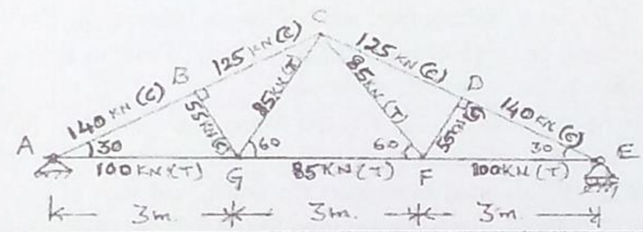
CMR INSTITUTE OF TECHNOLOGY	USN	1	C	R			C	V					CMR

Improvement Test

Sub:	DESIGN AND DRAWING OF STEEL STRUCTURES								Code:	10CV82
Date:	26/ 05 / 2017	Duration:	4 hours	Max Marks:	100	Sem:	VIII		Branch:	CIVIL

Note: Attempt any ONE full question from Part A and ONE question from Part B

		Marks	OBE	
			CO	RBT
	PART A			
1(a)	A cross beam ISLB350 @495N/m is connected to a main beam ISMB 500@869 N/m. The top of the flanges are at same level. The framed connection has the following details: i) Frame angle – 2ISA 150x115x10@200N/m. ii) The connection between the cleat angle leg of 115mm and web of the cross beam is 5mm fillet weld of length 250mm. The clearance between cross beam and web of main beam is 10mm. Draw to a suitable scale: i) Front view ii) Side view with all details	[15]	CIV802.1	L3
1(b)	A beam of ISMB 400 @ 61.6kg/m is connected to the flange of a stanchion ISHB 400@ 77.4 kg/m by a framed connections using 2 ISA 90x90x8mm angles. Five bolts of 20mm dia are used to connect the angles and web of ISMB 400 @ 61.6kg/m 12 bolts of 20mm dia are used to connect the angle and column ISHB 400@ 77.4 kg/m. Draw to a suitable scale i) Sectional plan ii) Any 2 views showing cross section details	[15]	CIV802.1	L3
2(a)	Draw to suitable scale the elevation and plan of the column splice having the following details: i) Bottom column: ISHB 300 @630N/m ii) Top column: ISHB 200 @400N/m ii) Splice plate : 8mm thick iv) Bearing plate : 50mm v) Use 8-20mm dia on each side of the joint in two rows of 4 bolts each for connecting flanges of the columns to flange splice plate. Draw to a suitable scale : i) Sectional elevation ii) Side view with details.	[15]	CIV802.2	L3
2(b)	A column base connection for axially loaded is to be made for the following data: i) Base plate – 900 x600 x25mm ii) Column – 1 no. ISHB 300, bf =300mm, tf=12mm, tw=7.5mm iii) Web cleat – 2 nos. ISA 80x80x8, 250mm long iv) Gusset plates – 2 nos. 900x600x12 mm (suitably tapered) v) Flange cleat – 2 nos. ISA 80x80x8, 900mm long vi) Bolts – 16mm ϕ Choose suitable pitch and number of bolts. Also show 4 – holding down bolts of 25mm ϕ. Draw the elevation (front), side view and plan (top view) of the above members connected suitably.	[15]	CIV802.3	L3

	PART B			
3	The centre line of a roof truss is shown in Figure below. The magnitude and nature of forces in each member under service load condition is shown against each member. Design top chord members, bottom chord members and interior members. Design the bearing plate to resist support reaction of 160kN. Also design the anchor bolts for a pull of 40kN to connect the truss to a RCC column 300 x 300 of M20 grade concrete. Use bolts of property class 4.6 for connections. Draw to a suitable scale. i) Elevation of the truss greater than half span. ii) Enlarged view of left hand support. iii) Enlarged view of joint C.  	[70]	CIV802.4	L3
4	Design a simply supported crane Gantry girder for the following data: i) Span of crane girder = 20m ii) Span of Gantry girder = 7m iii) Capacity of the crane = 220kN iv) Self weight of crane excluding the crab = 200 kN v) Weight of crab = 60kN vi) Wheel base distance = 3.4m vii) Minimum hook approach = 1.1m viii) Self weight of rail = 0.3kN/mm ix) Height of rail = 70mm Draw to a suitable scale: i) Plan details ii) Side Elevation iii) Section through the Gantry	[70]	CIV802.4	L3

