BHARTIYA INSTITUTE OF ENGINEERING & TECHNOLOGY SIKAR

DEPARTMENT OF CIVIL ENGINEERING

LAB MANUAL

6CE4-22 : STEEL STRUCTURES DESIGN

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LAB MANUAL – STEEL STRUCTURES DESIGN (6CE4-22)

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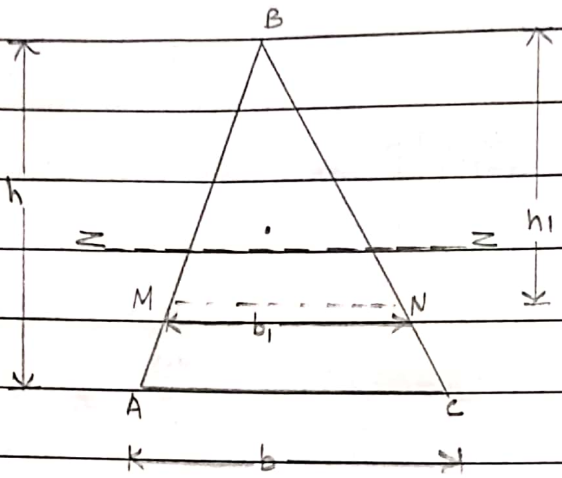
Topic	**Page no.**

Q-1) Find the S.F. of the triangular section of base 'b' and height 'h'.



$$\text{M.I. of section} = \frac{bh^3}{36}$$

$$\text{then, } Z = \frac{I}{y} = \frac{bh^3}{36 \times \frac{2}{3}h}$$

$$Z = \frac{bh^2}{24}$$

Plastic modulus (section modulus) $Z_p = \frac{A}{2}(y_1+y_2)$

let us consider a equal area axis MN.

$$AC = At = \frac{A}{2} = \frac{1}{2} b_1 h_1 = \frac{1}{2}\left(\frac{bh}{2}\right)$$

$$\Rightarrow \quad b_1 h_1 = \left(\frac{bh}{2}\right) \quad \text{— (i)}$$

$$\frac{h_1}{b_1} = \frac{h}{b} \quad \text{— (ii)} \quad \text{similarity of triangles.}$$

from eqn (i) & (ii) :

$$h_1 = \frac{h}{\sqrt{2}}$$

$$b_1 = b/\sqrt{2} \qquad A = \frac{bh}{2}$$

$$y_1 = \frac{h_1}{3} = \frac{h}{3\sqrt{2}}$$

$$y_2 = \left(\frac{h-h_1}{3}\right)\left(\frac{b_1+2b}{b_1+b}\right)$$

$$= \frac{h(\sqrt{2}-1)}{3\sqrt{2}} \times \frac{2\sqrt{2}+1}{(\sqrt{2}+1)}$$

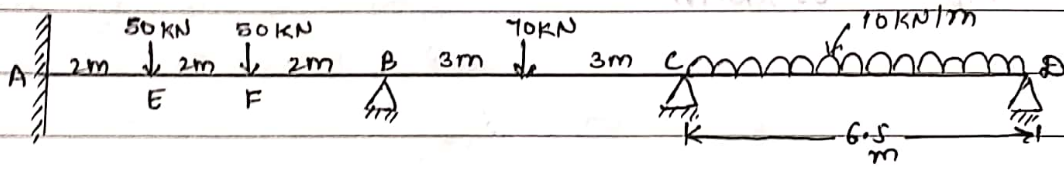
$$Z_p = \frac{A}{2}(y_1+y_2) \Rightarrow \frac{1}{2} \times \frac{bh}{2}\left[\frac{h}{3} \times \frac{1}{\sqrt{2}} + \frac{h}{3} \times 0.4643\right]$$

$$Z_p = \frac{bh^2}{12}\left[0.7072 + 0.4643\right] = 0.0976\, bh^2$$

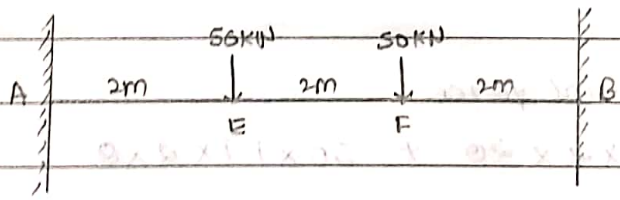
$$S.F. = \frac{Z_p}{Z} = \frac{0.0976\, bh^2}{bh^2/24}$$

$$S.F. = 2.343$$

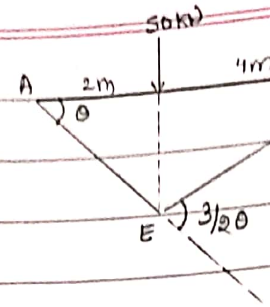
Q-2) Design a continous beam ABCD is loaded as shown in figure. taking load factor 1.70. Calculate the value of fully plastic moment for which it may be designed as uniform section.



Sol :- For span AB :



i)



→ workdone by external force :

$we = 50 \times 1.7 \times 2\theta + 50 \times 1.7 \times 2 \times \frac{\theta}{2}$

$we = 255\theta$

→ workdone by plastic hinge :

$wi = M_p \times \frac{3}{2}\theta$

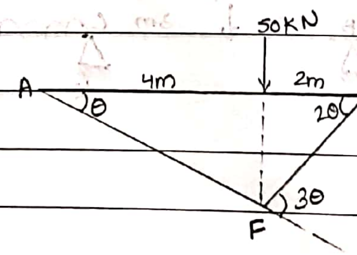
→ By principle of virtual work done :

$we = wi$

$255\theta = \frac{3}{2} M_p \cdot \theta$

$M_p = 85\ KN \cdot m$

ii)



→ workdone by external force :

$we = 50 \times 1.7 \times 2 \times 2\theta + 50 \times 1.7 \times 2 \times \theta$

$= 570\theta$

→ workdone by external hinge :

$wi = mp(3\theta + \theta + 2\theta)$

$wi = 6mp\theta$.

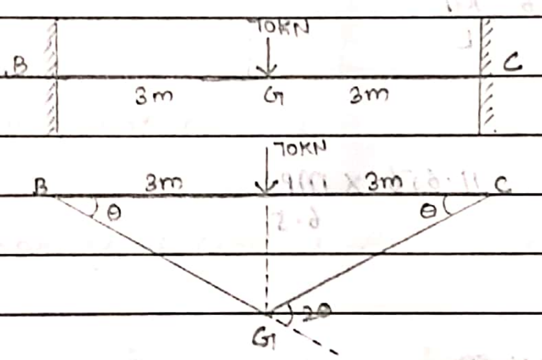
→ By principle of Virtual work :

$we = wi$

$mp = 85 KN\cdot m$.

collapse load is lower of both values.

→ For Span BC :



→ workdone by external force :

$we = 70 \times 1.7 \times 3 \times \theta$

$= 357\theta$

→ workdone by plastic hinge :

$wi = mp(\theta + 2\theta + \theta)$

$wi = 4mp\theta$

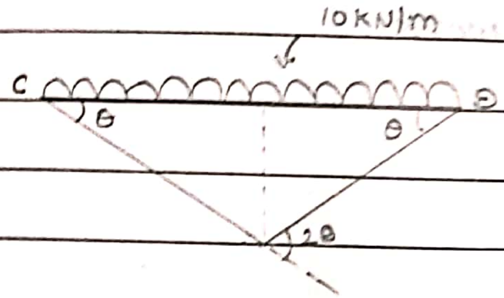
→ By principle of virtual workdone :

$We = wi$

$$35.7\theta = 4M_p\theta$$

$$M_p = 89.25 \text{ KN·m}.$$

⇒ Span CD :



we know that for propped cantilever with UDL collapse load :

$$W_c = 11.656 \frac{M_p}{L}$$

$$10 \times 1.7 \times 6.5 = 11.656 \times \frac{M_p}{6.5}$$

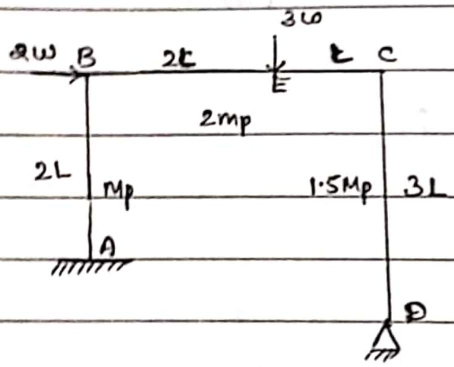
$$M_p = 61.61 \text{ KN·m}$$

For above all plastic moment value we use the max. plastic moment.

$M_p = 89.25$ KN·m (given by BC span).

Q-3)

Compute the true value of collapse load for the portal frame as shown in Figure : Comment on the type of failure.



