VI - Unit

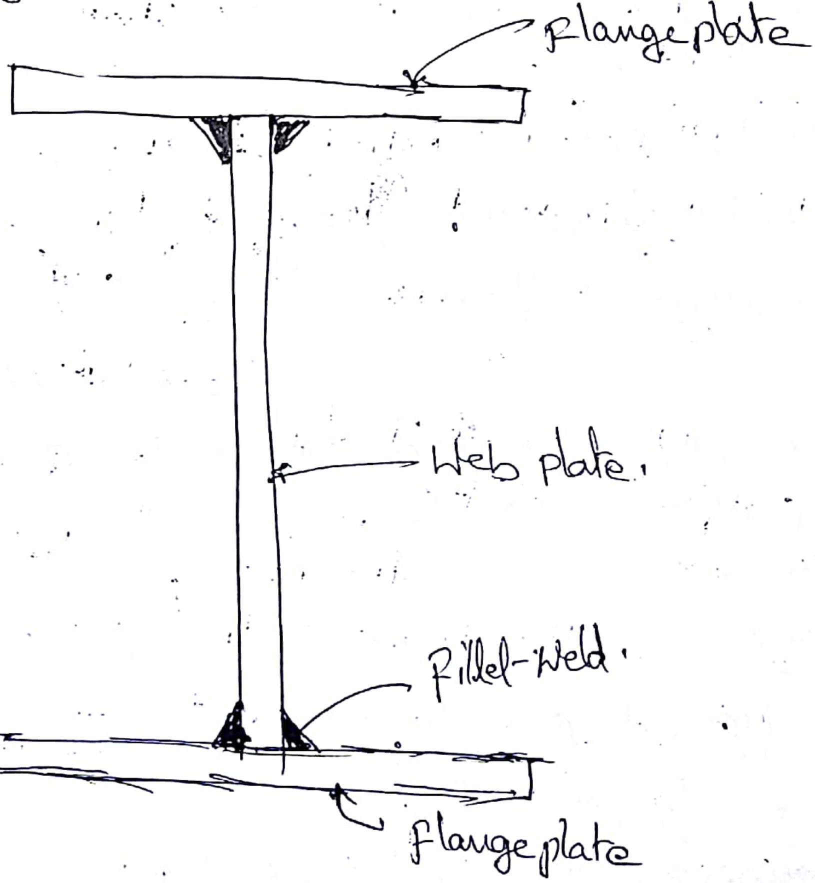
Design of plate girder :-

When ~~the~~ span and load increase the available rolled steel section may not be sufficient, even after strengthening with cover plates. Such situations are common in the following. ① large column free halls are required in the ~~in the~~ lower floor of a multistory building.
② In a work shop, where girders are required to carry ~~cara~~ crane beams. ③ In road (or) railway bridges.

In such situations one of the remedies is to go for a built up I-section with two flanges plates connected to a web plate of required depth. The depth of such I-beams may vary from 1.5 m to 5.0 m length. This type of I-beams are known as plate girder. as shown in fig.

Disadvantage of Riveted Connections:-

1. It is associated with high level of noise pollution
2. It needs heating the rivit to red hot
3. Inspection of connection requires a skilled person.
4. Removing poorly installed rivet is costly
5. Installation cost is high.



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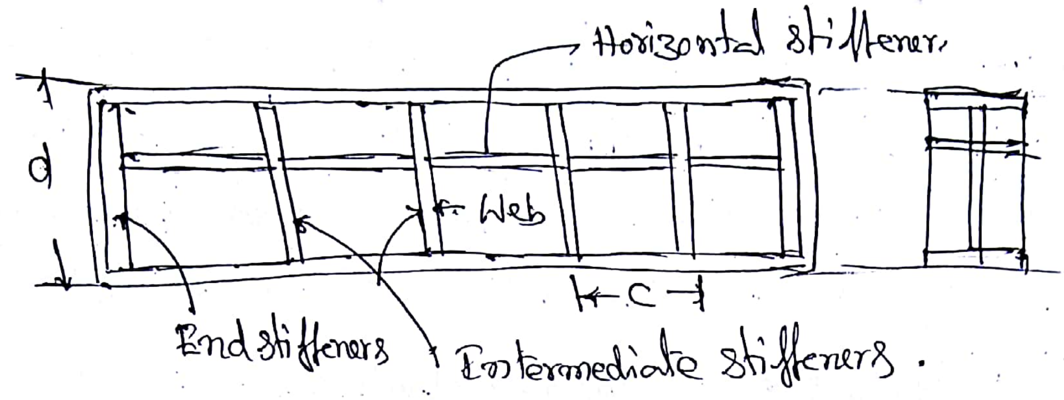
Elements of plate girders 6, 29/33 W

1) Web (2) Flanges (3) Stiffeners. 6 – 9, 10, 11



Webs of required depth and thickness are provided to
a) Keep flanges plates at required distances.
b) Resist the shear in the beam.

· Flanges of required width and thickness are provided to resist bending moment acting on the beam by developing Compressive force in one flange and tensile force in another flange.

- Stiffeners are provided to safeguard the web against local ~~bulking~~ buckling failure. The stiffeners provided may be classified as-

a) Transverse (Vertical) ~~sti~~ stiffeners.
b) Longitudinal (horizontal stiffeners

Transverse stiffeners → Bearing stiffener
Transverse stiffeners → Intermediate stiffener

Q Design a welded plate girder of span 24 mts to carry super imposed load of 35 kN/m Avoid use of Bearing and Intermediate stiffeners.

Solution Use Fe 410 grade steel.

· (1) Moment and shear force:

Span = 24 mts.

Super imposed load = 35 kN/m

∴ Factored load = 35 × 1.5 = 52.5 kN/m

Self Weight = $\frac{\text{Total load}}{200} = \frac{52.5 \times 24}{200} = 6.3$ kN/m

$\therefore$ Total load $= 52.5 + 6.3 = 58.8$ kN/m

$\therefore$ Maximum moment $= M = \frac{WL^2}{8} = \frac{58.8 \times 24^2}{8} = 4233.6$ kN-m

B.M. Max = 4233.60 kN-m (or) 4233.60×10^6 N-mm

Maxi S.F = End reaction.

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

(2) Depth of Web plate :-

If Stiffeners are to avoided

$K = \frac{d}{t_w} \leq 67$ (or) $K = 67$

$\therefore$ Economical depth of Web $= d = \sqrt[3]{\frac{M \times k}{f_y}} = \left(\frac{4233.6 \times 10^6 \times 67}{250}\right)^{1/3}$

$d = 1043$ mm

Use 1000 mm plates.

$t_w \geq \frac{d}{k}$. ie $\geq \frac{1000}{67} = 14.92$

Select $t_w = 16$ mm

Thus Web plate selected is 1000 mm × 16 mm.

(3) Selection of flange

Neglecting the moment capacity of Web, area of flange required is $\frac{A_f f_y d}{\gamma_{m0}} \geq M$

$\frac{A_f \times 250 \times 1000}{1.1} \geq 4233.6 \times 10^6$

$A_f = \frac{4233.6 \times 10^6 \times 1.1}{250 \times 1000} = 18628$ mm^2

To keep the flange in plastic category $\frac{b}{2t_f} \leq 8.4$.