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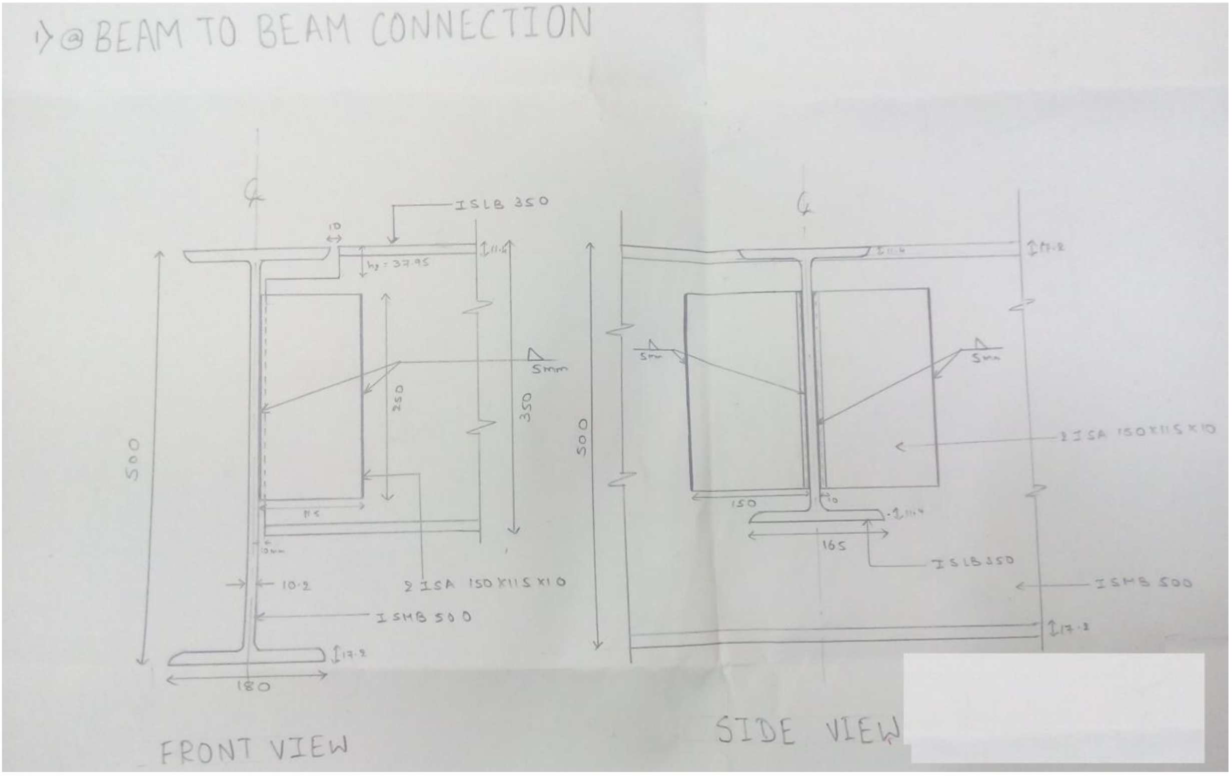
