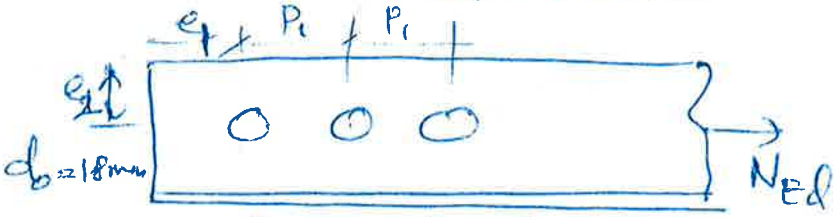
Question (1) -(a)

(i). $A = 1910\ mm^2$

$N_{ed} = 180\ kN$



UA 120 x 80x10

$e_2 = e_1 = P_1 = 60\ mm$

S 275 Steel.

$t_f = 10\ mm < 40\ mm$ From Table 3.1

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

(*) Check for the Cross Sectional yielding ;

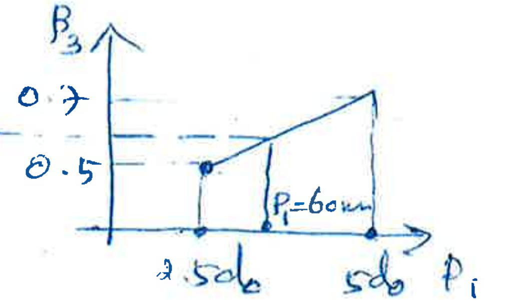
$$N_{Pl,Rd} = A f_y/\gamma_{M0} = \frac{1910 \times 275}{1.05 \times 10^3} = 500.23\ kN$$

$N_{Ed} = 180\ kN < N_{Pl,Rd} = 500.23\ kN$.

No cross Sectional yielding.

(*). Check for Local fracture near the holes.

$$N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{m2}}$$



$$\beta_3 = 0.5 + \frac{(0.7 - 0.5)}{(5d_0 - 2.5d_0)} \times (b_0 - 2.5d_0)$$

$= 45mm$ $= 90mm$

$$= 0.5 + \frac{0.2}{45} \times 15$$

$$= 0.567$$

$A_{net} = (A - d_0 t)$

$= 1910 - 18 \times 10$

$= 1730\ mm^2$

$$N_{u,Rd} = \frac{0.567 \times 1730 \times 430}{1.25 \times 10^3}$$

$$= 337.2\ kN$$

$N_{Ed} = 180\ kN < N_{u,Rd} = 337.2\ kN$

10

No Local fracture. SECTION is SUITABLE

(ii). If the spacing between 2 bolts;

$p_1 = 40\,mm$ β_2 decreasing

$\beta_2 = 0.5$

$$N_{u,Rd} = \frac{0.5 \times 1730 \times 430}{1.25 \times 10^3} = 297.56\,kN$$

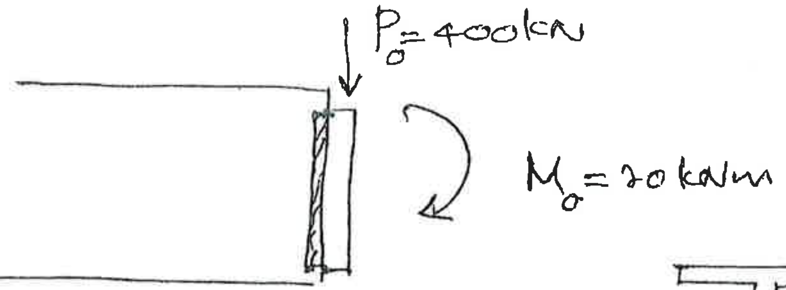
$N_{Ed} = 180\,kN < N_{u,Rd} = 297.56\,kN$

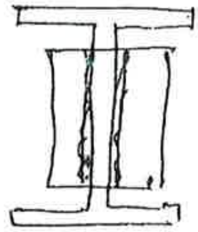
No local fracture of c/section

still can carry above loading //

3

(b).





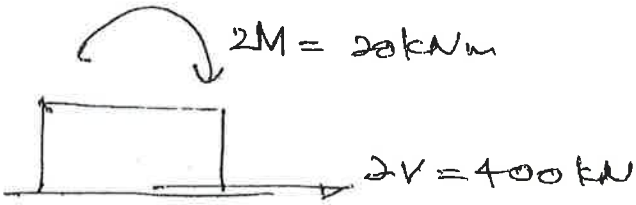
Weld length $L = 280 - 2 \times a$
$= 271.5$ mm.

plate material
S 355 steel.

$a = 6/\sqrt{2} = 4.24mm$

it is o.k $a > 0.5t_p = 4mm$

$f_y = 355\ N/mm^2$
$f_u = 510\ N/mm^2$
$\beta_w = 0.9$ (Table 6.2)



$M = 10\ kNm$
$V = 200\ kN$

Table 6.2 - page 49

$$\sigma_\perp = \tau_\perp = \frac{M}{\frac{\sqrt{2}}{12} a L^3} \cdot \frac{L}{2} = \frac{10 \times 10^6 \times 12}{\cancel{\sqrt{2}} \times 6/\cancel{\sqrt{2}} \times (271.5)^2 \cdot 2}$$
$$= 135.65\ N/mm^2$$

$$\tau_\parallel = \frac{V}{La} = \frac{200 \times 10^3}{(271.5) \times 6/\sqrt{2}} = 173.62\ N/mm^2$$

Check ①:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)} = \sqrt{135.65^2 + 3(135.65^2 + 173.62^2)}$$
$$= 405.01 < \frac{f_u}{1.25\beta_w} = \frac{510}{1.25 \times 0.9} = 453$$