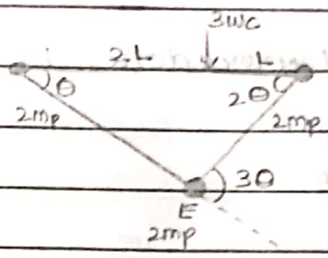
Sol :- No. of plastic hinges required for collapse, H = at point A, B, E, C

$H = 4$

→ Redundancy = 5 - 3 = 2

→ no. of indedended mechanism, N = H - R
= 4 - 2 = 2 < Beam / panel.

i) Beam mechanism :



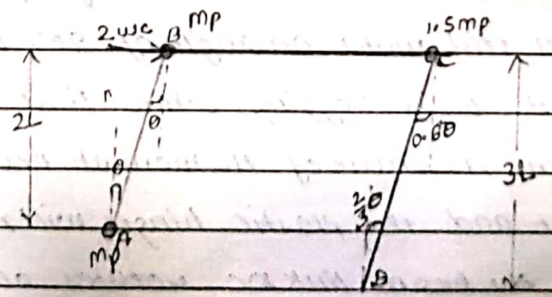
→ workdone by external force :
$$w_e = 3w_c \times 2L \times \theta$$

→ Internal work done :
$$w_i = 2m_p\cdot\theta + 2m_p\cdot 2\theta + 2m_p\cdot 3\theta = 12\,m_p\cdot\theta$$

→ By principle of Virtual work : $w_e = w_i$
$$6w_c L\theta = 12m_p\theta$$

$$w_c = \frac{2m_p}{L} \quad \text{— (i)}$$

ii) Panel mechanism :



$\because \theta\frac{2}{3} = 0.67\theta$

→ workdone by external force :

$we = 2wc \times 2L \times \theta = 4wcL\theta$

→ internal workdone :

$wi = mp\theta + mp\theta + 1.5\,mp \cdot 0.67\theta$

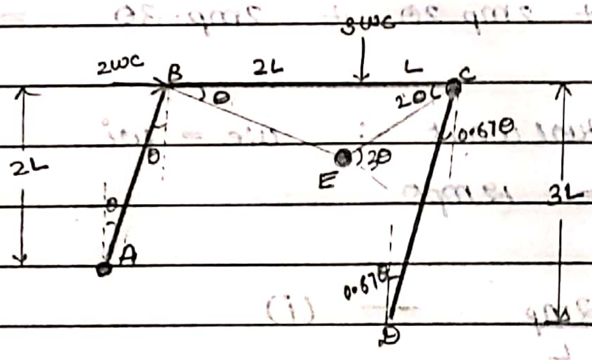
$wi = 3.005\,mp\theta \simeq 3mp\theta$

→ By principle of Virtual workdone : $we = wi$

$$4L\theta wc = 3mp\theta$$

$$wc = \frac{0.75\,mp}{L} \quad \text{— (ii)}$$

iii) Combined mechanism :



A plastic hinge does not develop at B because as the link BC of the beam rotation clockwise, link BA also rotates clockwise by the same amount, therefore there is no change of angle at joint B and the two links still make an angle of 90°. The moment at B, in the beam link BC is hogging while that in column AB is sagging. therefore the value of the moment never reaches the plastic moment and the plastic hinges will not develop. On the other hand, at D the beam link DC rotates anti-clockwise and the column DE rotates clockwise.

→ workdone by external force :

$$w_e = 2w_c \times 2L\theta + 3w_c \times 2L \times \theta = 10w_c L\theta$$

→ Internal work done :

$$w_i = m_p \cdot \theta + 2m_p \cdot 3\theta + 2m_p \cdot 2\theta + 1.5m_p \cdot 0.67\theta$$

$$w_i = 12m_p\theta$$

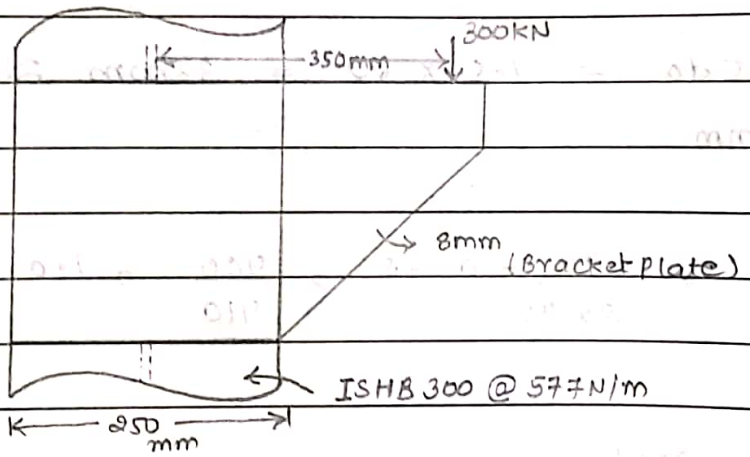
→ By principle of virtual work done : $w_i = w_e$

$$10w_c L\theta = 12m_p\theta$$

$$w_c = \frac{1.2\, m_p}{L} \quad \text{— (iii)}$$

From eqn (i), (ii) and (iii) ; lowest value of collapse load is $0.75\, m_p/L$ which is given by panel mechanism.
Here the failure is from panel mechanism so it is a partial failure.

Q-4) A bracket is bolted to the flange of column as shown in figure using 8mm thick bracket plate and M20 bolts of grade 4.6, design the connection.



Sol:- Given :

$d = 20mm$, $d_0 = 22mm$

$P = 300KN$

$e = 350mm$

Bolt grade = 4.6

f_{ub} = 400 N/mm²

f_{yb} = 240 "

f_u = 410 "

no. of bolts, $n = \sqrt{\frac{6M}{mpBv}}$

Now first find Bv;

i) design strength of bolt in shearing (cl. 10.3.3 pg - 75)

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_m}(n_n \cdot A_{nb} + A_{sb} \cdot n_s)$$

$$V_{dsb} = \frac{400 \times 1 \times 0.78 \times \frac{\pi}{4} \times 20^2}{\sqrt{3} \times 1.25}$$

$\because$ $n_n = 1$ (single shear) $n_s = 0$ (Assume)

V_{dsb} = 45.27 kN

ii) Design strength of bolt in bearing :

$$V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_m}$$

k_b is least of $\frac{e_2}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0

$e_2 = 1.5 d_0 = 1.5 \times 22 = 33 \approx 40$ mm

$p = 2.5 d = 2.5 \times 20 = 50$ mm

$$\Rightarrow \frac{40}{3\times22}, \frac{50}{3\times22} - 0.25, \frac{400}{410}, 1.0$$

$$\Rightarrow 0.61, 0.51, 0.97, 1.0$$

So, $k_b = 0.51$

$t = 7.6$ (By Steel table)

$$V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 7.6 \times 410}{1.25}$$

$V_{dpb} = 63.56$ KN

So, $BV = 45.27$ KN.

→ Assume line of bolts $m = 2$

So, no. of bolt $= \sqrt{\frac{6M}{mp\,BV}}$

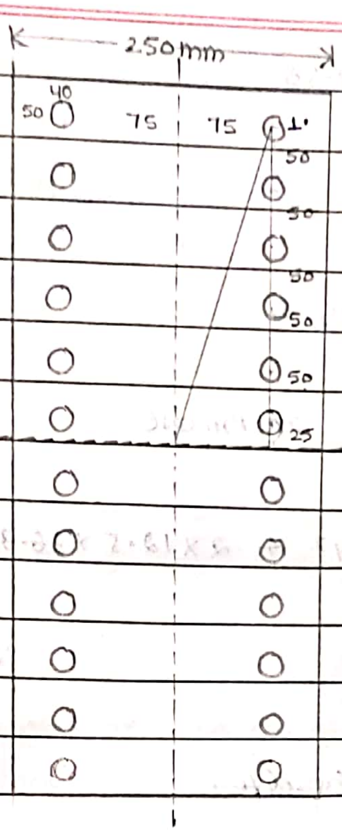
$$= \sqrt{\frac{6 \times 300 \times 10^3 \times 350}{2 \times 50 \times 45.27 \times 10^3}}$$

$= 11.79$ bolts

So, no. of bolts $= 12$

Hence,
provide 12 bolts in each row.

min. edge distance $(e_1) = 1.5 d_o = 1.5 \times 22 = 33 \approx 50$ mm



Now finding P_r ;

i) shear force due to axial load :

$$P_a = \frac{P}{n} = \frac{300}{24} = 12.5$$

ii) shear force due to torsional moment :

$$P_m = \frac{Mr}{\Sigma r^2} = \frac{P.e.r}{\Sigma r^2}$$

$$r = \sqrt{275^2 + 75^2} = 285.04 \text{ mm}$$

$$\Sigma r^2 = \Sigma x^2 + \Sigma y^2$$
$$= 24 \times 75^2 + \{(4 \times 25^2) + (4 \times 75)^2 + (4 \times 125^2) + (4 \times 175^2) + (4 \times 225^2)$$
$$= 856400 \text{ mm}^2$$

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$$Pm = \frac{105 \times 10^6 \times 282.58}{816400}$$

$$= 36.34 \text{ KN.}$$

Resultant shear force : Pr

$$Pr = \sqrt{Pq^2 + Pm^2 + 2Pq\, Pm \cos\theta}$$

$$Pr = \sqrt{12.5^2 + 36.34^2 + 2 \times 12.5 \times 36.34 \times 43.5}$$

$$Pr = 43.56 \text{ kN} \qquad \because \cos\theta = \frac{282.58}{65}$$

So, $BV > Pr$ Hence it is safe.

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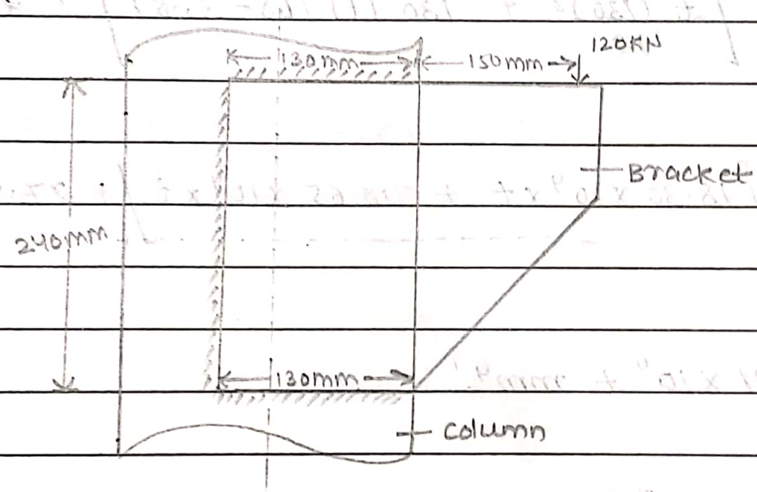
Q-5) Calculate the size of weld required to support a factored load of 120KN applied at a distance of 150mm from the edge of the flange of column on a plate bracket as shown in figure.