$\therefore \frac{b_f}{2t_f} \leq 8.4$

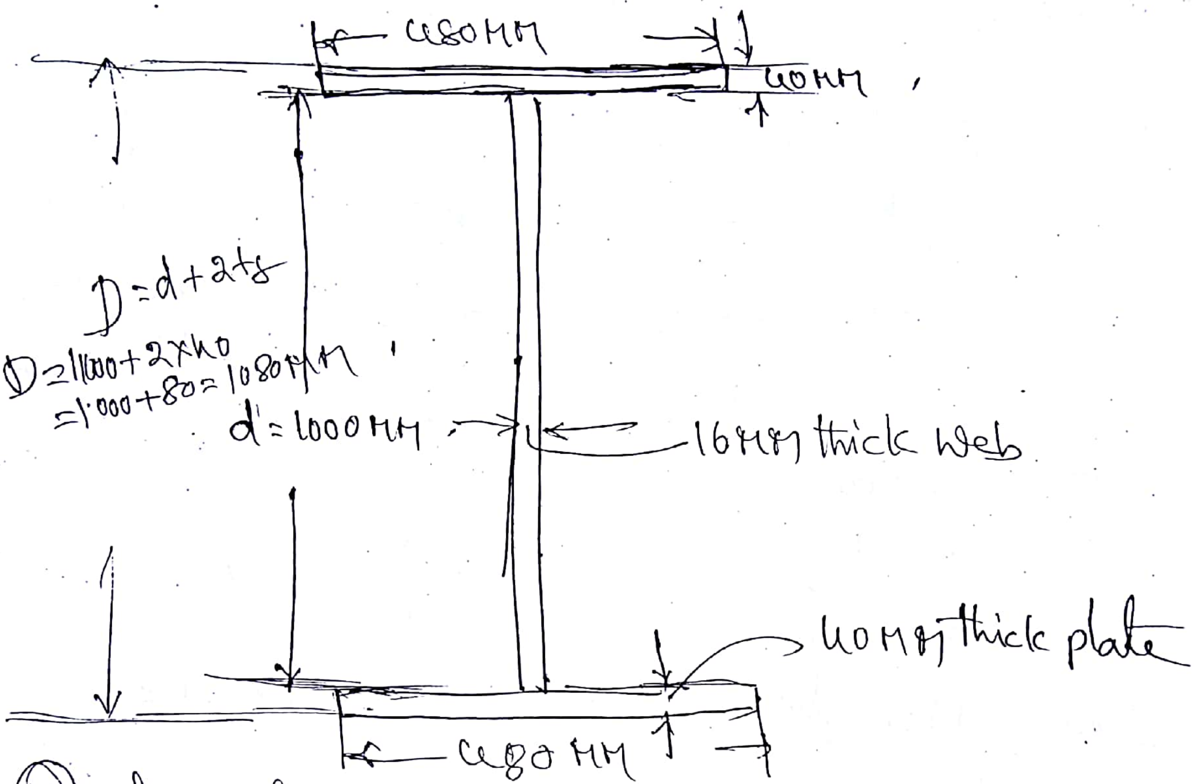
$$A_f = 16.8\, t_f \times t_f = 18628$$

$$t_f^2 = \frac{18628}{16.8}$$

$$t_f = 33.3\ mm,$$

Select 40 mm plates, Width of plate required $= \frac{18628}{40} = 465.7mm$

Hence use 480 X 40 MM thick plates. Section Selected as shown in fig.



(ii) Check for the moment Capacity of the girder

Since it is assumed that only flanges resist the moment and flange is a semi compact section, (clause 8.2.1.2)

$$M_d = \frac{Z_e f_y}{\gamma_{mo}}$$

$$I_{zz} = 2\left[\frac{b_f t_f^3}{12} + b_f t_f \left(\frac{D}{2}\right)^2\right] \quad \left[2\left(\frac{bd^3}{12} + ah^2\right)\right]$$

$$= 2\left[\frac{1}{12} \times 480 \times 40^3 + 480 \times 40 \left(\frac{1080}{2}\right)^2\right]$$

$$= 2 \times 5601.28 \times 10^6\ MM^4$$

$$Ze = \frac{I_{zz}}{y_{maxi}} = \frac{2 \times 5601.28 \times 10^6}{540} = 20.745 \times 10^6 \ mm^3 \quad \left(y_{maxi} = \frac{D}{2}\right)$$

$$\therefore M_d = \frac{20.745 \times 10^6 \times 250}{1.1} \quad \left(\frac{Ze \cdot f_y}{\gamma_{m0}}\right) = \cancel{20.745 \times 10^6 \ N-mm}$$

$$M_d = 4714.77 \times 10^6 \ N\text{-}mm$$

$$= 4714.77 \ KN\text{-}m > M = 4233.60 \ KN\text{-}m$$

Hence the section is safe.

⑤ Shear resistance of Web (clause 8.4)

$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v f_{yw}}{\gamma_{m0}\sqrt{3}} = \frac{d \times t_w f_{yw}}{\gamma_{m0}\sqrt{3}}$$

$$V_d = \frac{1000 \times 16 \times 250}{1.1\sqrt{3}} = 2099.455 \times 10^3 \ N$$

$$V_d = 2099.455 \ KN > 705.6 \ KN$$

Hence the section is adequate. No stiffeners are required.

⑥ Check for end bearing

$$\text{Bearing strength of Web} = F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$$

Assuming that the width of support is 200mm. ~~stiff~~ minimum stiff bearing provided by support = 100mm = b_1

$$\text{Dispersion length} = n_2 = 2.5 t_f = 2.5 \times 40 = 100 \ MM$$

$$F_w = (100 + 100) \times 16 \times \frac{250}{1.1} = 727 \times 10^3 \ N = 727 \ KN$$

$$F_w = 727 > 705.6 \ KN$$

Hence it is O.K. ∴ The end stiffeners is ~~also~~ not required.

(7) Design of Weld Connecting Web plate and flange.

Maximum Shear force = 705.6 kN (or) 705.6×10^3 N

Shear stress in flange at the level of junction of Web and flange.

$$q = \frac{F}{bI}(a\bar{y}) = \frac{705.6\times10^3}{480\times2\times5601.28\times10^6}\left[480\times40\times\left[500+\frac{40}{2}\right]\right]$$

$q = 0.512\ N/mm^2$

∴ Shear force per mm length in the junction.

$= 0.512\times480 = 245.76$ N ✓

If 'S' is the size of shop weld, throat thickness is 0.7S. providing Weld on both sides of Web strength per unit length

$$~~2\times0.75\times~~ = 2\times0.7S\times\frac{f_u}{\sqrt{3}}\cdot\frac{1}{\gamma_{mw}}$$

$\gamma_{mw} = 1.25$ for shop Weld.

$f_u = 410\ N/mm^2$

$$= 2\times0.7S\times\frac{410}{\sqrt{3}}\cdot\frac{1}{1.25} = 265\times S$$

equating it to Shear force, We get

$$265\ S' = 245.76$$

$\therefore S = 0.92$ mm

But a minimum of 5mm is to be provided since the thickness of Web is 16mm. Intermittent Welds may be provided

$$\therefore \%\text{ of Weld length} = \frac{0.92}{5}\times100 = 18.4\text{ mm}$$

Use 40 mm long Welds With a gap of 160 mm

a) Minimum Weld length 50mm,

b) Maximum unwelded length $= 12\times16 = 192$ mm ($=12t_s$)

Final Design →

Web = 1000 × 16 mm, Flange = 480 × 40 mm. No stiffeners are required.

Weld = 5mm intermittent of length 40mm and a gap of 160 mm.

