BHARTIYA INSTITUTE OF ENGINEERING & TECHNOLOGY SIKAR

DEPARTMENT OF CIVIL ENGINEERING

LAB MANUAL

8CE6A : STEEL STRUCTURES DESIGN – II

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BHARTIYA INSTITUTE OF ENGINEERING & TECHNOLOGY SIKAR

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LAB MANUAL – STEEL STRUCTURES DESIGN - II (8CE6A)

CONTENTS

2014

Q. 1. Design a gantry girder for an Industrial building to carry an EOT crane from following data

Crane capacity = 150 kN

Wt of crane excluding trolley = 100 kN

Wt of trolley = ~~40 kN~~ 40 kN

Span of crane = 12 m

Span of GG = 7 m

Min approach of hook = 1 m

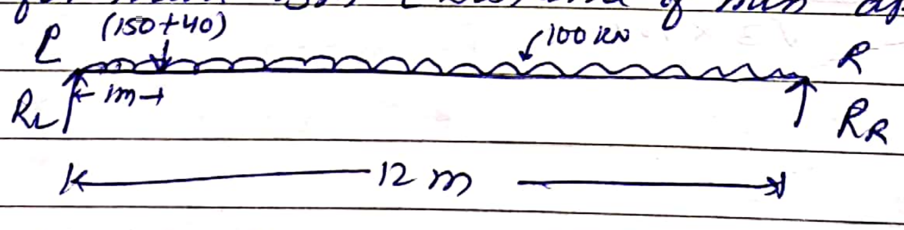
Wheel Base = 3 m.

Soln

BM and SF

A total wt of crane capacity and trolley i.e 150 kN + 40 kN (= 190 kN) is travelling on crane bridge

→ Its position should be nearest to any of the GG for max BM [Distance of min app of hook i.e. 1m]



Taking —— R

$$R_L \times 12 = 100 \times \frac{12}{2} + 190 \times 11 = 2690$$

$$\boxed{R_L = 224.16 \text{ kN}}$$

This load is divided into two wheel loads of equal amount i.e $= \frac{224.16}{2} = 112.08$ kN

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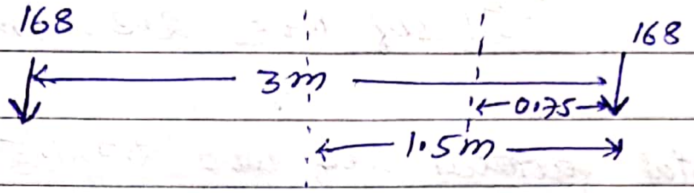
Factored value of each wheel load $= 112.08 \times 1.5$

$= 168.12\ kN$

$\simeq 168\ kN$

set of load is

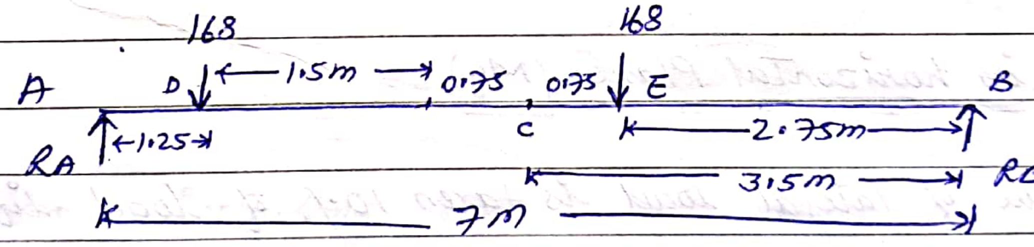


This set of load is moving on GG of span 7m. Their position should be such as to cause Absolute max BM in the GG.

Here, $0.586L = 0.586 \times 7 = 4.102m$

$4.102m > 3m$

For absolute max BM we have to apply TOS theorem. Acc. to theorem in "TOS" for absolute max BM, they should be so placed that their CG and the loads under as Mmemmax is occuring should be equidistance from the centre of span (GG)



Taking — A

$$R_B = \frac{168 \times 1.25 + 168 \times (3.5 + 0.75)}{7} = 132$$

$\boxed{R_B = 132\ kN}$

Absolute max BM occurs at E -

$M_E = 132 \times 2.75 = 363\ kN \cdot m$

$\boxed{M_E = 363\ kN \cdot m}$

Assume self wt of GG = 2 kN/m
assume self wt of rail = 0.3 kN/m

total self wt = 2.3 kN/m

Total factored self wt = 2.3×1.5
= 3.45 kN/m

Max BM due to self wt = $\frac{Wl^2}{8} = \frac{3.45 \times 7^2}{8}$

= 21.13 kN.m

Due to impact we take 25% allowance for EOT crane

Total BM including Impact = $363 \times 1.25 + 21.13$
= 474.88 kN.m

$\boxed{M_x = 474.88 \text{ kN.m}}$ BM in vertical Plane

BM in horizontal Plane (M_y)

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

$= \frac{10}{100} \times (150+40) = 19 \text{ kN}$

चूंकि there are 4 wheels in trolley
∴ lateral load per wheel = $\frac{19}{4} = 4.75$ kN

factored lateral load per wheel = $4.75 \times 1.5 = 7.125$ kN

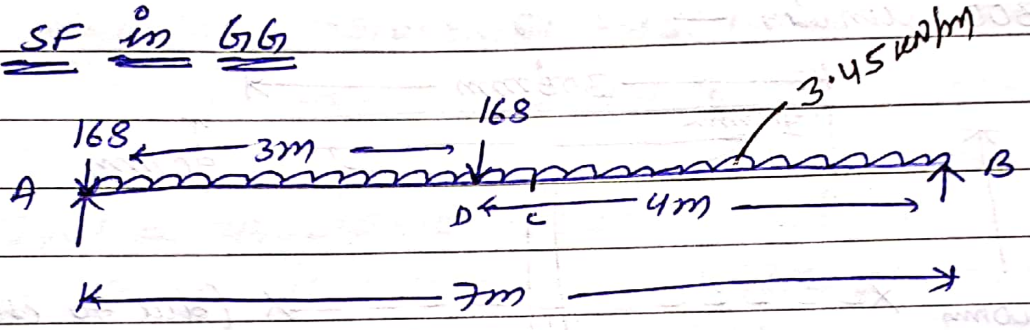
There position on GG for Max BM in horizontal plane will be same as for verticle load. It is found by proportion

$$\frac{M_y}{M_E} = \frac{f_y}{f_x}$$

$$M_y = \frac{f_y}{f_x} \times M_E = \frac{7.125}{168} \times 363$$

$$\boxed{M_y = 15.395 \text{ kN·m}}$$

Max SF in GG



Max SF will be at point A acc. to position of load

$$\text{Max SF} = R_A = \frac{(168\times7)+(168\times4)}{7} + \frac{\overset{3.45}{\cancel{4}}\times7\times7}{7\times2}$$

