465.3\,kN$

C/S is not suitable as it subjected to overall flexural buckling about z-z axis.

(a)(ii)

If c behave as fixed with sliding

$(l_{cr,z})_{BC} = 0.7 \times 5 = 3.5$ m.

$(l_{cr,z})_{AC} = 0.5 \times 10 = 5$ m

$(l_{cr,z}) = 5$ m.

only possible buckling z-z axis.

$$\bar{\lambda}_z = \frac{5000}{47.4} \cdot \frac{1}{93.9 \times 0.81} = 1.387$$

from Buckling curve "a" (Figure 6.4); $\chi_z \simeq 0.42$

$$N_{b,Rd} = 0.42 \times \frac{A f_y}{\gamma_{m1}} = 839 \text{ kN}$$

$> N_{Ed} = 500$ kN

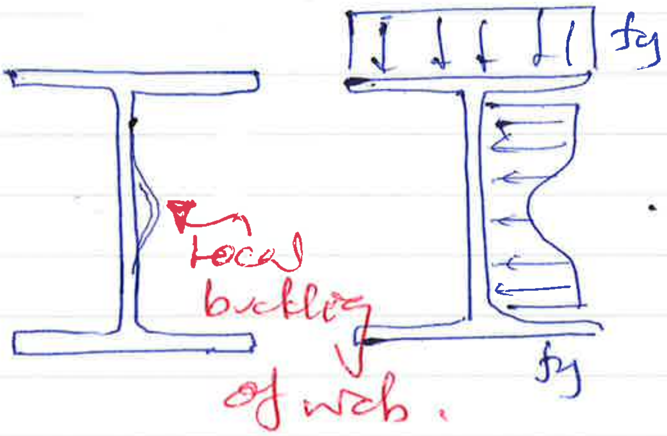
ABC Member can carry the load without overall flexural buckling.

ok

(b).(i) - "Local buckling" of compression part of c/s may reduce the effective area of c/s.

- Local buckling of c/s is the major behavior which reduces the resistance of class (4) c/s.

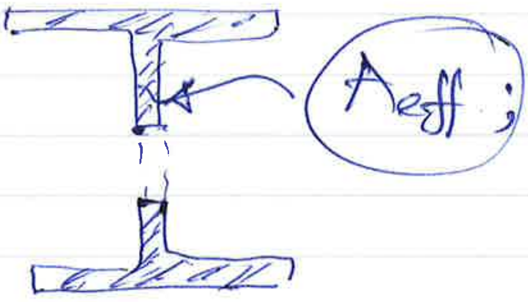
- for class (4) c/s ; local buckling occurs befor c/s reach to yielding state of stress.



$N_{c,Rd} < \frac{A f_y}{\gamma_{M0}}$

effective Area is not the gross Area.

⇓

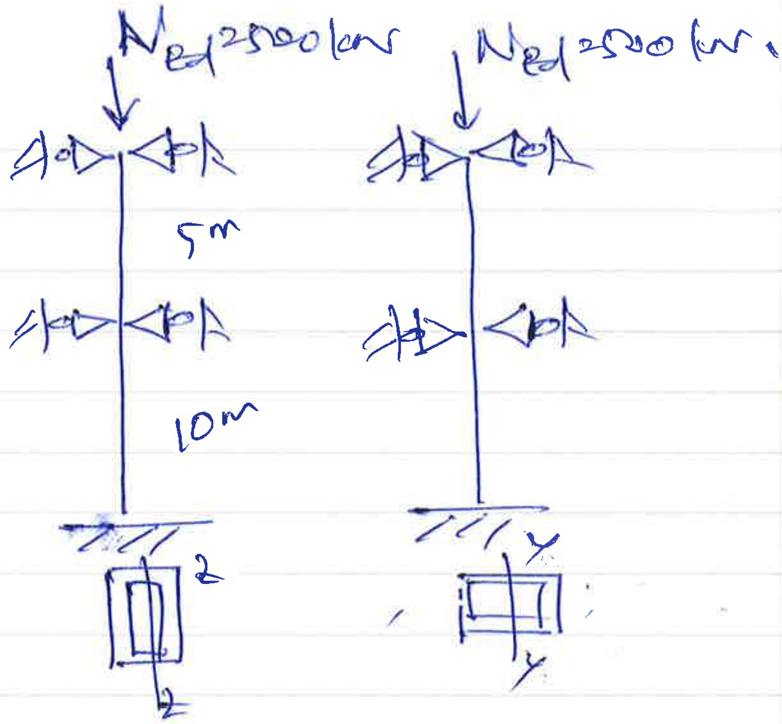


$$N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}}.$$

I_{eff} , Neutral axis changes.

$I_{T,eff}$ $C_{w,eff}$ //

(ii).



$L_{cr,y} = L_{cr,z} = 7$ m

S355 steel.