Sol:- let us assume throat thickness = t

Total area of weld $A = (2 \times 130 + 240)\,t$

$= 500t \ mm^2$

due to symmetry centroidal X-X axis is at the mid height of vertical weld.
let centroidal Y-Y axis be at a distance of $\bar{x}$ from the vertical weld.

$$\bar{x} = \frac{2 \times 130 \times t \times 65 + 240\,t \times 0}{500\,t}$$

$$= 33.8\ mm.$$

then moment of inertia about X-X axis ;

$$I_{XX} = I_{G} + Ah^2$$

$$= \frac{t\,(240)^3}{12} + 2 \times 130 \times t \times 120^2$$

$$= 489.6 \times 10^4 \cdot t\ mm^4$$

$$I_{YY} = 2 \times \left[\frac{t\,(130)^3}{12} + 130\,(t)\,(65-33.8)^2\right] + 240\,(t)\,(33.8)^2$$

$$= 2\left[18.30 \times 10^4 \times t + 12.65 \times 10^4 \times t\right] + 27.41 \times 10^4\,t$$

$$= 89.31 \times 10^4\,t\ mm^4.$$

$$I_{ZZ} = I_{XX} + I_{YY}$$

$$= 489.6 \times 10^4\,t + 89.31 \times 10^4\,t$$

$$= 578.94 \times 10^4\,t\ mm^4$$

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$$r_{max} = \sqrt{120^2 + 96.2^2}$$

$$= 153.8 \text{ mm}$$

eccentricity (e) $= 150 + 130 - 33.8$

$$= 246.2 \text{ mm.}$$

i) direct shear stress :

$$q_1 = \frac{P}{A}$$

$$q_1 = \frac{120}{500t} = \frac{0.24}{t} \text{ KN/mm}^2$$

$$q_1 = \frac{240}{t} \text{ N/mm}^2$$

ii) Shear stress at extreme edge due to torsional moment :

$$q_2 = \frac{P \cdot e \cdot r_{max}}{I_{zz}}$$

$$= \frac{120 \times 10^3 \times 246.2 \times 153.8}{578.94 \times 10^4 \; t}$$

$$= \frac{784.85}{t} \text{ N/mm}^2$$

→ Resultant shear stress :

$$q = \sqrt{q_1^2 + q_2^2 + 2 q_1 q_2 \cos\theta}$$

$$q = \sqrt{\left(\frac{240}{t}\right)^2 + \left(\frac{784.85}{t}\right)^2 + 2\times\left(\frac{240}{t}\right)\left(\frac{784.85}{t}\right) \times \frac{96.2}{153.8}}$$

$$q = \frac{1}{t}\, 953.43 \ N/mm^2$$

As per code : Design stress $= \frac{f_{uw}}{\sqrt{3}\,\gamma_{mw}}$

$$\Rightarrow \frac{410}{\sqrt{3}\times 1.25} = 189.37 \ N/mm^2$$

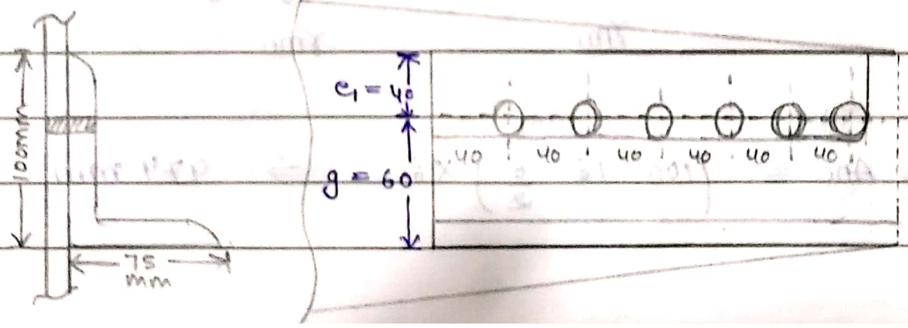
equating design stress and Resultant stress ;

$$\frac{953.43}{t} = 189.37$$

$$t = 5.05 \ mm$$

& $S = 7.22mm$

Q-5) A single unequal angle 100mm x 75mm x 6mm is connected to a 10mm thick gusset plate, at the end with 6 - 16mm ϕ bolts to transfer tension as shown in figure. determine the design tensile strength if the longer leg is connected take $f_y = 250 N/mm^2$ and $f_u = 410 N/mm^2$



Sol :- Given that :

ISA 100, 75, 6mm.

$p = 40mm$, $e_2 = 40mm$

$f_y = 250$ and $f_u = 410\ N/mm^2$

$d = 16$ $d_h = 18\ mm$

$n = 6.$

As per codel provision :

$g = 60mm$ (60% of channel leg length).

$\Rightarrow e_1 = 40\ mm$

i) Design strength due to yielding of gross section : (Cl. 6.2 pg. 32)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$

$$T_{dg} = \frac{1014 \times 250}{1.1}$$

where $A_g = 1014\ mm^2$ (From steel table)

$T_{dg} = 230.45\ KN$ ----- (i)

ii) Design strength due to rupture of critical section :
(Cl. 6.3.3 pg- 33 IS : 800- 2007)

$$T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} f_y}{\gamma_{m0}}$$

$\because A_{nc} = \left(100 - 18 - \frac{6}{2}\right) \times 6 \Rightarrow 474\ mm^2$

$$A_{go} = \left(75 - \frac{6}{2}\right) \times 6 = 432\ mm^2$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \geq 0.7$$

where, b_s = length from outstand endpoint to connection

$= 75 + 60 - 6 = 129\ mm$

$L_c = 5 \times 40 = 200mm$

So, $\beta = 1.4 - 0.076\left(\frac{75}{6}\right)\left(\frac{250}{410}\right)\left(\frac{129}{200}\right)$

$\beta = 1.026 > 0.7$ O.K.

$$T_{dn} = \frac{0.9 \times 474 \times (410)}{1.25} + \frac{1.026 \times 432 \times 250}{1.1}$$

$T_{dn} = 240.66\ KN$ ---- (ii)

Also, App. formula, $T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$

$$T_{dn} = \frac{0.8 \times 906 \times 410}{1.25}$$

$\alpha = 0.8$ (as per code)

$A_n = A_g - d_h \cdot t$

$A_n = 1014 - 18 \times 6 \Rightarrow 906\ mm^2$

So, $T_{dn} = 237.7\ KN.$

iii) Design strength due to block shear : (cl 6.4.1 pg - 33)

$$T_{db1} = \frac{A_{vg} \cdot f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9\,A_{tn} f_u}{\gamma_{m1}}$$

$\because A_{vg} = (l_c + e_2)\,t$

$= (200 + 40)\,6 = 1440\ mm^2$

$\because A_{vn} = A_{vg} - \left(n - \frac{1}{2}\right) d_h \cdot t$

$= 1440 - \left(6 - \frac{1}{2}\right) \times 18 \times 6 = 846\ mm^2$

$\because A_{tg} = e_1 t = 40 \times 6 = 240\ mm^2$

$\because A_{tn} = A_{tg} - \frac{d_h t}{2} = 240 - \frac{18}{2} \times 6 \Rightarrow 186\ mm^2$

also, $T_{db1} = \frac{1440 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$

$T_{db1} = 243.85\ kN.$ — (iii)

$$T_{db2} = \frac{0.9\,A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 846 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1}$$

$T_{db2} = 198.73\ kN.$ — (iv)

From eqn (i), (ii), (iii) and (iv) ; Design strength of tension m

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least value is 198.73 KN. which is given by block shear.

Therefore,

$$\boxed{T_d = 198.73 \text{ KN.}}$$

Q-7) Design a double angle discontinous strut to carry a factored load of 175 KN. the length of the strut is 3m b/w intersection the two angles are placed back to back and are take bolted. consider the following cases ;

a) Angles are placed on the same side of gusset plate.

b) Angles are placed on opposite sides of the gusset plate.

c) Two angles in star formation.

Assume grade fe-410 steel with $f_y = 250$ MPa.

Sol:- Given data : $P = 175$ KN

$l = 3m$

→ Assume, $f_{cd} = 90 N/mm^2$

→ Req. area, $A_{rq} = \frac{P}{f_{cd}} = \frac{175 \times 10^3}{90}$

$A_{req} = 1944.45 mm^2$ (This is of whole angle)

→ Area required for single angle $= \frac{1944.45}{2}$

$A_{req} = 972.23 mm^2$

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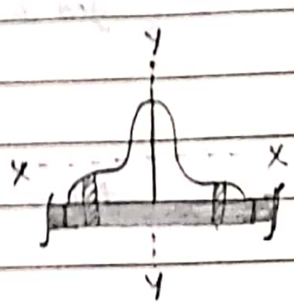
a) Angles are placed on same side of gusset plate :
let us try 2 ISA 55,55, 10mm giving Area :
$A = 1002\ mm^2$ (single angle)



→ Total Area, $A = 2 \times 1002 = 2004$ (double Angle)

→ Assume thickness of gusset plate : $t_g = 10mm$

→ Radius of gyration : $r_{min} = \sqrt{\frac{I_{yy}}{A}}$

M.O.I. ; $I_{yy} = 2\,[I_y + Ah^2]$ $\quad \because h = C_{xx}$

$= 2\,[26.3 \times 10^4 + (1002 \times 17.2^2)]$

$= 111.89 \times 10^4\ mm^4$

So, $r_{min} = \sqrt{\frac{111.89 \times 10^4}{2004}} = 23.62\ mm$

→ Slenderness Ratio (Sr) $= \frac{kL}{r}$

$Sr = \frac{0.80 \times 3000}{23.62}$

$\because k = 0.80$ for double angle and $L = 1L$ (for hinge)

$Sr = 101.61$

→ As per table 10 pg-44 buckling class (c) :
So, from table 9(c) pg-42 IS : 800-2007 ;

$$f_{cd} = \frac{(y_3 - y_1)(x_2 - x_1)}{(x_3 - x_1)} + y_1$$

KL/r	$f_y = 250$
100	107
101.61	?
110	94.6

$$f_{cd} = \frac{(94.6 - 107)(101.61 - 100)}{(110 - 100)} + 107$$

$f_{cd} = 105.66\ N/mm^2$

→ Design strength = $A \times f_{cd}$

$P_d = 2004 \times 105.66$

$= 211.75\ kN > 175\ kN$

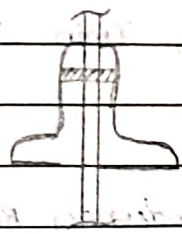
So, $P_d > P$ ∴ Hence safe.