$$= 264 + \cancel{14}\ \ \cancel{12.25}$$

12.075

Max SF including impact 25% for EOT crane

$$= 264\times1.25 + 14$$

$$\boxed{\text{Max SF} = \cancel{344 \text{ kN}}} \quad 342.075 \text{ kN}$$

Trial section - guidelines

① Depth $\simeq \frac{L}{12}$, width $\simeq \frac{L}{30}$

② section choice is as per crane capacity

→ I section vary from ISWB 500 to ISWB 600

→ [" " " – ISMC 300 to ISMC 400

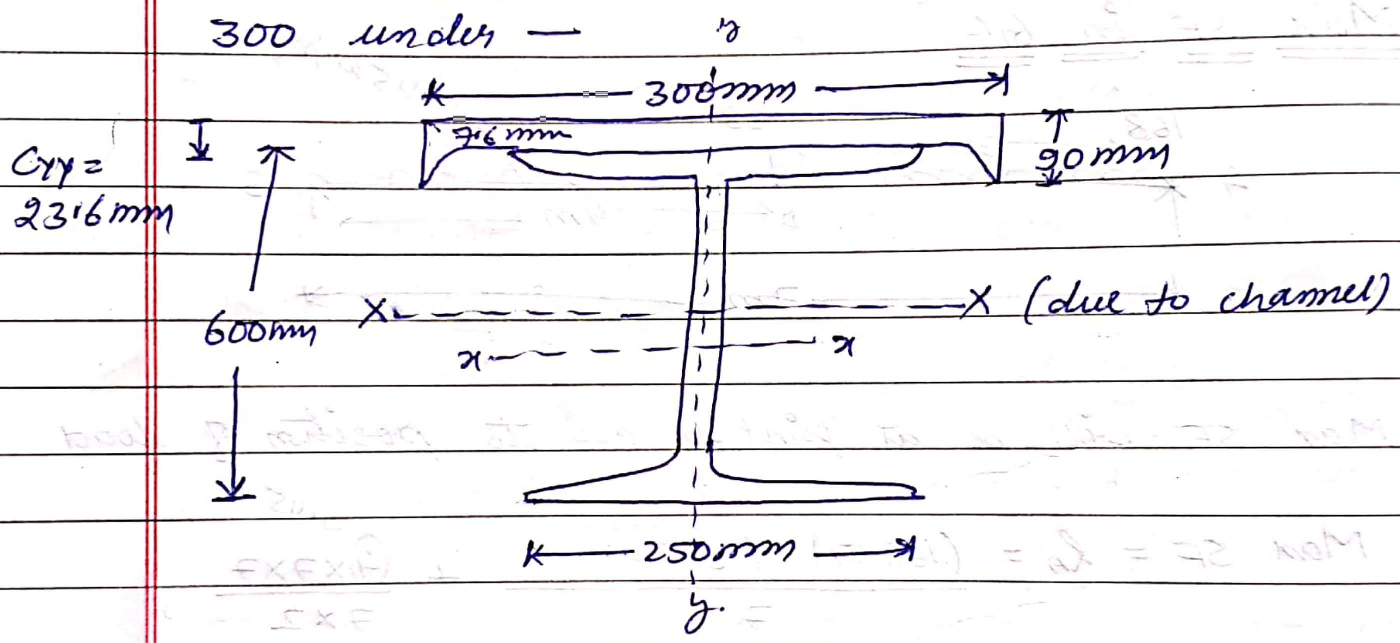
Take a pair (I, C section) to suit capacity ranging from 60 to 150 kN

Here,

$$\text{depth} = \frac{L}{12} = \frac{7000}{12} = 583.33 \text{ mm}$$

$$\text{width} = \frac{L}{30} = \frac{7000}{30} = 233.33 \text{ mm}$$

Let us try ISWB 600 @ 1.312 kN/m and ISMC 300 under —



	ISWB 600	ISMC 300
A	17038 mm²	4564 mm²
bf	250 mm	90 mm
tf	21.3 mm	13.6 mm
tw	11.2 mm	7.6 mm
Ixx	106198.5 cm⁴	6362.6 cm⁴
Iyy	4702.5 cm⁴	310.8 cm⁴
		Cyy = 23.6 mm

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

$$\bar{y} = \frac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2}$$

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

$$\boxed{\bar{y} = 360\,mm}$$

To find $I_{xx'} = 106198.5 \times 10^4 + 17038 \times (360-300)^2 + 310.8 \times 10^4 + 4564\ (607.6 - 360 - 23.6)^2$

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

$$I_{yy'} = 4702.5 \times 10^4 + 6362.6 \times 10^4$$
$$= 110651000\,mm^4 = 11065 \times 10^4\ mm^4$$

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

For compression flange about Y-Y axis

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

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

We should have intraction formula to be satisfied

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

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To calculate Mdz

Here, GG is laterally unsupported

$M_{dz} = Z_p \cdot f_{bd}$

for calculating Z_p

$\boxed{Z_p = \Sigma ay}$

Let EA axis lies at a distance Y_p from bottom

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

$5325 + 11.2 Y_p - 238.56 = 10801$

$Y_p \simeq 510 \, mm$

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

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

$Z_p = 4783247.528 \, mm^3$

$Z_p = 4783.25 \times 10^3 \, mm^3$

To calculate fbd

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

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 $= \frac{Kl}{r} = \frac{1 \times 7000}{71.57} = 97.81$

from IS 800:2007, pg-57 table 14

$\frac{Kl}{r}$	20	21	25
90	380.4		344.2
97.81		$f_{cr,b}$.	
100	325.8		291.4

$$f_{cr,b} = \frac{(25-21)(100-97.81)}{(25-20)(100-90)} \times 380.4 + \frac{(21-20)(100-97.81)}{(25-20)(100-90)} \times 344.2$$

$$+ \frac{(25-21)(97.81-90)}{(25-20)(100-90)} \times 325.8 + \frac{(21-20)(97.81-90)}{(25-20)(100-90)} \times 291.4$$

$= 203.06 + 15.076 + 203.56 + 45.516$

$= 467.21\ N/mm^2$

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

500	188.6
467.21	f_{bd}
450	186.4

$$f_{bd} = \frac{(188.6-186.4)(467.21-450)}{(500-450)} + 186.4 = 187.157\ N/mm$$

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

$= 4783.25 \times 10^3 \times 187.157$

$M_{dx} = 895.22$ kN·m

$M_{dx} > M_x$ (474.88 kN·m) OK

To Calculate Z_{py}

$Z_{py} = \frac{1}{4} bd^2$

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

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

$M_{dy} = Z_{py} \cdot \frac{f_y}{\gamma_{m0}} \not> 1.2 \frac{Z_{ey} f_y}{\gamma_{m0}}$

$= 824.76 \times 10^3 \times \frac{250}{1.1} \not> 1.2 \times 609.07 \times 10^3 \times \frac{250}{1.1}$

$= 187.44 \not> 166.11$ kN·m

The lesser value is 166.11 kN·m

$\boxed{M_{dy} = 166.11 \text{ kNm}}$

Interaction formula as per code

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

$M_x = 474.88$ kN·m

$M_{dx} = 895.22$ kN·m

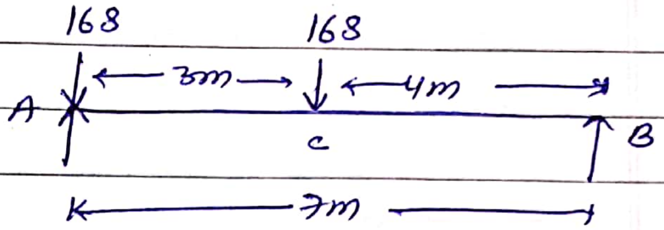
$M_y = 15.395$ kN·m

$M_{dy} = 166.11$ kN·m

$\therefore \dfrac{474.88}{895.22} + \dfrac{15.395}{166.11} < 1$

$= 0.62 < 1$ OK

Check for shear



$T \longrightarrow A$

$R_A = \dfrac{(168 \times 7) + (168 \times 4)}{7} = 264 \text{ kN}$

SF including Impact $= 264 \times 1.25 = 330$ kN

SF due to self Wt $= 3.45 \times \dfrac{7}{2} = 12.075$ kN

Total SF $= 330 + 12.075$

$= 342.075$ kN

OK

Design SF (Pg 59 IS 800:2007)