Q Design a welded plate girder of span 24mts to carry superimposed load of 35 kN/m. using end stiffeners and thin web, but avoid intermediate stiffeners. Use Fe415 (E250) steel.

Solution:

Span = 24 mts.

Superimposed load = 35 KN/m.

∴ Factored load = 35 × 1.5 = 52.5 KN/m.

f_u = 415 N/mm²

f_y = 250 N/mm²

(1) Self wt of plate girder = $\frac{\text{Total load}}{200}$ =

$= \frac{52.5 \times 24}{200} = 6.3$ KN/m.

∴ Total load (Factored) = 52.5 + 6.3 = 58.8 KN/m

Maxi B.M = $M = \frac{W_u l^2}{8}$

$M_{maxi} = \frac{58.8 \times 24^2}{8} = 4233.6$ KN-m

~~Maxi B.M~~ ⟹ Maxi S.F = $V_{maxi} = \frac{WL}{2} = \frac{58.8 \times 24}{2}$

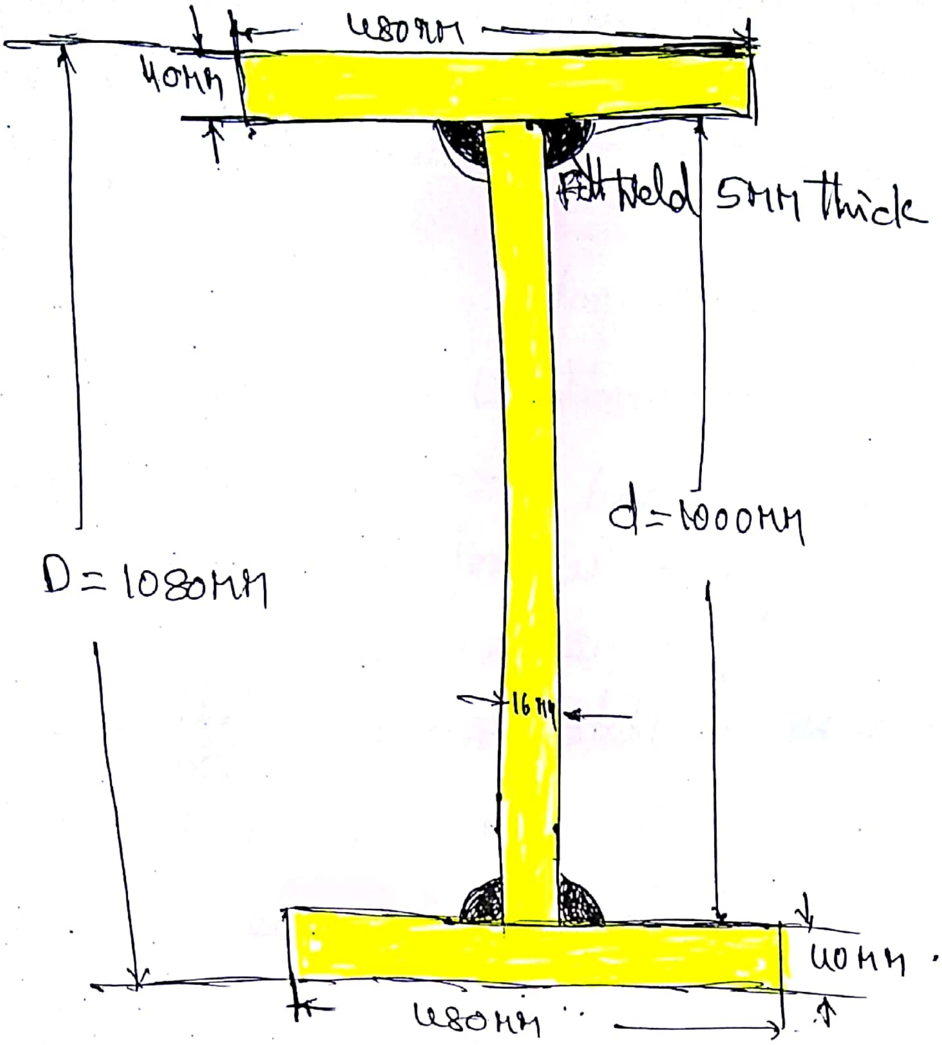
V_{maxi} 705.6 KN (or) 705.6×10^3 N

(2) Depth of Web: – i) If $\frac{d}{t_w} > 200$, Intermediate transverse stiffeners are to be provided.

ii) If $\frac{d}{t_w} \leq 67$, end as well as intermediate transverse stiffeners are not required but thick webs are required. It is around $K = \frac{d}{t_w} = 100$ that thin webs with end stiffeners.

try $K = \frac{d}{t_w} = 100$ in this case; Economical depth $= \sqrt[3]{\frac{MK}{f_y}}$

$= \left[\frac{4233.6 \times 10^6 \times 100}{250}\right]^{1/3} = 1192 \simeq 1200$ MM.



A Typical c/s of PLATE GIRDER

provide 1200 MM wide plates.

$$K = \frac{d}{t_w} = 100 \geq \frac{1200}{t_w} = 100$$

$$t_w = \frac{1200}{100} = 12MM.$$

Use 1200MM × 12MM plates for Web

3) Flanges

Assuming only flanges resist moment, area of flange A_f required is given by.

$$\frac{A_f \cdot f_y \cdot d}{\gamma_{mo}} \geq \text{Max B.M},$$

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$$\frac{A_f \times 250 \times 1200}{1.1} = 4233.6 \times 10^6$$

