* Gantry Girder :– overhead travelling Cranes are used in Industrial building to lift & transport heavy jobs machines and so on from one place to another. The Crane may be manually (hand) operated overhead travelling (motor Hot) Crane or an electrically operated over head travelling (EOT) Crane..

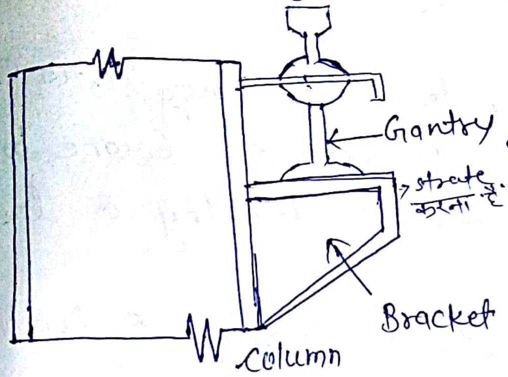


fig :– Elevation

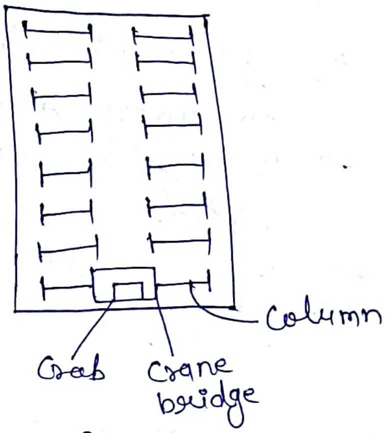
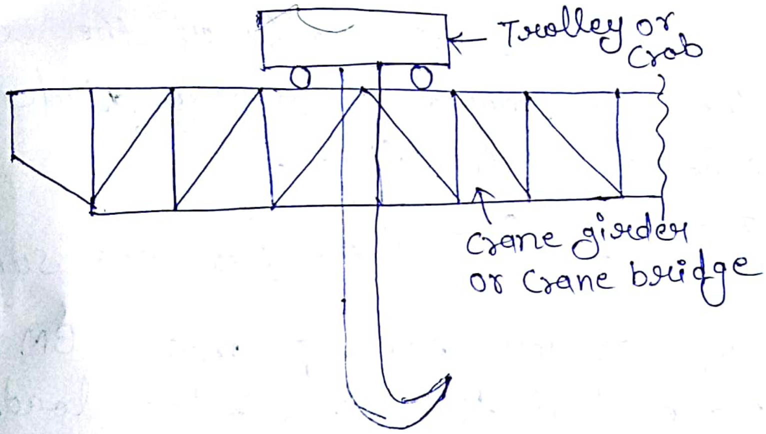


fig :– plan



→ Satisfying eqn of G.G. (Interaction formula)

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

where

M_x = Bending moment in vertical plane

M_y = BM in horizontal plane

M_{dx} = Design BM in vertical plan

M_{dy} = Design BM in horizontal plane

* LOADS :-

1. vertical loads :- wt. of Crane Girder or Crane bridge
→ wt. of trolley or Crab
→ wt. of Gantry girder (self wt.)

2. Impact allowance :- for EOT = 25 %.
for HOT = 10%.

3. Horizontal loads or lateral loads :- They are caused on gantry girder by movement of trolley on Crane bridge
→ for EOT its among is 10% of wt. of trolley & load lifted.
→ for HOT its among is 5% of wt. of trolley & load lifted.

4. Longitudnal loads :- Act longitudnally at the level of rail ~~etc~~. sectⁿ they are caused on gantry girder by movement of crane girder on gantry girder.

* Absolute max. BM Theorm :- (As per TOS sub)

According to theorm of TOS for absolute max. BM. They should be so place that their C.G. & the loads under which Mmax. is according should be Equidistance from Centre of span.

Condition for applying this theorm.

→ if $0.586L < b$
then one load at mid span gives absolute max. BM
b = distance b/w 2 points of loads (wheel)
L = Span (G.G.)

→ if $0.586L > b$
apply TOS theorm for absolute BM theorm.

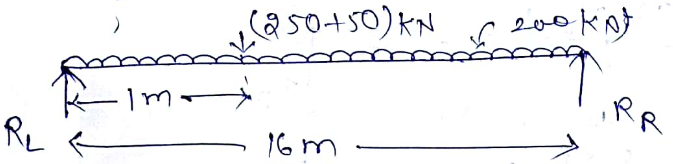
Ques. Design a gantry girder simply supported & carrying one EOT Crane from following data.

→ Capacity of Crane = 250 KN
→ Selfwt. of Crane bridge excluding trolley = 200 KN
→ wt. of trolley or crab = 50 KN
→ span of Crane bridge = 16 m
→ span of gantry girder = 6.5 m
→ wheel base = 3.5 m
→ min. approach of hook = 1 m
→ Self wt. of rail = 0.3 KN/m

Solutⁿ **BM & SF.**

B.M. in vertical plane :- A total wt. of Crane capacity trolley (i.e. 250 KN + 50 KN) is ~~to~~ travelling on Crane bridge. It positⁿ should be nearest to any of gentry girder (for max. BM)

∴ Distance of min. approch of hook i.e 1m)



to find R_L →

taking moment about point R

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

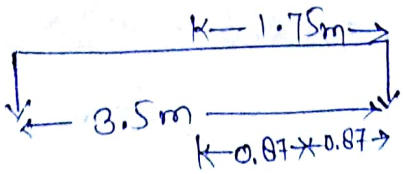
$$\boxed{R_L = 381.25\ KN}$$

∵ There is 2 wheels so ~~that~~ this load is devided into 2 equal wheel loads.

$$\text{i.e} = \frac{381.25}{2} = 190.625\ KN$$

→ factored ~~load~~ value of one wheel load $= 190.625 \times 1.5$
$= 285.937\ KN$
$= 286\ KN$

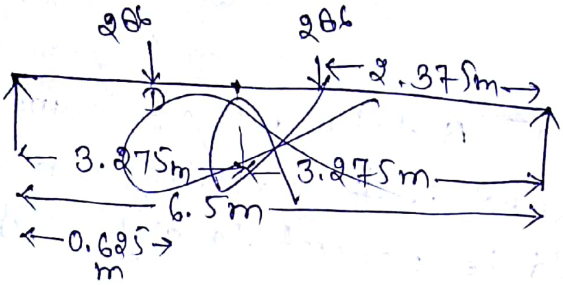
→ Set of load is

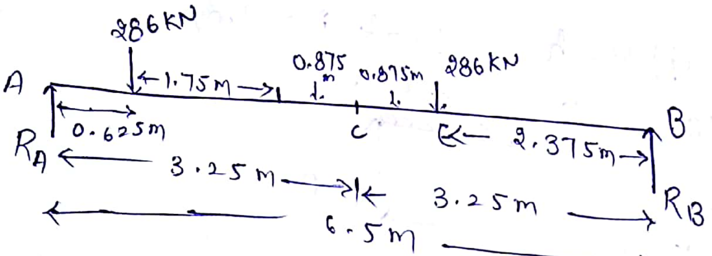


→ this set of loads is moving on gantry girder of span 6.5 m. There posit^n should be such as to cause absolute max. BM in the gantry girder.

check $0.586L = 0.586 \times 6.5 = 3.809 > b(3.5)$

Therefore we have to apply TOS thearm Mmax according to theorm in TOS for Mmax max. They should be so placed their C.G. & the load under which Mmax. max is occurring should equidistance from the Centre of span ℄ (G.G.)





To find R_B

taking moment about

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

$R_B = 209$ kN

So $M_{℄} = R_B \times 2.375$

$= 209 \times 2.375 = 496.38$ kNm

Assume Self wt. of gantry girder $= 2$ kN/m

Self wt. of rail $= 0.3$ kN/m

total Self wt. $= 2.3$ kN/m

factored Self wt. $= 1.5 \times 2.3 = 3.45$ kN/m

therefore max. Bending moment due to self wt.

$M = \frac{wl^2}{8}$

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$M = \frac{2.3 \times 6.5^2}{8} = 18.22$ kNm

Due to Impact we take 25% allowance for EOT Crane.

→ Total BM including impact (M_x) $= 496.38 \times 1.25 + 18.22$

$M_x = 638.69$ kNm

→ BM. in horizontal plane (M_y) :– value of lateral load is taken 10% of load lifted & trolley for EOT Crane.

$M_y = \frac{10}{100}(250+50)$

$= 30$ kN

There are 4 wheels in trolley.

lateral load per wheel $= \frac{30}{4} = 7.5$ kN

So factored value of one wheel lateral load.

$= 7.5 \times 1.5$

$= 11.25$ kN

their posit^n on gantry girder max. BM in horizontal plane will be same as for vertical load.

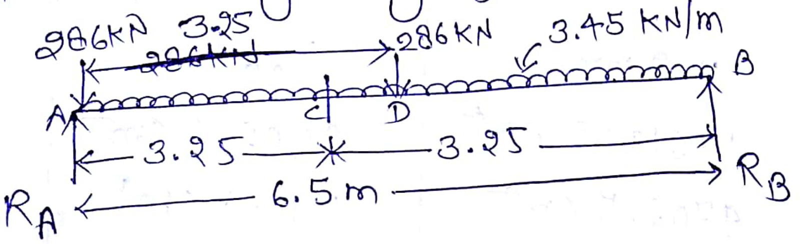
It's found by perportion.

$$\frac{M_y}{M_{x(E)}} = \frac{f_y}{f_x}$$

$$M_y = \frac{11.25}{286} \times 496.38$$

$$M_y = 19.525 \text{ kNm}$$

→ Max. SF in gentry girder



for max. SF, find R_A

Taking moment about point B.

$$R_A \times 6.5 = 286 \times 6.5 + 286 \times 3 + 3.45 \times 6.5 \times \frac{6.5}{2}$$

$$R_A = 418 + 11.2$$

max. SF including Impact, for E.O.T Crane 25%. allow

max. SF (V) = 418 × 1.25 + 11.2

V = 533.71 kN

NOTES :- Trial Section guidelines

i) Depth = $\frac{L}{12}$, width = $\frac{L}{30}$

ii) Section choice is as per Crane Capacity.

→ I-Section vary from ISWB 500 + ISWB 600

→ C - Section vary from ISMC 300 to ISMC400

→ This pair of I & C Section is suitable for Crane Capacity.

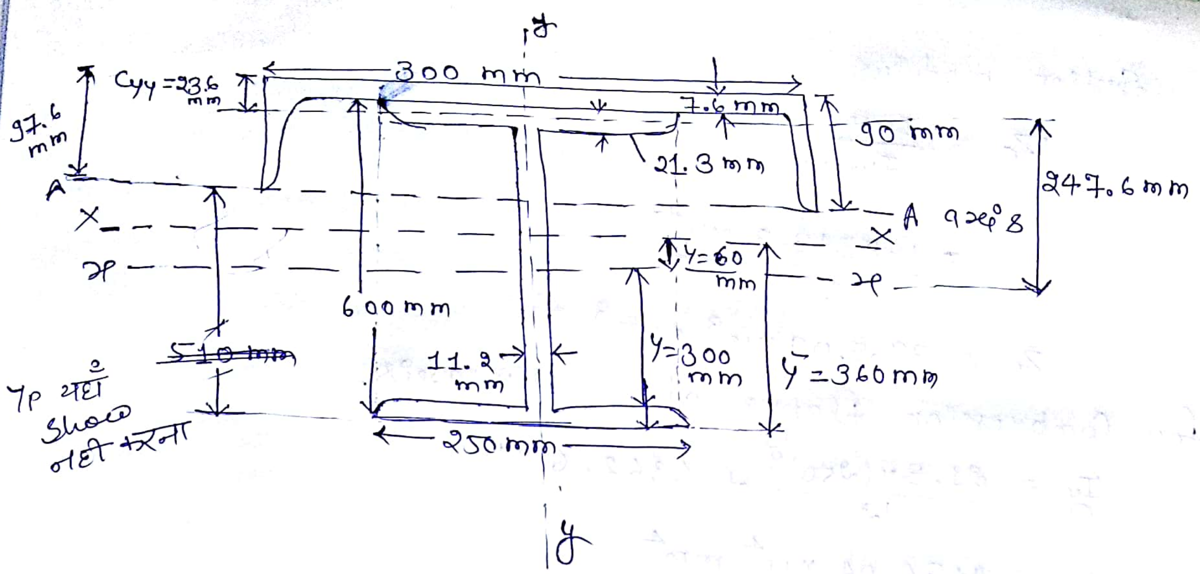
→ Trial Section.

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

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

Let's try ISWB 600 @ 1.33 kN/m & ISMC 300 as shown below.

	ISWB 600	ISMC300
A	17038 mm^2	4564 mm^2
b	250 mm	90 mm
t_f	21.3 mm	13.6 mm
t_w	11.2 mm	7.6 mm
I_{xx}	106198.5 cm^4	6362.6 cm^4
I_{yy}	4702.5 cm^4	310.8 cm^4
C_{yy}	—	23.6 mm



To get CG. from bottom

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2}$$

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

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

→ To find I'_{xx}

$$I'_{xx} = \underbrace{106198.5 \times 10^4 + 17038(360-300)^2}_{I\text{-sect}^n} + \underbrace{310.8 \times 10^4 + 4564\,(247.6 - 23.6)^2}_{C\text{-sect}^n}$$

$$I'_{xx} = 135543.31 \times 10^4 \text{ cm}$$

→ To find I'_{yy}

$$I'_{yy} = 4702.5 \times 10^4 + 6362.6 \times 10^4$$
$$= 11065.1 \times 10^4 \text{ mm}$$

Section modulus

$$Z_e = \frac{I_{xx}}{I_{ymax}}$$

$$= \frac{135543.31 \times 10^4}{360}$$

$$Z_e = 3765.09 \times 10^3 \ mm^3$$

for Compression flange about y-y axis.

$$I_y = \frac{21.3 \times (250)^3}{12} + 6362.6 \times 10^4$$

$$I_y = 9136.04 \times 10^4 \ mm^4$$

$$Z_{ey} = \frac{I_y}{y_{max}(Comp.)} = \frac{9136.04 \times 10^4}{150}$$

$$Z_{ey} = 609.07 \times 10^3 \ mm^3$$

23/1/19

* To calculate M_{dx} →

M_{dx} = Design Bm. in vertical plane

Here Gantry Girder is laterally unsupported

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

Let E.A. axis lies at a distance y_p from bottom

for computing Z_p.