$V_d = \dfrac{A_v f_y}{\sqrt{3}\, \gamma_{m0}}$ → area of web

$= \dfrac{600 \times 11.2 \times 250}{\sqrt{3} \times 1.1} = 881.77 \text{ kN} > 342.075 \text{ kN}$

OK

Date:
www.mirajmulticolour.com

Q. 2.

Quest Design a welded PG of span 24m to carry super-imposed load of 35 kN/m. Avoid use of bearing and ITs. use fe-415 steel.

soln

$L = 24m$

$w = 35\ kN/m$

step① factored load = $35 \times 1.5 = 52.5\ kN/m$

step② Total factored load on PG = $W' = 52.5 \times 24 = 1260\ kN$

self wt of PG = $w' = \frac{W'}{200} = \frac{1260}{200} = 6.3\ kN/m$

total intensity of udl on PG = $52.5 + 6.3 = 58.8\ kN/m$

step③ Max BM (M) and Max SF (V)

$$M = \frac{wL^2}{8} = \frac{58.8 \times (24)^2}{8} = 4233.6\ kNm$$

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

step④ Depth of Web of PG

$$d_w = \sqrt[3]{\frac{MK}{f_y}}$$

Here ITs is avoided therefore $K = 67$

$$d_w = \sqrt[3]{\frac{4233.6 \times 10^6 \times 67}{250}} = 1042.99\ mm$$

$d_w = 1042.99\ mm$

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

$= \frac{24000}{8}$ to $\frac{24000}{12}$

$d_w = 3000\,mm$ to $2000\,mm$

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

step ⑤ Acc. to serviceability condⁿ, thickness of web plate

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

also

$K \geq \frac{d_w}{t_w}$

$t_w \geq \frac{d_w}{K} \geq \frac{1050}{67} \geq 15.67\,mm$

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

size of web plate = $1050\,mm \times 16\,mm$

step ⑥ Design of flange plate

App. area of ~~footing~~ flange

$A_f = \frac{M}{d_w\left(\frac{f_y}{\gamma_{mo}}\right)} = \frac{4233.6 \times 10^6}{1050 \times \left(\frac{250}{1.1}\right)}$

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

Assume section is plastic

$$\frac{b}{t_f} \leq 8.4$$

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

$$b_f = 16.8\, t_f$$

Also

$$A_f = b_f \times t_f$$

$$17740.8 = 16.8\, t_f^2$$

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

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

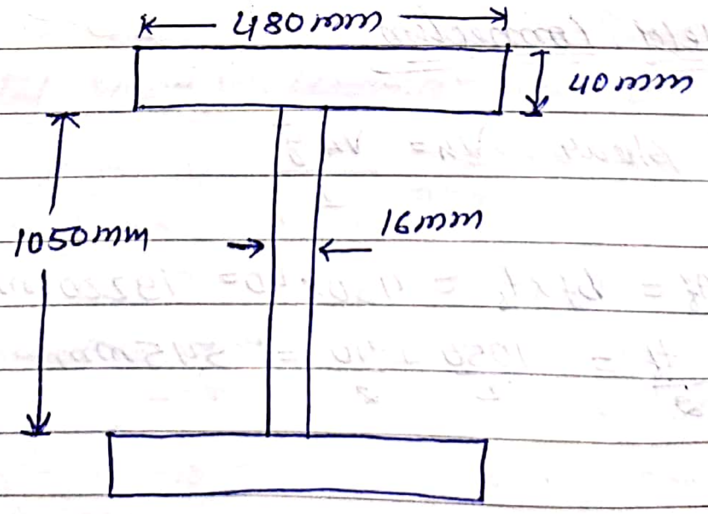
step ⑥ $b_f = 0.3 \times d_w = 0.3 \times 1050 = 315\text{ mm}$

also

$$b_f = \frac{A_f}{t_f} = \frac{17740.8}{40} = 443.5\text{ mm}$$

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

size of flange plate = 480mm x 40mm, therefore final section of PG is



step⑧ Design BM

$$\frac{b}{t_f} = \frac{b_f - t_w}{2\,t_f} = \frac{480-16}{2\times 40} = 5.83 < 8.4$$

section is plastic

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

$$Z_p = b_f\, t_f\,(d_w + t_f)$$

$$= 480\times 40\,(1050+40)$$

$$= 20928000 \text{ mm}^3$$

$$M_d = \frac{20928000\times 250}{1.1}$$

$$M_d = 4756.36 \text{ kN·m} > M\,(4233.6 \text{ kN·m}) \quad \text{OK}$$

step⑨ Design SF — 59.

$$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\,\gamma_{mo}} = \frac{1050\times 16\times 250}{\sqrt{3}\times 1.1}$$

$$V_d = 2204.42 \text{ kN} > V\,(705.6 \text{ kN}) \quad \text{OK}$$

Design of Weld Connection

step① Horizontal shear $V_h = \frac{V A \bar{y}}{I}$

here $A = A_f = b_f \times t_f = 480\times 40 = 19200 \text{ mm}^2$

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

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

$$= 1295438\times10^4 \ mm^4$$

$$V_h = \frac{705.6\times10^3\times19200\times545}{1295438\times10^4}$$

$$\boxed{V_h = 569.95 \ N/mm}$$

step ② Assume length of each intermittent weld = 75 mm
[min weld length = 40mm — pg 78 — 10.5.1.2]

Assume size of weld = 8mm ⟶ s = 8mm

strength of weld = $\dfrac{L_w\times t_t \cdot f_u}{\sqrt{3}\,\gamma_m}$ ($t_t = 0.7s$) ($\gamma_m = 1.25$)

$$= \frac{2\times75\times0.7\times8\times410}{1.25\sqrt{3}} = 159071 \ N \quad \text{①}$$

$$= 159 \ kN$$

step ③ Let a is the c-c spacing b/w two adjacent intermittent weld,

Horizontal shear @ distance 'a' = $569.95\times a$ N — ②

step ④ Equate eqn ② & ① $569.95\times a = 159071$

$$a = 279 mm$$

step ⑤ Clear spacing = $a - L_w$

$$= 279 - 75 = 204 mm$$

Acc. to pg — 79 — 10.5.5.2, the clear spacing b/w two adjacent intermediate weld should ≯ 12t or 200 mm so, $12t = 12 \times 16 = 192 mm$

<u>say</u> clear spacing = 190 mm

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

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Ques ③ Design a DT PG RB for single track BG main line loading for following date

(i) Eff span = 24 m

(ii) Spacing of plate girders = 1.9 m c/c.

(iii) Wt of stock rails = 440 N/m

(iv) Wt of guard rails = 260 N/m

(v) Wt of fastenings etc = 280 N/m of track

(vi) Size of sleepers (timber) = 2.8 m × 250 mm × 150 mm @ 0.4 m c/c.

(vii) Density of timber = 7.4 kN/m3

Permissible stresses as per railway steel bridge code.

soln Two plate girder at c/c distance of 2m will be provided to support the track.

→ Dead load of each girder from sleeper $= \frac{1}{2}\left[7.8 \times 0.25 \times 0.15 \times \frac{7.4}{0.400}\right]$

$= 0.97125$ kN/m.