b) Angle are placed on the opposite side of the gusset plate:

let us try 2 ISA 80, 50, 8mm.

given area;

$A = 978\ mm^2$ (single angle)



Total area, $A = 2 \times 978$

$= 1956\ mm^2$

→ Assume thickness of gusset plate $t_g = 10mm$

$$I = 2\left[I_{aa} + A \times \left(h + \frac{t_2}{2}\right)^2\right]$$

$$= 2\left[18.5 \times 10^4 + 978 \times \left(12.4 + \frac{10}{2}\right)^2\right]$$

~~96.22×10^4~~

$= 89.60 \times 10^4\ mm^4$

→ $I_{min} = \sqrt{\frac{I}{A}}$

$$= \sqrt{\frac{\overset{96.22}{89.60 \times 10^4}}{1956}} = \overset{22.18}{21.40\,mm}$$

→ effective length : KL

$$KL = 0.80 \times L \Rightarrow 0.80L$$

→ slenderness ratio : $\frac{KL}{r}$

$$S_r = \frac{0.80 \times 3000}{\underset{22.18}{21.40}} = \overset{108.21}{112.149}$$

→ Buckling class for angle section is (c). from table 9(c) page 42 for 250 N/mm² IS code 800 : 2007 ;

$$y_2 = \overset{107}{94.6} + \frac{(83.7 - \overset{94.6}{94.6})(\overset{107}{}\,\overset{108.21}{112.149} - \overset{100}{110})}{\underset{110-100}{(120-110)}}$$

KL/r	fcd
100 110	107 94.6
108.21 112.149	?
120	83.7
110	94.6

$$F_{cd} = \overset{96.82}{92.25}\ N/mm^2$$

→ Design strength Pd :

$$P_d = A_{req} \times f_{cd}$$

$$P_d = 1956 \times \overset{96.82}{92.25}$$

$$P_d = \overset{189.38}{180.44}\,KN > 175\,KN$$

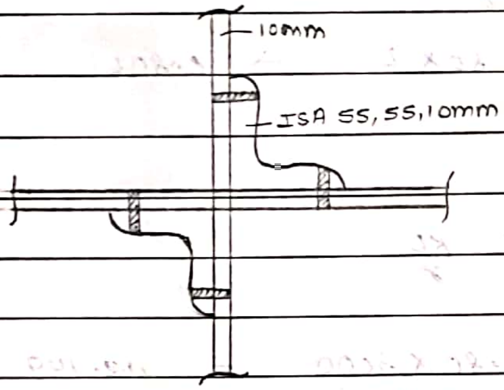
So, Pd > P. Hence design is safe.

c) Two angle in star formation :

let us try 2 ISA 55,55, ~~10~~mm from steel table giving data:

Area = 1002 mm^2 (single angle)

Total area, A = 2 X 1002 = 2004 mm^2



→ From steel table ;

I_{yy} = ~~26.3~~ X 10^4 mm^4

r_{yy} = 16.2 mm C_{xx} & C_{yy} = 17.2 mm

r_{vv} = 10.6 mm

r_{uu} = 20.3 mm

→ Assume thickness of gusset plate t_g = 10mm.

→ $$r_y = \sqrt{(C_{yy})^2 + \left(r_{yy} + \frac{t_g}{2}\right)^2}$$

$$r_y = \sqrt{(17.2)^2 + \left(16.2 + \frac{10}{2}\right)^2}$$

$$r_y = 27.3 \text{ mm}$$

→ $$r_v = \sqrt{r_v^2 + 2\left(C_y + \frac{t_g}{2}\right)^2}$$

$$r_v = \sqrt{(10.6)^2 + 2\left(17.2 + \frac{10}{2}\right)^2}$$

r_v = 33.13 mm

→ $r_u = r_{uu}$ = 20.3 mm

→ Therefore, Final radius of gyration, $r_{min} = r_u \approx 20.3$ mm

→ Slenderness ratio, $Sr = \frac{KL}{r}$

$$= \frac{0.80 \times 3000}{20.3} = 118.23.$$

∵ K = 0.80 (double Angle) and L = 1L

→ From table 10, pg - ~~44~~, buckling class is (c).
From table 9, pg - 42

$$f_{cd} = \frac{(83.7 - 94.6)(118.23 - 110)}{(120 - 110)} + 94.6$$

KL/r	fy = 250
110	94.6
118.23	? (fcd)
120	83.7

f_{cd} = 85.63 N/mm²

→ Design strength Pd;

Pd = Area x fcd

= 2004 x 85.63

Pd = 171.85 KN < 175 KN

Hence, Pd < P so design fail.

So, Redesign this steel formation by ISA 65, 65, 8mm giving Area;

$A = 978 \, mm^2$

Total area $A = 2 \times 978 = 1952 \, mm^2$

Assume $t_g = 10mm$

→ From steel table :

$C_{xx} = C_{yy} = 18.9 \, mm$

$r_{vv} = 12.5 \, mm$

$r_{uu} = 24.7 \, mm$

$r_{yy} = 19.6 \, mm$

→ $r_y = \sqrt{(C_{yy})^2 + \left(r_{yy} + \frac{t_g}{2}\right)^2}$

$= \sqrt{(8.9)^2 + \left(19.6^2 + \frac{10}{2}\right)^2}$

$= 31.02 mm$

→ $r_v = \sqrt{r_v^2 + 2\left(C_{yy} + \frac{t_g}{2}\right)^2}$

$= \sqrt{12.5^2 + 2\left(18.9 + \frac{10}{2}\right)^2}$

$= 36.03 \, mm$

→ $r_u = r_{uu} = 24.7 \, mm$

So radius of gyration, $r_{min} = 24.7 \, mm$

→ Slenderness ratio $(sr) = \frac{KL}{r}$

$sr = \frac{0.80 \times 3000}{24.7} = 97.166$

→ From table 10, pg - 44 buckling class (c)

From table 9, pg - 42

KL/r	$fy = 240$
90	121
97.166	? (f_{cd})
100	107

$$f_{cd} = \frac{(107-121)(97.166-90)}{(100-90)} + 121$$

$$= 110.97$$

→ Design strength $P_d = A \times f_{cd}$

$$= 1952 \times 110.97$$

$$P_d = 216.67 > 175 \text{ KN}$$

So, $P_d > P$ Hence design is safe.

Q-8) Design a laterally supported beam having effective span 6m the beam is simply supported at the two ends and carries uniform distributed factored load of 30 KN/m including its self weight.

Sol:- Given that : factored load = 30 KN/m

$$\text{load} = \frac{30}{1.5} = 20 \text{ KN/m}$$

span = 6m.

→ maximum bending moment (M) : $\frac{wl^2}{8}$

$$M = \frac{30 \times 6^2}{8} = 135 \text{ KN·m}$$

→ maximum shear force (V) = $\frac{wl}{2}$

$$V = \frac{30 \times 6}{2} = 90 \text{ KN.}$$

→ Assume section is plastic.

$$Z_{p.req.} = \frac{M.\gamma_{mo}}{f_y} = \frac{135 \times 10^6 \times 1.1}{250}$$

$$Z_{p.req.} = 594 \times 10^3 \text{ mm}^3.$$

→ let us use I section as beam :

$$Z_{xx.req} = \frac{Z_{p.req.}}{S.F.}$$

$$Z_{xx.req.} = \frac{594 \times 10^3}{1.13}$$

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Z_{xx} req. = $525.86 \times 10^3 \, mm^3$

→ let us try ISMB 300 from steel table, giving

$Z_{xx} = 573.6 \times 10^3 \, mm^3$

→ section classification (table-2 pg-18 IS 800:2007)

$\frac{b}{t_f} = \frac{b_f}{2t_f} = \frac{140}{2 \times 12.4} = 5.64 < 9.4\varepsilon$ (∵ ε=1

Hence section is plastic.

→ Design bending moment $M_d = \frac{Z_p \cdot f_y}{\gamma_{mo}}$

$M_d = \frac{Z_{xx} \times S.F. \times f_y}{\gamma_{mo}}$

$= \frac{573.6 \times 10^3 \times 1.13 \times 250}{1.1}$

$M_d = 147.31 \, kN \cdot m > M \, (135 \, kN \cdot m)$

Hence, safe in bending.

→ Design shear force (Cl. 8.4, pg-59, IS: 800-2007)

$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\, \gamma_{mo}} \Rightarrow \frac{h \times t_w \times f_y}{\sqrt{3}\, \gamma_{mo}}$

$= \frac{300 \times 7.5 \times 250}{\sqrt{3} \times 1.1}$

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$V_d = 296.05$ kN $> V$ (90 kN)

Hence safe in shear.

→ check as per cl. 9.2, page - 69 :

$0.6 V_d = 0.6 \times 296.05 \Rightarrow 177.63 > V$ (90 kN)

Hence, No reduction in bending moment is required.

→ check for deflection :

$$\delta max = \frac{5}{384} \frac{wl^4}{EI}$$

$$= \frac{5}{384} \frac{\times 20 \times 10^3 \times 6000^4}{2 \times 10^5 \times 2603.6 \times 10^4}$$

$dmax = 19.61$ mm

$$d_{permissible} = \frac{span}{300}$$

$$= \frac{6000}{300}$$

$d_{per} = 20$mm

So, $20 > 19.61$ i.e.

$d_{per.} > d_{max}$

Hence, section is safe.

Q-9) Design a slab base for a column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 KN. M20 concrete is used for the foundation provided welded connection b/w column and base plate.