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

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

$= A_c y_1 + A_t y_2$

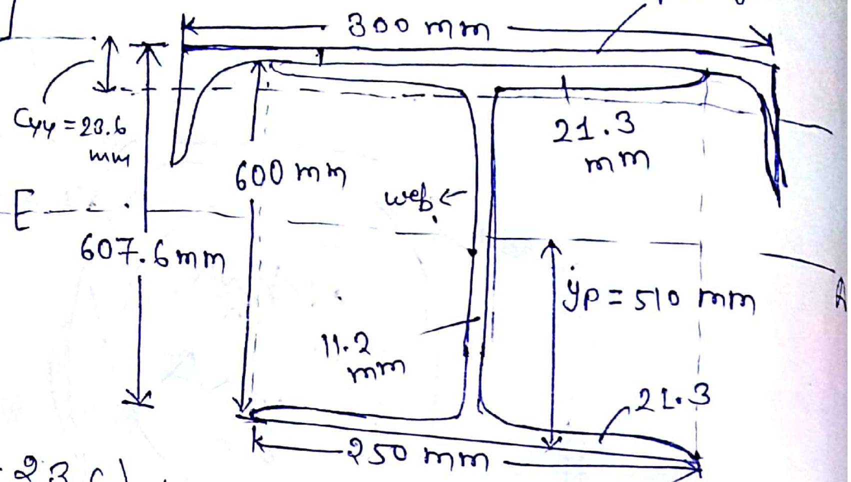
$Z_p = a_1 y_1 + a_2 y_2 + a_3 y_3 + \ldots$

$\boxed{Z_p = \Sigma a y}$

$= 250 \times 21.3 + (Y_p - 21.3) \times 11.2 = \frac{A}{2} = \frac{17038 + 4564}{2}$

$= 5325 + 11.2 Y_p - 238.56 = 10801$

$\boxed{Y_p = 510 \text{ mm}}$



~~$Z_p = \Sigma a y$~~

~~$= 4564 \times (607.6 - 510) - 23.6)$~~

~~$= 4564 \times 97.6$~~

~~$= 445.45$~~

$Z_p = \Sigma a y = 4564 \times (97.6 - 23.6) + 21.3 \times 250 \times (97.6 - 7.6 - \frac{21.3}{2})$

$+ (97.6 - 7.6 - \frac{21.3}{2}) \times 11.2 \times \frac{(97.6 - 7.6 - 21.3)}{2}$

$+ (510 - 21.3) \times 11.2 \times \frac{(510 - 21.3)}{2}$

$+ (250 \times 21.3) \times (510 - \frac{21.3}{2})$

$= 337736 + 422538.75 + 30527.532 + 1337435.64$

$+ 2659038.75$

$= \cancel{168}$

$\boxed{Z_p = 4383.17 \times 10^3 \text{ mm}^3}$

* To Calculate f_{cd} :—

$\frac{h}{t_f} = \frac{600 + 7.6}{21.3 + 7.6} = 21.02 \simeq 21$

$\frac{KL}{r} =$

Total area = 17038 + 4564 = 21602 mm^2

radius of gyration $r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{11065.1 \times 10^4}{21602}}$

$r_{yy} = 71.57$ mm

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

finding $f_{cr,b}$

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

h/t_f

	$\frac{KL}{r}$	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

$$f_{cr,b} = \frac{(25-21)(100-90.82)}{5 \times 10} \times (380.4) + \frac{(21-20).(100-90.82)}{50} \times 344.2$$

$$+ \frac{(25-21) \times (90.82-90)}{50} \times 325.8 + \frac{(21-20).(90.82-90)}{50} \times 291.4$$

$= 279.37 + 63.196 + 21.38 + 4.78$

$= 368.73$ N/mm^2 (Critical stress)

from pn. 55, table 13(a) of IS 800:20
single Interpolation

	$f_{cr,b}$	
&	400	184.1
	368.73	f_{cd}
	350	172.7

$$f_{cd} = 184.1 + \frac{(368.71-400)(172.7-184.1)}{(350-400)}$$

$= 176.96$ N/mm^2

24/1/19

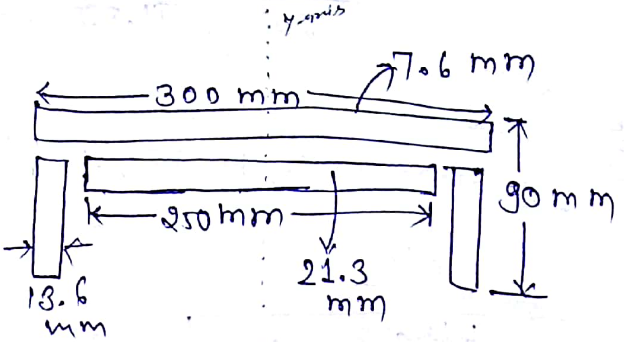
$\Rightarrow M_{dx} = Z_p \times f_{cd}$

$= 4763.17 \times 10^3 \times 176.96$

$= 846.44$ KNm $> M_x$ (638.69 KNm) ok.

$\Rightarrow$ To find Mdy $\rightarrow$

Calculating Zpy



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

$= \frac{1}{4} \times 21.3 \times 250^2$

$+$

$\frac{1}{4} \times 7.6 \times 300^2 + 2 \times 90 \times 13.6$

$Z_{py} = \frac{1}{4} \times 21.3 \times 250^2 + \frac{1}{4} \times 7.6 \times (300 - 2 \times 13.6)^2 + 2 \times 90 \times 13.6 \times \left(150 - \frac{13.6}{2}\right)^2$

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

compression वाला

$\Rightarrow M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}} \ngtr \frac{Z_e f_y 1.2}{\gamma_{mo}}$ {from pn - 70 IS 800:2007}

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

$= 187445.45 \ngtr 16611 \times 10^4$

$= 187.44$ KNm $\ngtr 166.11$ KNm

Therefore $\boxed{M_{dy} = 166.11 \ KNm}$ ~~and it's~~

$M_{dy} > M_y$ (19.525 KNm) ok.

So Interaction formula as per Code

$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} = \frac{638.69}{846.44} + \frac{19.525}{166.11}$

$= 0.873 < 1$ hence ok.

Design Safe

* Check for shear :- max. SF (already Calculated)

$= 533.71$ KN

~~max SF (already Calculated)~~

Design Shear force {Pn-59, IS 800 : 2007}

$$V_d = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m0}} = \frac{h . t_w . f_y}{\sqrt{3}\,\gamma_{m0}} = \frac{600 \times 11.2 \times 15}{\sqrt{3} \times 1.1}$$

$= 881.77$ KN. $> V$

Hence Safe in Shear.

2014

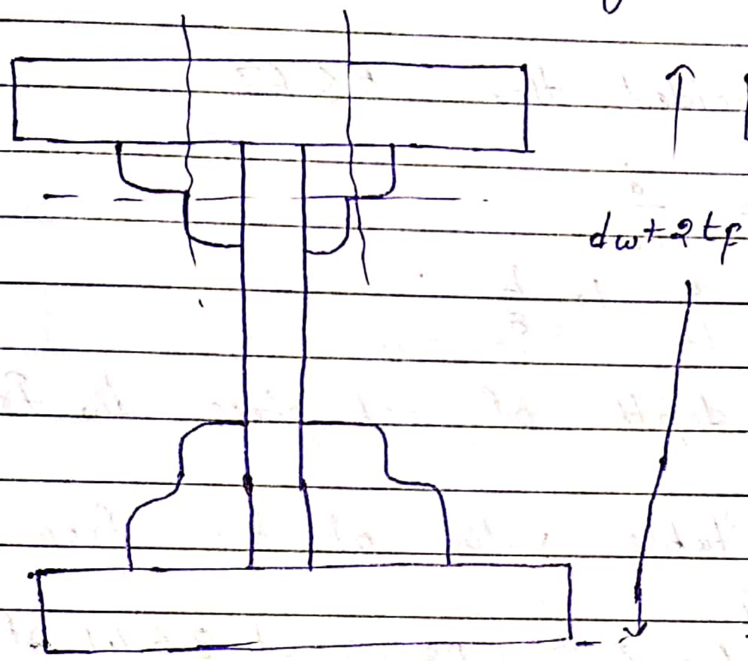
Ques. 2 Design a gantry girder for an Industrial building to Carry an EOT Crane from following data.

→ Crane Capacity 150 KN
→ wt. of Crane (bridge), Excluding trolley = 100 KN
→ Wt. of trolley = 40 KN
→ span of Crane (Bridge) = 12 m
→ span of gantry girder = 7m
→ min. approach of hook = 1m
→ Wheel base = 3m

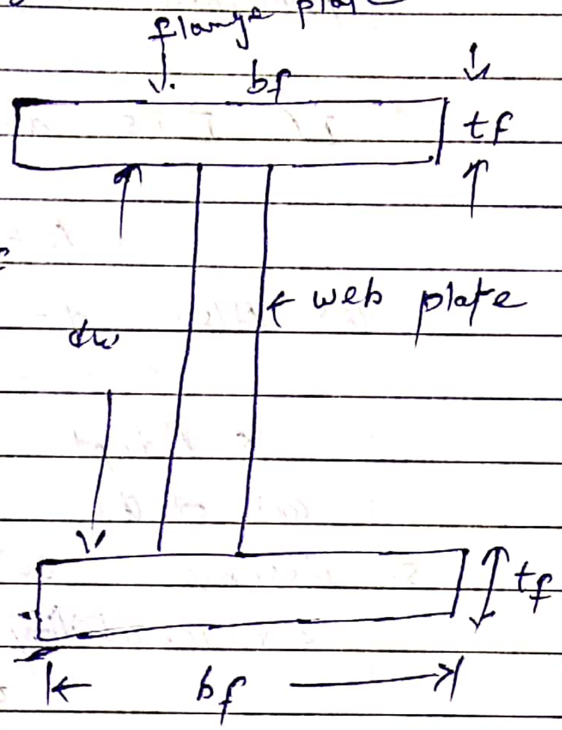
Unit - 2

Plate Girder :->

It is a beam consisting of plates when very heavy loads acts and span is very large, then max. B.M. & S.F. very very large. Rolled sections even with plates will be insufficient, so we use plate girders.



Rolled | Bolted P.G.



Welded P.G.

Steps to design a plate Girder :-

Given → span & loading

1. Convert to factored load.
2. Assume self weight $w' = \frac{W'}{200}$ KN/m

W' = Total factored load on P.G. in KN

3. find factored value of max. B.M. (M) and max. S.F. (V)

4. (a) Approximate depth of P.G.

$$d = \sqrt[3]{\frac{M.K}{f_y}}$$

* if ITs (Intermediate transverse stiffness) provided then $200 \geq K > 67$

* If ITs avoided then $K \leq 67$

$$k = \frac{d}{t_w}$$

(b) Also $d = \frac{L}{12}$ to $\frac{L}{8}$

* Adopt depth of web plate d_w from (a) & (b)

5. Assume suitable value of t_w from serviceability

$t_w > \frac{d_w}{200}$ (Pg. 63 Cl. 8.6.1.1 of IS:800-2007)

6. b_f is usually taken as approximately $0.3\, d_w$

7. Calculate value of flange area — approximate area of flange —

$$A_{f\,approximate} = \frac{M}{d_w\left(f_y/\gamma_{mo}\right)}$$

Also

$$A_f = b_f \times t_f$$

$$t_f = A_f/b_f$$

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8 Design B.M. (M_d) → section classification from table ② p.n. 18 of IS: 800-2007

$b/t_f < 8.3$ plastic, compact

$b = \frac{b_f - t_w}{2}$

→ for plastic section

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

$$\boxed{Z_p = b_f \cdot t_f (d_w + t_f)}$$

→ for semi-compact section

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

$$\boxed{Z_e = \frac{I_{xx}}{D/2}}$$ D = overall depth

if $M_d > M$ then design is safe in beam.

9. Design shear force

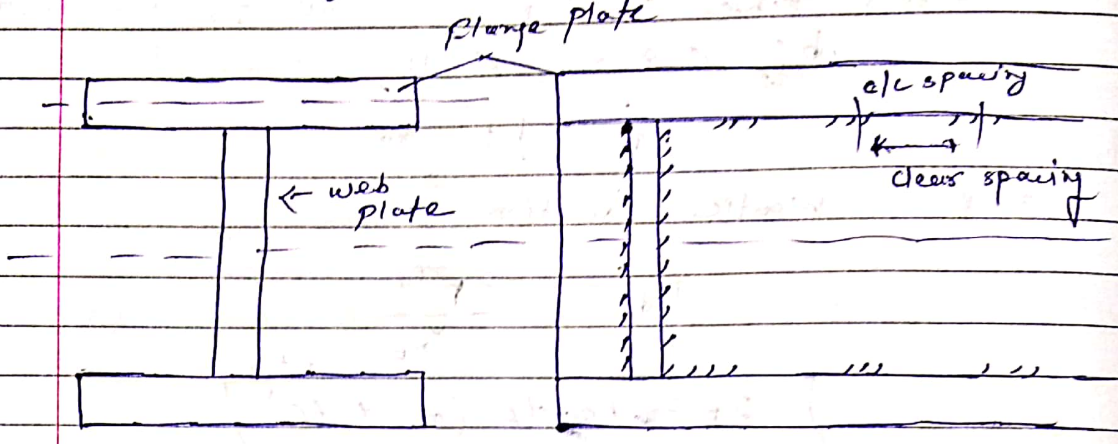
(i) Plastic shear Resistance - (p.n 59)

$$V_d = \frac{A_v f_y}{\sqrt{3}\, \gamma_{mo}}$$, A_v = Area of web = $d_w \cdot t_w$

if $V_d > V$ ok

(ii) Buckling Shear Resistance p.n. 60.
(If ITS provided)

* Weld Design in plate Girder :-



→ The continuous fillet weld be economics therefore we use intermittent weld.

→ As per code (Cl. 10.5.5.1, p.n. 79) length of each intermittent weld equal to 4 S (S → thickness of thinner part) or 40 mm, whichever is more.

→ Clear spacing b/w two adjacent intermittent weld

- should not greater 12t or 200mm for compression
- should not greater 16t or 200mm for tension

(Cl. 10.5.5.2. p.n. 79) (t = thickness of thinner plate)

* Procedure :-

The welds are designed to take horizontal shear at the level of junction of web plate and flange plate.

$$V_h = \frac{VA\bar{y}}{I} \quad N/mm$$

V = shear force (max. s.f)
I = M.O.I of whole section
A = Area above the level considered
(Af - Area of flange)
$\bar{y}$ = C.G. of weld connection.
$= \frac{d_w}{2} + \frac{t_f}{2}$

→ Assumption :-
→ length of each intermittent weld
= 75 mm
→ Size of Weld (if not given) = 8 mm

⇒ Weld value i.e. strength of weld at particular location.
$$c = \frac{L_w \, t_t \, f_u}{\sqrt{3}\, \gamma_{mo}}$$ (Cl. 10.5.7.1.1 p.n. 79)

• let 'a' is the c/c spacing b/w two locations of welds.