$$A_f = 15523 \text{ mm}^2$$

To keep the flange in semi ~~circular~~ compact category,

$$\frac{b}{t_f} \leq 13.6$$

Assuming $(b) = 13.6 \times t_f$, we get

$\cancel{A_f = 13.6\, A_f}$ $\quad A_f = b_f \times t_f = 13.6\, t_f \times t_f$

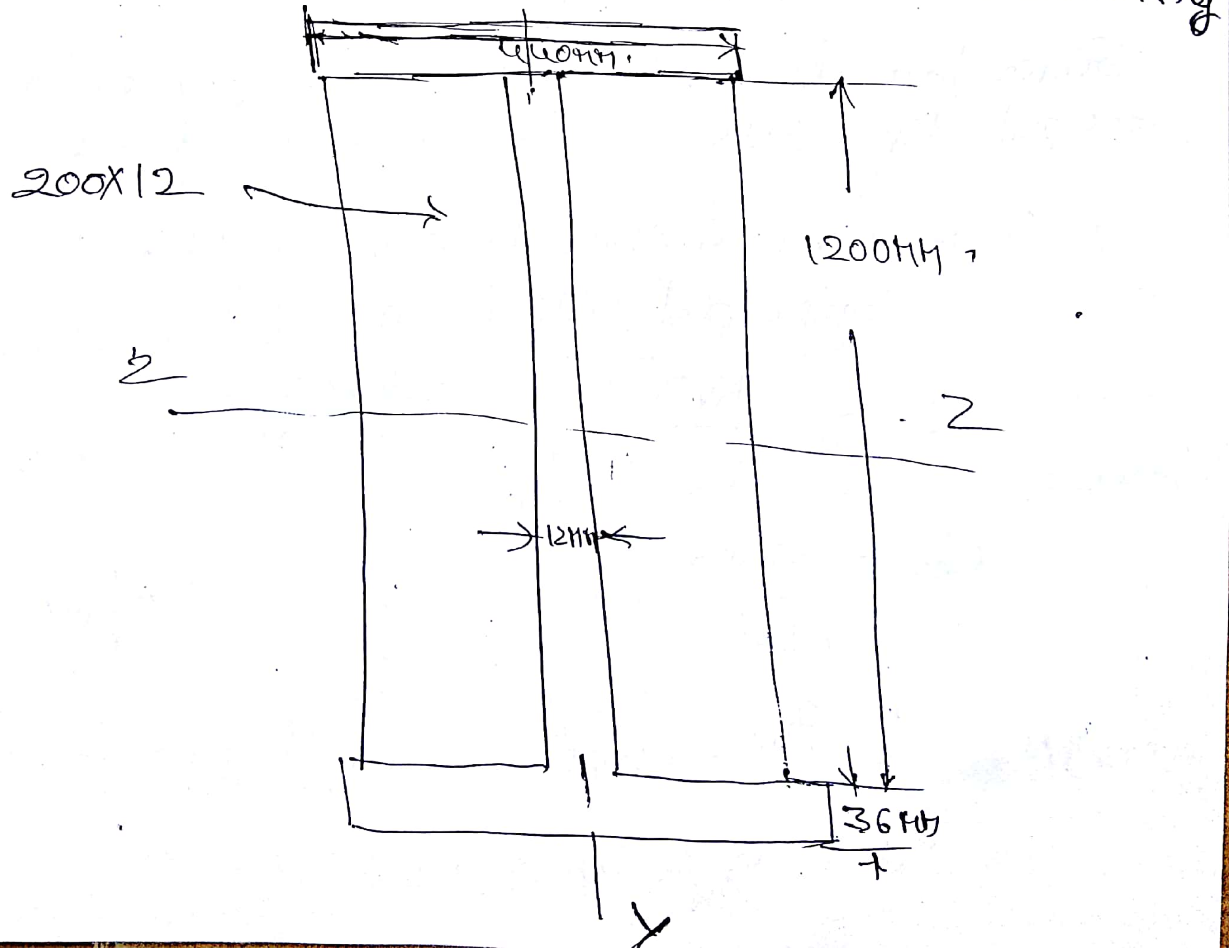
$$15523 = 13.6\, t_f^2$$

$$t_f = \sqrt{\frac{15523}{13.6}} = 33.8 \text{ mm}$$

$\therefore$ Select 36 mm plates.

$$b = \frac{15523}{36} = 431 \text{ mm}$$

Use 440 MM × 36 mm thick flange plates, as shown in fig



(a) Check for moment Capacity:

Assuming only flanges resist moment

$$I_{zz} = 2\left[\frac{bd^3}{12} + ah^2\right]$$

$$I_{zz} = 2\left[\frac{1}{12}\times 440\times 36^3 + 440\times 36\times \left(600+\frac{36}{2}\right)^2\right]$$

$$= 1.210\times 10^{10}\ mm^4$$

$$\therefore Z_e = \frac{I_{zz}}{y_{max}} = \frac{1.21\times 10^{10}}{y_{max}} = \frac{1.21\times 10^{10}}{636} = 19.03\times 10^6\ mm^3$$

$$Z_e = 19.03\times 10^6\ mm^3$$

Since it is assumed that only flanges resist the moment and flanges is a semi compact section.

$$M_d = \frac{Z_e \times f_y}{\gamma_{mo}} = \frac{19.03\times 10^6 \times 250}{1.1} = 4323.9\times 10^6\ N\text{-}mm.$$

$$M_d = 4324.9\ kN\text{-}m > M_{max}$$

Hence it is adequate.

Since transverse stiffeners are to be provided only at support, $K_v = 5.35$ and in this case $\frac{d}{t_w} > 67$

Hence resistance to shear buckling should be verified.

Consider simple post critical method (clause 8.4.2.2)

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

$K_v = 5.35$, $E = 2\times 10^5$, $\mu = 0.3$; $d = 1200mm$, $t_w = 12mm$.

$$\tau_{cr} = \frac{5.35\times \pi^2 \times 2\times 10^5}{12(1-0.3^2)\left(\frac{1200}{12}\right)^2} = 96.7\ N/mm^2$$

$$\therefore \lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3}\times 96.7}} = 96.97\ N/mm^2$$

$$\lambda_w = 96.97\ N/mm^2$$

$\therefore V_{cr} = \cancel{d t_w \tau_b} \; d\, t_w\, \tau_b$

$= 1200 \times 12 \times 96.97 = 1396.44 \times 10^3$ N

$\therefore V_{cr} = 1396.44$ KN

$$\therefore V_d = \frac{V_n}{\gamma_{m0}} = \frac{V_{cr}}{\gamma_{m0}} = \frac{1396.44}{1.1} = 1269.49 \text{ KN}$$

$V_d = 1269.49 \text{ KN} > 705.6 \text{ KN}$

Hence shear strength is adequate.

⑥ <u>Local capacity of the Web (clause 8.7.4)</u>

As per the clause 8.7.4 local capacity of Web

$$F_W = \frac{(b_1 + n_2)\, t_w \times f_{yw}}{\gamma_{m0}}$$

taking $b_1 = 0$

$n_2 = 2 \times (2.5\, t_f)$

$n_2 = 2 \times (2.5 \times 36) = 180$ mm.

$$\therefore F_W = \frac{(0 + 180) \times 12 \times 250}{1.1} = 6.60 \times 10^5 \text{ N}$$

$F_W = 660.0 \text{ KN} < 705.6$ N

Hence end stiffeners are required.

⑦ <u>Design of end stiffeners.</u>

Outstanding of flange $= \frac{440 - 12}{2} = 214$ mm

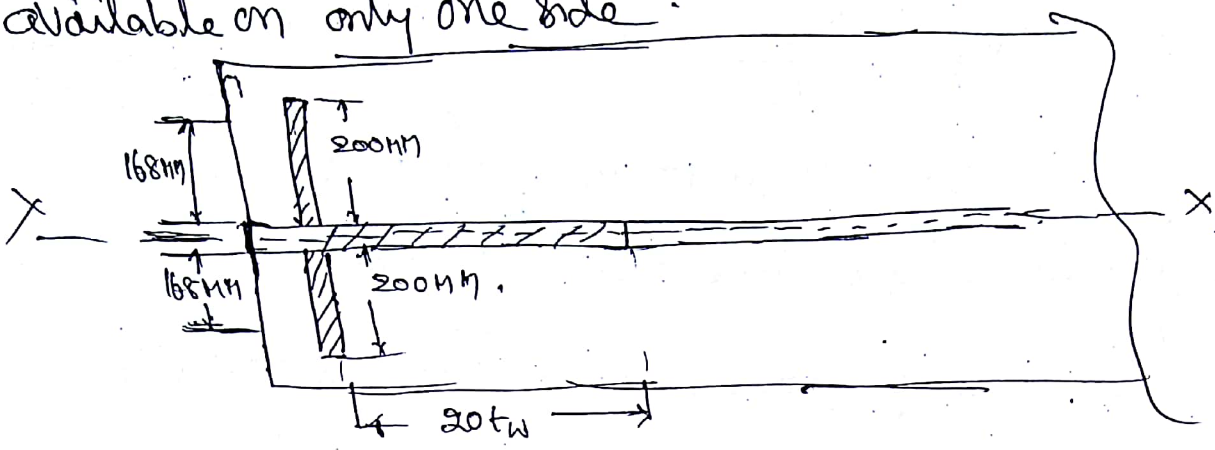
Try a pair of 200×12 mm flats.