Sol:- Given that:

column = ISHB 300 @ 577 N/m.

b_f = 250 mm (In steel table)

h = 300 mm "

t_f = 10.6 mm "

Axial factored load $P = 1000\,KN$.

grade of concrete - M20 (f_{ck}) = $20\,N/mm^2$

→ Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 20$

$= 9\,N/mm^2$

→ Required area of base plate (A_p) $= \dfrac{\text{load of column}}{\text{Bearing strength of column}}$

$$\Rightarrow \frac{1000 \times 10^3}{9} = 111111.11\ mm^2$$

→ let two projection are same;

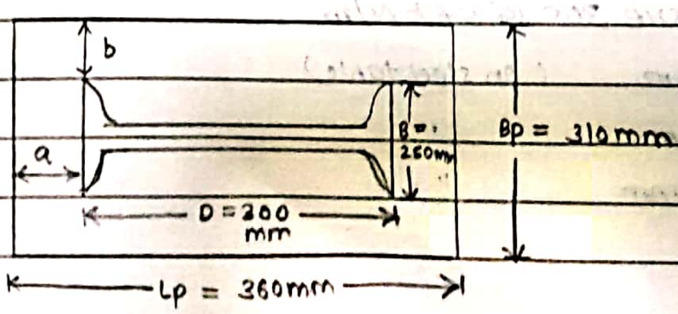
$$L_p = \frac{(D-B)}{2} + \sqrt{\left(\frac{D-B}{2}\right)^2 + A_p}$$

$$= \left(\frac{300-250}{2}\right) + \sqrt{\left(\frac{300-250}{2}\right)^2 + 111111.11}$$

$$L_p = 359.27\ mm.$$

→ we know that: $A_p = L_p \times B_p \Rightarrow B_p = \dfrac{111111.11}{359.27} = 309.27\,mm$

Provide size of base plate = 360×310 mm.



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Projection are ;

$$a = \frac{l_p - D}{2} = \frac{360 - 300}{2} = 30\,mm$$

$$b = \frac{B_p - B}{2} = \frac{310 - 250}{2} = 30\,mm$$

Therefore, $a = b$ (both projections are same) o.k.

→ Actual pressure from Concrete ;

$$w = \frac{P}{l_p \times B_p} = \frac{1000 \times 10^3}{360 \times 310} = 8.96\ N/mm^2$$

→ As per clause 7.4.3.1 pg - 47 IS 800 : 2007

→ thickness of base plate ;

$$t = \sqrt{\frac{2.5\,w\,(a^2 - 0.3b^2)\,\gamma_{mo}}{f_y}}$$

$$t = \sqrt{2.5 \times 8.96 \times (30^2 - 0.3 \times 30^2) \times \frac{1.1}{250}}$$

$t = 7.86 \nless t_f$

Take $t = 12mm$

→ Hence provide 360mm × 310mm × 12mm size of base plate.

→ Use 4 bolts of 20mm diameter and 400mm long to anchor the plate.

Weld connection b/w column of base plate :

Properly machined column is to be connected in base plate using fillet weld.

→ The length available for welding :

$= 2 \times [250 + (250 - 7.6) + (300 - 2 \times 10.6)]$

$= 1542.4 mm$

→ Strength of weld $= \dfrac{f_{uw}}{\sqrt{3} \cdot \gamma_{mw}}$ (pg-79, cl. 10.5.7.1.1)

$= \dfrac{410}{\sqrt{3} \times 1.25} \Rightarrow 189.37 \ N/mm^2$

→ let us 's' be the size of weld $\Rightarrow t_t = 0.7 \times S$

eff. area $= l_e \times t_t$

→ Design strength of weld $= L_e \times t_t \times \dfrac{f_{uw}}{\sqrt{3} \gamma_{mw}}$

$\Rightarrow l_e \times t_t \times 189.37 = 1000 \times 10^3$

$\Rightarrow l_e \times 0.7S \times 189.37 = 1000 \times 10^3$

take size of weld $S = 6mm$

$l_e = \dfrac{1000 \times 10^3}{0.7 \times 6 \times 189.37} = 1257.30 \ mm$

→ Available eff. length = Available length − 2 × 6 × no. of meter

$= 1542.4 - 2 \times 6 \times 12$

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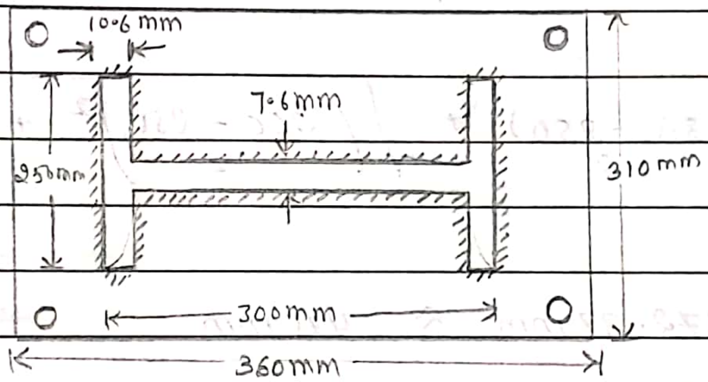
$\Rightarrow$ 1398.4 mm > 1257.30 mm.

Hence, 6 mm weld is adequate.



Q-106 Design a welded plate girder of span 24m to carry super Imposed load of 35 KN/m. Avoid use of bearing and intermediate transverse stiffners. use fe-415 steel.

Sol:- Given; span, $l = 24m$

super imposed load $= 35$ KN/m.

→ factored load $= 1.5 \times 35 = 52.5$ KN/m

→ Assume self weight of P.G ⇒ $w' = \frac{W'}{200}$ KN/m

$$w' = \frac{52.5 \times 24}{200} = 6.3 \text{ KN/m}$$

→ Total load on P.G $= 52.5 + 6.3 = 58.8$ KN/m

→ maximum B.M. (M) $= \frac{wl^2}{8}$

$$M = \frac{58.8 \times 24^2}{8} = 4233.6 \text{ KN·m.}$$

→ max. Shear force (S.F.), $V = \frac{wl}{2}$

$$V = \frac{58.8 \times 24}{2} = 705.6\ KN$$

Design of web plate :

→ Assume section is plastic.

→ depth of web plate, $d_w = \sqrt[3]{\frac{M.K.}{f_y}}$

$$d_w = \sqrt[3]{\frac{4233.6 \times 10^6 \times 67}{250}} \quad \{ \because K \leq 67,\ I.T.S.\ \text{avoided}$$

$$d_w = 1042.99\ mm.$$

Also; $d_w = \frac{L}{12}$ to $\frac{L}{8}$ ⇒ $\frac{24000}{12}$ to $\frac{24000}{8}$

$$= 2000 \text{ to } 3000$$

So, Adopt $\boxed{d_w = 1042.99\ mm}$

→ Thickness of web plate as per serviceability condition.

$$t_w \geq \frac{d_w}{200} \quad (Cl.\ 8.6.1.1\ pg-63\ IS\ 800-2007)$$

$$t_w \geq \frac{1050}{200} \Rightarrow t_w \geq 5.25\ mm$$

Also; $t_w \geq \frac{d_w}{67}$ ⇒ $t_w \geq \frac{1050}{67}$

$t_w \geq 15.67\,mm \approx 16\,mm$

therefor adopt, $\boxed{t_w = 16\,mm.}$

→ size of web plate is $1050\,mm \times 16\,mm$.

Design of flange plate :

→ Area of flange : $A_f = \dfrac{M}{d_w\left(\dfrac{f_y}{\gamma_{m0}}\right)}$

$$(\text{approx})\,A_f = \frac{4233.6 \times 10^6}{1050\left(\frac{250}{1.1}\right)}$$

$A_f\,(\text{approx}) = 17740.8\ mm^2$

→ As per table - 2 pg - 18 of IS : 800 - 2007

For plastic section ;

$$\frac{b}{t_f} \leq 8.4 \quad \Rightarrow \quad \frac{b_f}{2t_f} \leq 8.4$$

$$\frac{b_f}{t_f} \leq 16.8 \quad \Rightarrow \quad b_f \leq 16.8\,t_f$$

Also, $A_f = b_f \times t_f$ $\quad\quad \because A_f = A_f\,(\text{approx.})$

$17740.8 = 16.8\,t_f \times t_f$

$$t_f^2 = \frac{17740.8}{16.8}$$

$t_f = 32.49$ mm

adopt $\boxed{t_f = 40 \text{ mm}}$

therefore, $b_f = \dfrac{A_f}{t_f}$

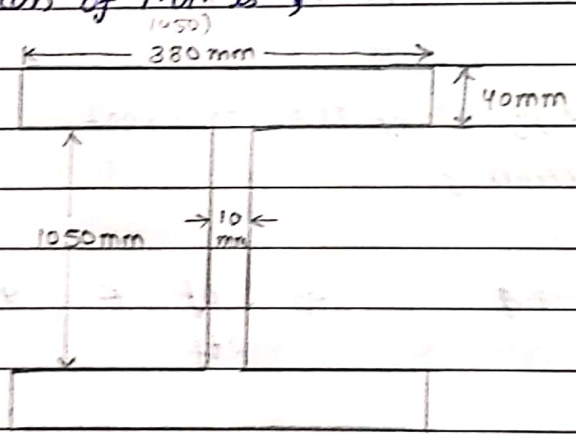
$b_f = \dfrac{17740.8}{40} \Rightarrow 443.52$ mm

Also, $b_f = 0.3\, d_w = 0.3 \times 1050 = 315$ mm

$\therefore$ Adopt $\boxed{b_f = \overset{450}{380} \text{ mm}}$

→ $\therefore$ Size of flange plate $= \overset{450}{380}$ mm $\times$ 40 mm

→ So, Final section of P.G. is ;



→ section classification $\dfrac{b}{t_f} = \dfrac{b_f - t_w}{2 t_f}$

$= \dfrac{\overset{450}{380} - 16}{2 \times 40} = \overset{5.24}{4.5} < 8.4$ OK//

Hence section is plastic.