→ DL on one girder from stock rail, guard rail and fasteners $= 440 + 260 + \frac{280}{2} =$ ~~980~~ ~~N/m~~ $840 = 0.84$ kN/m

Total DL on one girder $= 1.81125 \times 24 = 43.47$ kN — (1)

for self wt of plate girder, we must find EUDL and CDA

for 24m span and BG loading

For BM = 2280 kN

For SF = 2503 kN

CDA = 0.417

higher load value is 2503 kN

For one track on one girder, EUDL including impact

$= \frac{1}{2} \times 2503 \times 1.417 = 1773.38$ kN — (2)

Total DL + LL + IL on one girder

$W' = 43.47 + 1773.38$

$= 1816.85$ kN

→ Assuming self wt of one PG $= \frac{W'}{300} = \frac{1816.85}{300}$

$= 6.06$ kN/m

→ Total self wt of PG (one) = 6.06×24
$= 145.44$ kN —— (3)

↳ Let Wm is the total udl on one plate girder for BM and Wv is " " " " " " " " SF

Total load for BM on one PG

$$Wm = \frac{1}{2}(2280 \times 1.417) + (1) + (3)$$
$$= \frac{1}{2}(3230.76) + 43.47 + 145.44$$
$$= 1804.29 \text{ kN}$$

Total load for SF on one PG

$$Wv = (1) + (2) + (3)$$
$$= 43.47 + 1773.38 + 145.44$$
$$= 1962.29 \text{ kN}$$

⇒ Max BM on one PG

$$M = \frac{Wm\, L}{8} = \frac{1804.29 \times 24}{8} = \cancel{5586.87}\ 5412.87 \text{ kN.m}$$

$$V = \frac{Wv}{2} = \frac{1962.29}{2} = 981.145 \text{ kN.m}$$

step (3) Design of web plate

$$dw = \sqrt[3]{\frac{MK}{fy}} \qquad K=67$$

$$dw = 3\sqrt{\cancel{\frac{5412.87\times10^6\times67}{250}}} = \cancel{38087.39}\ mm \quad 1132.02\ mm$$

also

$$dw = \frac{L}{8} \text{ to } \frac{L}{12} = \frac{24000}{8} \text{ to } \frac{24000}{12} = 3000 \text{ to } 2000\ mm$$

adopt $\boxed{dw = 1150\ mm}$

$$tw \geq \frac{dw}{200} = \frac{1150}{200} = 5.75\ mm$$

also

$$tw = \frac{dw}{K} = \frac{1150}{67} = 17.16\ mm$$

adopt $\boxed{tw = 18\ mm}$

size of web plate = 1150 mm × 18 mm.

step② Design of flange plate

$$\text{App. area of footing} = Af = \frac{M}{dw\left(\frac{fy}{\gamma_{mo}}\right)} = \frac{5412.87\times10^6}{1150\left(\frac{250}{1.1}\right)}$$

$$= 20721.59\ mm^2$$

assume section is plastic

$$\frac{b}{tf} < 8.4$$

$$Af = 16.8\ tf^2$$

$$20721.59 = 16.8\ tf^2$$

$tf = 35 \simeq \boxed{tf = 40\ mm}$

$b_f = 0.3\, d_w = 0.3 \times 1150 = 345$ mm

also

$A_f = b_f \times t_f \rightarrow b_f = \frac{A_f}{t_f} = \frac{20721.59}{40} = \underline{518.04 \text{ mm}}$

adopt $b_f = 550$ mm

$\therefore$ size of flange plate $= 550$ mm $\times$ 40 mm

→ section classification

$\frac{b}{t_f} \leq 8.4 \rightarrow \frac{b_f - t_w}{2 t_f} = \frac{550 - 18}{2 \times 40} = 6.65 < 8.4$

Hence, section is plastic

→ Design Md

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

$Z_p = b_f t_f (d_w + t_f) = 550 \times 40 (1150 + 40)$

26180×10^3 mm3

$M_d = 26180 \times 10^3 \frac{\times 250}{1.1}$

$= 5950$ kN.m $> M$ ($\cancel{500}$ 5412.87 kN)

<u>OK</u>

→ Design Vd $\rightarrow$ Web.

$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\, \gamma_{m0}} = \frac{1150 \times 18 \times 250}{\sqrt{3} \times 1.1}$

$= 2716.17$ kN $> \underline{981.145 \text{ kN}}$

also

$V_d \rightarrow 0.6 V_d$

$= 0.6 \times 2716.17 = 1629.70$ kN

<u>OK</u>

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Q. 4.

First RTU 2014

Determine the increase in stresses in the flanges of leeward girder due to overturning effect of wind when

(a) bridge is unloaded

(b) bridge is loaded

For a deck Type PG RB, BG, from the following data

(i) eff. span of bridge = 25m

(ii) spacing of PG c/c = 2m

(iii) Overall depth of the section of girder = 2.1m

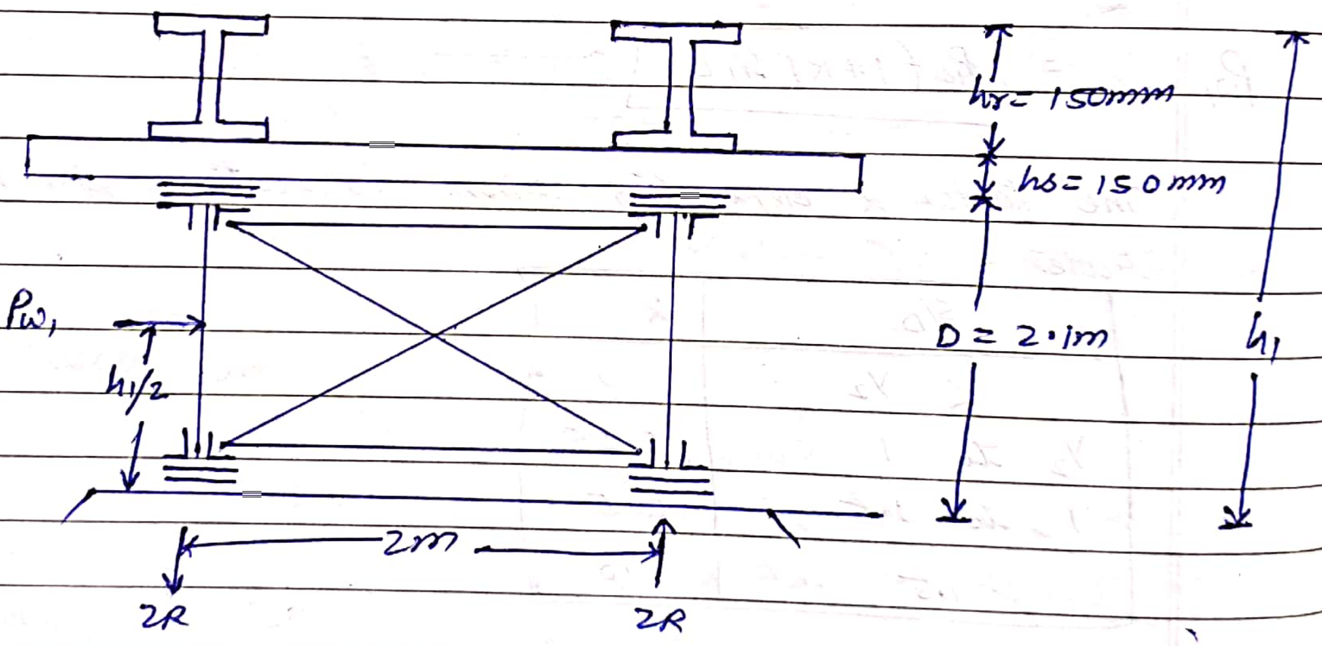
(iv) Height of rail section = 150mm

(v) Height of sleepers = 150 mm.