Now $14 t_f = 14 \times 12 = 168$ mm

$\therefore$ Core area of stiffener on each side $= 168 \times 12$ mm²

Core area $= 168 \times 12 = 2016$ mm²

Fig shows the core area of stiffener along with effective area of web ($20t_w$) assuming web area is available on only one side.



Area for buckling resistance $= (2\times168 + 20t_w)\times t_w$

$A = (2\times168+20\times12)12 = 6912\ mm^2$

$I_{xx} = \frac{12}{12}\times(168+168+12)^3 + \frac{1}{12}(20t_w)\times t_w^3$

$= \frac{12\times(168+168+12)^3}{12} + \frac{1}{12}(20\times12)\times12^3$

$= 42.179\times10^6\ mm^4$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.175\times10^6}{6912}} = 78.12\ mm$

$\lambda = 0.7\times\frac{d}{r} = 0.7\times\frac{1200}{78.12} = 10.75$

From table 9c in IS 800 (table 6.4 ~~in this~~ IS Code Book)

$f_{cd} = 226.5\ N/mm^2$ (Assumed)

$\therefore$ Buckling resistance $= 226.5\times6912 = 1565.5\times10^3\ N$

$= 1565.3\ kN > 705.6\ kN$.

Hence the stiffners is safe.

Check for bearing Capacity of Stiffeners

$$F_{psd} = \frac{A_q f_{yq}}{0.8\, \gamma_{m0}}$$

Where A_q = Area of stiffeners in contact with flange

$= 2 \times 200 \times 12 = 4800\ mm^2$

$$\therefore F_{psd} = \frac{4800 \times 250}{0.8 \times 1.1} = 1363.6 \times 10^3\ N$$

or $= 1363.6\ KN > 705.6\ KN$.

Hence the stiffeners is adequate.

⑧ Design of Weld Connecting Web ~~to~~ flange

Shear force $= V = 705.6\ KN = 705.6 \times 10^3\ N$

$I_{zz} = 1.21 \times 10^{10}\ mm^4$ (all ready found)

$$\therefore \text{Shear stress} = \frac{V}{b I_z}(a\bar{y})$$

$$\text{Shear force per unit length} = \frac{V}{I_z} a\bar{y} \quad (b = 1\ mm)$$

$$= \frac{705.6 \times 10^3}{1.21 \times 10^{10}}(440 \times 36 \times 618) = 570.71\ N/mm^2$$

The size of ~~...~~ field weld is 's', and it is provided on both sides of web.

Strength of weld per unit length =

$$f_{wd} = 2(0.7 s \times l \times f_{cd}) \quad \left(\because f_{cd} = \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25}\right)$$

$$f_{wd} = 2\left(\left(0.7 \times s \times 1 \times \frac{f_u}{\sqrt{3}} \cdot \frac{1}{1.25}\right)\right)$$

$$= 2\left(0.7 \times s \times 1 \times \frac{410}{\sqrt{3}} \cdot \frac{1}{1.25}\right) = 265.10s\ N/mm.$$

Equating S.F. we get:

$570.71 = 265.10\, s$

$$s = \frac{570.71}{265.10} = 2.153 \approx 3\, mm.$$

The minimum size of weld used = 5mm.
Minimum length of inter~~mittent~~ mediate welds = 40mm.
∴ Hence provide 40mm length with 5mm weld on both sides and then give a gap of 140mm.

Thus the final design

Web = 1200mm x 12mm.
Flange = 440 x 36 mm.
End stiffeners = 200 x 12m

Weld connecting flange and web = 40mm with a gap of 140 mm, size 5mm, on both sides weld connecting stiffener and web; 55mm and gap of 45mm size 5mm on both sides.

Q Design the plate girder using thin web and end stiffener, ~~but~~ and intermediate stiffeners.

Solution:-

(1) Maximum ~~moment~~ Moment & S.F = 4233.6 kN-m.
Maximum shear force = 705.6 kN.

2) Depth of web:-

The stiffeners spacing is 'c' & c/d is between 'd' and '3d' where 'd' is depth of web, the serviceability requirement is $k = \frac{d}{t_w} \leq 200$; taking k = 190 we get economical depth

$$d = \left(\frac{Mk}{f_y}\right)^{1/3} = \left(\frac{4233.6 \times 10^6 \times 190}{250}\right)^{1/3} = 1476 \text{ mm}$$

Use 1500mm wide plates.

∴ Use 1500 mm wide, 8 mm thick plates.

provide stiffener's at every 2 mts intervals.

③ Flange plates :- $(3d \geq c \geq d.)$

Assuming flange alone resists the moment

$$\frac{A_f \times f_y \times d}{1.1} \geq M$$

$$\frac{A_f \times 250 \times 1500}{1.1} \geq 4233.6 \times 10^6$$

$$A_f = 12418 \text{ mm}^2$$

to keep flange in plastic class:

when $f_y = 250$

$\epsilon = 1$

∴ $b < 8.4 t_f$ (ie) ~~$b \leq 8.4 t_f$~~ ϵ $b_f/2 \leq 8.4 t_f$

taking $b_f = 16.8\, t_f$

~~we get~~. $A_f = 12418 = 16.8 t_f \times t_f$

i.e $t_f \geq 27.1$

provide 32 mm plates

$$\therefore b_f = \frac{12418}{32} = 388 \text{ mm}.$$

Use 400 mm wide 32 mm thick plates.

Check for shear bulking

using simple post critical method.

$c/d \geq 1.0$ $(3d \geq c \geq d)$; $c = 2000$!

$$K_v = 5.35 + \frac{4}{(c/d)^2} = 5.35 + \frac{4}{\left[\frac{2000}{1500}\right]^2} = 7.6$$

$$\tau_{cr} = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

$$= \frac{7.6 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1500}{32}\right)^2} = 39.08$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3} \times 39.08}} = 1.92$$

$\cancel{\lambda_w \geq 1.2}$ $\lambda_w \geq 1.2$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.92^2} = 39.15\ N/mm^2$$

$$\therefore V_n = V_{cr} = A_v \tau_b$$

$$= 1500 \times 8 \times 39.15$$

$$= 469.800 \times 10^3 N = 469.8\ KN < 705.6N$$

Hence Intermediate stiffeners are to be used to improve buckling strength to be slender web ~~and shear capacity of end panel should be checked.~~

Design of Intermediate stiffeners :-

As the shear force goes on reducing towards mid span, the first stiffener from end is critical. Since first intermediate stiffeners is at $c = 2m$ from end

Shear on this stiffener is $= R - 2w$

$$= 705.6 - 2 \times 58.8 = 588\,kN.$$

In this case $= c = 2$ Mts (or) 2000 MM.

$d = 1.5$ Mts (or) 1500 MM.

$$\therefore c/d = \frac{2000}{1500} = 1.33 < \sqrt{2} < 1.41$$

Hence Minimum $I_s = \dfrac{1.5 \cdot d^3 t_w^3}{c^2} = \dfrac{1.5 \times 1500^3 \times 8^3}{2000^2}$

$I_s = \underline{648000 \text{ mm}^4}$

Try intermediate stiffeners of size 120x10 mm on each side.
This is not violating outstand clause ($< 20 t_q$)

$I_s = \frac{1}{12} \times 10 \, (120+8+120)^3 - \frac{1}{12} \times \cancel{10} \times 8^3$

$= 12.71 \times 10^6 \text{ mm}^4 > I_s$ required. Hence adequate.

Check for buckling

$\frac{1448}{11.218} = 120.$

Shear buckling resistance of the Web alone.