$f_y = 355$ N/mm^2

$\varepsilon = 0.81$

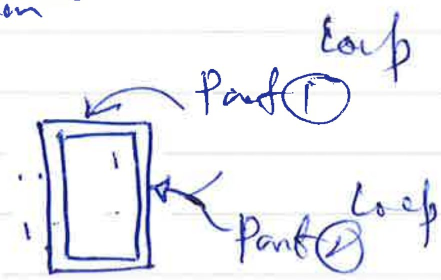
RHS 250×150×6.3

$i_y = 92.4$ mm

$i_z = 62.2$ mm

$A = 4820$ mm^2

Class classification ;



Part ① (Comp)

$$\frac{c_1}{t\varepsilon} = \frac{B-3t}{t\varepsilon} = \frac{150-3\times6.3}{6.3\times0.81} = 25.69 < 33$$

Class ①

Part ② (Comp)

$$\frac{c_2}{t\varepsilon} = \frac{H-3t}{t\varepsilon} = \frac{250-3\times6.3}{6.3\times0.81} = 45.28 > 42$$

Class ④

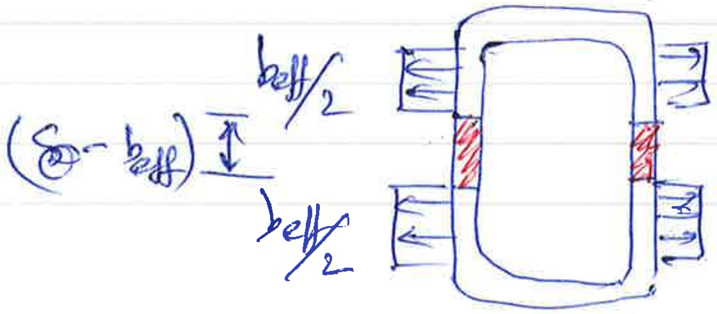
Part ② only subjected to local buckling.

C/S is class ④ //

From Table 4.1 : Internal Compression elements.

$\psi = \sigma_2/\sigma_1 = 1.0$

$k_\sigma = 4$



$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{②/s}{28.4\times0.81\times\sqrt{4}} = \frac{231.1/6.3}{28.4\times0.81\times\sqrt{4}}$$

$$= 0.797 > 0.5 + \sqrt{0.085 - 0.055\psi} = 0.673$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{0.797 - 0.055(3+1)}{0.797^2}$$

$$= 0.908.$$

$$b_{eff} = \rho\,② = 0.908 \times 231.1 = 209.9 \text{ mm}$$

$$A_{eff} = A - 2(② - b_{eff})s$$

$$\therefore A_{eff} = 4820 - 2(231.1 - 209.9)\times 6.3 = \underline{\underline{4553 \text{ mm}^2}}$$

(iii). $N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}}$ class ④ c/s.

$$= \frac{4553\times 355}{1.05} = 1539.3 \text{ kN} > N_{Ed} = 580 \text{ kN}$$

ok //

Colum can withstand axial load without c/s yielding. //

(iv). $N_{b,Rd} = \chi \frac{A_{eff} f_y}{\gamma_{M1}}$

<u>Determination of χ</u>;

Hot finished RHS c/s buckly curve "a" from Table 6.2.

$\alpha_y = \alpha_z = 0.21$

Lowest i gives the largest $\bar{\lambda}$;

$i_z = 62.2\,mm < i_y = 92.4\,mm.$

$\therefore$ Buckling occurs about z-z axis.

$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \cdot \frac{\sqrt{A_{eff}/A}}{\lambda_1} = \frac{7000}{62.2} \times \frac{\sqrt{\frac{4553}{4820}}}{93.9 \times 0.81}$$

$$= 1.438$$

$$\phi_z = 0.5\left[1 + 0.21\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^2\right]$$
$$= 0.5\left[1 + 0.21\left(1.438 - 0.2\right) + 1.438^2\right]$$
$$= 1.664$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = 0.3998 \simeq 0.4$$

$$N_{b,Rd} = \chi_z \frac{A_{eff} f_y}{\gamma_{M1}} = \frac{0.4 \times 4553 \times 355}{1.05}$$

$$= \underline{\underline{615.4}}\ kN > N_{Ed} = 500\,kN$$

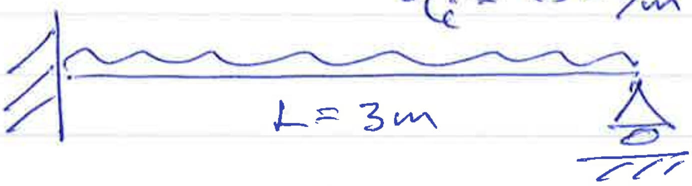
Column can withstand the axial load without overall flexural buckling //

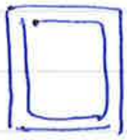
ok.

Question ③

(a).

$q_d = 30\ kN/m$
$q_i = 45\ kN/m$



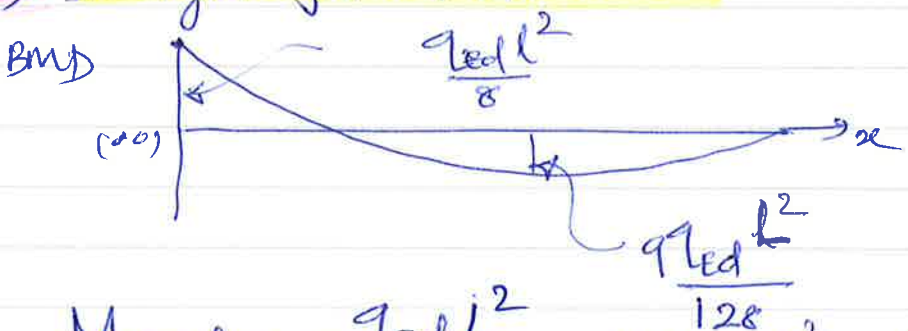


RHS Section
(Table 1.5)