Check ① is O.k

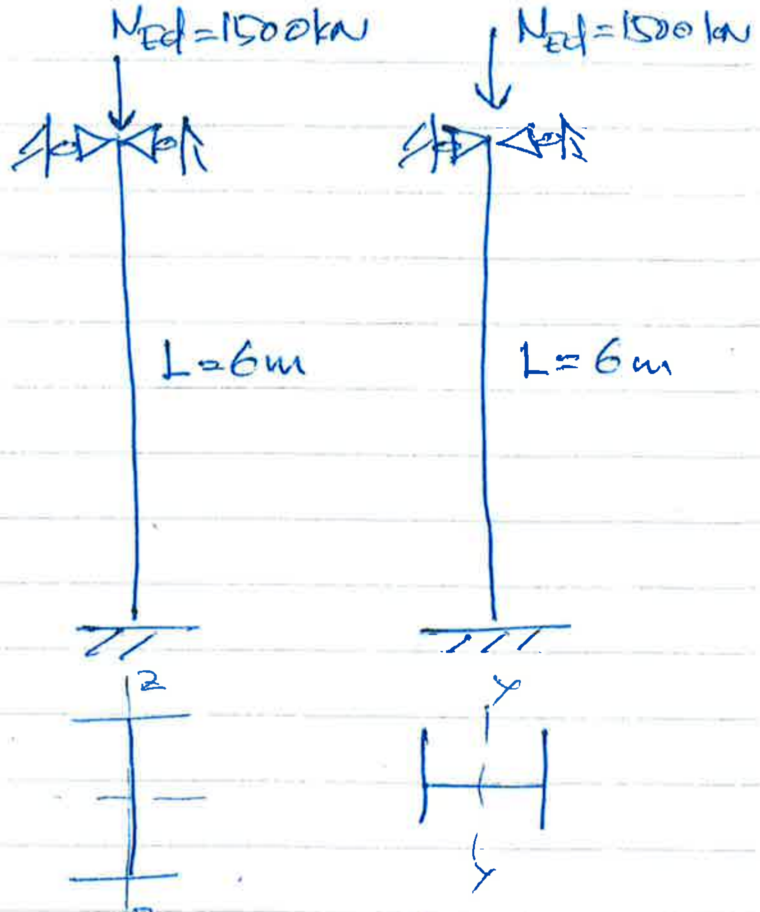
Check ②

$$\sigma_\perp = 135.65\ N/mm^2 < \frac{f_u}{1.25} = \frac{510}{1.25} = 408\ N/mm^2$$

Check ② is o.k.

Welded Joint Can carry above loadings.

Question (2)



(a)(i) S275 Steel

$f_y = 275\,N/mm^2$

$\varepsilon = 0.92$

HE 200 B

$A = 7810\,mm^2$

$b = 200\,mm$

$h = 200\,mm$

$t_f = 15\,mm$

$t_w = 9\,mm$

$r = 18\,mm$

$i_y = 85.4\,mm$

$i_z = 50.7\,mm$

$L_{cr,z} = 0.7L$ $\quad$ $L_{cr,z} = 0.7L$

$= 4.2m$ $\quad$ $= 4.2m$

Class Classification;

Flange (Compression)

$$\frac{c_f}{t_f\varepsilon} = \frac{(b - t_w - 2r)/2}{t_f\,\varepsilon} = 5.61 < 9$$

Class ①

Web (Compression)

$$\frac{c_w}{t_w\varepsilon} = \frac{(h - 2t_f - 2r)}{t_w\,\varepsilon} = 16.18 < 33$$

Class ①

Cross Section is Class ①//

(No Local buckling).

Check for Cross Sectional Yielding.

$$N_{c,Rd} = \frac{A f_y}{\gamma_{m0}} = \frac{7810 \times 275}{1.05 \times 10^3} = 2045.1\,kN$$

$N_{Ed} = 1500\,kN < N_{c,Rd} = 2045.1\,kN$. Ok//

No Cross Sectional Yielding.

Checking for overall flexural buckling.

$$N_{b,Rd} = \chi A f_y / \gamma_{M_1}$$

$$\bar{\lambda}_y = \frac{l_{cr,y}}{i_y} \cdot \frac{1}{\lambda_1}$$
$$= \frac{4200}{85.4} \cdot \frac{1}{93.9 \times 0.92}$$
$$= 0.566$$

$$\bar{\lambda}_z = \frac{l_{cr,z}}{i_z} \cdot \frac{1}{\lambda_1}$$
$$= \frac{4200}{50.7} \cdot \frac{1}{93.9 \times 0.92}$$
$$= 0.959$$

Table 6.2 ; Rolled ; $h/b = 1.0 < 1.2$; $t_f < 100mm$

y-y axis buckling

$$\alpha_y = 0.34$$
$$\phi_y = 0.5\left[1 + \alpha_y(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2\right]$$
$$\phi_y = 0.722$$
$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^2 - \bar{\lambda}_y^2}} = 0.853$$

z-z axis buckling

$$\alpha_z = 0.49$$
$$\phi_z = 0.5\left[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2\right]$$
$$\phi_z = 1.140$$
$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = 0.567$$

χ = Smaller of $[\chi_y : \chi_z] = 0.567$

$$\therefore N_{b,Rd} = \frac{0.567 \times 7810 \times 275}{1.05 \times 10^3} = 1160 \text{ kN}$$

$N_{Ed} = 1500\,kN > N_{b,Rd} = 1160\,kN$

Not ok //

Column can subjected to buckling failure. //

HE 200 B Cross Section is not Suitable //

(a)(ii). To improve ;

(1). Cross Sectional Resistance ($N_{c,Rd}$)

$A \longrightarrow$ larger $\longrightarrow$ higher Cross Section.

$f_y \longrightarrow$ larger. $\longrightarrow$ higher Steel grade.

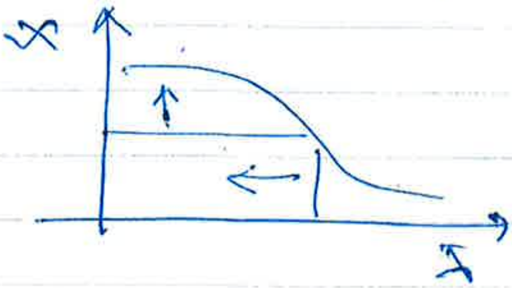
(But S460 is not Suitabel for plastic Design).

low ductility.