$V_{cr} = 469.800 \times 10^3 \text{ N} = \underline{469.8 \text{ KN}}$

$\therefore$ Shear strength of stiffeners alone required $= \dfrac{V - V_{cr}}{\gamma_{m0}}$

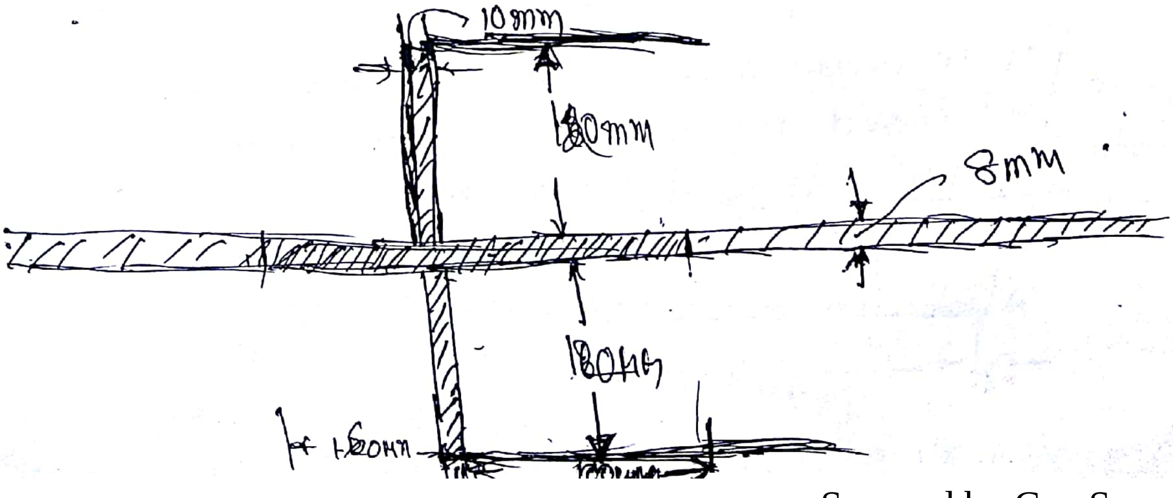
$= \dfrac{588 - 469.8}{1.1} = \underline{107.45 \text{ KN}}$

Buckling resistance of Intermediate Stiffeners (clause 8.7.15)

Considering $20 \times t_w = 20 \times 8 = 160$ mm ~~width~~ width of Web on both side along with stiffeners

$I_x = I_s + 2 \times \frac{1}{12} \times 160 \times 8^2$

$= 12.71 \times 10^6 + 13653 = 12.73 \times 10^6 \text{ mm}^4$



Area = $2 \times 120 \times 10 + 2 \times 160 \times 8 = 4960\ mm^2$

$$\therefore r = \sqrt{\frac{12.73 \times 10^6}{4960}} = 50.66$$

$KL = 0.7 \times 1500 = 1050\ mm$

$$\lambda = \frac{KL}{r} = \frac{0.7 \times 1500}{50.66} = 20.73$$

from table 9c in IS 800

$$f_{cd} = 224\ N/mm^2$$

$\therefore$ Buckling resistance $= \overset{f_{cd} \times A}{224 \times 4960} = 1111 \times 10^3\ N$ (or) 1111 KN

Shear strength required = 105.45 KN $<$ Buckling strength 1111 KN

This is more than required resistance ~~of~~ Hence the stiffeners is safe

Gantry Girder

Design a simply supported gantry girder to carry one electric overhead travelling crane given.

Span of granty girder = 6.5 mts ; span of crane girder = 16 mts ; Crane Capacity = 250 KN = Wt of lifted. Self Wt of crane girder excluding trolly = 200 KN ; Self Wt of trolly = 50 KN

Minimum hook approach = 1.00 mts.

Distance between wheels = 3.50 mts.

Self Weight of rails = 0.3 KN/m.

Solution:-

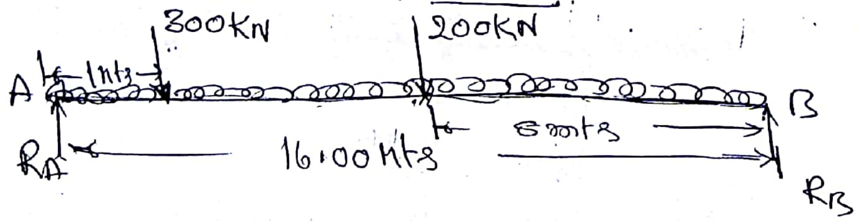
load for maximum moment = Weight of trally + lifted load

= 50 + 250 = 300 KN

Self wt of crane girder = 200 KN.

For maximum reaction on granity girder, the moving load should be as close to granty as possible.

fig shows the load position.



Taking moment about B.

$R_A \times 16 = 15 \times 300 + 200 \times 8$

$\cancel{R_A} = \frac{\cancel{4500} + 1600}{\cancel{16}}$ $R_A = \frac{4500 + 1600}{16}$

$R_A = \frac{6100}{16} = 381.25\ KN \approx 382\ KN$

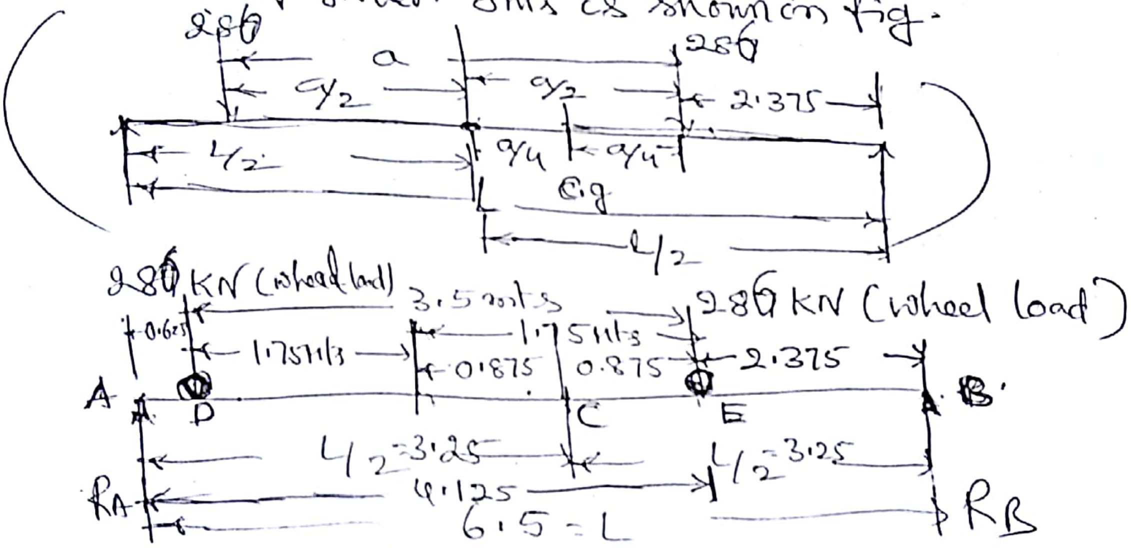
This load is transfered to girder through wheels, the wheel base being 3.5 mts. =

Load on grantry girder from each wheel $= \frac{381.25}{2} = 190.60\ KN$

Factored wheel load $= 1.5 \times 190 = 285.5 \approx 286\ KN$.

Span of granty girder = 6.5 mts.

Maximum moment due to moving loads occur under wheel when the C.G of wheel load and the wheel are equi distance from the centre of girder. This is shown in fig.