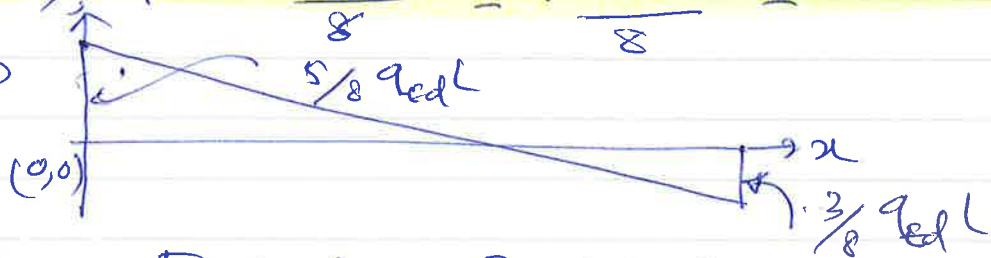
$$q_{Ed} = 1.35(q_d) + 1.5(q_i)$$
$$= 1.35(30) + 1.5(45) = 108\ kN/m$$

S 355 Steel
$f_y = 355\ N/mm^2$
$\varepsilon = 0.81$

(i) Design for ULS



$$M_{y,Ed} = \frac{q_{Ed} L^2}{8} = \frac{108 \times 3^2}{8} = 121.5\ kNm$$
(at x=0)



$$V_{Ed} = \frac{5}{8} q_{Ed} L = \frac{5}{8} \times 108 \times 3 = 202.5\ kN$$
(at x=0)

Trial Section $W_{pl,y} \geq \gamma_{M0} \frac{M_{y,Ed}}{f_y}$

$$\therefore W_{pl,y} \geq \frac{1.05 \times 121.5 \times 10^6}{355}$$

$$W_{pl,y} \geq 359.366 \times 10^3\ mm^3$$

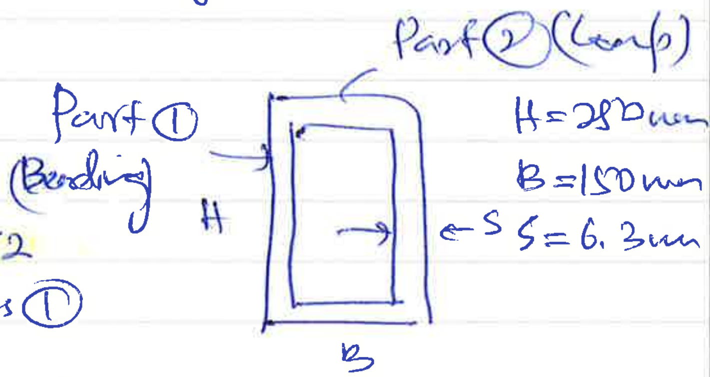
Can select RHS 200×120×10 ; Mass = 45.9 kg/m
Can Select RHS 250×150×6.3 ; Mass = 37.8 kg/m
Most optimized Trial C/S ; RHS 250×150×6.3

$W_{pl,y} = 400 \times 10^3 \ mm^3$

$A = 4820 \ mm^2$

$I_y = 41.1 \times 10^6 \ mm^4$

Class Classification



Part ① (Bending)

$$c/_{S\varepsilon} = \left(\frac{H-3S}{S\varepsilon}\right) = 45.28 < 72$$

Class ①

Part ② (Comp)

$$c/_{S\varepsilon} = \left(\frac{B-3S}{S\varepsilon}\right) = 25.69 < 33$$

Class ①

Cross Section is Body Class ①.

Checking for bending Moment

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{400 \times 10^3 \times 355}{1.05} = 135.23 \ kNm$$

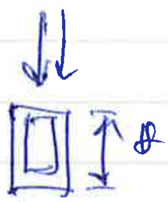
$M_{y,Ed} = 121.5 \ kNm < M_{c,Rd} = 135.23 \ kNm$ ok

No cross sectional yielding due to bending.

No LTB (Lateral Torsion buckling) as RHS.
(i.e Negligible LTB).

$\therefore$ Bending check is Ok //

Checking for shear force.



$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v \ f_y/\sqrt{3}}{\gamma_{M0}}$$

$$A_v = \frac{AH}{(B+H)} = 3012.5 \ mm^2$$

load parallel to depth

$$\therefore V_{c,Rd} = \frac{3012.5 \times 355/\sqrt{3}}{1.05} = 588.04 \ kN$$

$\therefore V_{Ed} = 202.5\ kN < V_{c,Rd} = 588.04\ kN$ ok

No c/s shear yielding //

checking for Combined effect (Bending + Shear)

$V_{Ed} = 202.5\ kN < 0.5 V_{c,Rd} = 294.02\ kN$

So; There is not significant effect of V_{Ed} to reduce the Moment Capacity. (i.e no effect of V_{Ed} on $M_{c,Rd}$)

$\therefore$ Not necessary check for Bearing failure of class (1) webs. //

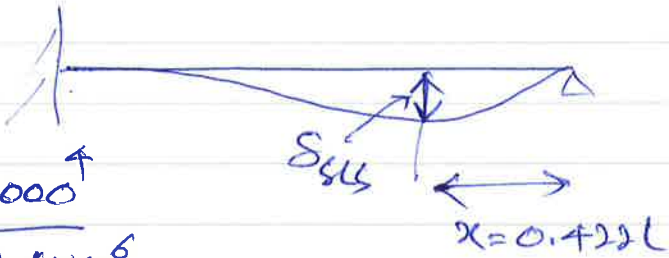
RHS 250×150×6.3 is the Most suitable C/s for ULS. //