(2). Buckling Resistance ($N_{b,Rd}$)

A — larger $f_y \rightarrow$ larger

Smaller $\bar{\lambda}\downarrow \longrightarrow$ larger $\chi\uparrow$

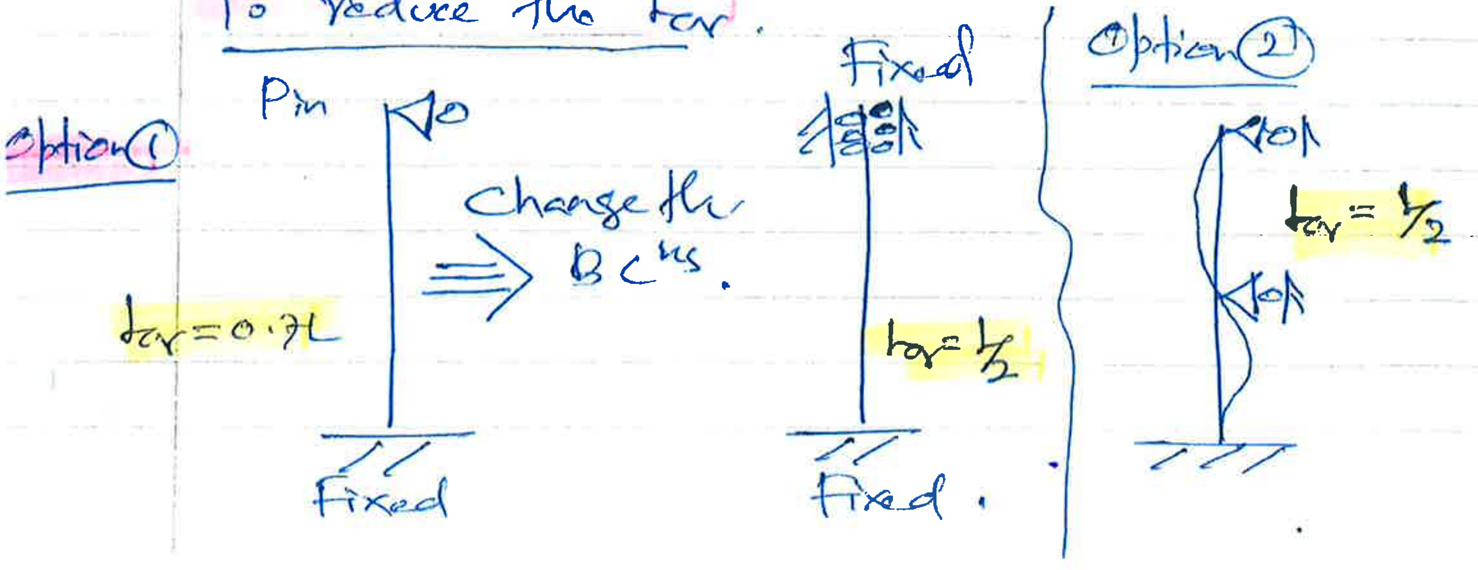


$$\bar{\lambda} = \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1}$$

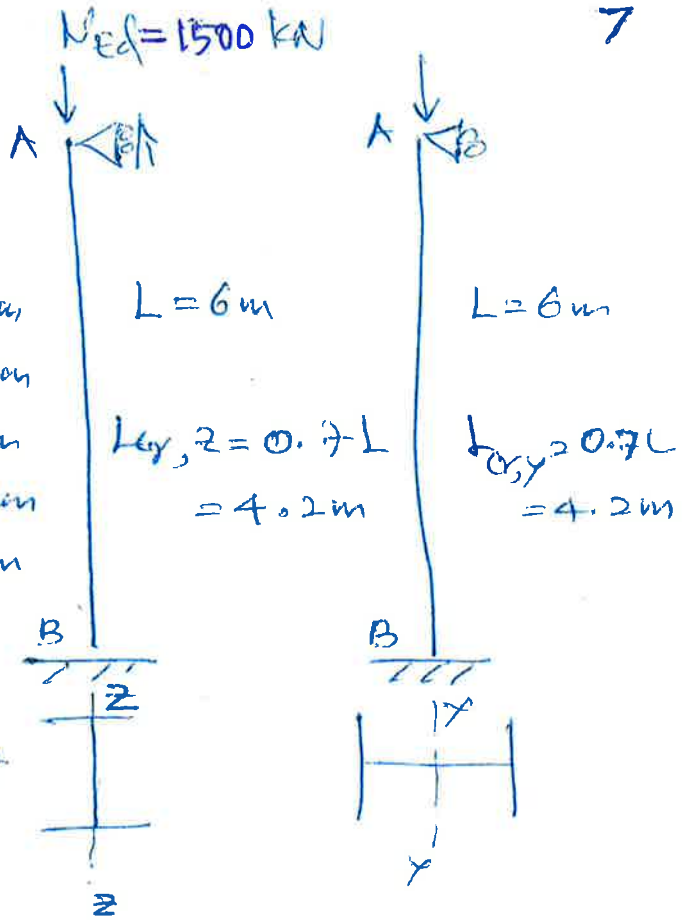
larger cross section "i" larger. $\longrightarrow \bar{\lambda}$ small.

L_{cr} Smaller $\longrightarrow \bar{\lambda}$ small.

To reduce the L_{cr}.



(b)



$h = 474$ mm
$b = 190$ mm
$t_f = 12$ mm
$t_w = 10$ mm
$r = 8$ mm

S 275 Steel
$\varepsilon = 0.92$
$f_y = 275\ N/mm^2$

$A = 2bt_f + (h - 2t_f)t_w = 9060\ mm^2$

$i_y = 187.7$ mm $\qquad i_z = 38.9$ mm

(i). Class Classification

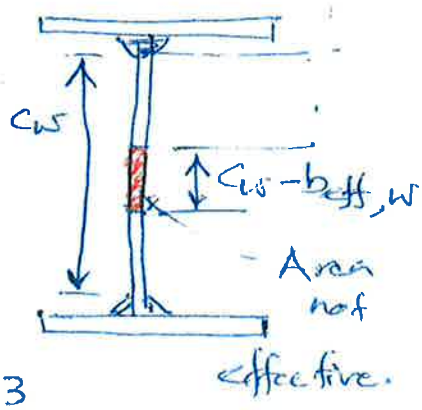
Flange (Compression): $c_f = (b - t_w - 2r)/2 = 82$ mm

$c_f / t_f\varepsilon = 7.4 < 9$ class ①.