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→ Design B.M. $(M) = \frac{Zp \cdot fy}{\gamma mo}$

$Zp = bf\ tf\ (dw + tf)$

$Zp = \overset{450}{\cancel{380}} \times 40\ (1050 + 40)$

$Zp = \overset{19620}{\cancel{16568}} \times 10^3\ mm^3$.

So, $Md = \frac{\overset{19620}{\cancel{16568}} \times 10^3 \times 250}{1.1}$

$Md = \overset{4459.1\ >\ (4233.6\ KN \cdot m)\ \text{safe}}{3765.45\ KN \cdot m < 4233.6\ KN \cdot m)\ \text{Not safe}}$.

⇒ so if we take bf = 380 then design is not safe in bending but if bf = 450 then design is safe in bending.

→ Design shear force (Cl. 8.4 pg-59, IS: 800-2007)

$Vd = \frac{Av \cdot fy}{\sqrt{3}\ \gamma mo} = \frac{dw \times tw \times fy}{\sqrt{3}\ \gamma mo}$

$Vd = \frac{1050 \times 16 \times 250}{\sqrt{3} \times 1.1}$

$Vd = 2204.42\ KN > V$

i.e. $2204.42 > 705.6\ KN$

$Vd > V$

Hence, design is safe in shear.

* Design of weld Connection :

Horizontal shear $V_n = \frac{V \cdot A\bar{y}}{I}$

$V = 705.6\,kN$ (Already calculated)

→ $A = A_f = b_f \times t_f = 450 \times 40 = 18000\,mm^2$

→ $\bar{y} = \frac{d_w}{2} + \frac{t_f}{2} = \frac{1050}{2} + \frac{16}{2} = 533\,mm$

→ $I = \frac{450 \times (40 \times 1050 \times 40)^3}{12} - \frac{(450-16) \times 1050^3}{12}$

$I = 1224120 \times 10^4\,mm^4$

→ $V_n = \frac{705.6 \times 10^3 \times 1800 \times 533}{1224120 \times 10^4}$

$V_n = 553\,N/mm$

→ Assume length of each intermittent weld = 75mm

→ Assume size of weld, s = 8mm for length of weld.

→ check as per cl. 10.5.5.1, pg-79, IS: 800-2007

$4s = 4 \times 8 = 32mm \simeq 40mm$

$75mm > 40mm$ O.K.

→ Design strength of weld $= l_w \times t_t \times \frac{f_{uw}}{\sqrt{3}\,\gamma_{mw}}$

$= \frac{2 \times 75 \times 0.7 \times 8 \times 410}{\sqrt{3} \times 1.25}$ { 2 intermittent are present for weld }

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$= 159$ KN.

let 'a' is the c/c spacing b/w two adjacent intermittent weld.

→ horizontal ~~distan~~ shear at distance 'a' $= 553 \times a$. N

→ For safe design, Also intermittent shear will taken by weld (design strength) $= 553 \times a = 159 \times 10^3$

$a = 287.5$ mm

→ clear spacing b/w two adjacent intermittent weld $= 287.5 - 75$
$= 212.5$ mm

→ According to Cl. 10.5.5.2 pg - 79 ;
clear spacing $\not> 12t$ or 200mm
$\not> 12 \times 16 \approx 196$ mm
196 ~~or~~ 200mm

→ Adopt clear spacing $= 190$ mm
Hence provide, 8mm intermittent weld of 75mm length at clear spacing 190 mm throughout the length of the plate girder.

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Q-11) Design a welded plate girder of span 18m, to carry factored load of 70 KN/m. Also design intermiddiate transverse stiffners. connection are need not to be designed. use Fe-415 ($f_y = 250$) steel.

Sol:- Given, $L = 18\,m$

Factored load = 70 KN/m

→ Assume self weight of P.G., $w' = \frac{w}{200}$

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$= \frac{70 \times 18}{200} = 6.3$ KN/m.

Total load $= 6.3 + 70 \Rightarrow 76.3$ KN/m

Part I :

→ max. bending moment : $M_{max} = \frac{wl^2}{8} = \frac{76.3 \times 18^2}{8}$

$M_{max} = 3090.15$ KN.m.

→ maximum shear force $V = \frac{wl}{2} = \frac{76.3 \times 18}{2}$

$V = 686.7$ KN.

→ Design of web plate :
Assume section is plastic

depth of web plate $d_w = \sqrt[3]{\frac{M.K}{f_y}}$

$d_w = \sqrt[3]{\frac{3090.15 \times 180 \times 10^6}{250}}$ { for ITS provided $67 < K \leq 200$
$K = 180$

$d_w = 1305.48$ mm

also, $d_w = \frac{L}{12}$ to $\frac{L}{16}$

$= \frac{18000}{12}$ to $\frac{18000}{8}$

$= 1500$ to 2250

Adopt $d_w = 1400$ mm

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→ Thickness of web plate as per serviceability condition
pg-63, cl. 8.6.1.1.

$$t_w \geq \frac{1400}{200}(d_w)$$

$t_w \geq 8mm$

also, $k > 67 \Rightarrow \frac{d_w}{t_w}$

$$t_w = \frac{1350}{180} = 7.5\,mm$$

adopt $t_w = 8mm$

→ Size of web plate (1400 × 8) mm.

→ Design of flange Plate:
Approximate area of flange

$$A_f\,(app.) = \frac{M}{d_w\left(\frac{f_y}{\gamma_{m0}}\right)} = \frac{3090.15 \times 10^6}{1400\left(\frac{250}{1.1}\right)}$$

$A_f(app.) = 9711.9\ mm^2 \approx 9720\,mm^2$

→ as per table-2, pg-18 for plastic section

$\frac{b}{t_f} \leq 8.4$, $\frac{b_f}{2t_f} \leq 8.4$

$\frac{b_f}{t_f} \leq 16.8$

$b_f \le 10.8\, t_f$

also, $A_f = b_f \times t_f$

$$t_f^2 = \frac{9720}{16.8}$$

$$t_f = 24.05 \approx 30\,mm$$

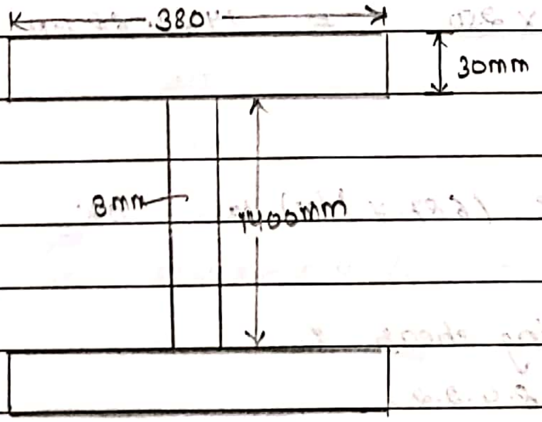
$$b_f = \frac{A_f}{t_f} = \frac{9720}{30}$$

$$b_f = 324\ mm$$

$b_f = 0.3 d_w = 1400\,mm \times 0.3 = 420\,mm$

Adopt $b_f = 380\,mm$ (intermediate of 324 mm & 420 mm)

→ Size of flange plate 380 x 30 mm.



→ Section classification ;

$$\frac{b}{t_f} < 8.4 \Rightarrow \frac{b_f - t_w}{2t_f} = \frac{380-8}{2\times 30} = 6.2 < 8.4$$

Hence section is plastic.

→ Design Bending moment : $Md = \frac{Zp \cdot fy}{\gamma m_0}$

$Zp = bf \cdot tf (dw + tf)$
$= 380 \times 30 (1400 + 30) = 16302000\ mm^3$

$Md = \frac{16302000 \times 250}{1.1} = 3705\ kN \cdot m$

$Md \geq 3705\ kN \cdot m > M\ (3090.15\ kN \cdot m)$

Hence ok.

→ Design SF : pg-59

$Vd = \frac{Av \cdot fy}{\sqrt{3}\, \gamma m_0} = \frac{dw \cdot tw \cdot fy}{\sqrt{3}\, \gamma m_0}$

$= \frac{1400 \times 8 \times 250}{\sqrt{3} \times 1.1} = 1469.69\ kN$

$Vd = 1469.69 > (687.7)(V)\ kN.$ OK

→ Check for buckling shear :
(pg-59, 60 cl. 8.4.2.2)

$Vcr = \frac{Av \cdot \tau_b}{\gamma m_0}$

$\tau_{cr,e} = \frac{kv\, \pi^2 E}{12 (1 - \mu^2) (d/tw)^2}$

assume c/c spacing b/w ITS $= c = 2m = 2000\ mm$

$$\Rightarrow \frac{c}{d} = \frac{2000}{1400} = 1.43 > 1$$

$$K_v = 5.35 + \frac{4.0}{(c/d)^2} = 5.35 + \frac{4.0}{(1.43)^2} = 7.306$$

assume $\mu = 0.3$, $E = 2 \times 10^5 \ N/mm^2$

$$\tau_{cr,e} = \frac{7.306 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1400}{8}\right)^2} = 43.12 \ N/mm^2$$

$$\tau_{cr,e} = 43.12 \ N/mm^2$$

$$\Rightarrow \lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\frac{250}{43.12 \times \sqrt{3}}}$$

$$\lambda_w = 1.83 > 1.2$$

$$\Rightarrow \tau_b = \frac{f_{yw}}{\sqrt{3}\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.83^2} = 43.10 \ N/mm^2$$

$$\Rightarrow V_{cr} = \frac{A_v \tau_b}{\gamma_{mo}} = \frac{1400 \times 8 \times 43.10}{1.1} = 438.84 \ kN$$

$$V_{cr} = 438.84 \ kN < V \ (686.7 \ kN)$$

fails in buckling shear.

here, intermediate transverse stiffeners (ITS) are to be used to improve buckling strength of the slender web.