(iii). Check for SLS. $q_{SLS} = 1.0(q_d) + 1.0(q_i)$

$= 75\ kN/m.$

$$\delta_{SLS} = \frac{2}{369} \frac{q_{SLS} L^4}{E I_y}$$

(at $x = 0.422L$)



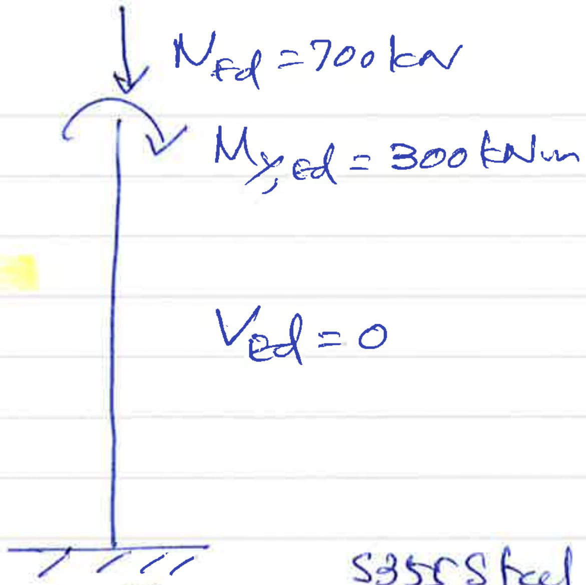
$$\delta_{SLS} = \frac{2}{369} \times \frac{75 \times 10^3 / 10^3 \times 3000^4}{210000 \times 41.1 \times 10^6}$$

$= 3.82\ mm$

$\delta_{SLS} = 3.82\ mm < \delta_{allowable} = L/200 = 15\ mm$

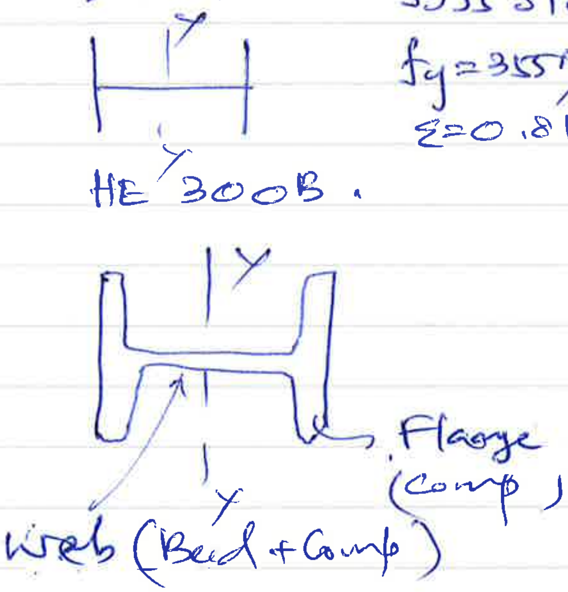
$\therefore$ SLS check is ok //

(b)(ii) Check the Column for C/s yielding shown in Figure 3(b)-(i).



HE 300B
$A = 14900$ mm^2
$S_y = 934 \times 10^3$ mm^3

S355 Steel
$f_y = 355$ N/mm^2
$\varepsilon = 0.81$



Class classification.

Flange (Compression)

$$\frac{c_f}{t_f\varepsilon} = \frac{(b - t_w - 2r)/2}{t_f\varepsilon}$$

$$= \frac{(300 - 11 - 2\times 27)/2}{19\times 0.81} = 7.63 < 9 \quad \text{class 1}$$

Web (Bending + Compression)

$$\frac{c_w}{t_w\varepsilon} = \frac{(h - 2t_f - 2r)}{t_w\varepsilon} = \frac{(300 - 2\times 19 - 2\times 11)}{11\times 0.81}$$

$$= 23.34 < 33$$

class 1. Lower bound of (M+N) class 1

$$\left[\,33 \leq \frac{396}{13\alpha - 1} \leq 72\,\right]$$

$\alpha = 1.0$ fully compressed. $\alpha = 0.5$ Bending

$\therefore$ C/s is class 1.

Checking of the level of effect on M_{Ed} to M_{Rd}.

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{m0}} = \frac{14900 \times 355}{1.05} = 5037.62 \text{ kN}$$

$$M_{pl,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{m0}} = \frac{2S_y f_y}{\gamma_{m0}} = \frac{2 \times 934 \times 10^3 \times 355}{1.05}$$

$$= 631.56 \text{ kNm}$$

$$\frac{h_w t_w f_y}{\gamma_{m0}} = \frac{(h - 2t_f) t_w f_y}{\gamma_{m0}} = \frac{(300 - 2 \times 19) \times 11 \times 355}{1.05}$$

$$= 974.39 \text{ kN}$$

$N_{Ed} = 700$ kN $<$ $0.25 N_{pl,Rd} = 1259$ kN

and $N_{Ed} = 700$ kN $>$ $\frac{0.5 h_w t_w f_y}{\gamma_{m0}} = 487.2$ kN

$\therefore$ N_{Ed} has an effect reduce the M_{Rd} of Column.