Web (Compression) $c_w = (h - 2t_f - 2r) = 434$ mm

$c_w / t_w\varepsilon = 47 > 42$ class ④ (Slender)

C/S is Class ④ //



Web Compression Member (Internal) Table 4.1

$\psi = \dfrac{\sigma_2}{\sigma_1} = 1 \quad ; \quad k_{\sigma,w} = 4.0$

$$\bar{\lambda}_{p,w} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{c_w/t_w}{28.4\varepsilon\sqrt{k_\sigma}} = 0.83 > 0.673$$

$$\rho_w = \frac{\bar{\lambda}_{p,w} - 0.055(3+\psi)}{\bar{\lambda}_p^2} = 0.88$$

$$b_{eff,w} = \rho_w \bar{b} = \rho_w c_w = 382 \sim 385 \text{ mm}$$

$$A_{eff} = A - (c_w - b_{eff,w})t_w = \underline{\underline{8540 \sim 8573}} \text{ mm}$$

(ii). $N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} = 2236 \sim 2245$ kN

$N_{Ed} = 1500$ kN $< N_{c,Rd} = 2236$ kN O.K

∴ Column is suitable to withstand the load with c/s yielding.

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

Lowest i gives largest $\bar{\lambda}$ & smallest χ ; smallest $N_{b,Rd}$.

$i_z < i_y$ $\qquad L_{cr,y} = L_{cr,z}$

$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \frac{\sqrt{A_{eff}/A}}{\lambda_1}$$

$$= \frac{4200}{38.9} \frac{\sqrt{8540/9060}}{93.9 \times 0.92} = 1.21$$

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

Welded section. $t_f < 40$ mm ; z–z axis curve c ; $\alpha_z = 0.49$.

$\phi_z = 1.48$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = 0.43$$

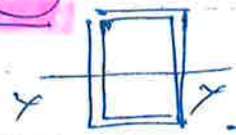
$$N_{b,Rd} = \frac{\chi_z \, A_{eff} \, f_y}{\gamma_{m_1}} = 962 \sim 964 \text{ kN}$$

$N_{Ed} = 1500 \text{ kN} > N_{b,Rd} = 962 \text{ kN}$ Not O.K

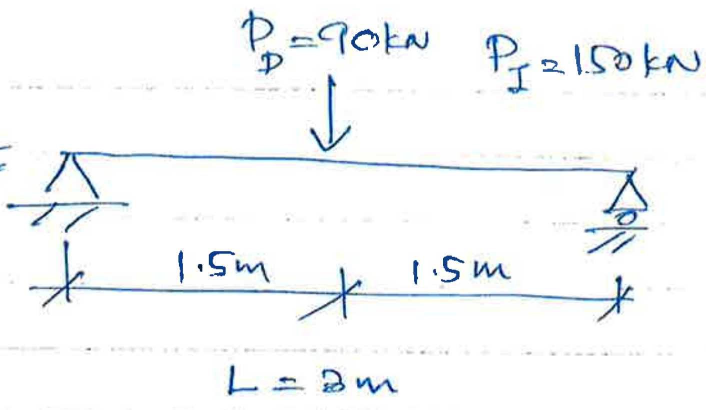
Column is NOT Suitable to withstand the load without buckling failure.

Question (3)

(a)(i)



RHS Section.



S 355 steel

$f_y = 355\,N/mm^2$ $\quad \varepsilon = 0.81$

$\therefore$ Design load $P_{Ed} = 1.35(P_D) + 1.5(P_I)$

$= 346.5\,kN$

$$M_{Ed} = \frac{P_{Ed}L}{4} = \frac{346.5 \times 3}{4} = 259.87\ kNm$$

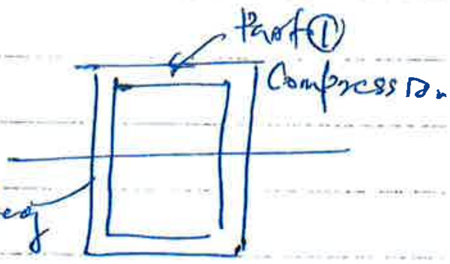
$$V_{Ed} = \frac{P_{Ed}}{2} = \frac{346.5}{2} = 173.25\ kN$$

Trail Section; $W_{pl,y} \geq \dfrac{\gamma_{mo} M_{Ed}}{f_y}$

$$W_{pl,y} \geq \frac{1.05 \times 259.87 \times 10^6}{355} = 768.6 \times 10^3$$

Select RHS $300 \times 200 \times 8$; $W_{pl,y} = 775 \times 10^3\ mm^3$

as Trail Section. $\quad A = 7650\ mm^2$



Class Classification;

Part (1); Compression.

$$c/t\varepsilon = \frac{(200 - 3\times8)}{8 \times 0.81} = 27.16 < 33$$

class (1).

Part (2): Bending

$$c/t\varepsilon = \frac{(300 - 3\times8)}{8 \times 0.81} = 42.59 < 72 \quad \text{class (1)}$$

$\therefore$ Cross Section is class (1) //

Checking for bending moment

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{775 \times 10^3 \times 355}{1.05 \times 10^6} = 262 \text{ kNm}$$

$\therefore M_{Ed} = 259.87$ kN $< M_{c,Rd} = 262$ kNm Ok.
No cross sectional yielding

RHS Sections negligible LTB. No LTB
$\therefore$ Bending check is Ok.

Checking for shear force

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

$$A_v = \frac{Ah}{b+h} = \frac{7650 \times 300}{(300+200)} = 4590 \text{ mm}^2$$

$$V_{c,Rd} = \frac{4590 \times 355/\sqrt{3}}{1.05 \times 10^3} = 895.96 \text{ kN}$$

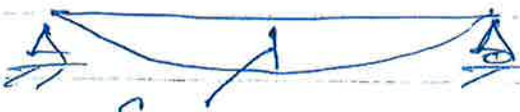
$V_{Ed} = 173.25$ kN $< V_{c,Rd} = 895.96$ kN ok
No cross Sectional Shear Yielding.

Checking for Combined effect (M+V)

$V_{Ed} = 173.25$ kN $< 0.5 V_{c,Rd} = 447.98$ kN.
No effect of V_{Ed} to $M_{c,Rd}$.