→ Design of I.T.S. :-

let us assume c/c spacing of ITS, $C = 2000\,mm$

shear force on I.T.S. nearest to support $V_1 = V - wc$

$$= 686.7 - 76.3 \times 2$$

$$V_1 = 534.1\ kN$$

→ For minimum MoI of ITS (8.7.2.4 pg-66)

$$\frac{c}{d} = \frac{2000}{1400} = 1.43 > \sqrt{2}$$

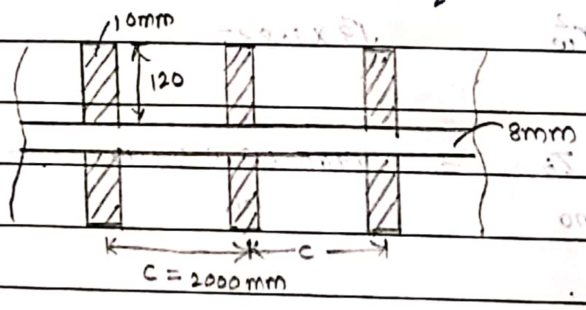
$$I_s = 0.75\, d\, t_w^3$$

$$= 0.75 \times 1400 \times 8^3 = 537600\ mm^4$$

let us try pair of plates 120mm x 10mm

$$14 t_q = 14 \times 10\,mm = 140\,mm \quad \text{(less then it)}$$

check for outstand $= 120\,mm \ngtr 14 t_q \ngtr 140\,mm$ o.k.



$$I_{provided} = \frac{bd^3}{12}$$

$$= \frac{10(120 + 8 + 120)^3}{12} - \frac{10 \times 8^3}{12}$$

$$= 12710400\ mm^4$$

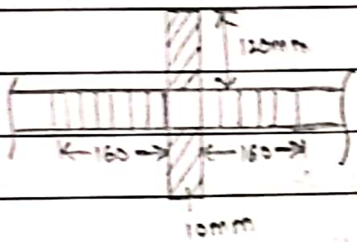
$I_{provide} >>>> I_s$ Hence O.K.

Required shear strength from ITS (pg-57):

$$F_q = \frac{V_1 - V_{cr}}{\gamma_{mo}} = \frac{534.1 - 432.84}{1.1} = 135.15 \text{ kN}.$$

→ As per code, the section of I.T.S. consist of the plate and also web portion equal to 20 tw on other side of it (pg-66 clause 8.7.1.5)

$20\ t_w = 20 \times 8 = 160$ mm.



Total area of ITS portion $= 2 \times 120 \times 10 + 2 \times 160 \times 8$

$= 4960 \text{ mm}^2$

→ MoI of ITS portion:

$$I = \frac{10(120+8+120)^3}{12} - \frac{10 \times 8^3}{12} + \frac{2 \times 160 \times 8^3}{12}$$

$I = 1272.40 \times 10^4 \text{ mm}^4$

→ eff. length $= 0.7\ d_w$

$= 0.7 \times 1400 = 50.65$ mm

→ Slenderness ratio, $Sr = \frac{\text{eff. length}}{\text{radius of gyration}} = \frac{980}{50.65}$

$Sr = 19.35$

From pg - 42 table 9(c)

KL/r	fcd
10	227
19.35	fcd = ?
20	224

$$f_{cd} = 227 - \frac{(227-224)(19.35-10)}{(20-10)}$$

$$f_{cd} = 224.19 \simeq 224.2$$

→ Strength of ITS = Area x fcd

= 4960 x 224.2

= 1112.032 KN > V1 (534.1 KN)

Hence, OK.

Q-12) Design a gantry girder simply support and carrying one EOT crane from following data :

Capacity of crane = 250 KN

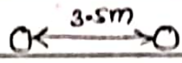
self wt. of crane bridge (excluding trolley) = 200 KN.

wt. of trolley or crab = 50 KN.

span of crane girder = 16 m

span of gantry girder = 6.5 m.

wheel base = 3.5 m



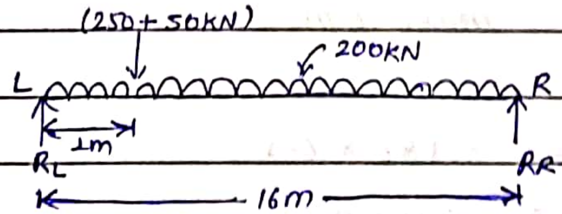
minimum approach of Hook = 1m.

self wt. of rail = 0.3 KN/m.

Sol:- A total wt. of crane capacity and trolley i.e. 250 KN + 50 KN is travelling on crane bridge.

Its position should be nearest to any of the a.c. (for max. B.M.)

[distance of min approch of hook i.e. 1m]



Taking moment at R ;

$$R_L \times 16 = 200 \times \frac{16}{2} + 300 \times 15$$

$$R_L = 381.25\ KN$$

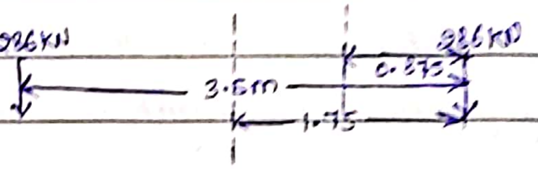
This load is divided in 2 wheels loads of equal amount i.e.

$\frac{381.25}{2} = 190.63$ KN

Factored value of one wheel load 190.63×1.5

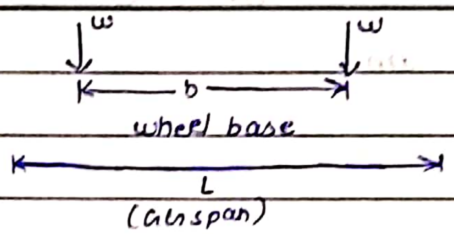
$\Rightarrow 285.94$ KN ≈ 286 KN

Set of load is ;



This set of loads is moving on gantry girder of span 6.5 m this position should be such as to cause absolute max B.M. in the gantry girder.

According to theorem in "TOS" for absolute max B.M., they should be so placed that their C.G. and loads under which Mmax is occuring should be equidistance from the centre of span (C.G.).



if $0.586\ L < b$, then one load at mid span gives absolute max. B.M.

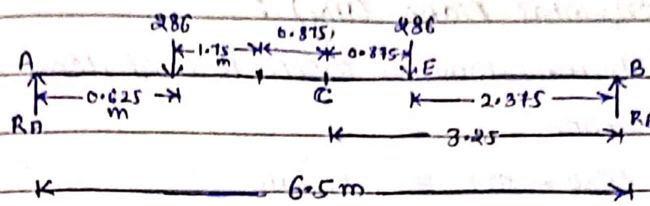
if $0.586\ L > b$, then apply TOS theorem for absolute max. B.M.

Here, $0.586\ L = 0.586 \times 6.5$

$= 3.809$ m

3.809 m > 3.5 m

$\therefore$ For absolute max. B.M. we have to TOS therom



Taking moment about point A :

$$RB = \frac{286 \times 0.625 + 286\ (3.25 + 0.875)}{6.5}$$

$RB = 209\ kN$

Absolute max. BM occure at E : $ME = 209 \times 2.375$

$ME = 496.375\ kN \cdot m$

Assume self wt. of GG = 2kN/m

Self wt. of rail = 0.3kN/m

Total weight = 2 + 0.3 ⇒ 2.3 kN/m

total factored self wt. = 2.3×1.5

= 3.45 kN/m

max. B.m due to self wt = $\frac{wl^2}{8}$

$$= \frac{3.45 \times 16.5^2}{8} = 18.22\ kN \cdot m$$

Due to impact we take 25% allowance for EOT crane total

BM including impact = $496.375 \times 1.25 + 18.22$

= 638.688 kN·m

Mx = 638.688 kN·m = B.m. in vertical plane.

→ BM in horizontal plane (My) :

Value of lateral load is taken 10% of load lifted and trolley for EOT crane.

$= \frac{10}{100} \times (250 + 50) = 30$ KN.

∵ there are 4 wheels in trolley

⇒ lateral load per wheel $= \frac{30}{4} = 7.5$ KN.

factored lateral load per wheel $= 7.5 \times 1.5$

$= 11.25$ KN.

There position on GG for max BM in horizontal plane will be same as for verticle load.

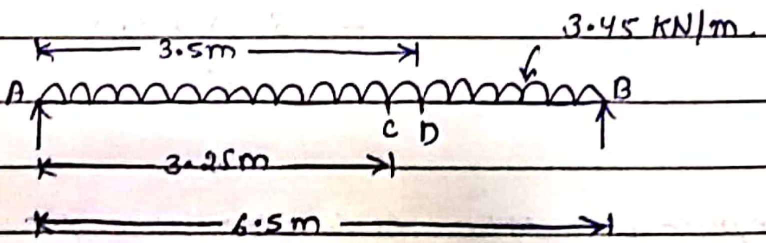
It is found by proportion.

$$\frac{M_y}{M_E} = \frac{F_y}{F_x}$$

$$M_y = \frac{F_y}{F_x} \times M_E = \frac{11.25}{286} \times 496.375$$

$$\boxed{M_y = 19.525 \text{ KN·m}}$$

→ Max. S.F. in GG :



Max. S.F. will be at point A acc. to position of load :

$$\text{Max. S.F.} = R_A = \frac{(286 \times 6.5) + (286 \times 3)}{6.5} + \frac{3.45 \times 6.5 \times 6.5}{6.5 \times 2}$$

$$R_A = 418 + 11.2125$$

max. S.F. including impact 25 % for EDT crane = $418 \times 1.25 + 11.21$

Max S.F. = 533.71 kN.

$$\boxed{\text{Max. S.F.} = 533.71 \text{ kN.}}$$

→ Trial section - Guidelines :

i) depth $\simeq \frac{L}{12}$, width $\simeq \frac{L}{30}$

ii) section choice is as per crane capacity.

→ I-section vary from ISWB 500 to ISLB 600

→ C section vary from ISMC 300 to ISMC 400.