→ Value of horizontal shear in distance 'a' = $a \times V_h$

→ Equate, $\frac{L_w \, t_t \, f_u}{\sqrt{3}\, \gamma_{mw}} = a \times V_h$

→ Clear spacing = a - length of weld $\not>$ 12t

Note- Clear spacing should not exceed 12 times of thickness of thinner plate joints

Q. Design a welded plate girder of span 24m to carry superimposed load of 35 kN/m avoid use of bearing and intermediate transverse stiffeners use fe-415 steel.

Sol^n Given data-

Span $L = 24m$

Super imposed load $\Rightarrow w = 35$ kN/m

Steel fe-415

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

→ Total factored load on P.G. $W' = 52.5 \times 24$
$= 1260$ kN

→ Self weight of P.G. $w' = \frac{W'}{200} = \frac{1260}{200}$
$= 6.3$ kN/m

→ Total intensity of factored U.d.l. on P.G.
$w = 52.5 + 6.3 = 58.8$ kN/m

→ Max B.M., $M = \frac{wl^2}{8} = \frac{58.8 \times 24^2}{8}$
$= 4233.6$ kN-m

→ Max. S.f. $V = \frac{wl}{2} = \frac{58.8 \times 24}{2}$

$$\boxed{V = 705.6 \text{ KN}}$$

→ Depth of web plate-

$$d_w = \sqrt{\frac{M_u K}{f_y}} \quad \left\{ \begin{array}{l} K = 67 \\ ITS \text{ avoided} \end{array} \right\}$$

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Anant Gyan
Date
Page

$$\Rightarrow \quad d_w = \sqrt[3]{\frac{4233.6 \times 10^6 \times 67}{250}}$$

$$d_w = 1042.99 \text{ mm}$$

and also $d_w = \frac{L}{12}$ to $\frac{L}{8} = \frac{24000}{12}$ to $\frac{24000}{8}$

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

Adopt $\boxed{d_w = 1050 \text{ mm}}$

- Acc. to service ability condⁿ, thickness of web plate (Cl. 9.6.1.1 p.n. 63)

$$t_w \geq \frac{d_w}{200} \geq \frac{1050}{200} \geq 5.25 \text{ mm}$$

Also $K \geq \frac{d_w}{t_w} \Rightarrow t_w \geq \frac{d_w}{K} \geq \frac{1050}{67} \geq 15.67 \text{ mm}$

therefore adopt $\boxed{t_w = 16 \text{ mm}}$

$\Rightarrow$ Size of Web plate $= 1050 \text{ mm} \times 16 \text{ mm}$

* Design of flange plate-

→ Area of flange, $A_f = \frac{M}{d_w (f_y/\gamma_{m0})}$

$$A_f = \frac{42.33 \times 10^6}{1050 \times \left(\frac{350}{1.1}\right)}$$

$$\boxed{A_f = 17740.8 \text{ mm}^2}$$

Assume section is plastic

$$\frac{b}{t_f} \leq 8.4 \quad \left(b = \frac{b_f}{2}\right)$$

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2) $\frac{b_f}{2t_f} \le 8.4 \Rightarrow \frac{b_f}{t_f} < 16.8$

$b_f \le 16.8 t_f$

We know that

$A_f = b_f \times t_f = 17740.8 = 16.8 \times t_f \times t_f$

$\boxed{t_f = 32.496 \text{ mm}}$

Adopt $t_f = 40$mm

$\because A_f = b_f \times t_f$

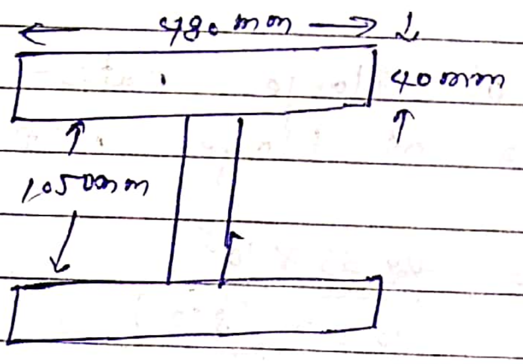
$b_f = \frac{17740.8}{40}$

$\boxed{b_f = 443.52 \text{ mm}}$

Adopt $\boxed{b_f = 480 \text{ mm}}$

$\therefore$ Size of Flange plate = 480mm × 40mm

3) final secn of plate Girder is



Section Classification

$\frac{b}{t_f} = \frac{b_f - t_w}{2t_f}$

$= \frac{480 - 16}{2 \times 40} = 5.8 < 8.4$

Hence section is plastic

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* Design of Weld connection -

→ Horizontal Shear -

$$V_h = \frac{VA\bar{y}}{I}$$

Here

$V = 705.6$ KN (Already calculated)

A = Shear Area = Area of Flange

$= 480 \times 40 = 19200\ (mm)^2$

$$\bar{y} = \frac{d_w}{2} + \frac{t_f}{2} = \frac{1050}{2} + \frac{40}{2} = 545\ mm$$

I = MOI of whole secⁿ

$$= \frac{480 \times (1050 + 2 \times 40)^3}{12} - \frac{480 \times (1050)^3}{12} - \frac{16 \times (1050)^3}{12}$$

$= 1.29 \times 10^{10}\ (mm)^4$

$I = 1295438 \times 10^4\ (mm)^4$

$$\Rightarrow V_h = \frac{705.6 \times 10^3 \times 19200 \times 545}{1295438 \times 10^4}$$

$V_h = 569.95$ N/mm

→ Assume length of each intermittent weld = 75 mm

(min. length of weld = 40mm As per Cl. 10.5.1.2. pn.78)

Assume size of weld $S = 8$ mm

⇒ Therefore strength of weld (Cl. 10.5.7.1.1, p.n. 79)

$$= \frac{L_w \times t_t \times f_u}{\sqrt{3}\ \gamma_{mw}} = \frac{2 \times 75 \times 0.7 \times 8 \times 410}{\sqrt{3} \times 1.25}$$

$\gamma_{mw} = 1.25$ for shop welding p.n. 30.

$= 159071.5$ N —— ①

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→ Let 'a' is the c/c spacing between two adjacent intermittent weld.

⇒ Horizontal shear at distance 'a' = a × 569.95 N —②

→ Equate eqⁿ ① & ②

$$a \times 569.95 = 159071.5$$

$$\boxed{a = 279 \text{ mm}}$$

∴ Clear spacing = a − length of intermittent weld
= 279 − 75 = 204 mm

As per Cl. 10.5.5.2 pn. 79 the clear spacing b/w two adjacent intermittent weld $\not>$ 12t or 200 mm where t is the thickness of web plate
$\not> 12 \times 16 = 192$

therefore take clear spacing = 190 mm c/c

Hence provide 8 mm intermittent welds of 75 mm length at clear spacing 190 mm throughout the length of plate girder.

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* Design of intermediate Transverse Stiffness (ITS)

⇒ Buckling shear Resistance (Pg. 59, 60 of IS-800: 2007).

The Web may buckle before full plasticity. It is so because pure shear produces tension in web along one diagonal and compression along the other. This compression (It's magnitude equal to shear value) causes buckling of web. The web does start buckling ~~resistance~~ (before yielding) at a stress called the elastic critical shear stress ($\tau_{cr,e}$). Shear buckling resistance can be calculated by any one of the two method

A. Simple post critical method (SPC)
B. Tension field method (TF method)

We shall be using SPC method.

* Design Procedure of ITS :-

* Nominal shear strength, $V_n = V_{cr}$

V_{cr} = shear force corresponding to web buckling.

* for finding V_{cr}

$V_{cr} = A_v \tau_b$

τ_b = shear stress corresponding to web buckling

To find τ_b -

(1) find $\tau_{cr,e}$ (Pg. 60)

$c = 2m$ (Assume)

Poisson's Ratio $\mu = 0.3$ (Assume)

Teacher's Signature..............

(2) Find d_w (Pg. 60)

(3) by d_w find I_b

If $V_{cr} > V$

then design is safe.

Design of ITS (Cl. 8.7.2 pg. 66 IS-800:2007)

→ choose suitable spacing 'c' of ITS, i.e. 2m.

→ Use Indian standard flats for ITS.

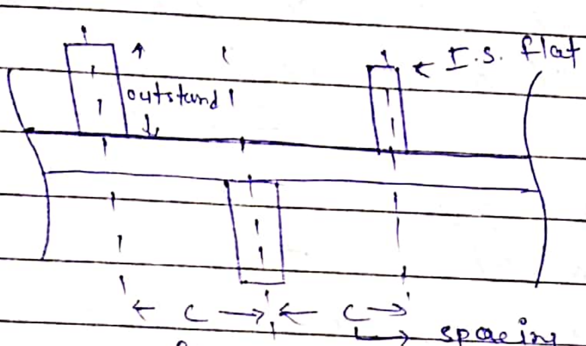


fig: single flat

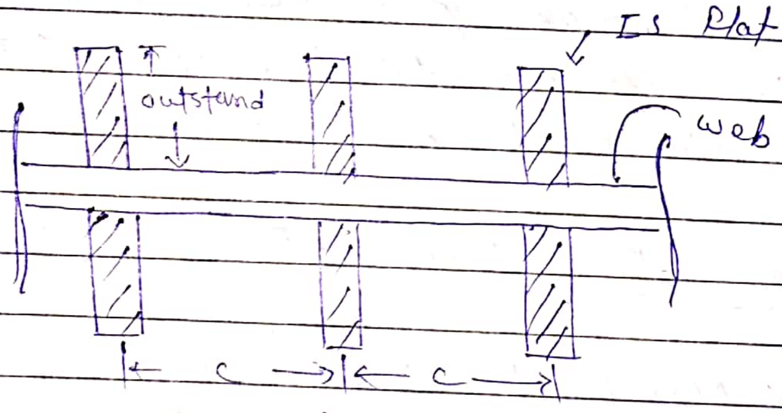


fig :- Pair of flats.

* The section of an ITS should be such that it must provide minimum moment of inertia (I_s). For this we use Clause 8.7.2.4, pg. no. 66 IS - 800 : 2007

* I_s has been calculated above the face of web of single flat section and

* Take size of flat so that projection OR outstand ≯ $20 t_q \epsilon$ (Cl. 8.7.1.2 pg. 65)
* for calulation purpose outstand should not exceed $14 t_q$.

Where t_q = thickness of stiffeners (always assume 10 mm)

* Check ITS for Buckling :-

→ find shear force at the location of ITS

$V_1 = V - w \cdot c$

$V \rightarrow$ max. s.f.

w = Total Udl

c = c/c spacing of ITS

* Shear strain required from ITS (Cl. 8.7.2.5 pg. 67)

$F_q = V - V_{cr} / \gamma_{m_0} \nleq F_{qd}$

* Find strength of ITS as column i.e.

strength of ITS > V_1

For finding strength of ITS

→ find effective length = 0.7d

→ find radius of gyration = $\sqrt{I/A}$

→ find slenderness ratio = KL/r

→ from tables for class c find f_{cd}

→ strength of ITS = Area × f_{cd}

Q. Design a welded plate girder of span 18m to carry factored load of 70 kN/m. Also design intermediate transverse stiffners connections are need not to be design. Use Fe-415 steel.

Solⁿ- Given → factored load = 70 kN/m

Span, $L = 18m$

steel = fe-415

→ self weight of P.G. $w' = W'/200$

$$w' = \frac{70 \times 18}{200} = 6.3 \text{ kN/m}$$

→ Total factored load on P.G. $= 70 + 6.3$

$= 76.3$ kN/m

→ Max. Bending moment $M = \frac{wL^2}{8} = \frac{76.3 \times 18^2}{8}$

$$\boxed{M = 3090.15 \text{ KN-M}}$$

→ Max. shear force

$$V = \frac{wL}{2} = \frac{76.3 \times 18}{2}$$

$$\boxed{V = 686.7 \text{ KN}}$$

* Design of web Plate-

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

here, $K = 180$

$$d_w = \sqrt[3]{\frac{3090.15 \times 10^6 \times 180}{250}}$$

$$\boxed{d_w = 1305.48 \text{ mm}}$$

Also $d_w = \frac{L}{12}$ to $\frac{L}{8}$ ⇒ $\frac{18000}{12}$ to $\frac{18000}{8}$ ⇒ 1500mm to 2250 mm

⇒ Adopt $\boxed{d_w = 1350 \text{ mm}}$

thickness of web plate →

As per cl. 8.6.1.1.(b) pg. 63 IS : 800-2007

$$t_w \geq \frac{d_w}{200} \geq \frac{1350}{200} \geq 6.75$$

Also $t_w \geq \frac{d_w}{k} \geq \frac{1350}{180} \geq 7.5$

$\Rightarrow$ Size of web plate

$$\boxed{1350\ mm \times 8\ mm}$$

Design of Flange Plate :-

Area of Flange $A_f = M / d_w (f_y/\gamma_{m0})$

$$= \frac{3090.15 \times 10^6}{1350(250/1.1)} \Rightarrow \boxed{A_f = 10071.6\ mm^2}$$

$\rightarrow$ Assume section is plastic

$b/t \leq 8.4 \Rightarrow b_f/2t_f \leq 8.4 \Rightarrow b_f = 16.8\ t_f$

Also $A_f = b_f \times t_f$

$A_f = 16.8\ t_f^2 \Rightarrow t_f = \sqrt{10071.6/16.8}$

$$\boxed{t_f = 24.48\ mm}$$

$$\boxed{t_f = 30\ mm}$$

Width / Breath of Flange $\Rightarrow$

$A_f = b_f \times t_f$

$b_f = A_f / t_f = \frac{10071.6}{30} = 335.72\ mm$

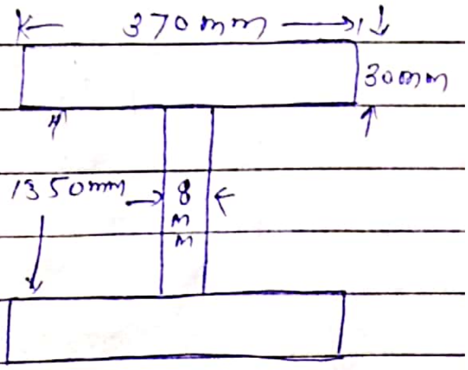
Also $b_f = 0.3 d_w = 0.3 \times 1350 = 405\ mm$

$\Rightarrow \boxed{b_f = 370\ mm}$

therefor siz of flange plate

$$\boxed{370\ mm \times 30\ mm}$$

Section classification



$b/t_f = \frac{b_f - t_w}{2t_f} = \frac{370-8}{2 \times 30}$

$= 6.03 < 8.4$

Hence section is plastic.