Taking moments about A

$R_B \times 6.5 = 286 \times 0.625 + 286 \times 4.125$

$R_B = \frac{286 \times 0.625 + 286 \times 4.125}{6.5} = 209\ KN$

Maximum moment at E point $= M_E = 209 \times 2.375 = 496.375\ KN\text{-}m$

Moment due to Impact = 25% Maxi moment at E due to static wheel load

$= 0.25 \times 496.375 = 124.10\ KN\text{-}m$

Assume the self wt of girder = 2 KN/M (Assume).

$\therefore$ Dead load due to self weight + self wt of rails $= 2 + 0.3 = 2.30\ KN$

Factored D.load $= 1.5 \times 2.30 = 3.45\ KN/M$

Moment due to D.L of G.G $= \underbrace{3.45 \times 6.5 \times 6.5}_{\text{Total load}} = 18.22\ KN\text{-}m$

Scanned by CamScanner

Total factored moment due to Vertical loads $\Big\} = M_2 = 496.38 + 124.09 + 18.22$

$M_2 = 638.689 \simeq \underline{639 \, KN-m}$

Maxi moment due to horizontal force.

a) Horizontal force transverse to rails = 10% (wt of trolly + load lifted)

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

Assuming Double flamed wheels, this is distributed over 4 wheels

$\therefore$ Horizontal force on each wheel $= \frac{30}{4} = 7.5 \, KN$

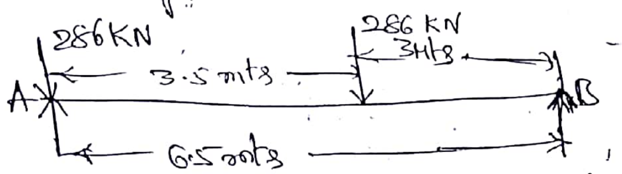
Factored horizontal force on each wheel $= \cancel{7.5 \, KN} \; 1.5 \times 7.5$

$= 11.25 \, KN$

$M_y = \frac{11.25 \, KN}{286 \, KN} \times 496.375 = 19.525 \, KN\text{-}m$

Shear force:- For Maximum shear force on the girder, the trailing wheel should be just on the girder.

taking moments about 'B'



The vertical shear due to wheel loads $= \frac{286 \times 6.5}{6.5} + \frac{286 \times 3.0}{6.5}$

$286 + \frac{286 \times 3}{6.5} = 418 \, KN$

Vertical shear due to $\cancel{\text{self weight}}$ Impact $= 25\%$ Vertical Shear

$= 0.25 \times 418 = 104.5 \, KN$

Vertical shear due to self wt Gantry girder Factored $\Big\} = 3.45 \times \frac{6.5}{2} = \cancel{27.6 \, KN}$ 11.21 KN

0.4467

$\therefore$ Total vertical shear $= 418 + 104.5 + 11.21 = 549.6 \, KN$

By proportioning lateral shear due to Surge $= \frac{11.21}{286} \times 418 = 16.38$

$= 16.38 \, KN$

Preliminary section

Depth of web $\frac{l}{12} = \frac{6500}{12} = 541.7\,mm$..

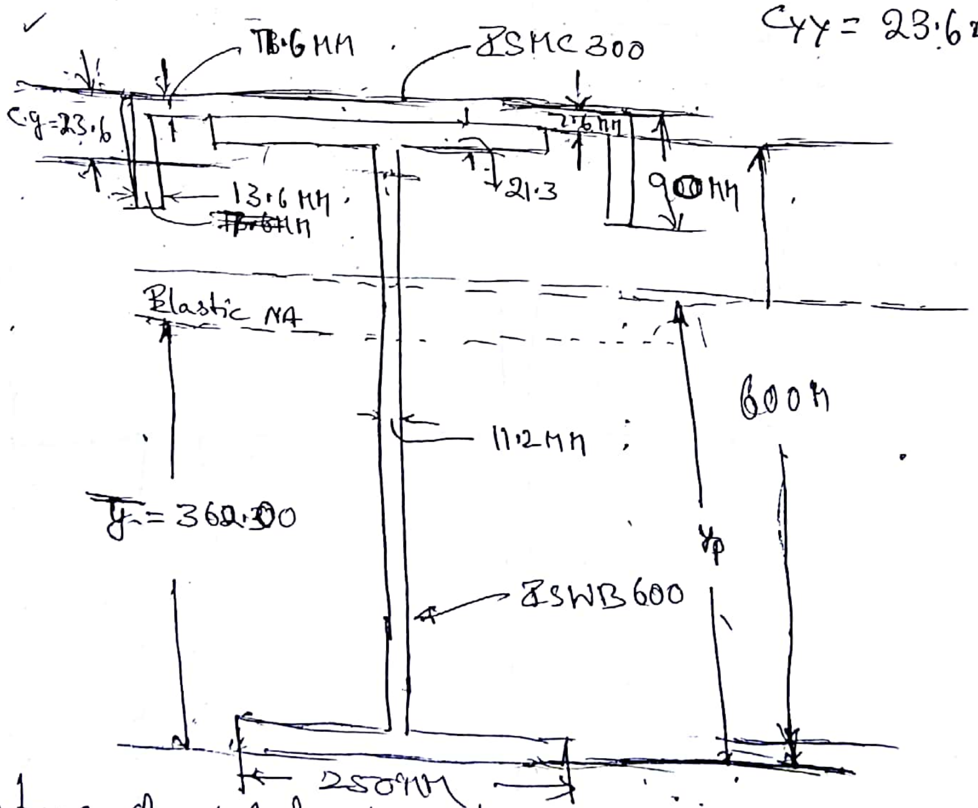
$\frac{l}{25} = \frac{6500}{25} = 260.00\,mm$.

Let us try ISWB 600 with ~~ISMC~~ ISMC 300 on compression flange as shown in fig

Properties of ISWB 600 @ 1.312 KN/m — Properties of ISMC 300.

$A = 17038\ mm^2$
$b = 250\ mm$
$t_f = 21.3\ mm$
$t_w = 11.2\ mm$.
$I_{zz} = 106198.5 \times 10^4\ mm^4$
$I_{yy} = 4702.50 \times 10^4\ mm^4$.

$A = 4564\ mm^4$
$b_f = 90\ mm$
$t_f = 13.6\ mm$
$t_w = 7.60\ mm$
$I_{zz} = 6362.60 \times 10^4\ m$
$I_{yy} = 310.8 \times 10^4\ mm^4$
$C_{yy} = 23.6\ ~~mts~~\ mm$



607.6
23.6
584.0

3.53
7.6
2

14.7
3.53
18.23

Let distance of N-A from the extreme fibre of tension flange be $\bar{y}$

Then, $\bar{y} = \dfrac{17038 \times 300 + 4564(600 + 7.6 - 23.6)}{17038 + 4564} = 360.00\,m$

$I_{zz} = 10619 8.5 \times 10^4 + 17038(360 - 300)^2 + (I_{yy})_{ISMC\ section}$.

$$I_{zz} = 106198.5\times10^4 + 17038(360-300)^2 + 3101.8\times10^4 + 4564(584-360)^2$$

$$I_{zz} = 1127.452\times10^6\ mm^4$$

$$Z_e = \frac{I_{zz}}{Y_{max}} = \frac{1127.452\times10^6}{360.00} = \underline{313.04\times10^4\ mm^3}$$