15/03/18
L-4

Soln

[A] BRIDGE UNLOADED



For bridge unloaded

Wind pressure $(P_w) = 2.4\ kN/m^2$

here, $h_1 = h_s + h_r + D = .150 + .150 + 2.1 = 2.4\,m$

and

spacing $s = 2m$

Now, $\frac{s}{D} = \frac{2}{2.1} = 0.95$

$\Rightarrow$ $\underline{K = 0.25}$ for $\frac{s}{D} = \frac{1}{2}$ to 1 i.e 0.95

$\rightarrow$ Wind force

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

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

$\boxed{P_{w_1} = 180\ kN}$

This P_{w_1} act $h_1/2$ distance from bottom

~~i.e~~ so

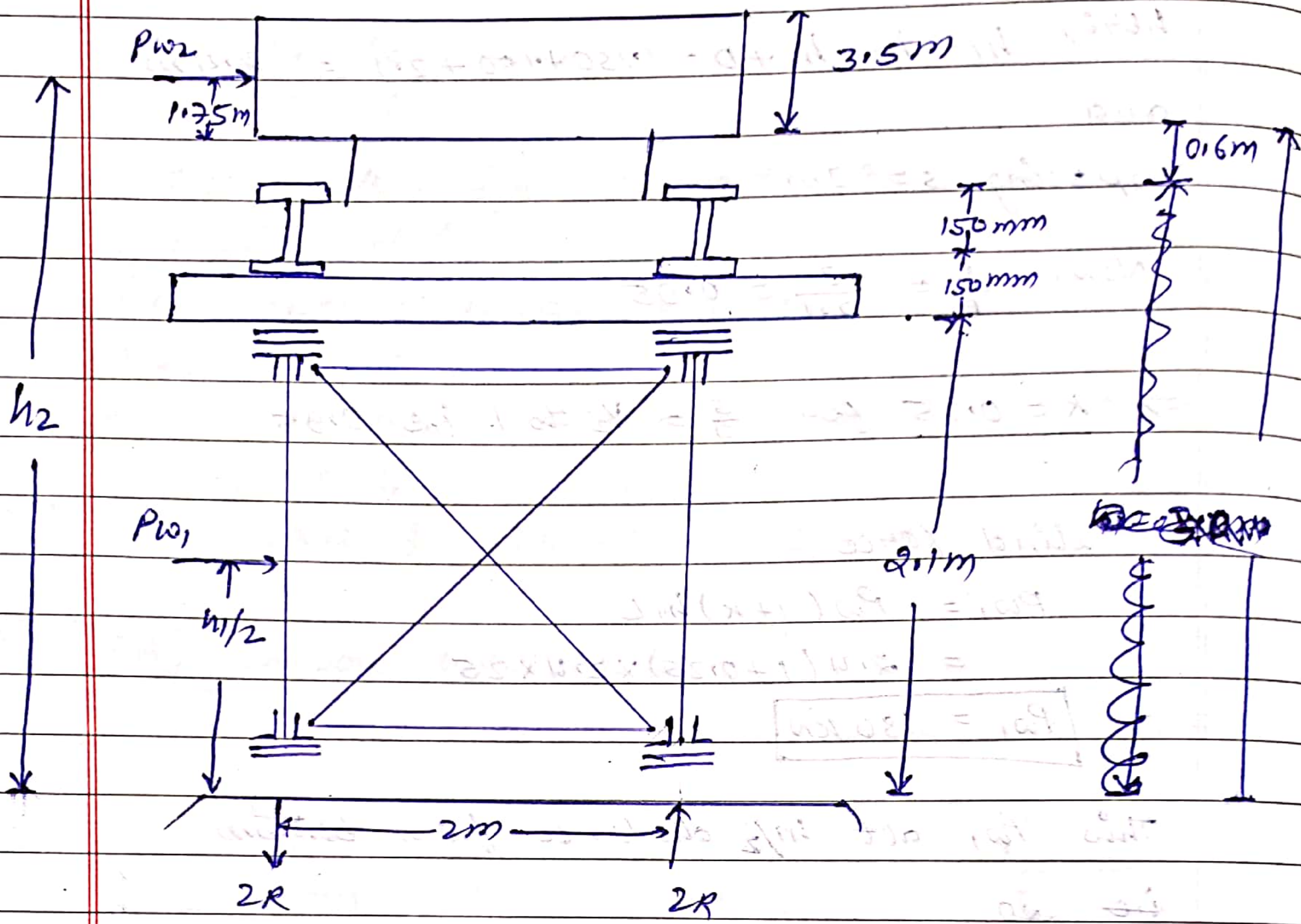
$P_{w_1} \times \frac{h_1}{2} = 2R \times s$

$180 \times \frac{2.4}{2} = 2R \times 2$

$\boxed{2R = 108\ kN}$

Extra $BM = \frac{(2R) \times L}{8} = \frac{108 \times 25}{8} = 337.5\ kN.m$

For [B] BRIDGE LOADED



For bridge loaded
wind pressure (P_w) = 1.5 kN/m²

let us assume the train occupies the whole span

$\Rightarrow P_{w_1} = P_w(1+K)h_1 L$

$= 1.5(1+0.25)\,2.4 \times 25$

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

($h_1 = 2.4$ for stable cond^n)

Now,

$$P_{w_2} = P_w \times \underbrace{(3.5 \times 25)}_{\text{area}} \quad \text{or} \quad P_w \times h_b \times L$$

$$= 1.5 \times 3.5 \times 25$$

$$\boxed{P_{w_2} = 131.25 \text{ kN}}$$

P_{w_1} acting at $\frac{h_1}{2}$ i.e. $\frac{2.4}{2} = 1.2m$ from bottom

P_{w_2} acting at $2.1 + 0.15 + 0.15 + 0.6 + 1.75$

$= 4.75 \, m$ from bottom.

So,

$$\left(P_{w_1} \times \frac{h_1}{2}\right) + \left(P_{w_2} \times h_2\right) = (2R) \times 5$$

$$= (112.5 \times 1.2) + (131.25 \times 4.75) = 2R \times 2$$

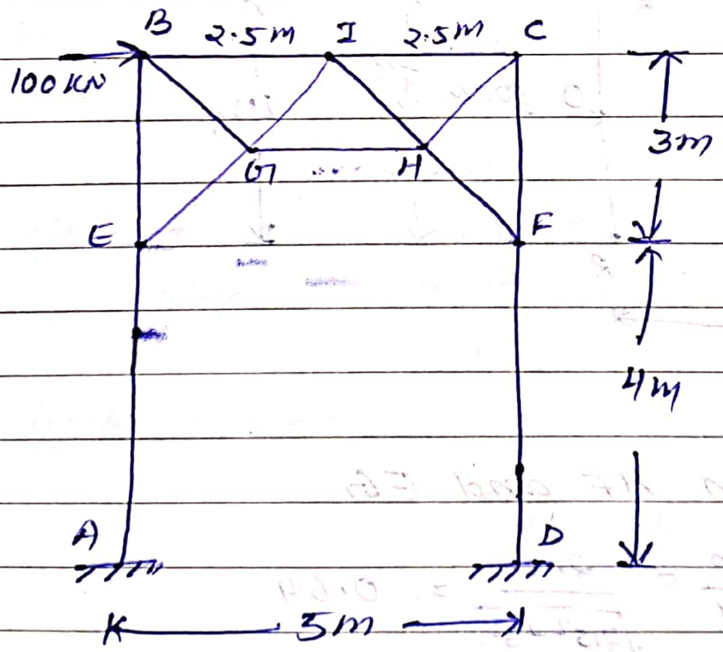
$$2R = 379.2 \text{ kN}$$

$$\text{Extra BM} = \frac{(2R) \times L}{8} = \frac{379.2 \times 25}{8} = \underline{\underline{1185 \text{ kN·m}}}$$

imp → Q. 5.