$\therefore$ RHS 300x200x8 is Suitable Cross Section

(ii) $P_{SLS} = 1.0(P_D) + 1.0(P_I)$
$= 240$ kN



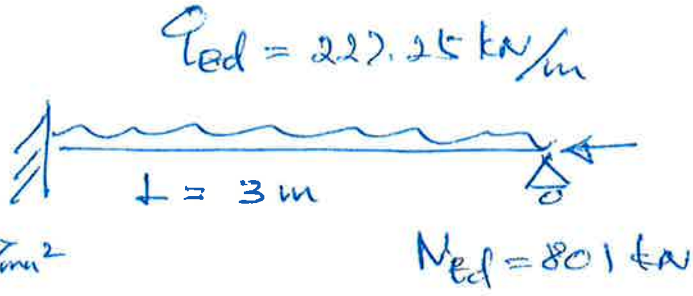
$$\delta_{SLS} = \frac{1}{48} \cdot \frac{P_{SLS} L^3}{EI}$$

δ_{SLS}

$$= \frac{1}{48} \cdot \frac{240 \times 10^3 \times 3000^3}{210 \times 10^3 \times 96.5 \times 10^6} = 6.66 \text{ mm} < \delta_{all} = L/200 = 15 \text{ mm}$$

$(\delta_{SLS})_{obtained} < \delta_{all}$: SLS check is ok

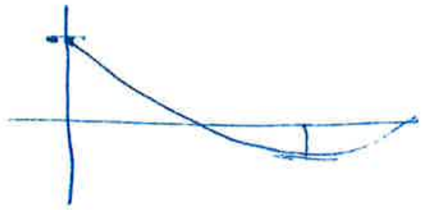
(b)



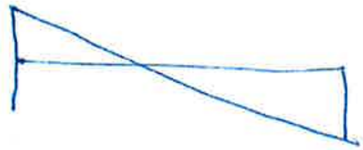
IPE 500 ;
S 275 steel ; $f_y = 275 N/mm^2$
$\varepsilon = 0.92$

IPE 500 $h = 500$. $b = 200$ $t_f = 16$ $t_w = 10.2$
$r = 21$ $A = 11600\ mm^2$ $I_y = 482 \times 10^6\ mm^4$
$S_y = 1100 \times 10^3\ mm^3$

$$M_{y,Ed} = \frac{1}{8} q_{Ed} l^2$$
$$= 255.7 \text{ kNm}$$

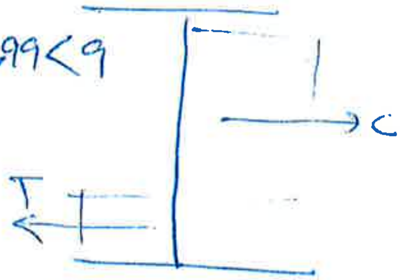


$$V_{Ed} = \frac{5}{8} q_{Ed} L = 426 \text{ kN}$$



$$N_{Ed} = 801 \text{ kN}$$

(i) Section classification (i.e Class)



Flange: $\frac{c_f}{t_f \varepsilon} = \frac{(b - t_w - 2r)/2}{t_f \varepsilon} = 4.99 < 9$
Class ①

Web: Combined action (N + M).
$c_w = 426$ mm $\quad c_w / t_w \varepsilon = 45.2$

$$(f_y t_w \alpha c_w) - (f_y (1-\alpha) c_w t_w) = N_{Ed}$$

$$\alpha = \left[\frac{N_{Ed}}{f_y t_w c_w} + 1 \right] / 2$$

$$= \left[\frac{801 \times 10^3}{275 \times 10.2 \times 426} + 1 \right] / 2$$

$$\alpha = 0.8352$$

Limit Class ① ; $\frac{396}{13\alpha-1} = 40.17$

Limit Class ② ; $\frac{456}{13\alpha-1} = 46.26$

∴ Web is class ②.

C/S is class ②

(ii). Check for buckling failure.

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{m1}}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{m1}}} \leq 1.0$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{m1}}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT}\cdot\frac{M_{y,Rk}}{\gamma_{m1}}} \leq 1.0$$

$k_{yy} = 0.54$
$k_{zy} = 0.32$

$N_{Ed} = 801$ kN $\quad M_{y,Ed} = 255.7$ kNm

$M_{z,Ed} = 0$; $N_{Rk} = Af_y = 3190$ kN

$M_{y,Rk} = f_y W_{pl,y} = 2S_y f_y$

$= \frac{2 \times 1100 \times 10^3 \times 275}{10^6} = 605$ kNm

$\gamma_{m1} = 1.05$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{2 \times 1100 \times 10^3 \times 275}{1634 \times 10^6}} = 0.61$$

$h/b = 500/200 > 2$ Curve b ; $\alpha_{LT} = 0.34$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2\right] = 0.75$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = 0.833$$

$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \cdot \frac{1}{\lambda_1} = \frac{0.7 \times 3000}{204 \times \lambda_1}$

$= 0.12 < 0.2$

No buckling along y-y axis

$\chi_y = 1.0$

$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z} \cdot \frac{1}{\lambda_1}$

$= \frac{2 \times 400}{43.1 \times 93.9 \times 0.92}$

$= 0.56$

Table 6.2 $h/b > 1.2$; z-z

$\alpha_z = 0.34$

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

$= 0.72$

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

$= 0.85$

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{801}{\left(\frac{1.0 \times 3190}{1.05}\right)} + 0.54 \times \frac{255.7}{\left(\frac{0.833 \times 605}{1.05}\right)} = 0.55 < 1$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}}$$

$$= \frac{801}{\left(\frac{0.85 \times 3190}{1.05}\right)} + 0.32 \times \frac{255.7}{\left(\frac{0.833 \times 605}{1.05}\right)} = 0.48 < 1$$

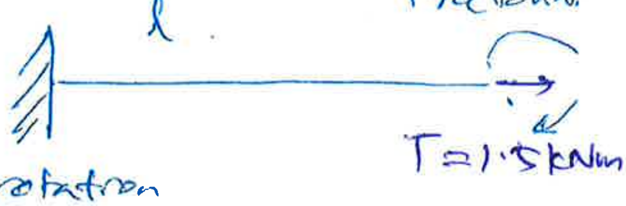
No buckling failure.

No LTB, flexural buckling //

Question (4)

(a)(i) $T_{w,max}$? ϕ_{max} ? $Z_{w,max}$? $\sigma_{w,max}$?

(neglect the effect of St Venant torsion)



Twist rotation + warping prevented

$\phi = 0$

$\phi' = 0$

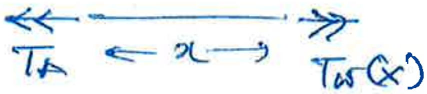
$\phi'' = 0$ Free to twist

Statically determinate structure.