(*) For Uniaxial bending; Reduced bending Moment resistance;

$$M_{N,y,Rd} = M_{pl,y,Rd} \left[\frac{1 - n}{1 - 0.5a} \right]$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{700}{5037.62} = 0.1389$$

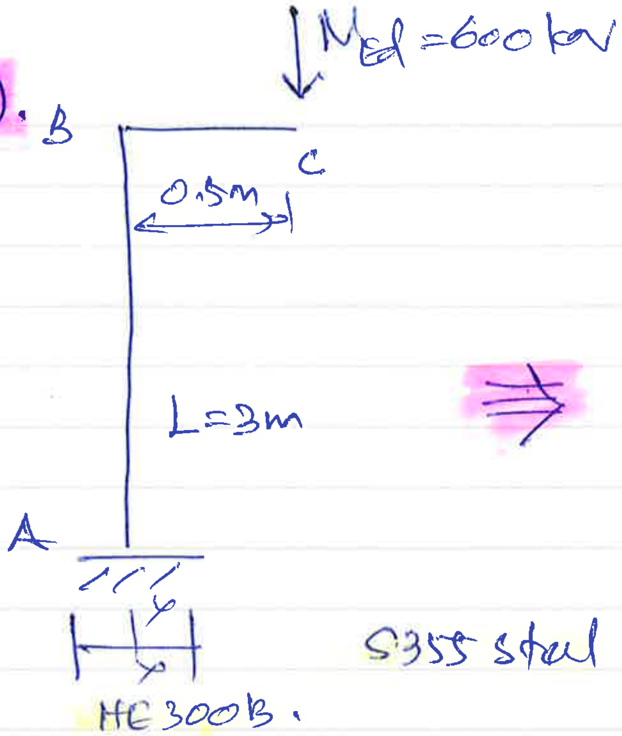
$$a = \frac{A - 2bt_f}{A} = \frac{14900 - 2 \times 300 \times 19}{14900} = 0.235 < 0.5$$

$$\therefore M_{N,y,Rd} = 631.56 \left[\frac{1 - 0.1389}{1 - 0.5 \times 0.235} \right] = 616.2 \text{ kNm}$$

$\therefore$ $M_{y,Ed} = 300$ kNm $<$ $M_{N,y,Rd} = 616.2$ kNm Ok

$\Rightarrow$ Column can withstand above loadings without cross sectional yielding. //

(ii).

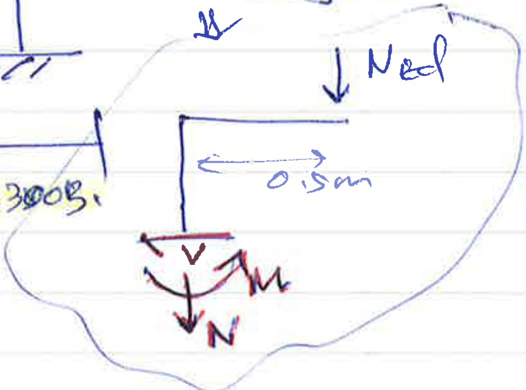


S355 steel.

$\Rightarrow$

$M_{y,Ed} = 600 \times 0.5$
$= 300$ kNm.

$V_{Ed} = 0$ as before.



Equilibrium;
$\Sigma F_x = 0$; $V = 0$
$\Sigma F_y = 0$; $N = -N_{Ed}$.
$\Sigma M = 0$; $M = N_{Ed} \times 0.5$
$= 300$ kNm

above Simplified structure can be imagine as the same column shown in Figure (3)(b)-(ii). No change of c/s class.

Only difference is; $N_{Ed} = 600$ kN [Figure 3(b)-(ii)] < 700 kN [Figure 3(b)(i)]

$M_{y,Ed}$ [Fig 3(b)(ii)] = $M_{y,Ed} = 300$ kNm [Figure 3(b)(i)]

$\therefore$ ($M_{N,y,Rd}$ of Fig. 3(b)-ii) > ($M_{N,y,Rd}$ of Figure 3(b)(i)) = 616.2 kNm

$\therefore$ c/s of AB not Subjected Yielding//

Question (4)

(a) Neglect the effect of Twisting

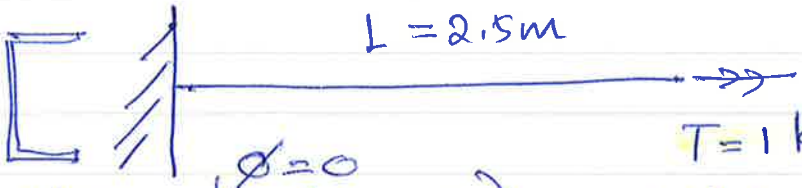
$GJ_T << EC_w$

S 355 Steel.

$f_y = 355 N/mm^2$

$f_u = 490 N/mm^2$

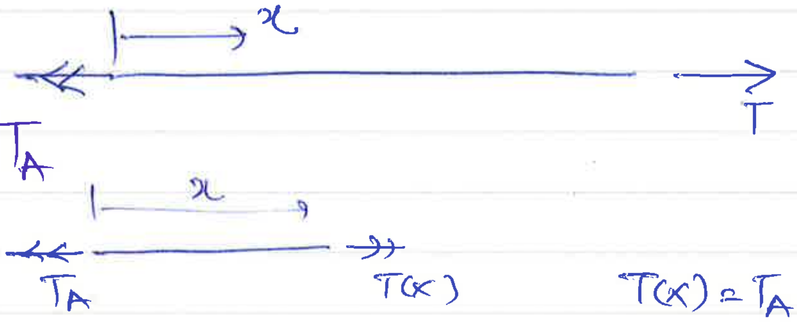
$\varepsilon = 0.81$



U 300 (Table 8.1)

$\left(\phi = 0 \quad \frac{d\phi}{dx} = \phi' = 0\right)$ Twist rotation + warping prevented.