Quest 2014

An A-type portal bracing has been used in a through type truss girder bridge. 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 ~~pol chords~~ posts are inclined at 50° to the horizontal.



soln

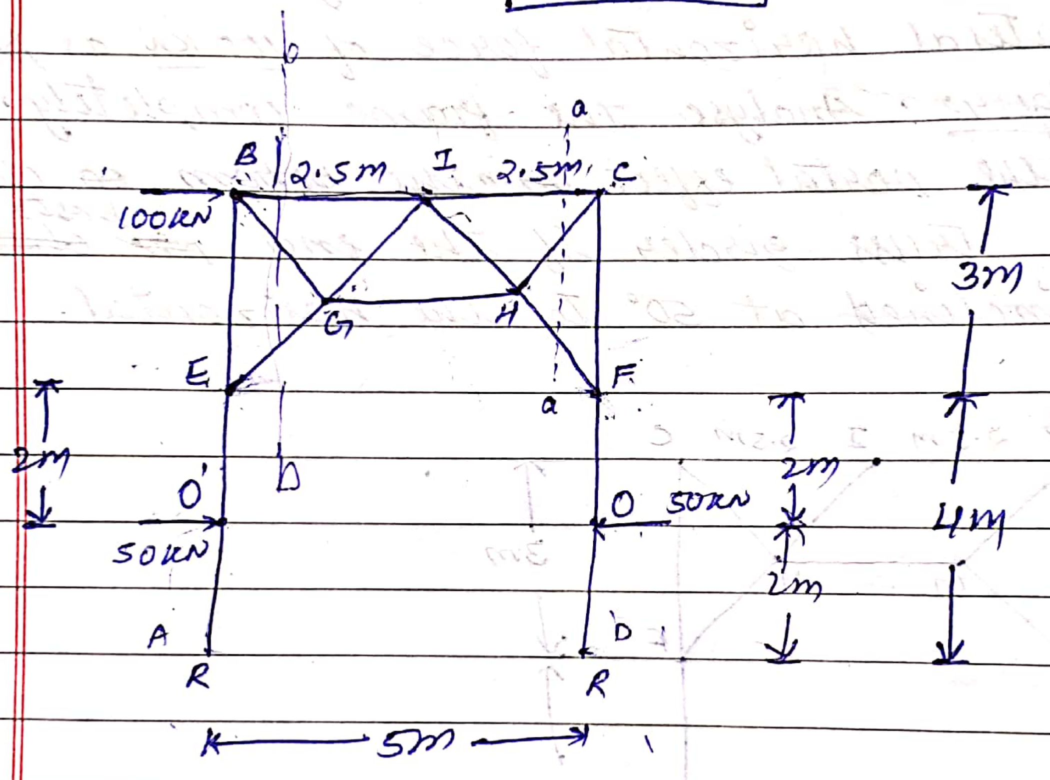
step ① Reaction

It is assumed that the points of contraflexure O' and O for the two end posts will be at the mid height of FD and EA respectively. The horizontal shear at O and O' will be 100/2 i.e 50 kN each.

The vertical reaction R is given by

$$R \times 5 = 100 \times (3+2)$$

$$\boxed{R = 100\ kN}$$



step ② 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

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

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

Similarly,

$P_{EG} = 130.21$ kN (tension) C.

17/03/2018
L-2

Step ③ Forces in HC and BG

Pass a section a-a and take the moment about I, we get

$P_{HC} = M_1 / Y_{HC}$, where Y_{HC} = perpendicular distance of HC from I.

Consider the eqlb of right hand position and take moments about I,

$M_1 = 50 \times (3+2) - 100 \times 2.5$

$\Rightarrow M_1 = 0$

Hence,

$P_{HC} = 0$, similarly $P_{BG} = 0$

Step ④ Forces in IH and GI

$P_{IH} = P_{HF} = 130.21$ kN (compression)

$P_{EG} = P_{GI} = 130.21$ kN (Tension)

step ⑤ Forces in IC and IB

Pass a section a–a and take moment about F

$P_{IC} \times 3 = 50 \times 2$

$\therefore P_{IC} = 33.33$ kN (Tension)

similarly pass a section b–b and Take moment about E

$P_{IB} \times 3 = 100 \times 3 + 50 \times 2$

$P_{IB} = 133.33$ kN (compression)

step ⑥ Force in GH

pass a section c–c and take moment about I

$P_{GH} = M_I / Y_{GH}$

where

Y_{GH} = perpendicular distance of GH from I

But $M_I = 0$ from step ③

Hence,

$P_{GH} = 0$

step ⑦ Moment at E, F, A, D.

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

$= 100$ kN.m

also,

$M_B = M_C = M_I = 0$

Q. 6.

Imp Design an overhead circular tank for capacity 2 lakh ltrs. It is supported on 6 columns uniformly spaced. Its bottom may be hemi-spherical.

Soln Given, ~~Velocity~~ volume = $200 m^3$, column = 6

step ① Dia and height of cylinderical portion

To find d and H.

$$\frac{H}{d} = 0.8$$

$$H = 0.8\,d$$

Volume of tank = $200 m^3$

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

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

$$200 = 0.62831853 d^3 + 0.261799387 d^3$$

$$200 = 0.890117917 d^3$$

$$\boxed{d = 6.08 m} \simeq \boxed{d = 6.1 m}$$

$(\because H = 0.8d$

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

step② thickness of plate

→ Thickness of cylinderical plate

$$t_{cyl} = \frac{WHd}{2\sigma_{at}\eta} = \frac{\cancel{9.81}\times\cancel{4.88}\times\cancel{6.1}}{\cancel{2}\times\cancel{120}\times\cancel{0.7}}$$

$$t_{cyl} = \frac{9.81\times10^{-6}\times4.88\times10^{3}\times6.1\times10^{3}}{2\times0.8\times0.6\times250\times0.70}$$

$\boxed{t_{cyl} = 1.74\,mm}$ but $\nless$ 6 mm

$t_{cyl} = 6\,mm$

→ Thickness of Spherical plate

$$t_{sph} = \frac{W(H+d/2)d}{4\sigma_{at}\eta} = \frac{9.81\times10^{-6}(4.88\times10^{3}+3.05\times10^{3})6.1\times10^{3}}{4\times0.8\times0.6\times250\times0.7}$$

$t_{sph} = 1.41\,mm \nless 6\,mm$

$\boxed{t_{sph} = 6\,mm}$

15/02/2018

L-4 step③ Conical Roof

provide 5 mm thick plate for conical roof.