* Design B.M. $(M_d) \rightarrow$

$$M_d = \frac{Z_p f_y}{\gamma_{m0}}$$

$$= b_f \times t_f (d_w + t_f) \frac{f_y}{\gamma_{m0}}$$

$$= 370 \times 30 (1350 + 30) \frac{250}{1.1}$$

$= 3481.36\ kN\text{-}m > M (3090.15\ kN\text{-}m)$ ok.

Hence design is safe in B.M.

Design s.f. (V_d) →

cl. 8.4.1 pg. 59 IS: 800-2007

$$V_d = A_v f_y / \sqrt{3}\, \gamma_{m_0}$$

$$= \frac{d_w \times t_w \times f_y}{\sqrt{3}\, \gamma_{m0}} = \frac{1350 \times 8 \times 250}{\sqrt{3} \times 1.1}$$

$$= 1417.13 \text{ KN} > V (686.7 \text{ KN}) \text{ OK}$$

Hence design is safe in shear.

* Check for buckling shear (Cl. 8.4.2.2 pg. 59, IS:800-2007)

Shear force corresponding to web buckling –

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

To find τ_b →

$$\tau_{cr,e} = \frac{K_v \pi^2 E}{12(1-\mu^2)(d_w/t_w)^2}$$

Assume c/c spacing between two adjacent TTS

$$c = 2m = 2000 \text{ mm}; E = 2 \times 10^5 \text{ N/mm}^2$$

for $K_v \Rightarrow c/d = \frac{2000}{1350} = 1.4821$

$$\Rightarrow k_v = 5.35 + 4/(c/d)^2 = 5.35 + \frac{4}{1.482} = 7.17$$

$$\Rightarrow \tau_{cr,e} = \frac{7.17 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1350}{8}\right)^2}$$

$$\boxed{\tau_{cr,e} = 45.51 \text{ N/mm}^2}$$

Now $\lambda_w = \sqrt{\dfrac{f_y}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\dfrac{250}{\sqrt{3} \times 45.51}}$

$$\boxed{\lambda_w = 1.78 > 1.2}$$

So $\tau_b = f_y / \sqrt{3}\, \lambda_w^2$

$$= \frac{250}{\sqrt{3} \times 1.78^2} \Rightarrow \tau_b = 45.55 \text{ N/mm}^2$$

$$V_{cr} = A_v \cdot \tau_b = 1350 \times 8 \times 45.55$$

$$\boxed{V_{cr} = 491.94 \text{ KN}} < V (686.7 \text{ KN})$$

$V_{cr} = 491.94\ KN < V\ (686.7\ KN)$

Hence, fails in Buckling shear.

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

* Design of ITS.

$c = 2000\ mm$ (Already assumed)

shear force on ITS nearest to support.

$V_1 = V - wc$

$= 686.7 - (76.3 \times 2)$

$V_1 = 534.1\ KN$

As per Cl. 8.7.2.4 pg. 66.

min. MOI

$\frac{c}{d} = \frac{2000}{1350} = 1.48 > \sqrt{2}$

$I_s = 0.75\ d\ t_w^3$

$I_s = 0.75 \times 1350 \times 8^3$

$I_s = 518400\ mm^4$

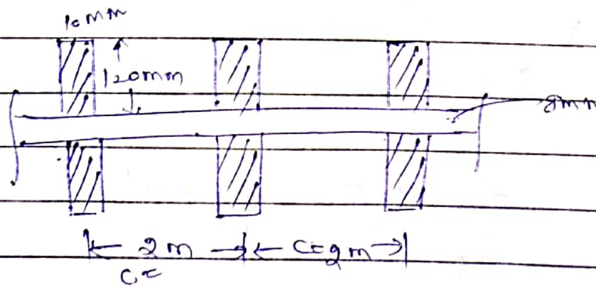
→ Let us try pair of flats of size 120mm X 10mm

Cl. 8.7.1.2 pg. 65

Check for outstand = 120 mm ⊅ $20 t_q$

$= 20 \times 10 = 200$

OK.



I provided $= \frac{10 \times (120 + 8 + 120)^3}{12} = 12710400\ mm^4$ >>> I_s

OK.

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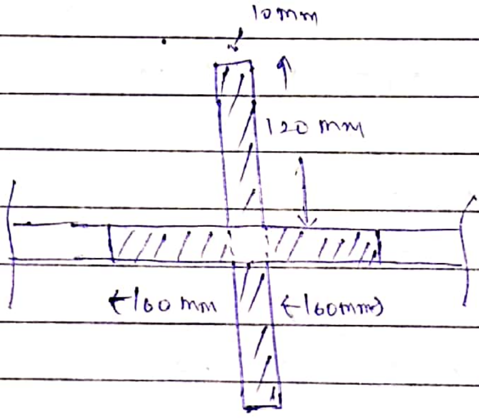
As per Cl. 8.7.2.5 pg. 67 of IS 800, 2007.
Req. shear strength from ITS.

$$F_q = V_1 - \frac{V_{cr}}{\gamma_{m_0}}$$

$$F_q = 534.1 - \frac{491.940}{1.1}$$

$$\boxed{F_q = 86.88\,KN.}$$

As per code Cl. 8.7.1.5 pg. 66. The secⁿ of ITS consist of Plat of also web friction portion equal to 20 tw on either side of it.



→ Total Area of ITS

$$A = (2 \times 120 \times 10) + (2 \times 160 \times 8)$$

$$A = 4960\ mm^2$$

→ MoI of ITS

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

$$I = 12724053.33\ mm^4$$

→ eff. length $KL = 0.7\,d_w$

$$KL = 0.7 \times 1350 = 945$$

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→ radius of gyration

$$r = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{12724053.33}{945}}$$

$r = 50.65$ mm

$Sr = \frac{KL}{r}$ $\quad Sr = \frac{945}{50.65} = 18.66$

from table 10, pg. 44 of IS 800: 2007.
buckling class C
from pg. 42 table 9(c) $f_{cd} = ?$

KL/r	$f_y = 250$ N/mm2
10	227
18.66	f_{cd}
20	224

$$f_{cd} = \frac{(224 - 227)}{(20-10)} \times (18.66 - 10) + 227$$

$= 224.40$ N/mm^2

So strength of TTS.

$= A \times f_{cd}$

$= 4960 \times 224.40$

$= 1113$ kN > V_1 (534.1 kN)

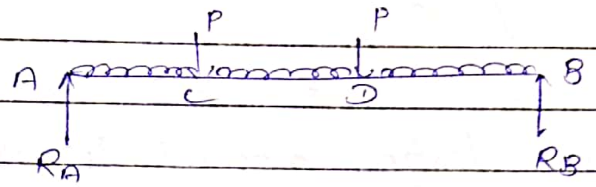
ok

06 Feb 2019

\+ **Bearing stiffeners (B.S.)** →

[Cl: 8.7.4, pg. 67 IS : 800 - 2007]

→ A bearing stiffeners is provided under concentrated load.



eg. here at point A and B for force R_A & R_B & at C and D for intern load. P.

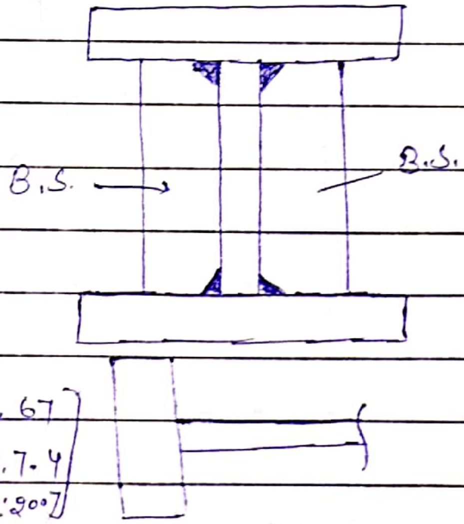
→ They transfer the load by bearing

→ B.S. most provide bearing area A_q, obtained from formula given at pg. No. 68 of IS : 800 - 2007.

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

Where F_{psd} = bearing strength of stiffener.

F_x - external load or Reaction



→ Load Capacity of the Web

$$F_w = (b_1 + n_2) \frac{t_w f_{yw}}{\gamma_{m0}}$$ [pg. no. 67 Cl. 8.7.4 IS-800:2007]

Where b_1 → stiff~~ening~~ bearing length ($b_1 = 0$ in numericals)

n_2 = length obtained by dispersion through the flange to the web junction at a slope of 1:2.5 to the plane of the flange.

So $n_2 = 2.5\, t_f$.

→ Bearing stiffeners are checked for the load $R - F_w$ as strut / column ($R \rightarrow$ Reaction)

→ It's strength (i.e. $A_q \times f_{cd}$) should greater than $R - F_w$

Q. In previous design numerical, also design Bearing stiffeners.

Sol^n Design of Bearing Stiffeners.

→ Already Calculated, Reaction

$F_x = R = V = 686.7$ KN

→ Bearing Area (Cl. 8.7.5.2 pg. 68 of IS 800 : 2007)

$$F_{psd} = \frac{A_q f_{yq}}{0.8 \gamma_{m_0}} \geq F_x$$

$$\Rightarrow A_q \geq \frac{F_x \times 0.8 \times \gamma_{m_0}}{f_{yq}}$$

$$\Rightarrow A_q \geq \frac{686.7 \times 10^3 \times 0.8 \times 1.1}{250}$$

$$\Rightarrow A_q \geq 2417.18 \text{ mm}^2$$

⇒ Let's Try 2 Flats of size 180 mm × 10 mm

→ check for outstand, ~~[illegible]~~

(Cl. 8.7.1.2, pg. 65 IS - 800 : 2007)

outstand $\not> 20 t_q \not> 20 \times 10 \not> 200$ mm

Here outstand = 180 mm < 200 mm

OK

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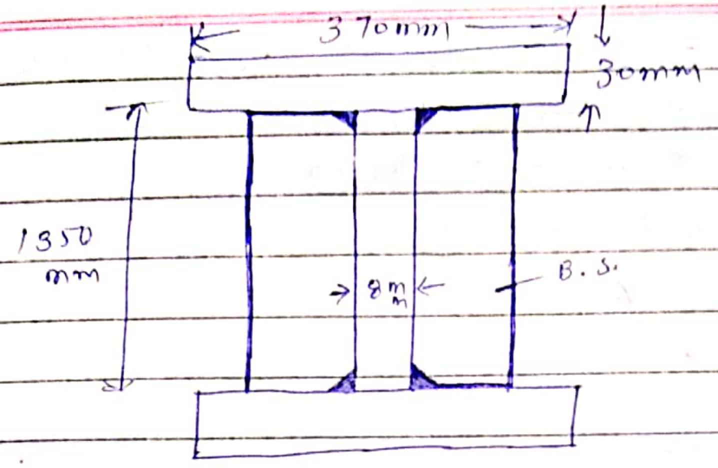


→ Actual Bearing Area $\left(2\times 8 = 16 \approx 15\right)$ for web

$A_q = 2\times(180-15)\times 10$

$= 3300\ mm^2 > 2417.18\ mm^2$

ok

* Check for B.S. As ~~[illegible]~~ column –

→ Capacity of Web along crippling (Cl 8.7.4 pg. 67 IS-800:2007)

$$F_w = \frac{(b_1+n_2)\,t_w \cdot f_y}{\gamma_{mo}}$$

b_1 = stiff portion of bearing, assume $b_1 = 0$

n_2 = for end reaction at dispersion 1:2.5

$n_2 = 2.5\, t_f = 2.5\times 30$

$n_2 = 75\ mm$

$$F_w = \frac{75\times 8\times 250}{1.1}$$

$$\boxed{F_w = 136.36\ KN}$$

→ Hence, compressive load on B.S. = $R - F_w$

$= 686.7 - 136.36 = 550.34$ KN.

Effective Area of section of B.S. (As per code

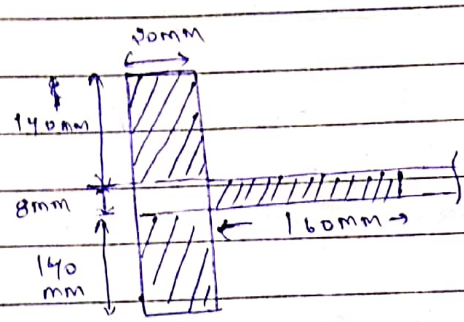
Width $\ngtr 14\, t_f$

$\ngtr 14 \times 10 \ngtr 140$ mm

Depth $\ngtr 20\, t_w \ngtr 20 \times 8$

$\ngtr 160$ mm

→ final Section of B.S :-



→ Eff. area is hatched portion as shown for calculation

→ If contribution of web plate is ignored.

$A = (140 + 8 + 140) \times 10 = 2880$ mm^2

→ MOI of the section, $I = \dfrac{10\,(140 + 8 + 140)^3}{12}$

$= 1990 \times 10^4$ mm^4

→ Radius of Gyration, $r = \sqrt{\dfrac{I}{A}}$

$= \sqrt{\dfrac{1990 \times 10^4}{2880}}$

$= 83.12$ mm

→ Slenderness Ratio, $S_r = \dfrac{KL}{r} = \dfrac{0.7 \times 1350}{83.12}$

$= 11.37$

→ Slenderness ratio, $S_r = 11.37$.

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→ for buckling class 'c' from table 9[c] pg. 42.