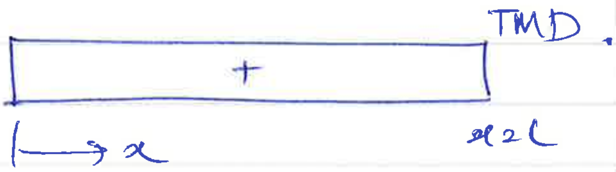
$\left(\frac{d^2\phi}{dx^2} = \phi'' = 0\right)$ Free to warp.



$T - T_A = 0$

$T_A = T$

$T(x) = T_A = T$



$$-EC_w \frac{d^3\phi}{dx^3} = T(x) = T$$

$$\frac{d^3\phi}{dx^3} = \left(-T/EC_w\right)$$

$$\frac{d^2\phi}{dx^2} = -\frac{T}{EC_w}x + C_1$$

$$\frac{d\phi}{dx} = -\frac{T}{EC_w}\frac{x^2}{2} + C_1 x + C_2$$

$$\phi(x) = -\frac{T}{EC_w}\frac{x^3}{6} + C_1\frac{x^2}{2} + C_2 x + C_3$$

bc^{n}s: at $x = L$; $\frac{d^2\phi}{dx^2} = 0$; $C_1 = \frac{TL}{EC_w}$

at $x = 0$; $\frac{d\phi}{dx} = 0$; $C_2 = 0$

at $x=0$; $\phi=0$; $c_3=0$

$$\therefore \phi(x) = \frac{Tx^2}{6EC_w}(3L-x)$$

$$\frac{d^2\phi}{dx^2} = -\frac{T}{EC_w}x + \frac{TL}{EC_w} = \frac{T}{EC_w}(L-x)$$

For $x=0$; $\left(\frac{d^2\phi}{dx^2}\right)_{max} = \frac{TL}{EC_w} = \phi_{xx}$

For $0 \leq x \leq L$; $\left(\frac{d^3\phi}{dx^3}\right) = -\frac{T}{EC_w} = \phi_{xxx}$

(i). $T = 1\ kNm$ $\quad L = 2.5\ m$

$b = 100 - S/2 = 95\ mm$

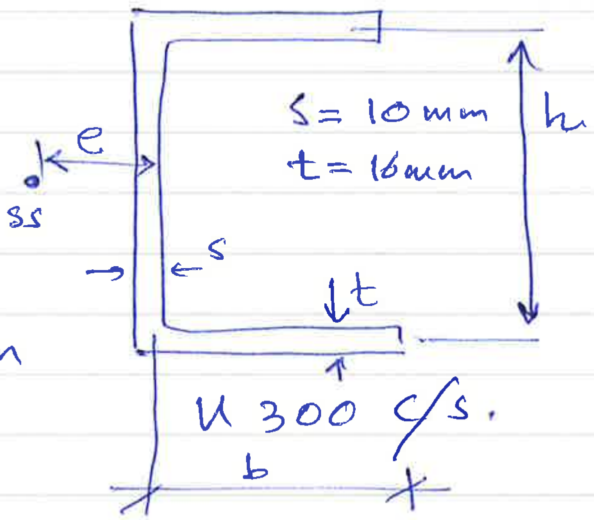
$h = 300 - t = 284\ mm$

$e_2 = 27\ mm$

$y_s = 54.1\ mm$

$e = (y_s - e_2 + S/2)$

$= (54.1 - 27 + 10/2) = 32.1\ mm$



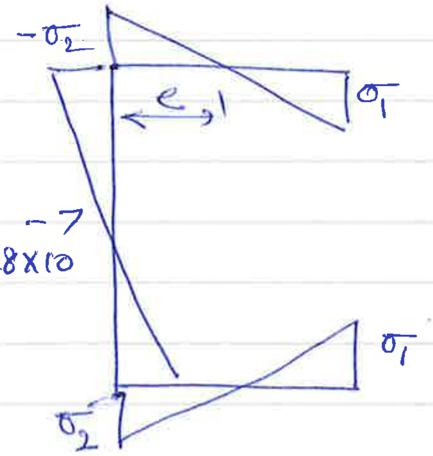
$C_w = 69.1 \times 10^9\ mm^6$

$I_t = 374 \times 10^3\ mm^4$

$$(\phi_{xx})_{at\ x=0} = \left(\frac{d^2\phi}{dx^2}\right)_{max} = \frac{TL}{EC_w} = \frac{1\times10^6 \times 2500}{210000 \times 69.1\times10^9} = 1.72283\times10^{-7}$$

$$(\phi_{xxx})_{0 \leq x \leq L} = \left(\frac{d^3\phi}{dx^3}\right)_{max} = -\frac{T}{EC_w} = \frac{-1\times10^6}{210000\times69.1\times10^9} = -6.8913\times10^{-11}$$