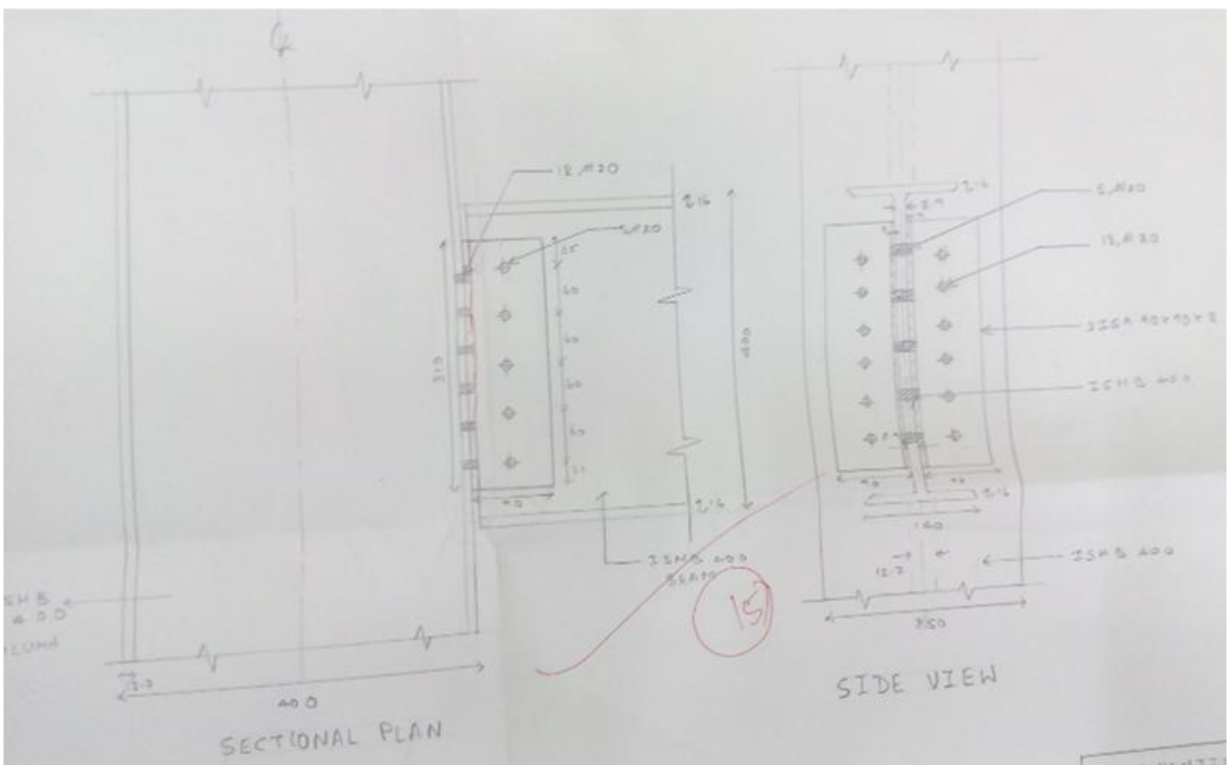
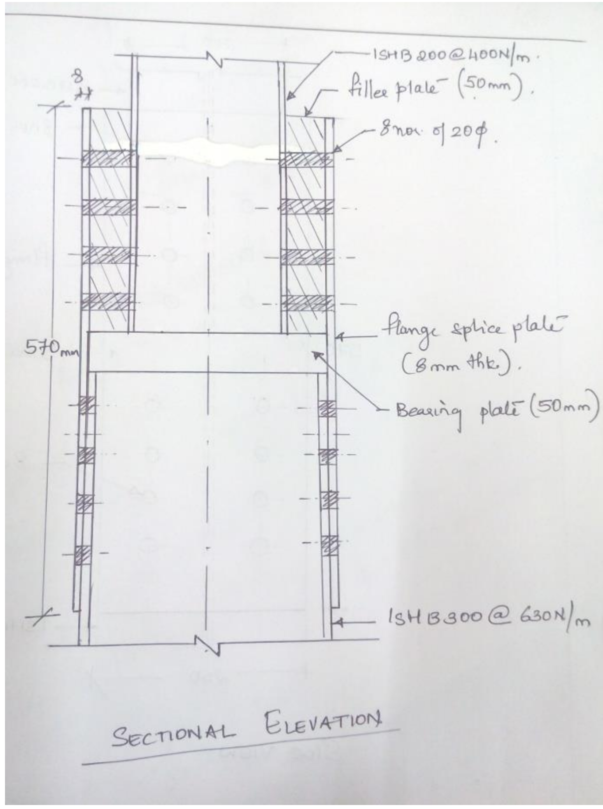
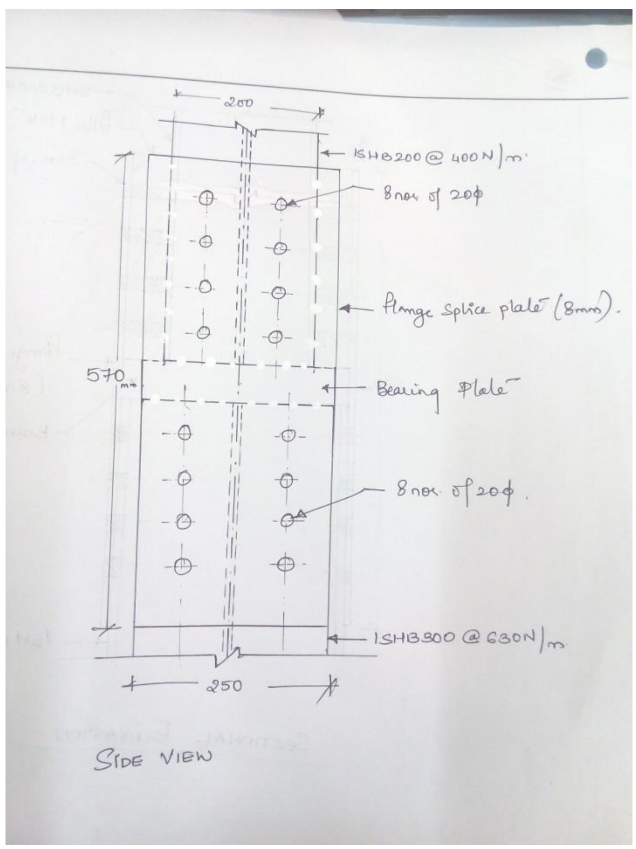