$\frac{KL}{r}$	$f_y = 250 N/mm^2$
10	227
11.37	f_{cd}
20	224

$$f_{cd} = \frac{(224-227)}{(20-10)} \times (11.37-10) + 227$$

$$\left[f_{cd} = 226.59 \ N/mm^2 \right]$$

→ strength of B.S. as column $= A \times f_{cd}$

$= 2880 \times 226.59$

$= 652.57 \ kN > 550.34 \ kN$

Hence ok

Design is safe

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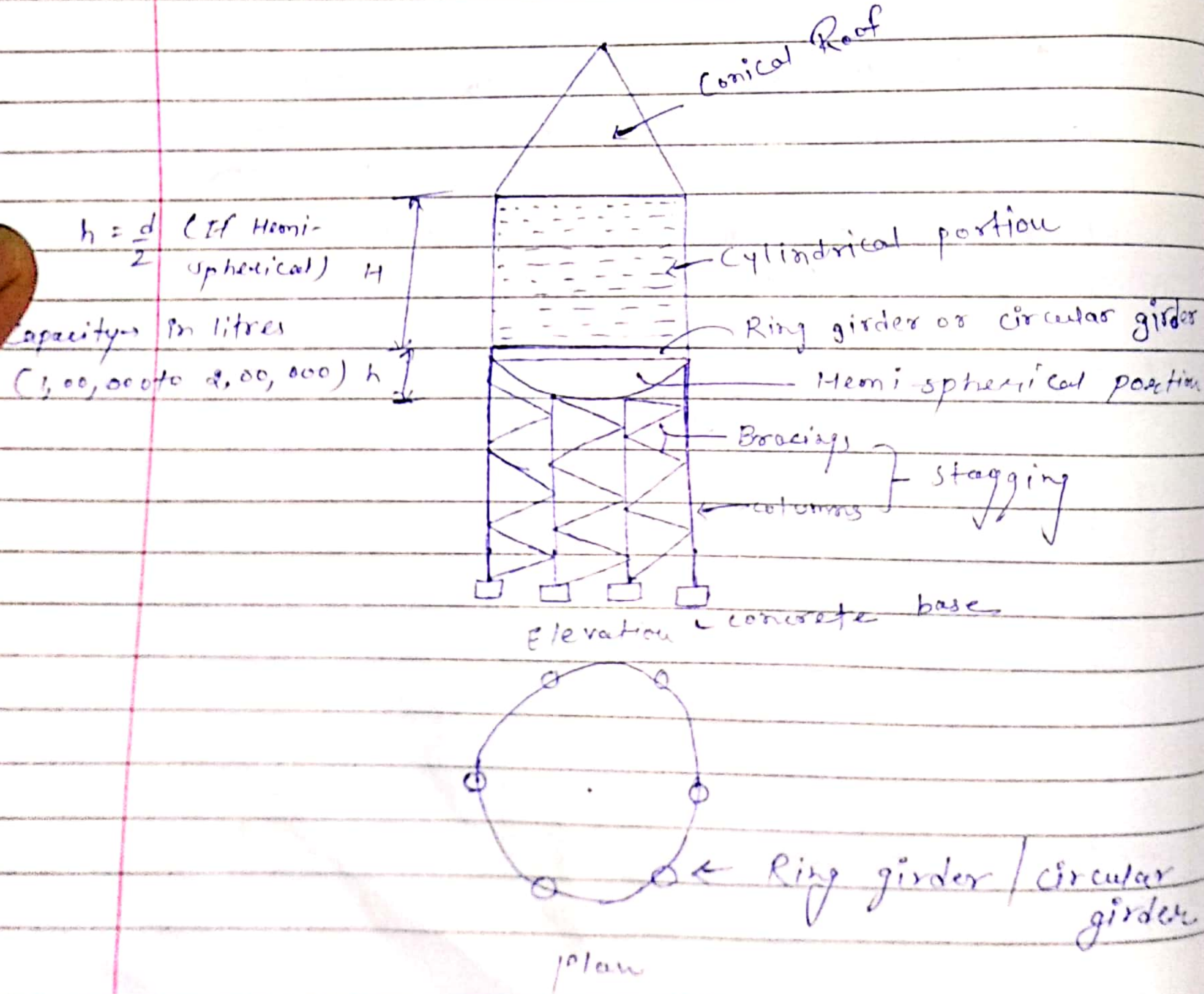
UNIT - 5. WATER TANKS.

* → Only elevated or overhead tanks.
  → They are used to store water.
  - Types of water tanks :-
    - I - Circular water tanks.
    - II - Rectangular water tanks.
      - Riveted / Bolted tanks
      - Pressed steel tanks.

Circular TANKS :-



Design Procedure :-

Step ① To decide dia. & height of cylindrical and spherical portion.

Let bottom is hemi-spherical ; $h = d/2$

$$V = \frac{\pi}{4} d^2 H + \frac{2}{3}\pi\left(\frac{d}{2}\right)^3$$

V = Volume of tank
d = dia of tank.
H = height of tank. (cylindrical portion)

* Assume $\frac{H}{d} = 0.8$ to 1 and find d & H.

step②. find thickness of cylindrical portion of the tank. -

We know that for thin cylindrical shell ~~containg~~ containing fluid under pressure, there are two hoop stresses (axial tension) produced

→ Circumferential stress $f_1 = \frac{pd}{2t}$

$$\Rightarrow f_1 = \frac{(wh')d}{2t}$$

→ longitudinal stress $f_2 = \frac{pd}{4t} = \frac{(wh')d}{4t}$

∴ thickness of cylindrical portion

$$\boxed{t_{cyl.} = \frac{WHd}{2\sigma_{at}\eta}}$$

similarly, thickness of spherical portion

$$\boxed{t_{sph.} = \frac{W(H + d/2)d}{2\sigma_{at}\eta}}$$

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Note - In no case $t_{sph.} < t_{cyl.}$

$W \rightarrow$ density of Water (9.81 kN/m^3)
$\eta \rightarrow$ Efficiency of Joint (70%)
$\sigma_{at} \rightarrow$ permissible axial tensile stress.
$\sigma_{at} = 0.8 \times 0.6 f_y$ [20% corrosion loss]
$= 120\ N/mm^2$

Step 3.

Conical Roof
→ It is not designed
→ It may be assumed 5 mm thick with pitch $\frac{1}{4}$ (pitch $= \frac{Rise}{span}$)

→ Riveted / Bolted Joints - These are double bolted lap joint and are designed accordingly.

Step 4. Ring girder or Circular Girder -

→ All loads are transfered to ring girder as UDL.
→ The ring girder is supported on even no. of columns equally spaced.
→ This is a special girder because it is subjected to torsion also in addition to B.M. and S.F.
→ Value of max B.M. (M) max. S.F (V) and max. torsion (T) depends on.
W - Total load on Ring girder
R → Radius d/2
n = Number of columns.

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Table showing values of M, V. & T

No. of Column	Load on each Column	Max. S.F V (KN)	B.M. of Column hogging M (KN-m)	B.M at Centre of span (KN-m) (M)	Max. Torsion T (KN-m)	ϕ at which T is max.
4	W/4	W/8	-0.03415WR	0.01762WR	0.0053WR	19°20'
6	W/6	W/12	-0.01482WR	0.00751WR	0.00151WR	12°44'
8	W/8	W/16	-0.00827WR	0.00416WR	0.00063WR	9°33'

M produces bending stress

$$\frac{M}{I} = \frac{f}{y}$$

$$f = \frac{M}{I} \times y = \frac{M}{Z}$$

V produces Shear stress

$$q_1 = \frac{V}{A}$$ Where A = area of cross-section

T also produces shear stress

$$q_2 = \frac{T\, t_{max}}{J \text{ (or } k)}$$ where J or k = torsional constant $= \sum \frac{1}{3} b t^3$

t_{max} = max. thickness anywhere in the cross-section.

$\therefore$ Total shear stress $q = q_1 + q_2 \leq \tau_{av} (= 0.4 F_y)$

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Q. Design an overhead circular tank for capacity 2lac. litres. It is supported on 6 columns uniformly spaced its bottom may be hemi-spherical.

Ans. Given

Volume = 2 lakh. litres

Columns = 6

Volume = 200 m^3.

STEP (1) Diameter and height of Cylindrical portion

To find d & H, assume $H/d = 0.8$

$\Rightarrow H = 0.8d$

Volume of tank, $V = \frac{\pi}{4} d^2 H + \frac{2}{3}\pi\left(\frac{d}{2}\right)^3$

$$200 = \frac{\pi}{4} d^2 (0.8d) + \frac{2}{3}\pi\left(\frac{d}{2}\right)^3$$

$$\boxed{d = 6.08\ m}$$

$$\boxed{\text{adopt } d = 6.1\ m}$$ *

$H = 0.8\ d$

$= 0.8 \times 6.1$

$$\boxed{H = 4.88\ m}$$ *

STEP (2) Thickness of Plate :-

→ Thickness of cylindrical portion

$$t_{cyl.} = \frac{WHd}{2\sigma_t \eta}$$

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$$t_{cyl.} = \frac{9.81 \times 10^{-6} \times 4.88 \times 10^{3} \times 10^{3} \times 6.1}{2 \times 0.8 \times 0.6 \times 250 \times 0.7}$$

$$\boxed{t_{cyl.} = 1.74 \text{ mm}} \not< 6\text{m}$$

$$\Rightarrow \boxed{t_{cyl.} = 6\text{mm}}$$

Thickness of spherical portion

$$t_{sph.} = \frac{w(H + d/2)\,d}{4\,\sigma_{at}\,\eta}$$

$$= \frac{9.81 \times 10^{-6}\left(4.88 \times 10^{3} + \frac{6.1 \times 10^{3}}{2}\right) \times 6.1 \times 10^{3}}{4 \times 0.8 \times 0.6 \times 250 \times 0.7}$$

$$t_{sph.} = 1.41 \text{ mm} \not< 6\text{m}$$

$$\boxed{t_{sph.} = 6\text{mm}}$$

STEP. 3.

Conical Roof :-

Provide 5mm thick plate for conical roof.
pitch may be taken as 1/4

$$\text{pitch} = \frac{\text{Rise}}{\text{span}}$$

$$\frac{1}{4} = \frac{\text{Rise}}{\text{span}}$$

$$\text{Rise} = \frac{\text{span}}{4} = \frac{6.1}{4}$$

$$\boxed{\text{Rise} = 1.525 \text{ m}}$$

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→ Riveted Joint :-

Bolt strength in shear
[~~Cl. 10.3.3 pg. 75 IS : 800-2007~~]

lets provide 16 mm dia bolt.

Shearing strength $= \frac{\pi}{4} d^2 \tau_v$ | τ_v = permissible stress in shear.

$= \frac{\pi}{4} \times 16^2 \times 0.90 \times 100$ [10% loss due to fluid. [1 − 0.1 = 0.90]]

$= 18.09$ kN ≃ $\boxed{18.1 \text{ kN}}$

Strength in bearing :-

Strength in bearing $= d.t.\sigma_b$ | σ_b = permissible bearing stress

$= 16 \times 5 \times 0.9 \times 300$

$= \boxed{21.6 \text{ kN}}$

There fore Bolt value = 18.1 kN (lowest Value)

Number of Bolts :-

Hoop force in vertical Joint

$f_1 = \frac{wHd}{2}$

$= \frac{9.81 \times 10^{-6} \times 4.88 \times 10^3 \times 6.1 \times 10^3}{2}$

$= 145.86 \simeq \boxed{146 \text{ N/mm}}$

There are two rivets to take this Hoop force

$\Rightarrow 146\, p = 2 \times 18.1 \times 10^3$

$\boxed{p = 247.95 \text{ mm} \not> 10t}$ ($10t = 10 \times 5 = 50$ mm)

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Hence provide 50 mm pitch throughout ~~beams~~ in circumferencial joint also.

Step 4. Ring Girder :->

W = Total udl on Ring girder

=> W = water load + [vol. of cylindrical portion + Vol. of spherical portion + Vol. of conical portion] × 1.2 × Density of steel + self weight

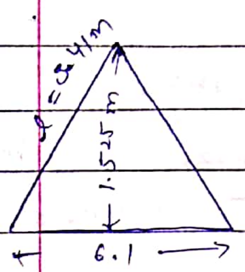
-> Water Load = $200\ m^3 \times 10\ KN/m^3$
= 2000 KN.

-> Self weight of Ring Girder = 1.6 KN/m (Assume)
Total self weight = $1.6 \times \pi d$
= $1.6 \times \pi \times 6.1$
= 30.66 KN

-> Volume of cylindrical portion = $\pi d t h$
= $\pi \times 6.1 \times 6 \times 10^{-3} \times 4.88$
= $0.56\ m^3$

-> Volume of spherical portion = $2\pi \times \left(\frac{d}{2}\right)^2 \times t_{sph}$
= $2 \times \pi \times \left(\frac{6.1}{2}\right)^2 \times 6 \times 10^{-3}$
= $0.35\ m^3$

-> Volume of conical portion = $\pi r l t$
= $\pi \times \frac{6.1}{2} \times 3.41 \times 5 \times 10^{-3}$
= $0.16\ m^3$



=> Total udl on Ring Girder =

$W = 2000 + (0.56 + 0.35 + 0.16) \times 1.2 \times 78.5 + 30.66$

= 2131.45 KN

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$\Rightarrow R = \frac{d}{2} = \frac{6.1}{2} = 3.05\,m$

From Table knowing W, T & Number of column we get values of max. B.M. (M), max S.F. (V) & max. Torsion (T) to which Ring girder is subjected.

$\rightarrow$ Max. S.F. $= \frac{W}{12} = \frac{2131.45}{12}$

$= 177.62$ KN

$\boxed{\Rightarrow V = 177.62 KN}$

$\rightarrow$ Max. B.M. (M.) $= 0.01482\ WR$

$= 0.01482 \times 2131.45 \times 3.05$

$\Rightarrow \boxed{M = 96.34\ KN-M}$

$\rightarrow$ Max. Torsion(T) $= 0.00151 \times W \times R$

$= 0.00151 \times 2131.45 \times 3.05$

$\Rightarrow \boxed{T = 9.82\ KN-M}$

$\rightarrow Z_{req.} = \frac{M}{0.66 f_y}$ $\leftarrow \sigma_{bt}$

$= \frac{96.34 \times 10^6}{0.66 \times 250}$

$= 583.87 \times 10^3\ (mm)^3$

let us try section of Ring Girder Consisting of two Angle ISA $200 \times 200 \times 25$ connecting as shown.