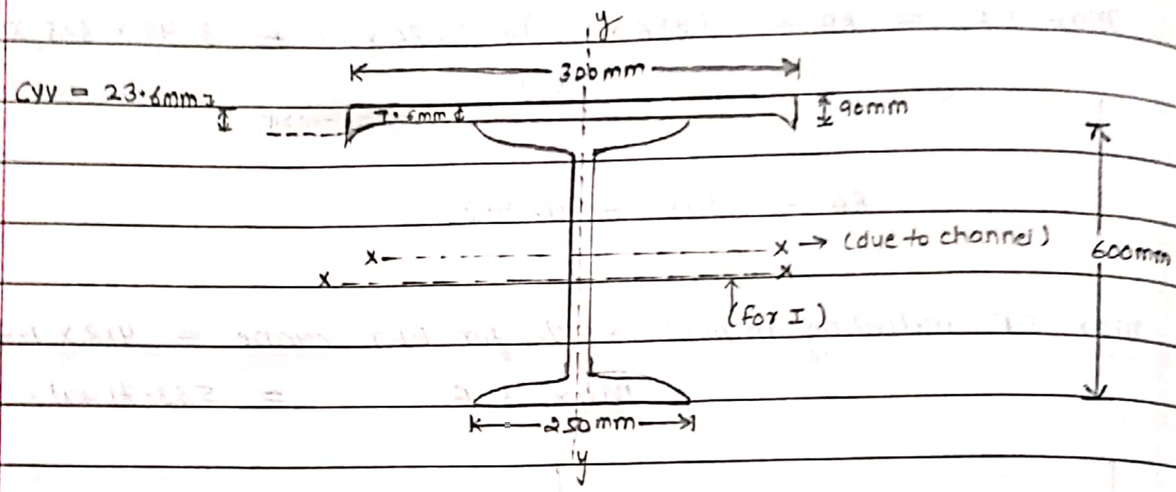
Take a pair (I, C section) to suit capacity ranking from 150 kN to 250 kN

Here,

$$\text{depth} = \frac{L}{12} = \frac{6500}{12} = 541.66 \text{ mm.}$$

$$\text{width} = \frac{L}{30} = \frac{6500}{30} = 216.66 \text{ mm.}$$

let us try ISWB 600 [541.66 se bda 550 tha but due to section this is very low value that's why ISWB 600] @ 1.312 kN/m and ISMC 300 @ under -



	ISWB 600	ISMC 300
A	$17038\ mm^2$	$4564\ mm^2$
bf	250	90 mm
tf	21.3	13.6 mm
tw	11.2 mm	7.6 mm
Ixx	$106198.5\ cm^4$	$6362.6\ cm^4$
Iyy	$4702.5\ cm^4$	$310.8\ cm^4$
Cyy		23.6 mm

To get CG $\bar{y}$ from bottom :

$$\bar{y} = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$$

$$= \frac{17038 \times 300 + 4564 \times (600 + 7.6 - 23.6)}{17038 + 4564}$$

$$\bar{y} = 360\ mm.$$

To find $I_{xx}' = I_{self} + A\underbrace{h^2}_{(y-\bar{y})^2} + I_{xx\,self} + Ah^2$

$$\Rightarrow 106198.5 \times 10^4 + 17038\,(360-300)^2 + 310.8 \times 10^4 + 4564\,(247.6 - 23.6)^2$$

$= 135543.30 \times 10^4\ mm^4$

$I_{yy'} = 4702.5 \times 10^4 + 6362.6 \times 10^4$

$= 110651000\ mm^4$

$Z_e = \frac{I_{xx'}}{y_{max}} = \frac{135543.30 \times 10^4}{360} = 3765.09 \times 10^3\ mm^3$

For compression flange about Y-Y axis ;

$I_y = \frac{21.3 \times 250^3}{12} + 6362.6 \times 10^4 = 9136.04 \times 10^4\ mm^4.$

→ $Z_{ey} = \frac{I_y}{y} = \frac{I_y}{h} = \frac{9136.04 \times 10^4\ mm^4}{150} = 609.07 \times 10^3\ mm$

we should have interaction formula to be satisfied.

$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} < 1$

→ To calculate M_{dx} :

Here CG is laterally supported

$M_{dx} = Z_p \cdot f_{cd}$

for calculating Z_p :

$Z_p = \frac{A}{2}(y_1 + y_2)$

$= \frac{A}{2} y_1 + \frac{A}{2} y_2$

$= A_c y_1 + A_t y_2$

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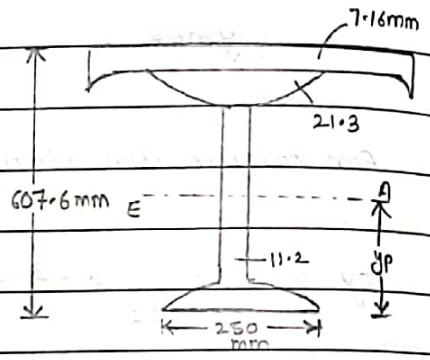
$$Z_p = a_1y_1 + a_2y_2 + a_3y_3 + \cdots + a_ny_n.$$

$$\boxed{Z_p = \Sigma ay}$$

let EA axis lies at a distance yp from bottom:

$$250 \times 2.13 + (y_p - 21.3)\,11.2 = \frac{A}{2}$$



$$5325 + (y_p - 21.3)\,11.2 = 10801$$

$$y_p = 510.22$$

$$\boxed{y_p \approx 510 \text{ mm}}$$

Zp will be obtained by taking moments of individuals component in compression and tension about EA axis.

$$M_p = a.y.f_y$$
$$Z_p = \frac{M_p}{f_y} = \frac{a.y.f_y}{f_y}$$

$$Z_p = a.y.$$

$$Z_p = 4564 \times (607.6 - 510 - 23.6) + 21.3 \times 250\left(600 - 510 - \frac{21.3}{2}\right) +$$
$$(600 - 510 - 21.3) \times 11.2 \times \left(600 - 510 - \frac{21.3}{2}\right) + (510 - 21.3) \times 11.2 \times$$
$$\left(\frac{510 - 21.3}{2}\right) + 21.3 \times 250\left(510 - \frac{21.3}{2}\right)$$

$$Z_p = 760343.45 + 1363865.328 + 2659038.75$$

$$Z_p = 4783.25 \times 10^3 \text{ mm}^3$$

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→ To calculate f_{bd} :

$$\frac{h}{t_f} = \frac{600+7.6}{21.3+7.6} \simeq 21$$

Total area, $A = 17038 + 4564 = 21602 \ mm^2$

Radius of gyration $r_{yy} = \sqrt{\frac{I_{yy}'}{A}} = \sqrt{\frac{11065 \times 10^4}{21602}} = 71.57 mm$

Slenderness ratio $= \frac{KL}{r} = \frac{1 \times 6500}{71.57} = 90.82$

from IS 800:2007, pg-57, table-14

KL/r	h/t_f		
	x_1 20	x 21	x_2 25
y_1 90	380.4		344.2
y 90.82		$f_{cr,b}$	
y_2 100	325.8		291.4

$$f_{cr,b} = \frac{(25-21)(100-90.82)}{(25-20)(100-90)} \times 380.4 + \frac{(21-20)(100-90.82)}{(25-20)(100-90)} 344.2$$

$$+ \frac{(25-21)(90.82-90)}{(25-20)(100-90)} \times 325.8 + \frac{(21-20)(90.82-90)}{(25-20)(100-90)} \times 291.4$$

$f_{cr,b} = 368.7 \ N/mm^2$

from table -13 (a) pg-55 IS 800:2007

$$\Rightarrow \frac{((184.1)-172.7)(368.7-350)}{(400-350)} + 172.7$$

$f_{cr,b}$	$f_y = 250$
400	184.1
368.7	f_{bd}
350	172.7

$$\Rightarrow 176.96 = f_{bd}$$

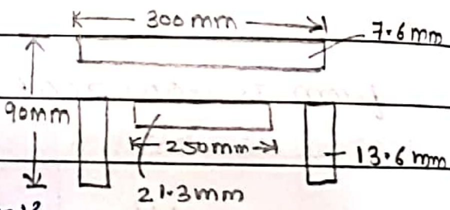
So, $M_{dx} = Z_p \cdot f_{bd}$

$= 4783.25 \times 10^3 \times 176.96$

$M_{dx} = 846.44 \text{ kN·m}$

$M_{dx} > M_x$ (638.688 kN·m) O.K.

To calculate $Z_{p \cdot y}$ (compression part):



$$Z_p = \frac{1}{4} bd^2$$

$$= \frac{1}{4} \times 21.3 \times 250^2 + \frac{1 \times 7.6}{4} (300 - 13.6 \times 2)^2$$

$$+ 2 \times 90 \times 13.6 \left(150 - \frac{13.6}{2}\right)$$

$$Z_{py} = 824.76 \times 10^3 \text{ mm}^3$$

$M_{dy} = \frac{Z_{py} \cdot f_y}{\gamma_{mo}} \ngtr \frac{1.2\, Z_{ey} f_y}{\gamma_{mo}}$ (From pg - 70)

$$= \frac{824.76 \times 10^3 \times 250}{1.1} \ngtr \frac{1.2 \times 609.07 \times 10^3 \times 250}{1.1}$$

$$= 187.44 \ngtr 166.11 \text{ kN·m}$$

The lesser value is 166.11 kN.m

So, $M_{dy} = 166.11 \text{ kN·m}$

Interaction formula as per code ;

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} < 1$$

$M_x = 638.68$ KN·m

$M_y = 19.525$ KN·m

$M_{dx} = 846.44$ KN·m

$M_{dy} = 166.11$ KN·m

$$\Rightarrow \frac{638.68}{846.44} + \frac{19.525}{166.11} < 1$$

$\Rightarrow$ $0.87 < 1$ OK

Check for shear :

As we already calculate the maximum shear force for G.G.

Max. S.F. = 533.7 KN

→ Design shear force (pg-59 IS : 800-2007)

$$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\, \gamma_{m0}} \qquad \because A_v = \text{Area of web.}$$

$$= \frac{600 \times 11.2 \times 250}{\sqrt{3} \times 1.1}$$

$= 881.77$ KN $>$ 533.7 KN OK

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