Pitch may be taken as 1/4

$$pitch = \frac{rise}{span} = \frac{1}{4}$$

Scanned with CamScanner

$\Rightarrow$ rise $= \frac{1}{4} \times$ span $= \frac{1}{4} \times 6.1$

rise $= 1.525 m$

<u>Riveted Joint</u> - Let us provide 16mm dia bolt

$\rightarrow$ <u>Bolt strength in shear</u> $= \frac{\pi}{4} d^2 \tau_v$ permissible stress in shear

~~Vdsb = 1/ fub/√3 (nn Anb + ns Asb)~~

dia of bolt & 16 mm

$= \frac{\pi}{4} \times 18^2 \times 0.90 \times 100$ (fix)

(10% loss in field)

$= 22.90$ kN

$\rightarrow$ <u>strength in bearing</u> $= d \cdot t \cdot \sigma_b$ (fix due to 10% loss)

$= 18 \times 5 \times 0.9 \times 300$

$= 24.30$ kN

Rivet value = least value = <u>22.90 kN</u>

$\rightarrow$ Hoop ~~stress~~ force in vertical joint

$f_1 = \frac{WHd}{2} = \frac{9.81 \times 10^{-6} \times 4.88 \times 10^3 \times 6.1 \times 10^3}{2}$

$= 146$ N/mm

there are 2 rivets to take this hoop force

$\Rightarrow 146 \times P = 2 \times 22.90 \times 10^3$

$P = 313.7$ mm $\not> 10t$

minimum plate thickness

$\Rightarrow 10 \times 6 = 60$

$\therefore P = 60$ mm

Hence provide 60mm pitch throughout in circumferential joint also.

step ④ Ring girder

W = Total udl on ring girder

$\Rightarrow$ W = water + [cylinderical portion + spherical + conical portion] × 1.2 + self wt.

→ Water load = $200 \times \overset{(w \text{ i.e. } 9.81)}{10}$ = 2000 kN

Self wt of ring girder = 1.6 kN/m (assume)

→ Total self wt = $1.6 \times \pi \times d$

$= 1.6 \times \pi \times 6.1$

$= 30.66$ kN

→ volume of cylinderical portion = $\pi d t h$

$= \pi \times 6.1 \times 6 \times 4.88 \times 6 \times 10^{-3}$

$= \cancel{561.11}$ 0.56 m3

→ vol of spherical portion = $2\pi \times \left(\frac{d}{2}\right)^2 \times t_{sph}$

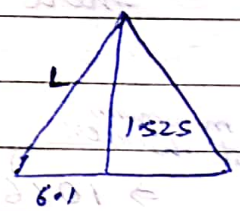
$= 2\pi \times \left(\frac{6.1}{2}\right)^2 \times 6 \times 10^{-3}$

$= 0.35$ m3

→ vol of conical portion = $\pi r l t$ (t = 5mm)

$= \pi \times \frac{6.1}{2} \times 3.41 \times 5 \times 10^{-3}$

$= 0.16$ m3



L = 3.41 m

wt of cylinderical + spherical + conical portion including fasterners and overlap

$= [(0.56 + 0.35 + 0.16] \times 1.2)] \times \underbrace{78.5}_{\text{wt of steel in kN}}$

$= 100.79$ kN

~~total wt = 2000 kN~~

total wt on Ring girder $= 2000 + 30.66 + 100.79$

$W = 2131.45$ kN

and $R = \frac{d}{2} = \frac{6.1}{2} = 3.05$ m.

From table, knowing W, R and no of columns we get values of M, V and T to which ring girder is subjected

~~Max SM~~ = Max SF $= \frac{W}{12} = \frac{2131.45}{12} = 177.62$ KN

Max BM at hogging $= 0.01482\, WR = 0.01482 \times 2131.45 \times 3.05$

$= \cancel{963.44 \text{ kN.m}}$

$= 96.34$ kN.m

Max Torsion (T) $= 0.00151\, WR = 0.00151 \times 2131.45 \times 3.05$

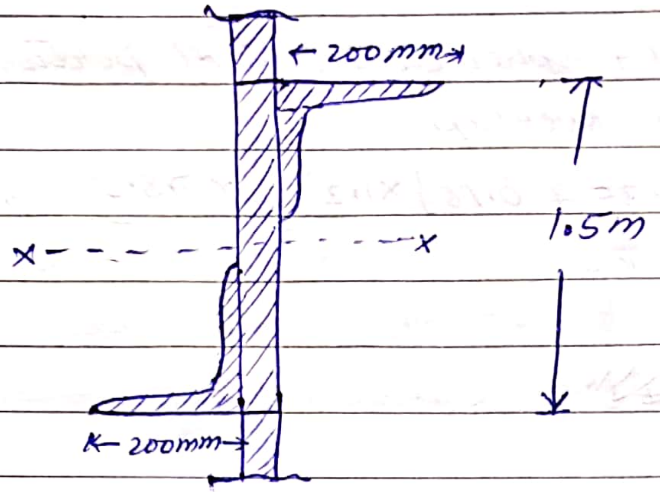
$= 9.82$ kN.m

16/02/2018
L-2

$Z_{req} = \frac{M}{0.66 f_y} = \frac{96.34 \times 10^6}{0.66 \times 250} = 583.87 \times 10^3$ mm3

from steel table

Let us try section of ring girder consisting of two angles, ISA 200x200x25 connecting as shown



(approx $\frac{d}{4} = \frac{6.00}{4} = 1.505$

$\simeq 1.50$

from steel table for ISA 200200,25

$A_{req} = 93.80\ cm^2$

$C_{xx} = 5.88\ cm$

$I_{xx} = 3436.3\ cm^4$

Check for shear –

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

$q_1 = 6.4\ N/mm^2$

shear stress due to torsion $q_2 = \frac{T}{J} t_{max}.$

$J = \frac{1}{3} bt^3$

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

$= \cancel{108000 +}$

$= 4014250\ mm^4$

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

$$q_2 = \frac{9.82 \times 10^6 \times 31}{4014250} = 75.83 \ N/mm^2$$

$\Rightarrow$ total shear stress $= 6.4 + 75.83$

$$q = 82.23 \ N/mm^2$$

and $\tau_{av} = 0.4 f_y$

$$= 0.4 \times 250 = 100 \ N/mm^2$$

$$q \ (82.23 \ N/mm^2) < \tau_{av} \ (100 \ N/mm^2)$$

OK

17/02/18

L-1 $\rightarrow$ check for Bending stress –

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

$\sigma_{bt} = 0.66 f_y$.

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

$$\sigma_{bt} = 0.66 f_y = 0.66 \times 250 = 165 \ N/mm^2 \quad \text{—} (1)$$

$I_{self} + Ah^2$

$\rightarrow$ ~~now~~ $I_{xx} = \frac{6 \times 1500^3}{12} + 2\left[3436.3 \times 10^4 + 9380 \ (750 - 58.8)^2\right]$

~~1071895.56 $\times 10^4$~~ $1071895.56 \times 10^4 \ mm^4$

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

$$= \frac{96.34 \times 10^6 \times 750}{1071895.56 \times 10^4} = 6.74 \ N/mm^2 \quad \text{—} (2)$$

from eq (1) and (2)

$$\sigma_{bt,cal} \ (6.74 \ N/mm^2) < \sigma_{bt} \ (165 \ N/mm^2)$$

OK

Q. 7.

Nov/Dec 2013

Design an elevated two tier RPST (rectangular pressed steel tank) having capacity 125K ltrs. Design the stays also and draw their arrangements. Show loads transfered to an intermediate top tier beam. Do not design the beam.