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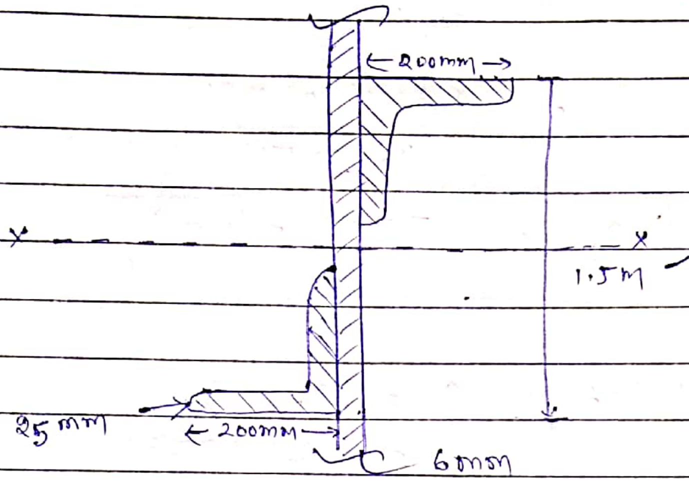

app. $\frac{d}{4} = \frac{6.1}{4} = 1.525$

from steel Table for ISA 200, 200, 25 mm

$A = 93.80\ cm^2 = 9380\ mm^2$

$C_{xx} = 5.88\ cm = 58.8\ mm$

$I_{xx} = 3436.3\ cm^4 = 3436.3 \times 10^4\ mm^4$

→ check for shear →

→ Shear stress due to shear force

$$q_1 = \frac{V}{A} = \frac{177.62 \times 10^3}{(1500 \times 6) + (2 \times 9380)}$$

$$\boxed{q_1 = 6.4\ N/mm^2}$$

→ shear stress due to torsion

$$q_2 = \frac{T}{J}\, t_{max}$$

J = Torsional Constant $= \Sigma \frac{1}{3} b t^3$

$$\Rightarrow J = \frac{1}{3} \times 1500 \times 6^3 + 2\left[\frac{1}{3} \times 200 \times 25^3 + \frac{1}{3} \times (200-25) \times 25^3\right]$$

$$\boxed{J = 4014250\ mm^4}$$

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Also $t_{max} = 6 + 25 = 31\ mm$

$\Rightarrow q_2 = \frac{T}{J} t_{max}$

$= \frac{7.82 \times 10^6 \times 31}{4014250}$

$$\boxed{q_2 = 75.83\ N/mm^2}$$

Total stress $q = q_1 + q_2$

$= 6.4 + 75.83$

$= 82.23\ N/mm^2$

and $\tau_{av} = 0.4 f_y = 0.4 \times 250 = 100\ N/mm^2$

$\therefore q < \tau_{av}$ ok//

Check for bending stress $\rightarrow$

$\sigma_{bt,cal.} < \sigma_{bt}$

$\rightarrow \sigma_{bt,cal.} = \frac{M}{I_{xx}} \cdot y$

$\sigma_{bt} = 0.66 f_y = 0.66 \times 250$

$= 165\ N/mm^2$

$$\boxed{I_{xx} = I_{self} + Ay^2}$$

$I_{xx} = \frac{6 \times 1500^3}{12} + 2\left[3436.3 \times 10^4 + 9380(150 - 58.8)^2\right]$

$= 107189.56 \times 10^4\ mm^4$

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$$\sigma_{bt\,cal.} = \frac{96.38 \times 10^6 \times 750}{1071895.56 \times 10^4}$$

$$\boxed{\sigma_{bt,cal.} = 6.74 \ N/mm^2}$$

Comparing $\sigma_{bt\,cal.}$ & σ_{bt}

$$\boxed{\sigma_{bt,cal.} < \sigma_{bt}}$$ ok//

Q. Design an overhead circular steel tank with hemispherical bottom, for capacity 1,80000 lit. It is supported on 8 columns uniformly placed along periferry, for which

$M = 0.00827 \ WR$

$T = 0.00063 \ WR$

& $F = W/16$ may be taken.

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Rectangular TANK

(1) Riveted Rectangular Tank.
(2) Pressed Steel Tank.

(1) Rectangular pressed steel TANK :-

→ They are rectangular in shape.
→ They are fabricated by joining together, the pressed steel (M.S. (mild steel)) plates by loading.
→ The plates are heated uniformly in a furnace and formed into required shape by pressing.

Advantages :-

→ Ease in errection
→ Facility in transportation
→ Standard Construction
→ Ease in dis-mentoling and re-errection.

Pressed STEEL Plates

→ They are square in shape of size 1.25m X 1.25M
→ Their thickness should be 3mm, 5mm, 6mm and 8mm
→ Flanges of plate can be pressed square or at partly 45° to the face of plates.

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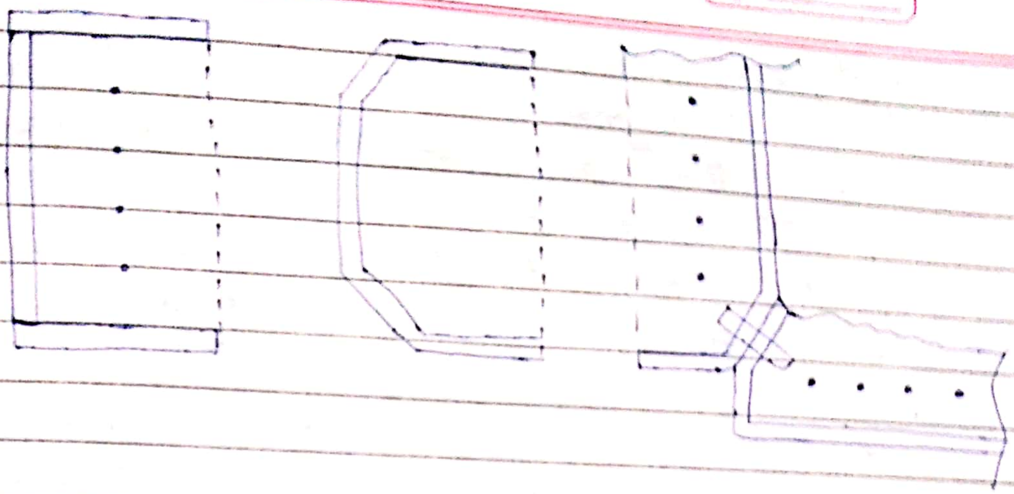

* Size of the TANK :-

→ Depends on capacity.
→ Smallest size 1.25m x 1.25m x 1.25m of capacity 1950 liters.

Thickness of Pressed Steel Tank :-
As per IS : 804

Depth of tank	Location of Plate	thickness
1.25 m	Bottom side	5 mm
2.5 m	Bottom & first tier of plate	6 mm
	Top Tier	5 mm
3.75 m	Bottom & first tier	8 mm
	second tier	6 mm
	Top Tier	5 mm
5 m	Bottom & first Tier	8 mm
	Second & Third Tier	6 mm
	Top Tier	5 mm

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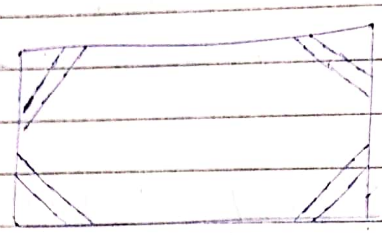
* Stays:-
→ The joints are single line bolts and as such they can not resist bending moment.
→ The sides of the tanks have to be supported by stays at junction of two plates.
→ Stays are equally inclined at the ends (at 45°)
→ Stays in vertical plane may be called diagonals of those in horizontal plane called bracing / horizontals.
→ Stays are of section either round or flat.



Columns:
→ The tank is supported on even no. of columns (may be 4 only) with 1.25m overhang of the tank on either side from concrete of columns.

Q. Design an elevated two tier rectangular pressed steel tank having capacity 125 kliters. Design the stays also and draw their arrangements. Show loads transferred to an intermediate top tier beam and Design the beam.

Sol^n

Step 1. Dimension of the tank = ~~$500m^3$~~

given Capacity of tank = $125 \ m^3$

→ let us provide $1.25 \ m \times 1.25 \ m$ size of plates.

→ let the overall height of the tank be

$= 2 \times 1.25$

$= 2.50 \ m$

→ Providing a free board of '0.15 m', therefore effective depth available for water storage

$= 2.50 - 0.15$

$= 2.35 \ m$

Base Area of the tank

$= \frac{V}{h} = \frac{125}{2.35}$

$= 53.19 \ m^2$

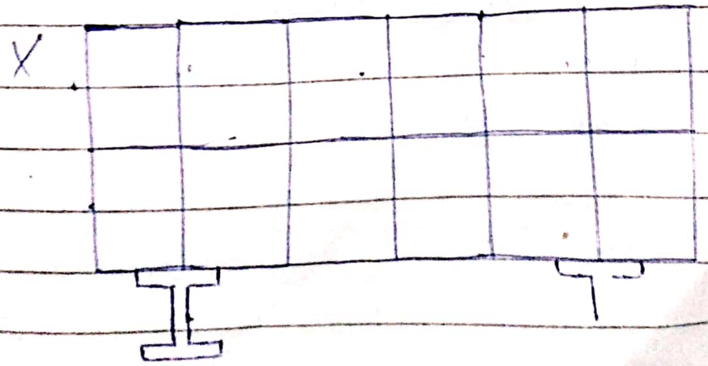
Provide square tank size

→ one dimension of the tank $= \sqrt{53.19}$

$= 7.29 \ m$

→ Size of tank Provided $= 7.5 \ m \times 7.5 \ m \times 2.5 \ M.$

→ The tank will be supported on 4 column spaced at 5m $[7.5 - 2 \times 1.25]$
$= 5m$



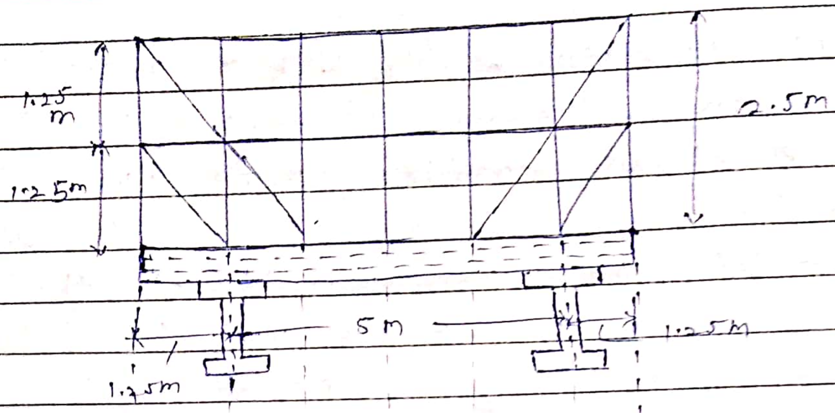


fig. Elevation

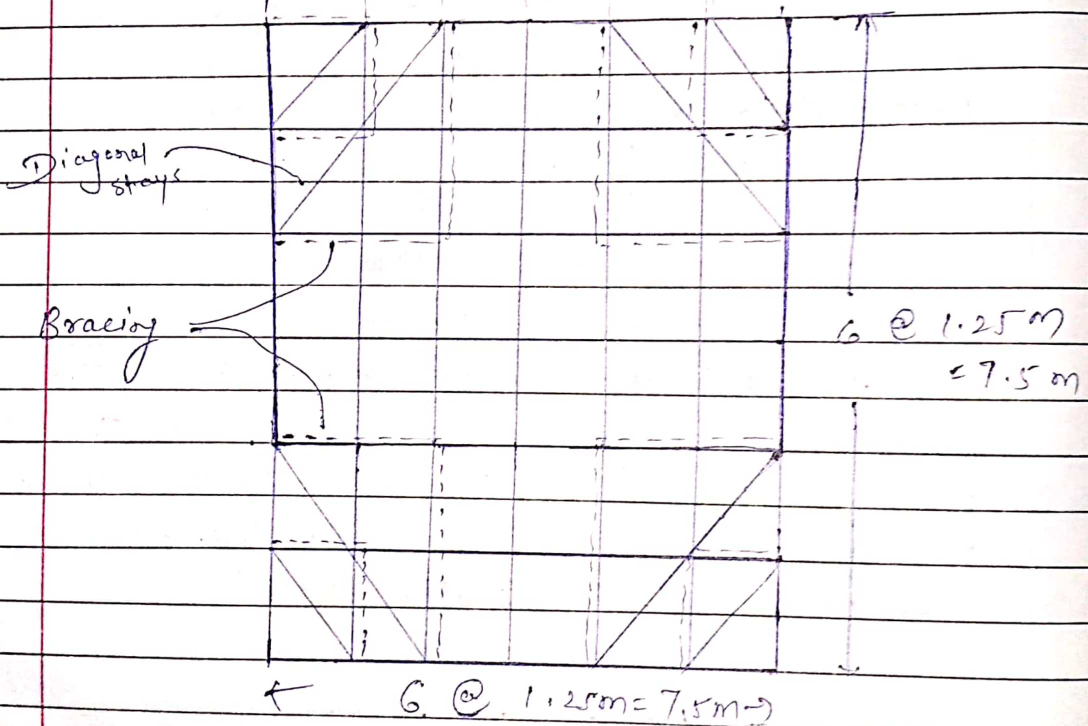


fig: Plan

Thickness of plates as per IS : 804 for 2.5m depth of tank →

Thickness of bottom & first tier plates = 6m

→ Thickness of Top tier plate = 5mm

* ① Design of stays →

→ Assuming that water level upto top of the tank

→ Water pressure at 1.25m from top

$= \gamma_w \times h$

$= 9.81 \times 1.25 \times 1.25$

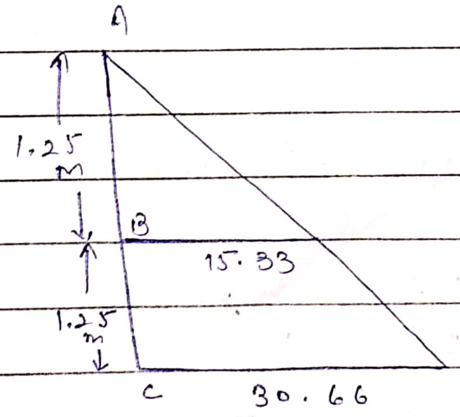
$= 15.33$ KN/m

→ Water pressure at 2.5 m from top.

$= \gamma_w \times h$

$= 9.81 \times 1.25 \times 2.5$

$= 30.66$ KN/M



Pressure P_1 on top plate

$= \frac{1}{2} \times 15.33 \times 1.25 = 9.58$ KN

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and act at $\frac{1.25}{3}$ (i.e. $h/3$) $= 0.42m$ from B,

- Pressure P_2 on bottom plate

$$= \frac{1}{2} \times (15.33 + 30.66) \times 1.25$$
$$= 28.74 \text{ KN}$$

and act at $\left(\frac{2a+b}{a+b}\right) \times \frac{h}{3}$

$$= \frac{2 \times 15.33 + 30.66}{15.33 + 30.66} \times \frac{1.25}{3}$$

$= 0.56m$ from C

→ Reaction of upper plate

$$R_A = \frac{P_1}{3} = \frac{1}{3} \times 9.58 = 3.193 \text{ KN}$$

$$R_{B_1} = \frac{2}{3} P_1 = \frac{2}{3} \times 9.5 = 6.386 \text{ KN}$$

→ Reaction of lower plate

$$R_{B_2} = \frac{\bar{y}}{h} \cdot P_2 = \frac{0.56}{1.25} \times 28.74$$
$$= 12.783 \text{ KN}$$

R_c at bottom of tank so not necessary.

$$\boxed{R_A = 3.193 \text{ KN}}$$

$$R_B = R_{B_1} + R_{B_2}$$
$$= 6.386 + 12.783$$
$$\boxed{R_B = 19.169 \text{ KN}}$$

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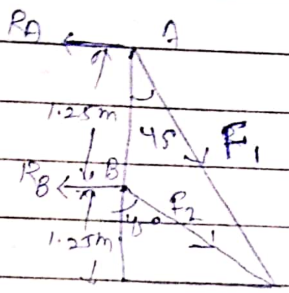
* forces in stays →



$$f_1 \sin 45^\circ = R_A$$

$$F_1 = R_A / \sin 45^\circ$$

$$= \frac{3.193}{1/\sqrt{2}}$$

$$\boxed{f_1 = 4.515 \text{ KN}}$$

→ Force f_2 in bottom stays

$$f_2 \sin 45^\circ = R_B$$

$$f_2 = \frac{19.169}{1/\sqrt{2}}$$

$$\boxed{f_2 = 27.109 \text{ KN}}$$

→ Net area required for top stay

$\sigma_{at} = 0.6 f_y$

$\sigma_{bt} = 0.66 f_y$

$$= \frac{f_1}{\sigma_{at}} = \frac{4.515 \times 10^3}{0.8 \times 0.6 \times 250}$$

80% loss due to corrosion

$$= 37.625 \text{ mm}^2$$

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→ Net Area Required for bottom stay

$$= \frac{f_2}{\sigma_{at}} = \frac{27.109 \times 10^3}{0.8 \times 0.6 \times 250}$$

$$= 225.908 \text{ mm}^2$$

Provide 85 mm × 6mm mild steel plate for top stay and 60 mm × 6mm mild steel plate for bottom stay and connect these by 14 mm dia rivet.

for 14 mm dia rivet, hole dia = 15.5 mm

→ Design net Area for top stay

$$= (85 - 15.5) \times 6$$

$$= 57 \text{ mm}^2 > 87.625 \text{ mm}^2$$

ok

→ Design net for bottom stay

$$= (60 - 15.5) \times 6$$

$$= 267 \text{ mm}^2 > 225.908 \text{ mm}^2$$

ok//

Unit - 3,4 Bridges.