Table 5.3 Profile book



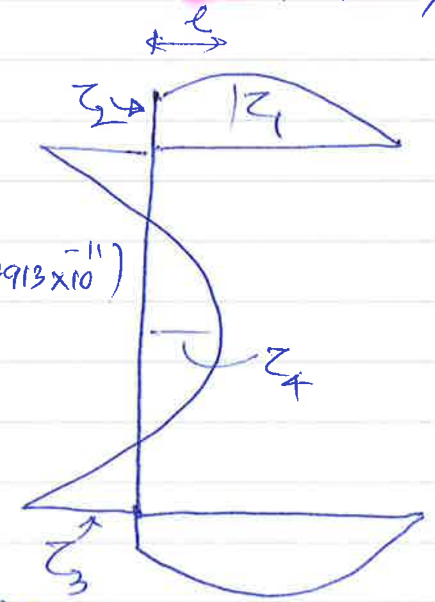
$$\sigma_1 = \left(\frac{b}{2} - e\right) h\, E\, \phi_{xx}$$

$$= \left(\frac{95}{2} - 32.1\right) \times 284 \times 210 \times 10^3 \times 1.7228 \times 10^{-7}$$

$$= 323.15 \text{ N/mm}^2$$

$$\sigma_2 = \frac{eh}{2} E\, \phi_{xx}$$

$$= \frac{32.1 \times 284 \times 210 \times 10^3 \times 1.7228 \times 10^{-7}}{2} = 164.91 \text{ N/mm}^2$$



$$\tau_1 = \left(\frac{b}{2} - e\right)^2 \frac{h}{4} E\, \phi_{xxx}$$

$$= \frac{\left(\frac{95}{2} - 32.1\right)^2}{4} 284 \times 210 \times 10^3 \times (-6.8913 \times 10^{-11})$$

$$= -4.07 \text{ N/mm}^2$$

$$\tau_2 = \frac{(b - 2e)}{4} b h E\, \phi_{xxx}$$

$$= \frac{(95 - 2 \times 32.1)}{4} 95 \times 284 \times 210 \times 10^3 \times (-6.8913 \times 10^{-11})$$

$$= -3.01 \text{ N/mm}^2$$

$$\tau_3 = t/s\ \tau_2 = 16/10 \times (-3.01) = -4.81 \text{ N/mm}^2$$

$$\tau_4 = \frac{eh^2}{24} E\, \phi_{xxx} = \frac{32.1 \times 284^2}{24} \times 210 \times 10^3 \times (-6.8913 \times 10^{-11})$$

$$= -1.56 \text{ N/mm}^2$$

Maximum value of warping normal stress $(\sigma_{w,max}) = \underline{\underline{323.15 \text{ N/mm}^2}}$ at $x = 0$

Maximum value of warping shear stress $\tau_{w,max} = \underline{\underline{-4.81 \text{ N/mm}^2}}$ at $0 < x \leq L$

(ii). Check the suitability of the beam for (ULS).

For Normal stress;

$$(\sigma_{w,max}) = 323.15\ N/mm^2 < f_y/\gamma_{M0} = \frac{355}{1.05} = 338\ N/mm^2$$

at $x=0$

ok//

For shear stress;

$$|\tau_{w,max}| = 4.81\ N/mm^2 < \tau_y/\gamma_{M0} = f_y/\sqrt{3}\gamma_{M0} = 195\ N/mm^2$$

$(at\ 0 \leq x \leq L)$

ok//

Member can withstand the design torque//.

(iii).

Free.

Torsionally Pinned.

at $x=0$

$\phi = 0$; $u \neq 0$

$\frac{d^2\phi}{dx^2} = 0$;

Warping stress is zero.

$\frac{d^2\phi}{dx^2} = 0$

No where of the member has restrained against warping displacement (i.e $u \neq 0$).

Free to warp:

No warping stress induced.

$\sigma_w \longrightarrow 0$; only $\tau_t = \checkmark$

$\tau_w \longrightarrow 0$

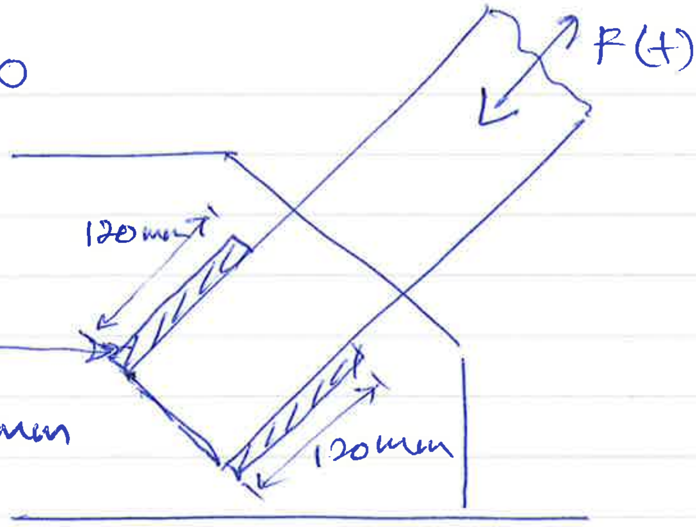
$$GI_T \frac{d\phi}{dx} - \cancel{EI_w \frac{d^3\phi}{dx^3}} = T(x) \quad ; \quad GI_T \frac{d\phi}{dx} = T(x)$$

$\cancel{EI_w \frac{d^3\phi}{dx^3}} \to 0$

governing formula.