From Rotational Equilibrium ; $\gg$ $T - T_A = 0$

$T_A = T = 3\text{ kNm}$

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



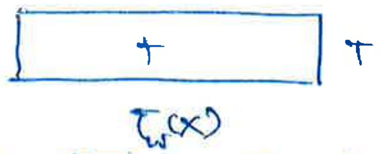
$T_w(x) = T_A = T$

$$\frac{d^3\phi}{dx^3} = \left(\frac{-T}{EC_w}\right) \quad \text{—} \circledast$$

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

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

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



$T_{w,max} = T = \underline{\underline{3\text{kNm}}}$

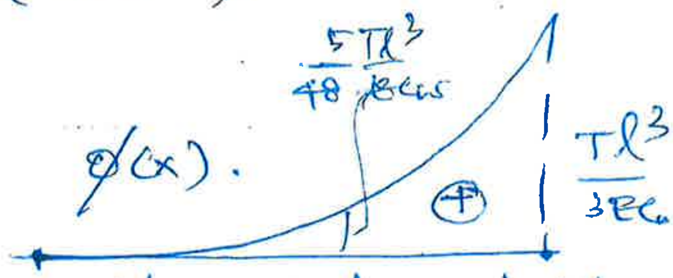
BC^{ns}; at $x = l$; $\frac{d^2\phi}{dx^2} = 0$; $C_1 = \frac{Tl}{EC_w}$

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

at $x = 0$; $\phi = 0$; $C_3 = 0$

$$\phi(x) = \frac{T}{6EC_w}\left[3lx^2 - x^3\right] = \frac{Tx^2}{6EC_w}(3l - x)$$

at $x = l$; $\phi_{max} = \left(\dfrac{Tl^3}{3EC_w}\right)$



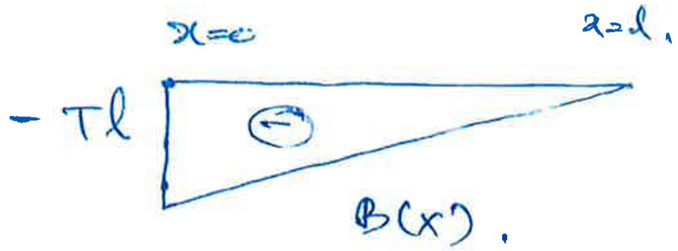
Non Uniform twist
(Non-Uniform torsion)

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

$$\frac{d^2\phi}{dx^2} = \frac{T}{EC_w}(l-x) \quad \text{———} \quad (*)$$

$$\therefore \quad B(x) = -EC_w\frac{d^2\phi}{dx^2} = -T(l-x)$$

$B_{max} = -Tl$

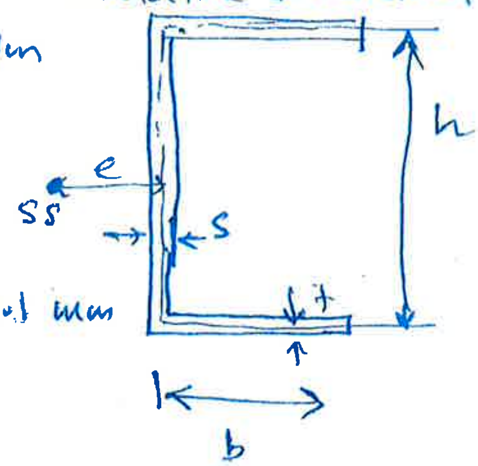


$$\left(\frac{d^2\phi}{dx^2}\right)_{max} = \frac{Tl}{EC_w} \quad (\text{at } x=0).$$

$$\left(\frac{d^3\phi}{dx^3}\right)_{max} = -\frac{T}{EC_w} \quad (\text{Throughout the beam.})$$

U 400 channel Section.

middline Dimension.



$b = 103$ mm
$h = 382$ mm
$s = 14$ mm
$t = 18$ mm

$T = 1.5$ kNm
$l = 3$ m

$e_z = 26.5$ mm $\quad y_s = 51.1$ mm

$C_w = 221 \times 10^9$ mm^6

$I_t = 816 \times 10^3$ mm^4

$e = (y_s - e_z + s/2) = (51.1 - 26.5 + 14/2) = 32$ mm

$$\phi_{xx} = \left(\frac{d^2\phi}{dx^2}\right)_{max} = \frac{Tl}{EC_w} = \frac{1.5\times10^6 \times 3000}{210000 \times 221\times10^9} = 9.7 \times 10^{-8}$$

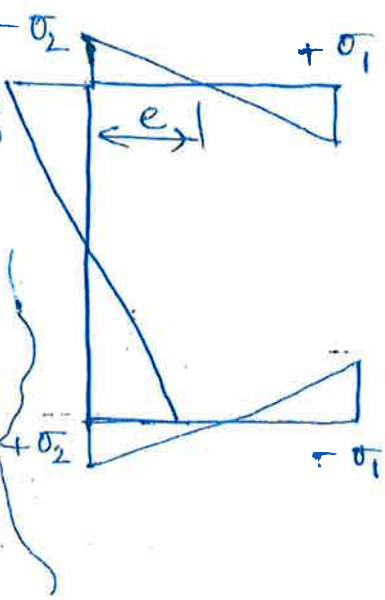
$$\phi_{xxx} = \left(\frac{d^3\phi}{dx^3}\right)_{max} = \frac{-T}{EC_w} = \frac{-1.5\times10^6}{210000 \times 221\times10^9} = -3.23 \times 10^{-11}$$

$$\phi_{max} = \frac{Tl^3}{3EC_w} = \frac{1.5\times10^6 \times 3000^3}{3\times 210000 \times 221\times10^9} = 0.290 \text{ Rad}$$

$$= \underline{16.67^\circ}$$

From Table C.3 Profile book.