Types or Classification

I → Materials → R.C.C. / → Steel

II → Traffic → Highway / → Railway

III → Floor Position → Deck Type (DT) / → Through type (TT)

IV → Main Girder → Plate Girder (P.G.) / → Truss Girder (T.G.)

→ There are four types of railway ~~brigl~~ bridges to be considered

→ DT PG RB
→ DT TG RB
→ TT TG RB
→ TT PG RB

PG → DT > 80% / → TT < 20%

TG → DT < 20% / → TT > 80%

UNIT-3 # BRIDGE

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* Deck Type Plate Girder Railway bridge (DT PG RB)

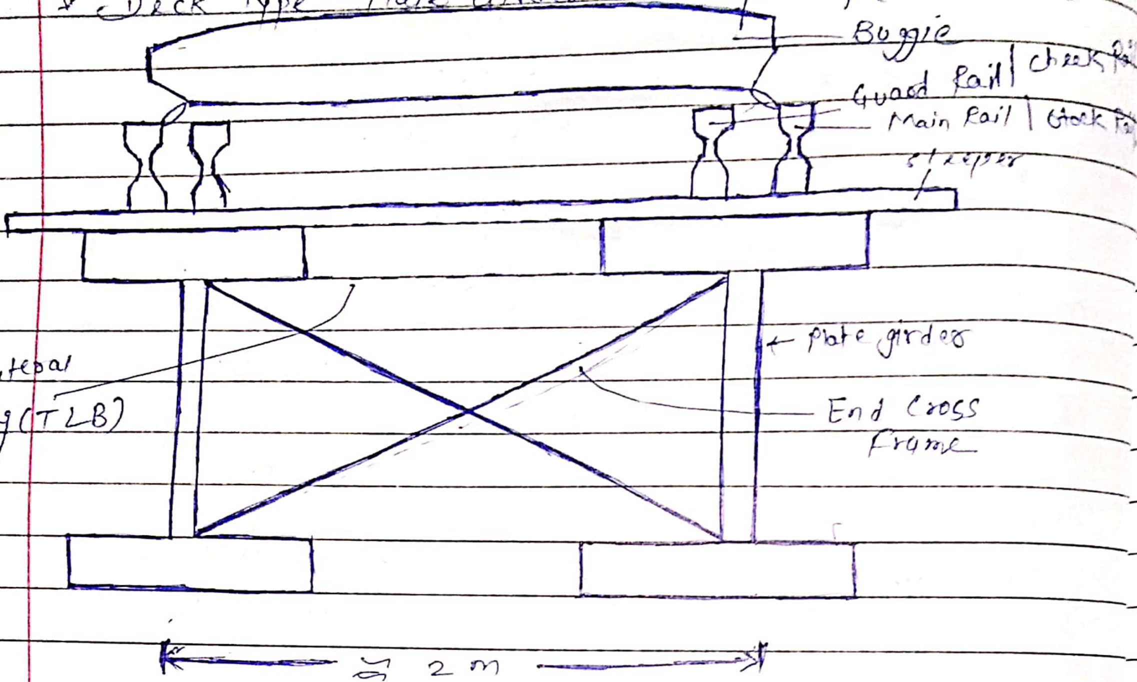

Fig: DT PG RB

* LOADS ON BRIDGES →

1. Dead Load.

(i) Loads of Rails, sleepers, fastening etc.

(ii) Self weight of Plate Girder

2. Live Load →

Train and people in it.

It is given in Bridge rule book published by railway board. It is given in terms of E.U.D.L [Equivalent uniformly distributed Load]

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It is given in tabular form for BM & SF.

3. Impact Load

Trains move with speed on a bridge and go away they produce impact effect and is calkulated in terms of CDA (Coefficient of Dynamic Augmentation.)

$$CDA = 0.15 + \frac{8}{(6+L)} \ngtr 1.0$$

L is the span of bridge in M.

4. Wind Load :-

It is also given in Bridge Rule. The wind pressure (p) is taken as

$p = 1.5 \ KN/m^2$ for bridge loaded

$p = 2.4 \ KN/m^2$ for bridge unloaded.

5. Racking Forces :-

It is a lateral force producing due to a lurching effect (side way movement). It is taken as 6 kn/m udl acting horizontally on lateral bracings connecting loaded Flanges.

6. Tractive effort and Braking force :-

These are longitudinal forces and act parallel to the track their values are given in bridge rule.

7. Centrifugal Force → It is produce when a train is moving on curved track.

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Q. A Deck type P.G RB for single track broad gauge main line loading has following data.

effective span = 24m, spacing of Girder c/c = 2m ; Size of timber sleeper = 3 m × 0.3m × 0.15
→ spacing of sleeper c/c = 600 mm
Density of timber = 9 kN/m^3
Weight of Main Rails = 0.5 kN/m
Weight of Guard Rails = 0.25 kN/m
Weight of fastening = 0.3 kN/m of track.

Calculate the max. BM and max. SF for which one main plate Girder will be design.

Soln:- → There will be two plate Girders for one track.

→ DEAD LOAD (D.L.) on one Girder from main rail, guard Rail and fasteners

$$= \left(0.5 + 0.25 + \frac{0.3}{2}\right) = 0.9 \text{ kN/m}$$

→ Dead load (D.L.) on one girder from sleeper

$$= \frac{1}{2}\left[3 \times 0.3 \times 0.15 \times \frac{9}{0.600}\right]$$

$$= 1.0125 \text{ kN/m}$$

→ Total D.L. on one Girder

$$= (0.9 + 1.0125) \times 24 = 45.9 \text{ kN}$$

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For self weight of plate Girder must find EUDL and CDA

→ From modified BG loading -1987 (Bridge Rule) for 24m span.

<u>Live loads</u> -

For BM = 2280 kN

For SF = 2503 kN

C.D.A. = 0.417

Higher value is 2503 kN (L.L for S.F.)

→ For one track on one girder EUDL including impact.

$= \frac{1}{2} \times 2503 \times 1.417$

$= 1773.4$ kN —(2)

⇒ Total D.L + L.L. + I.L. on one girder

from eqⁿ (1) & (2)

$= 45.9 + 1773.4$

$= 1819.3$ kN

Assuming self weight of one plate Girder

$= \frac{W'}{300} = 6.06$ kN/m

Total self weight $= 6.06 \times 24 = 145.44$ kN —(3)

Let Wm is the total Udl on one plate girder for BM and Wv is the total Udl on one plate girder for S.F..

Total load for BM on one plate Girder

$W_m = \frac{1}{2}\{(2280 \times 1.417) + 45.9 + 145.44\}$

{live load including impact) + D.L. + self weight

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$$\boxed{W_M = 1806.72 \text{ kN}}$$

→ Total load for S.f. on one P.G.

W_v = L.L. including impact + D.L + self weight

⇒ $W_v = \frac{1}{2} \times 2503 \times 1.417 + 45.9 + 145.44$

$$\boxed{W_v = 1764.71 \text{ kN}}$$

max. BM and S.f on one plate Girder

$$M = \frac{W_m L}{8}$$

$W_m = \frac{1806.72}{24}$

$$= \frac{1806.72 \times 24}{8}$$

$$\boxed{M = 5420.16 \text{ kN-m}}$$

$$V = \frac{W_v}{2} = \frac{1964.71}{2}$$

$$\boxed{V = 982.35 \text{ kN}}$$

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Q. Design a deck type plate girder bridge for single track B.G. main line loading for following data.

→ Eff. span = 24m
→ Spacing of plate Girder = 1.9m c/c
→ Weight of stock Rails = 440 N/m
→ Weight of Guard Rail = 260 N/m
→ Weight of fastenings = 280 N/m of track
→ Size of sleepers (Timber) = 2.8 m × 250 mm × 150 mm @ 0.4 m
→ Density of timber = 7.4 KN/m^3

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✗ Checking the section for wind load →

→ When wind loads are considered in design then as per code IS : 800 - 1984, the permissible stresses are increase to 4/3 times. (i.e. increase by 33%.)

→ When bridges are design then this increase as per code is 25% (i.e. permissible stress is increase by 5/4 times.

Bridges		Building	
BIS	Railway Board	BIS	R.B.
25%	16.67%	33%	25%

⇒ Formula for wind force P_{w_1} or $P_{w_2} = P_w (1+K) h_1 L$

The factor K depending upon S/D ratio as per bridge rule.

S - spacing of girders c/c
D - overall depth of girder

S/D	K
< 1/2	0
1/2 to 1	0.25
1 to 1.5	0.5
> 1.5	1.0

→ Wind pressure P_w is taken as
= 1.5 kN/m² for bridge loaded
= 2.4 kN/m² for " unloaded.

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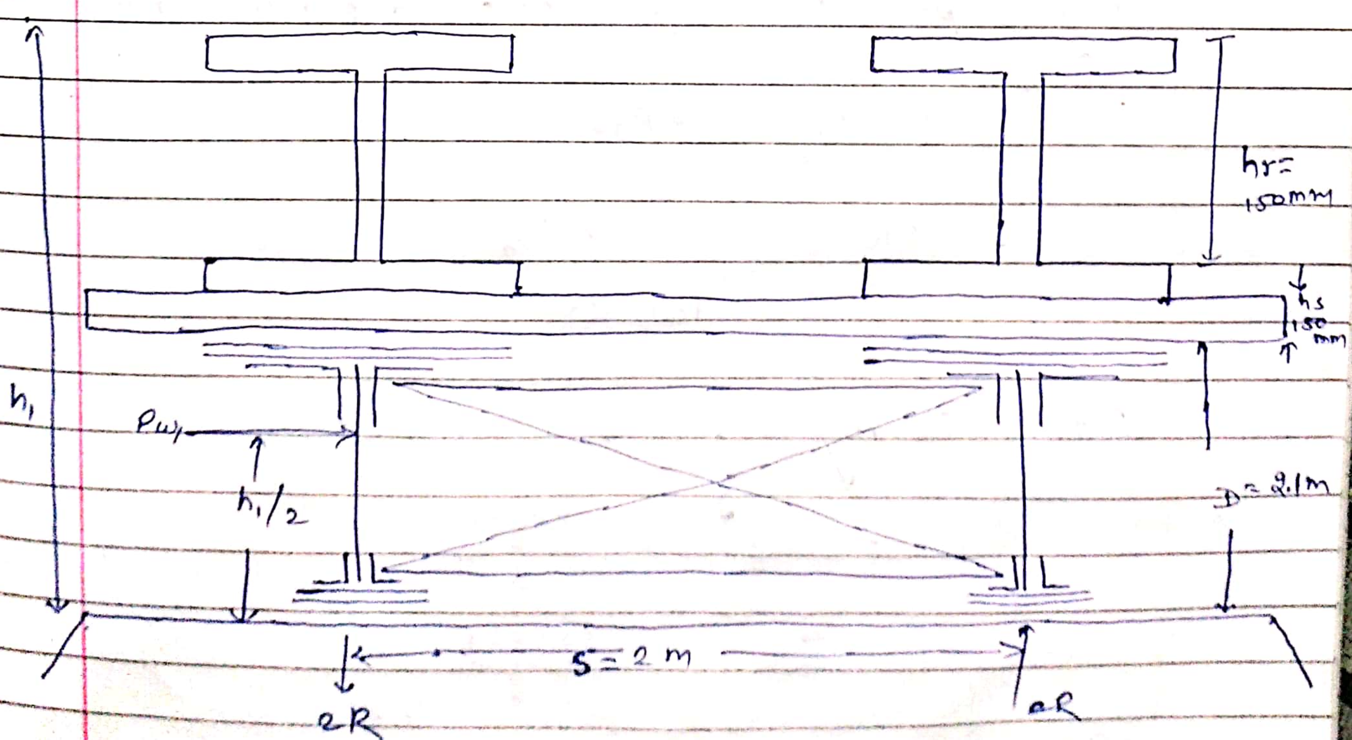
Q. Deter^e the increase in stresses in the flanges of ~~windward~~ leeward girder due to over turning effect of wind when
(i) Bridge is unloaded.
(ii) Bridge is loaded.
For a deck type plate girder railway bridge, B.G., from following data.

→ eff. span of bridge = 25 m
→ Spacing of P.G. c/c (s) = 2 m
→ Overall depth of the secⁿ of girder = 2.1 m
→ Height of Rail section = 150 mm = 0.15 m
→ Height of sleepers = 150 mm = 0.15 m.

Solⁿ (A) Bridge Unloaded :-



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For bridge unloaded wind pressure (P_w) as per bridge rule

$$P_w = 2.4 \ kN/m^2$$

$\rightarrow$ $h_1 = h_r + h_t + D_1$

$= 0.150 + 0.150 + 2.1$

$h_1 = 2.4 m$

and spacing of Girder c/c $S = 2m$.