1) @ BEAM TO BEAM CONNECTION
ISLB 350
5mm
250
350
500
2 ISA 150 X 115 X 10
ISMB 500
10.2
180
FRONT VIEW
150
165
ISLB 350
ISMB 500
SIDE VIEW



12 M20
400
ISMB 400
400
SECTIONAL PLAN
140
ISMB 400
350
SIDE VIEW
15


ISHB 200 @ 400N/m.
filler plate (50mm).
8nos. of 20ϕ.
8
570mm
flange splice plate (8mm thk).
Bearing plate (50mm)
ISHB 300 @ 630N/m
SECTIONAL ELEVATION


200
ISHB 200 @ 400N/m.
8nos. of 20ϕ
flange splice plate (8mm).
570mm
Bearing plate
8nos. of 20ϕ.
ISHB 300 @ 630N/m
250
SIDE VIEW

4) Step 1 :-

Given -

span of crane girder = 20 m

span of Gantry girder = 7 m

capacity of crane = 220 kN

Self weight of cran = 200 kN

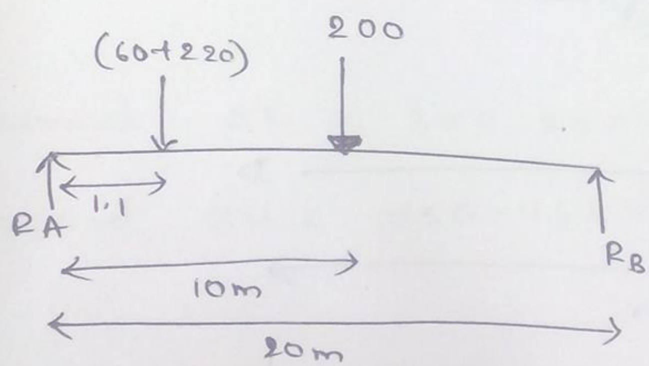
wt. of crab = 60 kN

wheel base = 3.4 m

Minimum hook distance = 1.1 m

Self wt. of rail = 0.3 kN/mm

Step 2 :-



$\Sigma V = 0$

$R_A + R_B = 60 + 220 + 200$

$R_A + R_B = 480$ —— (1)

$\Sigma M_A = 0$

$(280 \times 1.1) + (200 \times 10) - R_B \times 20 = 0$

$20 \times R_B = 2308$

$\therefore R_B = 115.4$ kN

$\Rightarrow R_A + R_B = 480$

$\therefore R_A = 364.6$ kN

$\Rightarrow$ wheel load $= \frac{364.6}{2} = 182.3$ kN

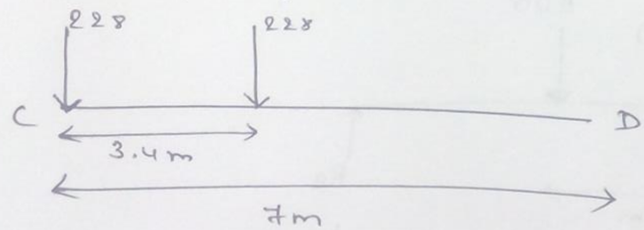
$\Rightarrow$ On considering Impact factor of 25%, we get the horizontal force as, $= 182.3 \times 1.25$