~~Zey for Compression flange~~ = For Com flange about y-y axis

$$I_{yy} = \frac{1}{12}\times21.3\times250^3 + 6362.6\times10^4$$

$$I_{yy} = 9136.04\times10^4\ mm^4$$

$$Z_{ey}\text{ for Compression flange} = \frac{9136.04\times10^4}{150} = \frac{I_{yy}}{y}$$

$$\therefore Z_{ey} = \underline{609.069}\times10^3\ mm^3$$

Plastic Modulus of Section:-

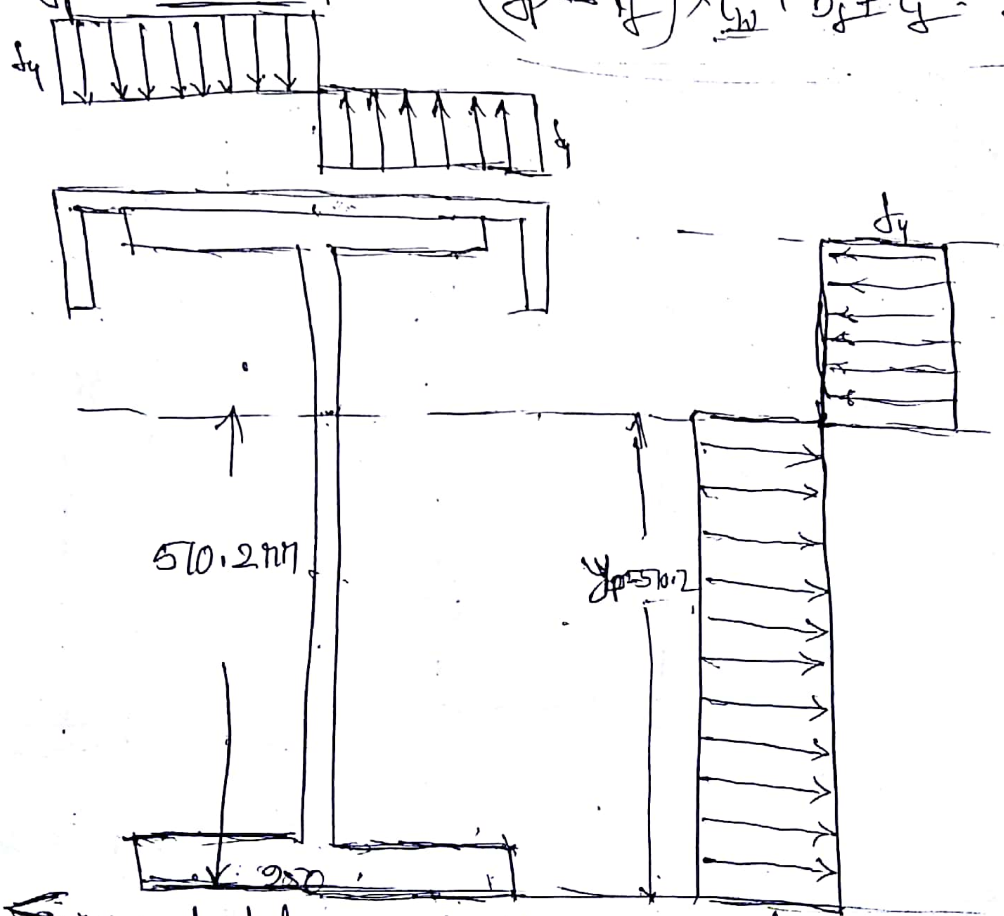
$$\text{Total area of the Section} = 17038 + 4564 = 21602\ mm^2$$

Let plastic N.A be at a distance Y_p from ~~tension~~ tension flange - Then

$$(Y_p - 21.3)\times11.2 + 250\times21.3 = \frac{A}{2} = \frac{21602}{2}$$

$$\therefore Y_p = \underline{510.20\ mm}$$

$$\left(Y_p - t_f\right)\times t_w + b_f\times t_f = \frac{A}{2}$$



$\therefore M_p = \cancel{\Sigma}$ Moment of forces at yield about plastic N.A

$$M_p = 21.3\times250\left(510.2 - \frac{21.3}{2}\right)f_y + \frac{(510.2-21.3)^2}{2}\times11.2\,f_y$$

$$+\frac{(600-21.3-510.2)^2}{2}\times 11.2 f_y + 21.3\times 250\left(600-\frac{21.3}{2}-510.2\right).$$
$$+4564\,(600+13.6-23.6-510.2)\,f_y$$
$$= 46864 50 f_y$$

For top flange $\therefore Z_p = \frac{M_p}{f_y} = \frac{4686450 f_y}{f_y} = 4686450\ mm^3$

$$Z_{py} = \frac{M_p}{f_y} = \frac{1}{4}\times 21.3\times 250^2 + \frac{1}{4}(300-2\times 13.6)^2\times 7.6 + 2\times 90\times 13.6\left[150-\frac{13.6}{2}\right]$$
$$= 824.764\times 10^3\ mm^3$$

$$\boxed{\frac{M_z}{M_{dz}}+\frac{M_y}{M_{dy}}\le 1}$$

Check for Moment Capacity :-

$\frac{b_f}{2t_f}$ of flange of ISWB 600 $= \frac{250-11.2}{2\times 21.3} = 5.6 < 8.4$,

$\frac{d_w}{t_w}$ of web of ISWB 600 $= \frac{600-2\times 21.3}{11.2} = 49.76 < 84$

and $\frac{b}{t_f}$ of flange of channel $= \frac{90-7.6}{13.6} = 6.06 < 8.4$.

Hence it is a plastic section.

Local moment capacity for bending in vertical plane

$$M_{dz} = \frac{f_y Z_p}{1.1} = \frac{250}{1.1}\times 4686450 = 1065.1\times 10^6\ N\text{-}mm$$

$$M_{dz} = \frac{1.2 Z_e f_y}{1.1} = \frac{1.2\times 313.18\times 10^4\times 250}{1.1} = 854.127\times 10^6\ N\text{-}mm$$

whichever is less.

$$\therefore M_{dz} = 854.127\times 10^6\ N\text{-}mm$$
(or) $854.12\ kN\text{-}m$

For top flange :- which ever is less

$$M_{dy} = \frac{f_y Z_{py}}{1.1} = \frac{250}{1.1}\times 824.764\times 10^3$$
$$= 187.446\times 10^6\ N\text{-}mm\ (or)\ 187.446\ kN\text{-}m$$

$$M_{dy} = \frac{1.2 Z_e f_y}{1.1} = \frac{1.2\times 609.069\times 10^3\times 250}{1.1} = 166.11\times 10^6\ N\text{-}mm$$
$$= 166.11\ kN\text{-}m$$

$\therefore$ For top flange $M_{dy} = 166.11\ kN\text{-}m$

Check for Combined local capacity.

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{638.689}{854.127} + \frac{19.525}{166.11} = 0.865 < 1$$

Hence it is adequate.

Check for shear:-

$V_z = 533.71\,kN$.

$$\text{Shear capacity} = \frac{A_v f_{yw}}{\sqrt{3} \times 1.1} = \frac{\overset{A_v}{(600 \times 11.6)} \times 250}{\sqrt{3} \times 1.1}$$

$= 913 \times 10^3\,N = 913\,kN > 549.9\,kN$

Hence it is o.k.

Weld Design

a) provide 6mm intermittent weld on both sides.

b) provide 6mm weld in the Gantry Girder.

$(250 \times 250) \cancel{\times 2}$

$\cancel{4}\ 2$