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



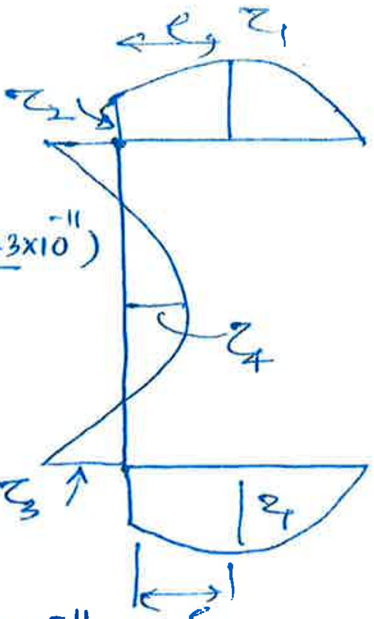
$$= \frac{(103-32)382}{2} \times 210000 \times 9.7\times10^{-8}$$

$$= 276.13 \text{ N/mm}^2$$

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

$$= \frac{32 \times 382 \times 210000 \times 9.7\times10^{-8}}{2}$$

$$= 124.45 \text{ N/mm}^2$$



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

$$= \frac{(103-32)^2 \times 382 \times 210000 \times (-3.23\times10^{-11})}{4}$$

$$= -3.27 \ N/mm^2$$

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

$$= \frac{(103-2\times32) \times 382 \times 210000 \times (-3.2\times10^{-11}) \times 103}{4}$$

$$= -2.6 \ N/mm^2$$

$$\tau_3 = \frac{t}{s}\tau_2 = \frac{18}{14} \times (-2.6) = -3.35 \ N/mm^2$$

$$\tau_4 = \frac{e h^2}{24} E \phi_{xxx} = \frac{32 \times 382^2}{24} \times 210000 \times (-3.2\times10^{-11})$$

$$= -1.32 \ N/mm^2$$

$$\therefore \quad \sigma_{w,max} = \underline{\underline{276.13}} \ N/mm^2$$

$$\tau_{w,max} = \underline{\underline{-3.35}} \ N/mm^2$$

(ii). Check for ULS;

$\sigma_{w,max} = 276.13\ N/mm^2 < f_y/\gamma_{M0} = \frac{355}{1.05} = 338\ N/mm^2$

O.k; No warping tensile failure.

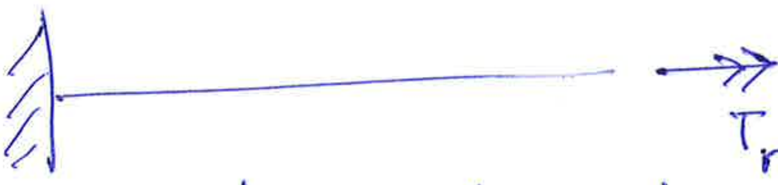
$\tau_{t,max} + \tau_{w,max} < \tau_y/\gamma_{M0} = f_y/\sqrt{3}\,\gamma_{M0}$

$\rightarrow \tau_{w,max} = 8.35\ N/mm^2 < \frac{355}{\sqrt{3} \times 1.05} = 195\ N/mm^2$

O.k; No warping shear failure.

U400 is suitable for ULS//

(iii). If opposite Torque applied at end B.



$T_{max} = (1.5 - 0.5)\ kNm$

$= \underline{1\ kNm}$

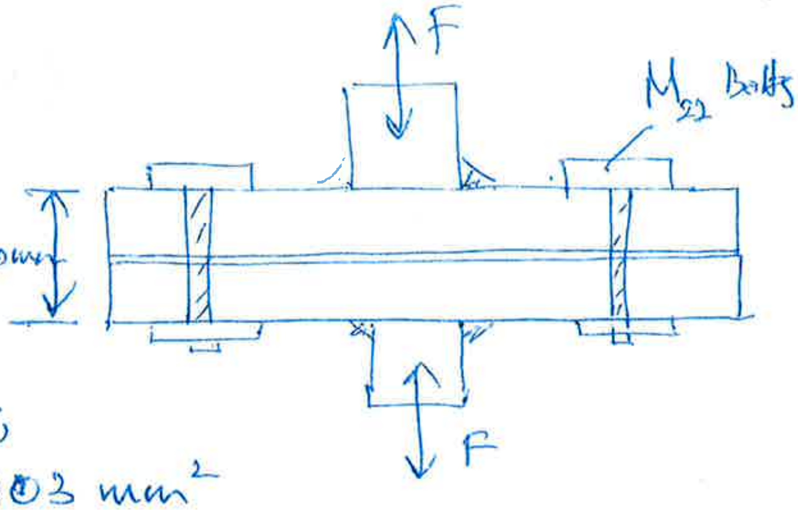
When $T \downarrow$; $\left.\begin{matrix}\phi''_{xx} \\ \phi'''_{xxx}\end{matrix}\right\} \downarrow$ Reduce.

$\therefore$ σ_w and τ_w reduces/decreases.

still U400 c/s can carry the Torques//

(b)

(i)