$\therefore$ wheel load $= 227.875$ kN

≈ 228 kN

Step 3:-

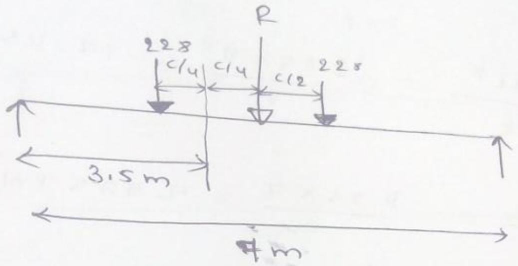
$\Rightarrow$ Consider gantry girder:-



$$\text{Max SF} = w\left[2 - \frac{c}{L}\right]$$

$$= 228\left[2 - \frac{3.4}{7}\right] = 345.25 \text{ kN}$$

⇒ In girder to have max BM, the wheels should be placed as shown below,



Max BM = $\frac{2w}{L}\left[\frac{L}{2} - \frac{c}{4}\right]^2$

$= \frac{2 \times 228}{7}\left[\frac{7}{2} - \frac{3.4}{4}\right]^2$

= 457.46 KNm //

⇒ Factored SF = 345.25×1.5 = 517.875 KNm //

Factored BM = 457.46×1.5 = 686.19 KNm //

⇒ Consider SF and BM due to dead load :-

Assume self wt. of = 1.6 KN/m

self wt. of rail = 0.3 KN/m

1.9 KN/m

Factored value = 1.9×1.5

$= 2.85$ kN/m

$$BM = \frac{wL^2}{8} = \frac{2.85 \times 7^2}{8} = 17.45 \text{ kNm}$$

$$SF = \frac{wL}{2} = \frac{2.85 \times 7}{2} = 9.975 \text{ kN}$$

$\therefore$ Total SF $= 9.975 + 517.875 = 527.85$ kN

Total BM $= 17.45 + 686.19 = 703.64$ kNm

Step 4:-

Lateral force = 10% of (60+220)

$= \frac{10}{100} \times 280$

$= 28$ kN

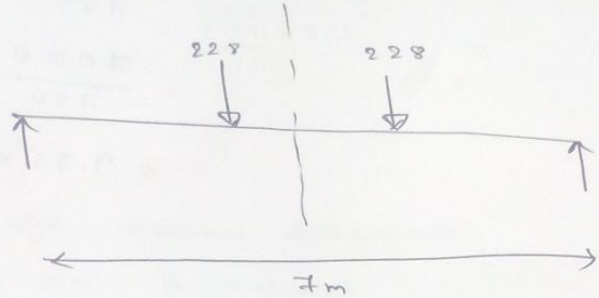
$\Rightarrow$ On each wheel $= \frac{28}{4} = 7$ kN

Moment due to lateral force $= \frac{7 \times 686.19}{228 \times 1.5}$

$= 14.04$ kNm

Step 5:-

Consider a trial section



WKT,

$$\delta max = \frac{W_c L^3 \left[\left(3a/_{4L} \right) - \left(a^3/_{L^3} \right) \right]}{6EI}$$

Where $a = \frac{L - C}{2}$

$= \frac{7000 - 34}{2}$

$= 1800$

$$\delta max = \frac{228 \times 10^3 \times 7000^3 \left[\left(\frac{3 \times 1800}{4 \times 7000} \right) - \left(\frac{1800^3}{7000^3} \right) \right]}{6EI}$$

WKT,

Max. permissible deflection $= \dfrac{span}{750}$

$= \dfrac{7000}{750}$

$= 9.33$ mm //

WKT,

Max. permissible deflection = Deflection due to self weight + δ_{max}

$$9.33 = 1 + \frac{1.146 \times 10^{10}}{I}$$

NOTE

Assume Δ due to self wt = 1mm

$$\frac{1.146 \times 10^{10}}{I} = 8.33$$

$$\therefore I = 1.37587 \times 10^{9} \text{ mm}^4$$

On Increasing I value by 30%, we get,

$$\therefore I = 1.78853 \times 10^{9} \text{ mm}^4 \quad //$$

$$= 178853 \times 10^{4} \text{ cm}^4 \quad //$$

$$\Rightarrow \delta_{max} = \frac{1.3752\times10^{16}}{6EI}$$

$$= \frac{1.146\times10^{10}}{I}$$

Step 6:

Based on I value,

deciding a section,

[from Pg:- 42 Steel table

ISWB 500,

ISMC 350

Top plate = 250 × 16

Bottom plate = 320 × 32

Area = 31728 mm^2

C_{xx} = 285.2 mm

Mean thickness { top flange = 30 mm, Bottom flange = 43.5 mm }

e_{xx} = 270.9

I_{xx} = 184963.8 × 10^4 cm^4

= 1.849638 × 10^9 mm^4

r_{yy} = 8.66

$\Rightarrow$ ISMC 350

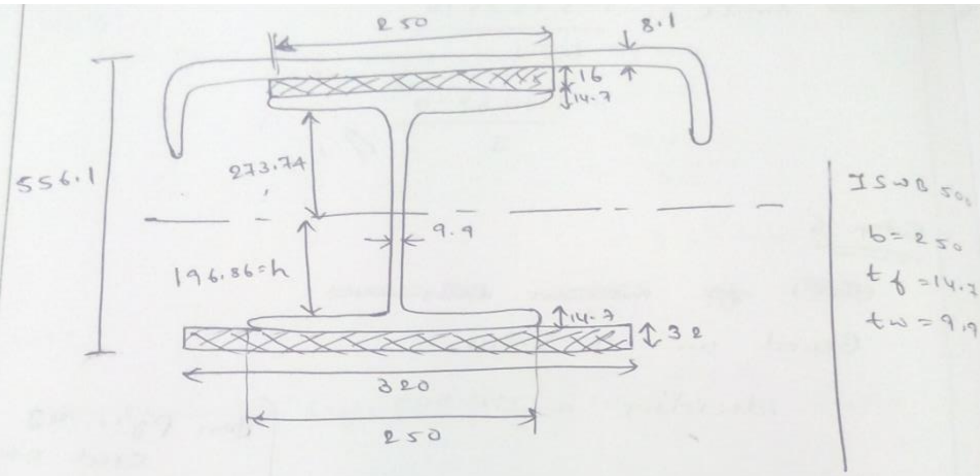
A = 5366 mm^2

t_f = 8.1

r_{min} = 28.3 mm

C_{yy} = 24.4

[from Pg:- 1[illegible] SP-6



Area of shaded portion $= \frac{1}{2}$ [Total area]

$$(h \times 9.9) + [250 \times 14.7] + [320 \times 32] = \frac{31728}{2}$$

$$(h \times 9.9) + 3675 + 10240 = 15864$$

$$\therefore h = 196.86 \text{ mm}$$

WKT, $Z_P = \Sigma(a\bar{y})$

$$\Rightarrow [(320 \times 32) \times 227.56] +$$

$$[(250 \times 14.7) \times 204.21] +$$

$+ \left[(196.86 \times 9.9) \times 98.43\right] +$

$\left[(273.74 \times 9.9) \times 136.87\right] +$

$\left[(250 \times 14.7) \times 281.09\right] +$

$\left[(250 \times 16) \times 296.44\right] +$

$\left[5366 \times 288.14\right]$

$(8.1 + 16 + 14.7 + 273.74) - Cyy$
288.14

$\therefore Z_P = (2330214.4 + 750471.75 + 191831.60 + 370921.25 + 1033005.75 + 1185760 + 1546159.24)$

$= 7.408 \times 10^6 \ mm^3$ //

Step 7:-

Check for moment resistance.