Soln

Fixation of dimension of the tank –

Given,

Capacity of the 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

$= 2 \times 1.25 m = 2.5 m$

Providing a free board of 0.15m, therefore eff depth available for water storage $= 2.5 - 0.15$

$= 2.35 m$

Base area of the tank

$$\frac{V}{h} = \frac{125}{2.35} = 53.19 m^2$$

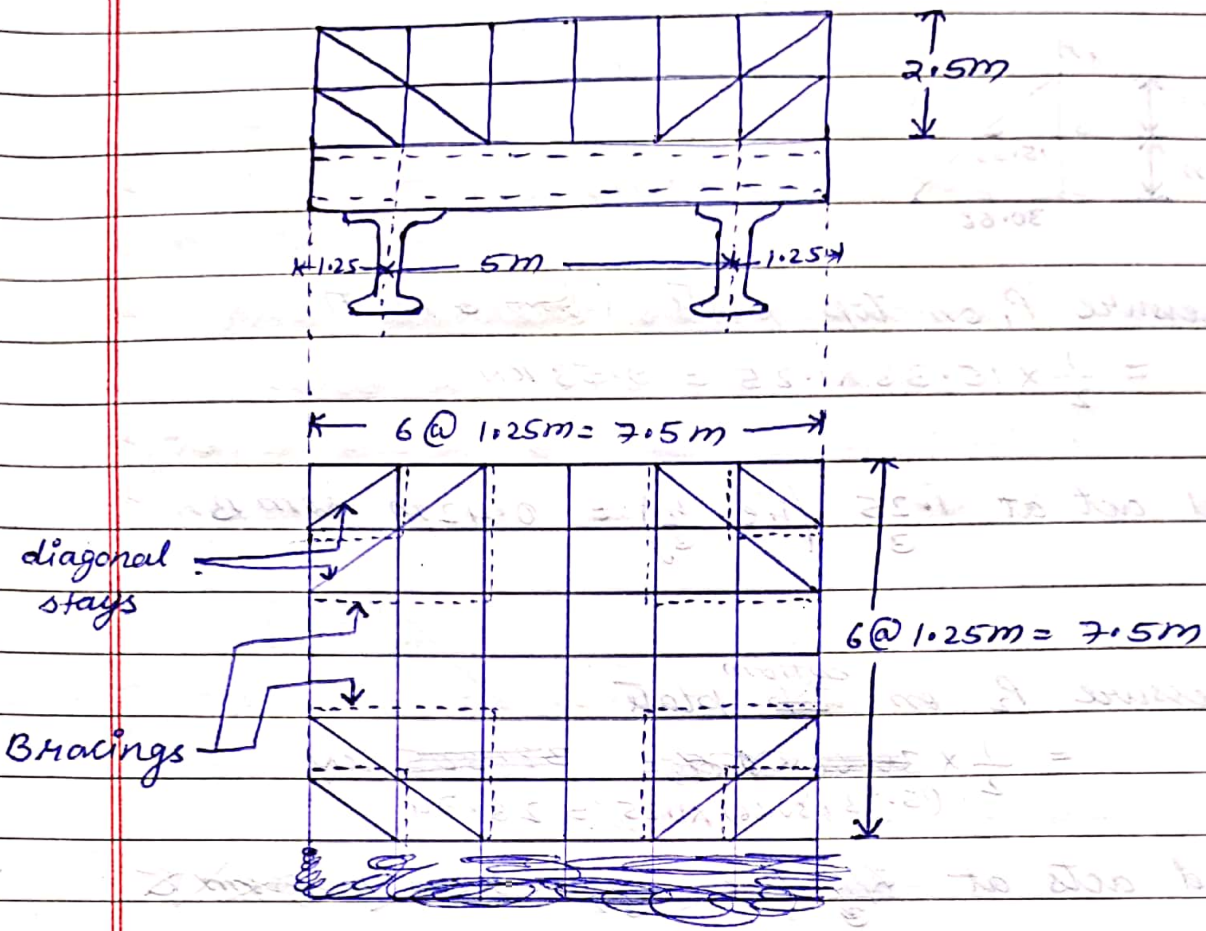
Proving square tank size

⇒ one side of the tank $= \sqrt{53.19}$

$= 7.29 m$

⇒ size of tank provided $= 7.50 m \times 7.50 m \times 2.5 m$

The tank will be supported on 4 columns spaced at 5m $[7.5 - (2 \times 1.25m)]$ c/c.



⇒ Thickness of plates as per IS:804

thickness of bottom plates = 6 mm

" " first tier plates = 6 mm

" " Top tier plates = 5mm

Design of stays –

- Assuming that water level upto top of the tank

Water pressure at 1.25m from top $= \gamma_w \cdot h_w$

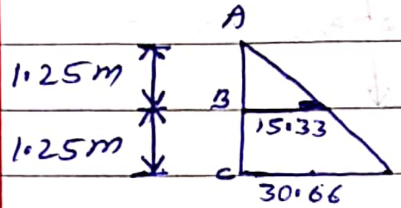
$= 9.81 \times 1.25 \times 1.25$

$= 15.33$ kN/m.

Water pressure at 2.5 m from top

$= 9.81 \times \cancel{2.5}^{1.25} \times \cancel{1.25}\ 2.5$

$= 30.66$ kN/m



Pressure P_1 on top plate (~~from B point~~)

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

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

Pressure P_2 on ~~top~~ bottom plate

$= \frac{1}{2} \times \cancel{30.66 \times 2.5} = \cancel{38.325}$ kN

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

and acts at ~~$\frac{2.5}{3}$ (i.e. $\frac{2h}{3}$) = 0.83 m from A~~

$\frac{2a+b}{a+b} \times \frac{h}{3} = \frac{2 \times 15.33 + 30.66}{15.33 + 30.66} \times \frac{1.25}{3} = 0.556$ m fr C

22/02/18

L-4

Reactions of upper plate

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

$R_{B_1} = \frac{2}{3} \times P_1 = \frac{2}{3} \times 9.58 = 6.386$ kN

Reaction of lower plate

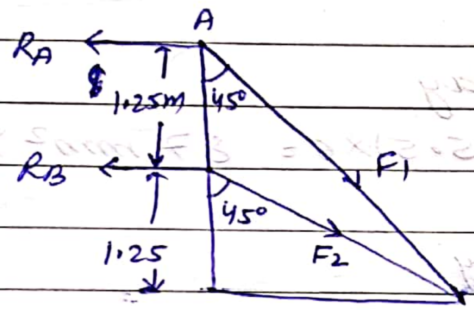
$$R_{B_2} = \frac{\bar{x}}{h} \times P_2 = \frac{0.556}{1.25} \times 28.74 = 12.783 \text{ kN}$$

R_c at bottom of tank, so not necessary

$\Rightarrow$ $R_A = 3.193$ kN

$R_B = R_{B_1} + R_{B_2} = 6.386 + 12.783 = 19.169$ kN

Forces in stays



Force F_1 in top stay

$F_1 \sin 45 = R_A$

$$F_1 = \frac{3.193}{\sin 45} = \frac{3.193}{1/\sqrt{2}} = 4.515 \text{ kN}$$

force F_2 in bottom stay.

$F_2 \sin 45 = R_B$

$$F_2 = \frac{19.169}{1/\sqrt{2}} = 27.109 \text{ kN}$$

Net area required for top stay

$$= \frac{F_1}{\sigma_{at}} = \frac{4.515 \times 1000}{0.8 \times 0.6 \times 250} = 37.625 \text{ mm}^2$$

Net area required for bottom stay

$$= \frac{F_2}{\sigma_{at}} = \frac{27.109 \times 1000}{0.8 \times 0.6 \times 250} = 225.908 \text{ mm}^2$$

Provide 30mm x 6mm mild steel flat for top stay and 60mm x 6mm M.S. plate for bottom stay and connect these by 14mm dia revet

for 14mm dia revet, hole dia = 15.5 mm

⇒

Design area of top stay

$$= (30 - 15.5) \times 6 = 87 \text{ mm}^2 > 37.625 \text{ mm}^2$$

design area of bottom stay

$$= (60 - 15.5) \times 6 = 267 \text{ mm}^2 > 225.908 \text{ mm}^2$$

OK