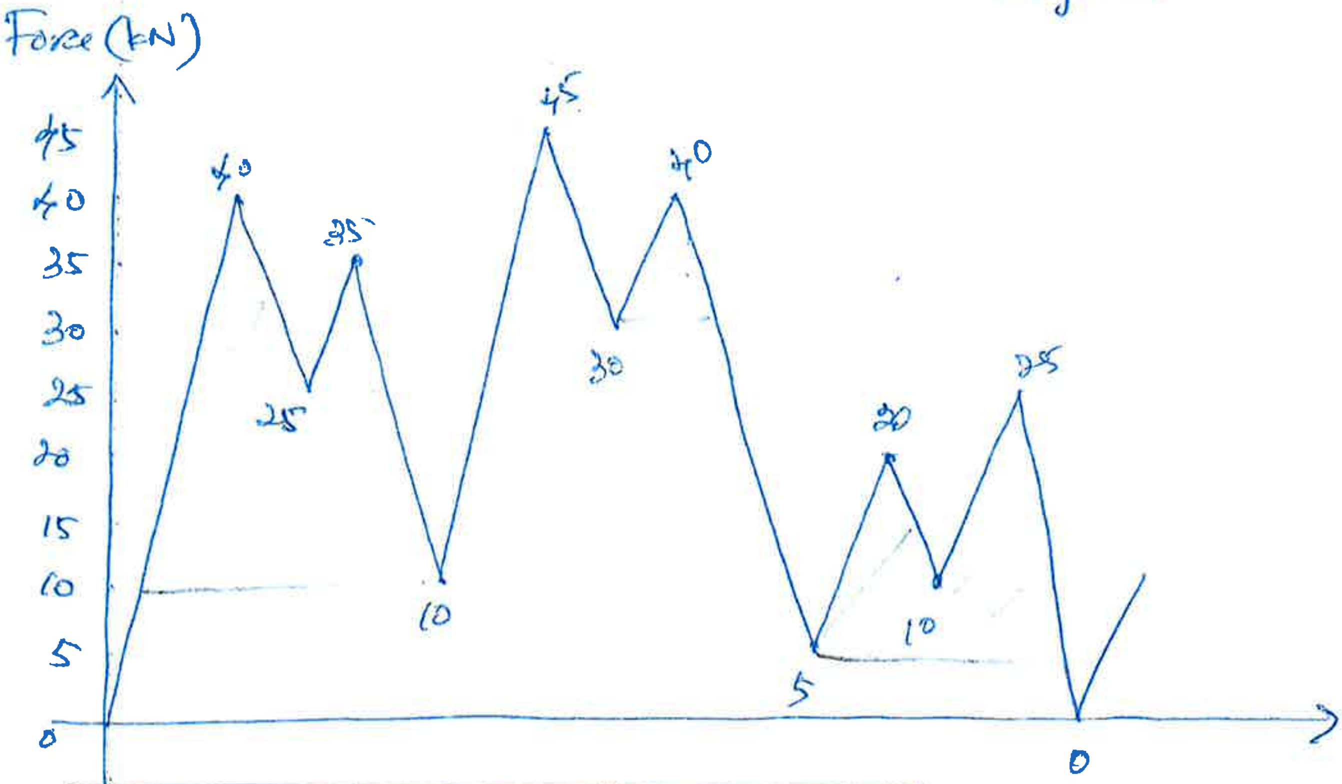


M 22 bolt;

$A_s = 303\ mm^2$

Class 8.8

15 Nos of cycles.



Fmax	Fmin	ΔF (kN)
45	0	45
40	10	30
35	25	10
40	30	10
25	5	20
20	10	10

F_{max}	F_{min}	ΔF	$\Delta\sigma_i = \Delta F / 2A_s$
45	0	45	74.26
40	10	30	49.5
35	25	10	16.5
40	30	10	16.5
25	5	20	33.03
20	10	10	16.5

Force Rages for Single bolt $= (\Delta F/2)$

Stress rage for Single bolt $= \Delta\sigma_i = \Delta F / 2A_s$

$\gamma_{Mf} = 1.15$

$\gamma_{Ff} = 1.0$

$A_s = 303\ mm^2$

M22 bolts.

$\Delta\sigma_R = \gamma_{Mf}\gamma_{Ff}\Delta\sigma_i$

Size effect

$k_s = (30/d)^{0.25}$

$= 0.93$

DC = 50

$\Delta\sigma_c = 50\ N/mm^2$

$\Delta\sigma_D = 0.737 \times \Delta\sigma_c$

$= 36.85\ N/mm^2$

$\Delta\sigma_L = 0.549\,\Delta\sigma_D$

$= 20.23\ N/mm^2$

Modified $\Delta\sigma_c \Rightarrow$

$\Delta\sigma_{c,red} = k_s\Delta\sigma_c$

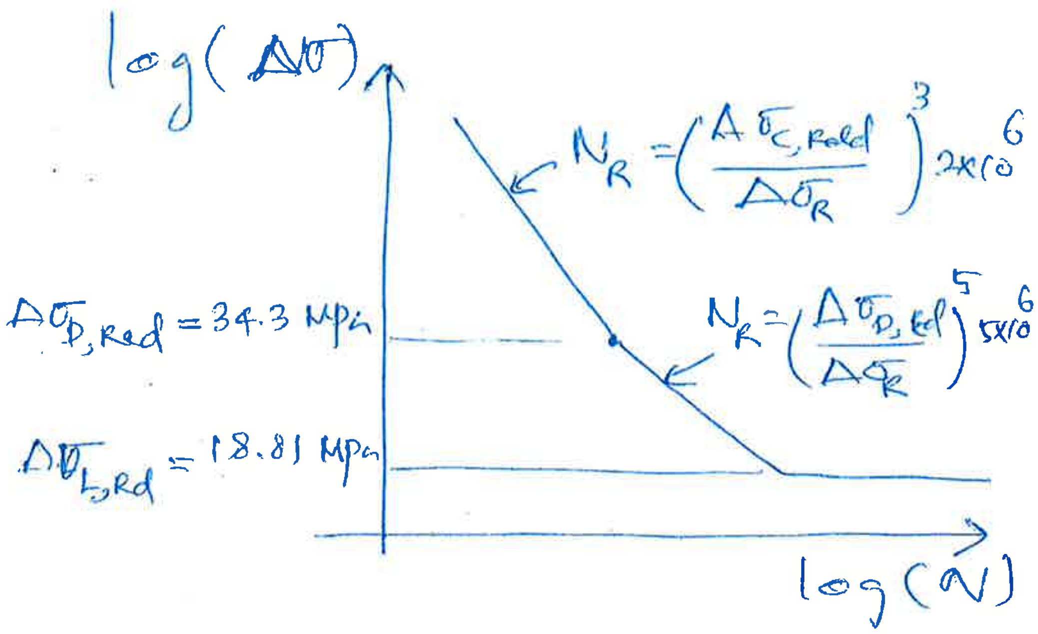
$= 0.93 \times 50$

$= 46.53\ N/mm^2$

$\Delta\sigma_{D,red} = 0.93 \times 36.85$

$= 34.3\ N/mm^2$

$\Delta\sigma_{L,red} = 18.81\ N/mm^2$

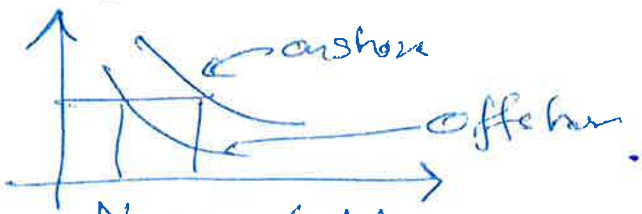


$\Delta\sigma_i$	n_{Ei}	$\Delta\sigma_R$	N_{Ri}	n_{Ei}/N_{Ri}
74.26	15	85.4	3.25×10^5	4.65×10^{-5}
49.5	15	56.93	1.09×10^6	1.38×10^{-5}
16.5	15	18.98	9.6×10^7	1.56×10^{-7}
16.5	15	18.98	9.6×10^7	1.56×10^{-7}
33.03	15	37.95	3.68×10^6	4.08×10^{-8}
16.5	15	18.98	9.6×10^7	1.56×10^{-7}

$$D_{d,day} = 6.48\times10^{-5}$$

$$\text{Fatigue life} = \frac{1}{6.48\times10^{-5}} = \underline{\underline{42.3 \text{ Years}}}$$

(iii). No. Fatigue life should be reduce as offshore structure subjected to deterioration (corrosion) & considered fatigue curve are as bellow the onshore curves.



$N_{offshore} < N_{onshore}$.