And overall depth of P.G. $D = 2.1 m$

$$S/D = 2/2.1 = 0.95$$

for $S/D = 0.95$; $K = 0.25$

$$P_{w1} = P_w (1+K) h_1 L$$

$$= 2.4 (1+0.25) \times 2.4 \times 25$$

$$P_{w1} = 180 \ kN$$

this P_{w1} act $h_1/2$ distance from bottom

So,

$$P_{w1} \times \frac{h_1}{2} = 2R \times S$$

$$\Rightarrow 2R = \frac{P_{w1}}{S} \times h_1/2$$

$$= 108 \ kN$$

Extra Bending Moment

$$= \frac{wl^2}{8} = \frac{2R \times L}{8}$$

$$= \frac{108 \times 25}{8}$$

$$= 337.5 \ kN\text{-}m$$

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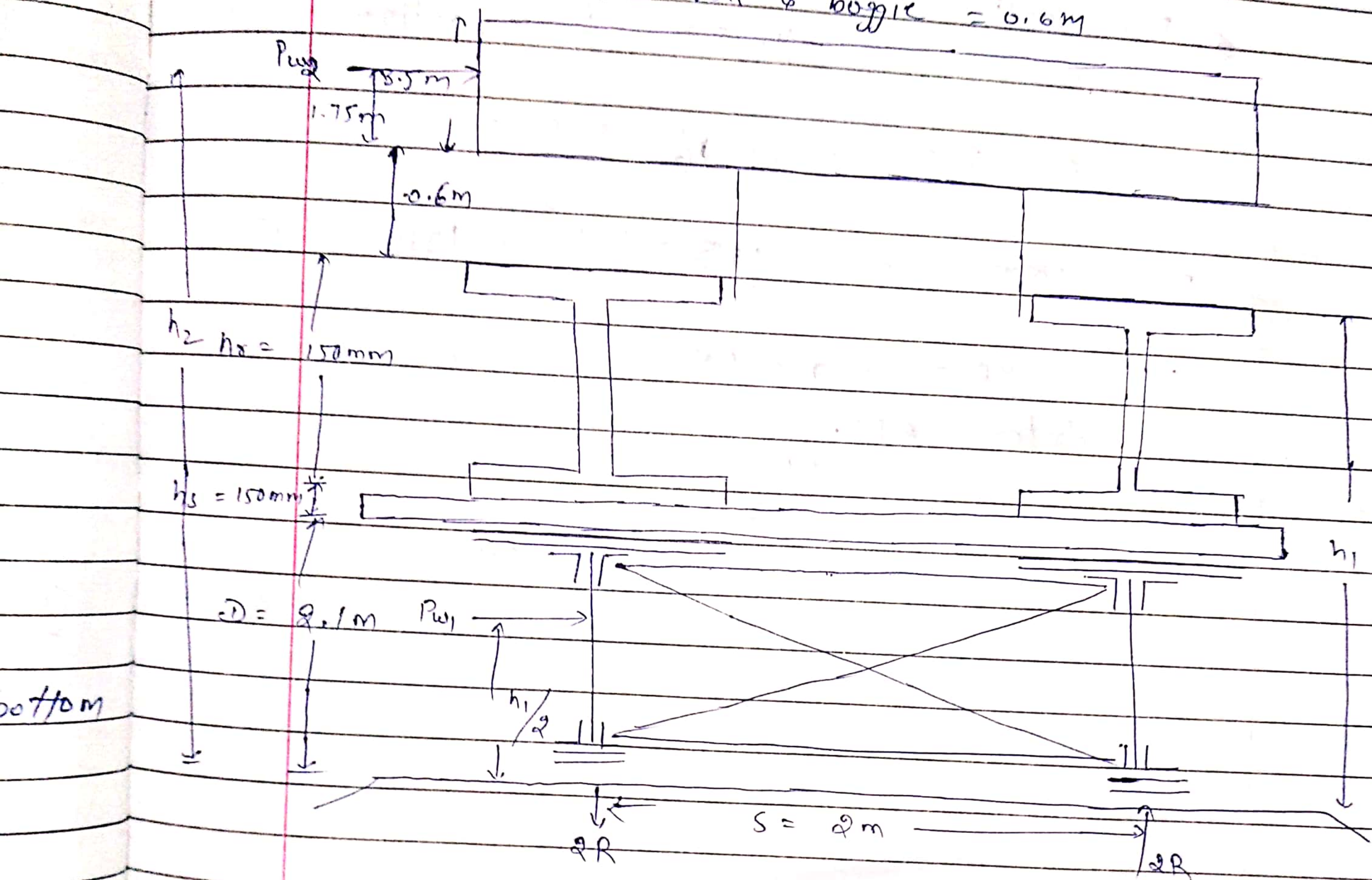
2.5", 0.6m

(B) Bridge loaded

Boggie Height = 3.5m

distance b/w rail & boggie = 0.6m



for bridge loaded, wind pressure as per bridge rule

$P_w = 1.5 \ kN/m^2$

→ let us assume the train occupies the whole span.

$P_{w_1} = P_w(1+k)h_1 L$

$P_{w_1} = 1.5(1+0.25) \times 2.4 \times 25$

$\boxed{P_{w_1} = 112.5 \ kN}$

Bridge loaded
$P_{w_2} = P_w \times h_b \times L$

$P_{w_2} = \cancel{P_w(1+k)} \ P_w \times h_b \times L$

$= 1.5 \times 3.5 \times 25$

$P_{w_2} = 131.25 \ kN$

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P_{w_1} acting at $h/2 = \frac{2.4}{2} = 1.2$ m from bottom. and

P_{w_2} acting at $2.1 + 0.15 + 0.15 + 0.6 + 1.75$
$= 4.75$ m from bottom

$$2R \times S = P_{w_1} \times h_1/2 + P_{w_2} \times h_2$$

$$2R \times 2 = 112.5 \times \frac{2.4}{2} + 131.25 \times 4.75$$

$$2R = 379.2 \text{ kN}$$

Extra B.M.

$$\frac{2R \times L}{8} = \frac{379.2 \times 25}{8}$$

$$= 1185 \text{ kN-m}$$

Sol^n

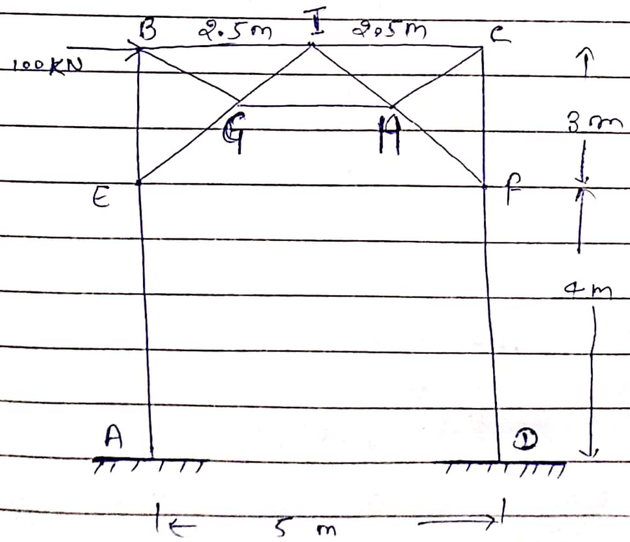
Unit - 4.

* T T T G R B (Through type truss Girder Railway bridge)

→ T T T G R B Diagram.

→ Difference between portal bracing and sway bracing used in T T T G R B

Q. An 'A'- type portal bracing has been used in a through type truss Girder bridge if it is subjected to a lateral horizontal force of 100 KN as shown in figure. Analyse the frame completely. Also find the portal effect in the bottom chord of the truss Girder if the end posts are inclined at 50° to the horizontal.



Sol^n Step I. Reaction

→ It is assumed that the points of contraflexure O' and O for the two end posts will be at the mid height of EA and FD respectively.

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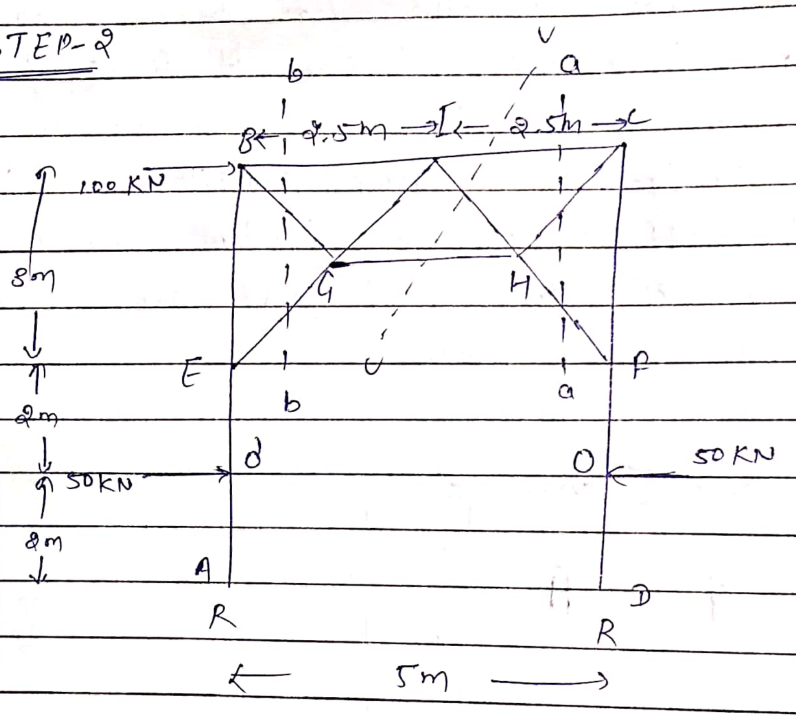
The horizontal shear at O' and O will be $\frac{100}{2}$
i.e. 50 KN each.
Vertical Reaction R is given by

[4 not taken because of point of contraflexure, i.e. mid of $4/2 = 2$.

$$R \times 5 = 100 \times (3+2)$$
$$\boxed{R = 100 \text{ KN}}$$

STEP-2



- Tension
+ compression

Forces in HF and EG.

$$\sin\theta = \frac{P}{H} = \frac{2.5}{\sqrt{2.5^2+3^2}} = 0.64$$

Pass a section a-a and take moment about C.

$$P_{HF} \times \sin\theta \times 3 = 50 \times 5 \quad (3+2)$$

$$P_{HF} = \frac{50 \times 5}{3 \times 0.64} = 130.21 \text{ KN (compression)}$$

$$\boxed{P_{HF} = 130.21 \text{ KN}}$$

Similarly

$$\boxed{P_{EG} = 130.21 \text{ KN}}$$ (Tension ~~compression~~)

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STEP-3 Forces in HC and BG.

Pass the section a-a and take the moment about point I. We get.

$$P_{HC} = M_1 / y_{HC}$$; where y_{HC} is perpendicular distance of H from I

$M_1 \rightarrow$ Consider the equilibrium of right hand posⁿ and take moment about I.

$$M_1 = 50 \times (3+2) - 100 \times 2.5$$
$$= 250 - 250$$
$$M_1 = 0$$
$$\Rightarrow \boxed{P_{HC} = 0}$$

and Similarly $\boxed{P_{BG} = 0}$

STEP-4 →

Forces in IH and GI

$$\boxed{\begin{aligned} P_{IH} &= P_{HF} = 130.21 \text{ KN (compression)} \\ P_{GI} &= P_{EG} = 130.21 \text{ KN (Tension)} \end{aligned}}$$

STEP-5 →

Forces in IC and IB

Pass a section a-a and take moment about point F.

$$P_{IC} \times 3 = 50 \times 2$$
$$P_{IC} = 100/3 \; \boxed{= 33.33 \text{ KN}} \text{ (Tension)}$$

Similarly, for IB →

Pass a section b-b and take moment about point E

$$P_{IB} \times 3 = 100 \times 3 + 50 \times 2$$
$$P_{IB} = \frac{400}{3} = 133.33 \text{ KN (compression)}$$

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STEP- 6

Force in GH →

Pass a section C-C and take moment about point I.

$$P_{GH} = \frac{M_I}{y_{GH}}$$

where y_{GH} = perpendicular distance of GH from I but $M_I = 0$, from step-7.

$\Rightarrow \boxed{P_{GH} = 0.}$

Step-7. moment at E, F, A, D.

$M_E = M_F = M_A = M_D = 50 \times 2$

$= 100 \text{ kN-m}$

Also $\boxed{M_B = M_C = M_I = 0}$

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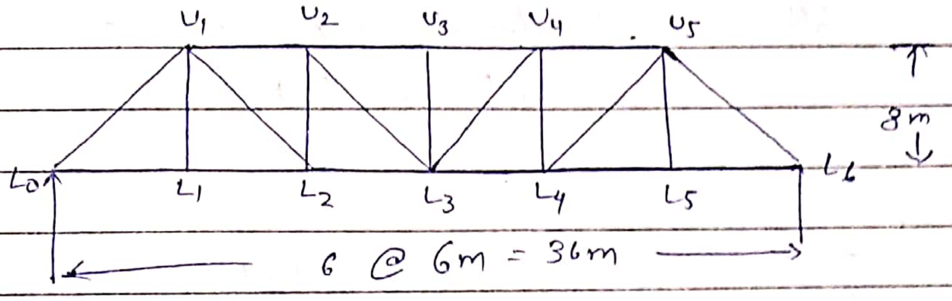
Q. Draw ILD for forces in member due U_2U_3 & L_2L_3 and U_1L_1 of a Pratt truss as shown in figure.



Solⁿ → ① Influence line for U_2U_3 →

$$P = \frac{x(L-x)}{Lh}$$

$$M = \frac{x(L-x)}{L}$$

$$P_{U_2U_3} = \frac{M_{L_3}}{h} \; ; \; M_{L_3} = \frac{18(36-18)}{36} = 9$$

$$P_{U_2U_3} = \frac{9}{8} = 1.125$$

② Influence line for L_2L_3

$$P_{L_2L_3} = \frac{M_{U_2}}{h}$$

$$M_{U_2} = \frac{12(36-12)}{36}$$

$$M_{U_2} = \frac{12 \times 24}{36} = 8$$

$$P_{L_2L_3} = \frac{M_{U_2}}{h} = \frac{8}{8} = 1$$

③ Influence line for $U_1 L_1$ →

When the unit load act at L_0

$P_{U_1L_1} = 0$

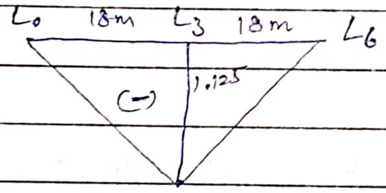
When unit load act at L_1

$P_{U_1L_1} = 1$

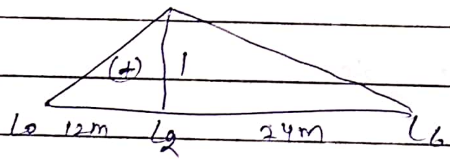
When unit load act at L_2

$P_{U_1L_1} = 0$

ILD → ① ILD for $U_2 U_3$



② ILD for $L_2 L_3$



③ ILD for $U_1 L_1$