No failure due to warping Normal stress ($\sigma_{w,max}$)//

(b). Member 150x100

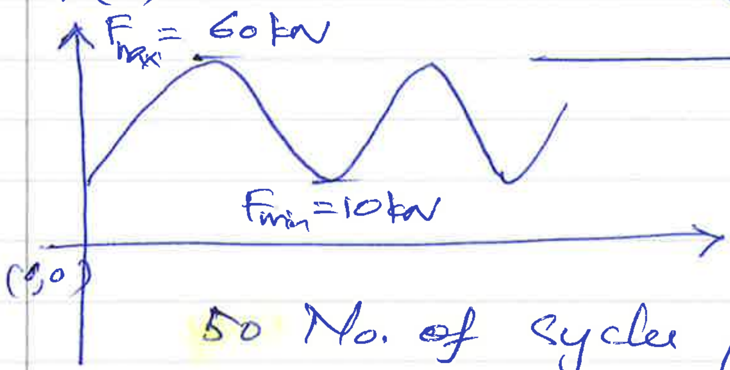


Fillet weld leg

$a = 8/\sqrt{2} = 5.66\,mm$

$L = 120 - 2a$

$= 108.68\,mm$



$\Delta F = F_{max} - F_{min}$

$= 60 - 10 = 50\,kN$

50 No. of cycle per day.

$\left.\begin{array}{l}\text{Assessment method} = \text{Safe life} \\ \text{Consequence of failure} = \text{High}\end{array}\right\}$ Table NA 3.1 $\gamma_{Mf} = 2.0$

$\gamma_{Ff} = 1.0$

(i). Fatigue life of the fillet weld.

$V_{Tz} = 0 \quad M = 0 \quad V_{Ty} = 0 \quad V_L = F(t)/2$

$\therefore \sigma_{\perp f} = 0 \quad \tau_{\perp f} = 0 \quad \therefore \sigma_{wf} = \sqrt{\sigma_{\perp f}^2 + \tau_{\perp f}^2} = 0$

$\tau_{\parallel f} = V_L / aL = \tau_{wf}$

$$\therefore \Delta \tau_{wf} = \Delta V_L / aL = \frac{\Delta F}{2aL} = \frac{50 \times 10^3}{2 \times 8/\sqrt{2} \times 108.68} = 40.66\,N/mm^2$$

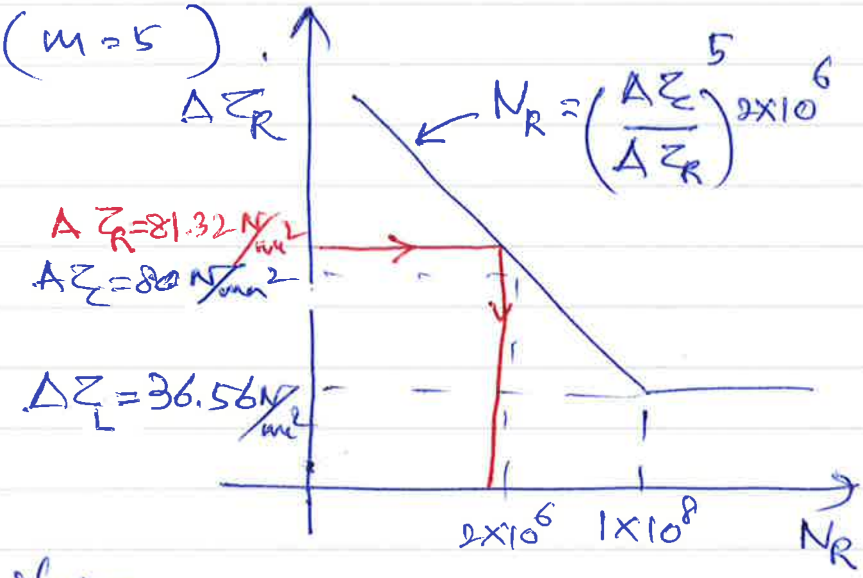
$$\Delta \tau_R = \gamma_{Mf}\gamma_{Ff}\Delta\tau_{wf} = 2.0 \times 1.0 \times 40.66 = 81.32\,N/mm^2$$

Detail Category, Figure (9) in Table 8.5

$\Delta\tau_c = 80\ N/mm^2$ $(m=5)$.

$\Delta\tau_L = 0.457\,\Delta\tau_c$

$= 36.56\ N/mm^2$



No of cycles to failure

$$N_R = \left(\frac{\Delta\tau_c}{\Delta\tau_R}\right)^5 2\times10^6 = \left(\frac{80}{81.32}\right)^5 2\times10^6$$

$$= 1\,842\,863 \text{ cycles.}$$

Fatigue life $= \left(\frac{N_R}{50\times365}\right) = 100.97$ Years.

$\simeq 100$ years

(ii). If we have to determine of Joint B; we have to calculate the fatigue life all the components which connected to the joint B. i.e;

(1). Fatigue life of AB Member.
(Subjected to Tension; D.C = 45*)

(2). Fatigue life of fillet weld. (Section (i))
(Subjected to shear D.C = 80 N/mm² m=5)

(3). Fatigue life of the Connecting plate.
(Subjected Combined stresses; Table 8.5, Figure (4))

$\therefore$ Fatigue life of Joint B = Min[(1); (2); (3)]