$M_d = \beta_b \times Z_P \times f_{bd}$

From IS-800
Pg:- 54
CN → 8.2.2

But,

$$f_{crb} = \frac{1.1 \times \pi^2 \times E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

$\Rightarrow L_{LT} = 7000$ mm

$E = 2 \times 10^5$

$r_y = r_{min} = 86.6$

$h_f = 556.1 - \frac{1}{2}[30 + 43.5] = 519.35$ mm

$t_f = 30$

$$\therefore f_{crb} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(7000/86.6)^2}\left[1 + \frac{1}{20}\left[\frac{7000/86.6}{519.35/30}\right]^2\right]^{0.5}$$

$$= 332.3243\left[1 + \frac{1}{20}(4.669)^2\right]^{0.5}$$

$$= 332.3243\,[1 + 1.090]^{0.5}$$

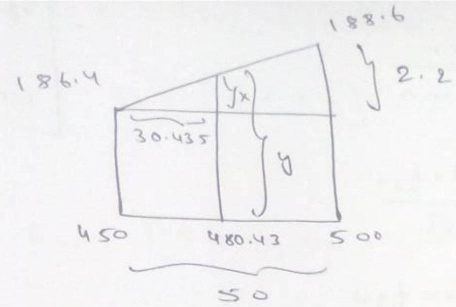
$$= 480.435$$

we can get value of f_{bd} from → [from Pg:- 55 IS-800 Table 13 (a)]

500 ⟶ 188.6

480.435 ⟶

450 ⟶ 186.4



$$\frac{x}{30.435} = \frac{2.2}{50}$$

$x = 1.33914$

$\therefore y = 186.4 + 1.33914$

$\therefore f_{bd} = 187.7391$ //

⇒ On substituting in,

$M_d = \beta_b \times Z_p \times f_{bd}$

$= 1 \times 7.408 \times 10^6 \times 187.7391$

$= 1390.77 \times 10^6 > 703.64 \times 10^6$

[Hence safe]

Step 8 :-

Check for shear resistance

$$V_d = \frac{V_n}{\gamma_{m0}}$$

[from IS-800 Pg :- 59 CN - 8.4.1]

But, $V_n = \frac{A_v f_{yw}}{\sqrt{3}}$

$$\therefore V_d = \frac{A_v \times f_{yw}}{\sqrt{3} \times \gamma_{m0}}$$

$$= \frac{4658.94 \times 250}{\sqrt{3} \times 1.1}$$

$$= 611.32 \text{ kN} > 527.85 \text{ kN}$$

[Hence safe].

$A_v = h \times t_w$

$= [500 - (2 \times 14.7)] \times 9.9$

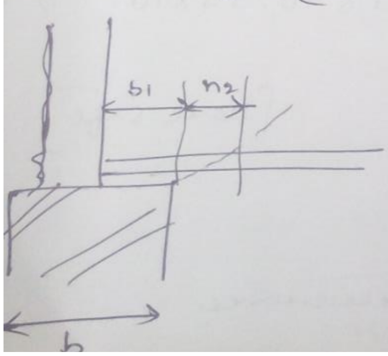
$= 4658.94 \text{ mm}^2$

Step 9:-

Check for web cripling

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

[from IS-800 Pg :- 67 CN - 8.7.4]



$b_1 = b/2 = 300/2 = 150$ mm

$n_2 = 2.5\,[t + t_f] = 2.5\,[32 + 14.7]$
$= 116.75$ mm

$\therefore\ F_w = (150 + 116.75) \times 9.9 \times \frac{250}{1.1}$

$= 600.18$ kN > 527.85 kN

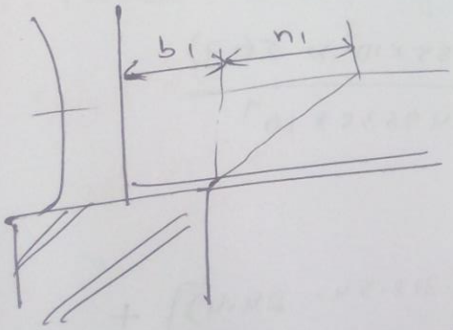
(Hence safe)

Step 10:-

Check for web buckling

$\Rightarrow F_w = (b_1 + n_1)\, t_w \times f_{cd}$

[from IS-800 Pg:-67 CN - 8.7.4]



$b_1 = b/2 = 300/2 = 150$ mm

$n_1 = e_{xx} = 270.9$ mm

$t_w = 9.9$

r_{min} → [from steel table Pg:-14]

$\Rightarrow$ To find f_{cd},

$\frac{KL}{r_{min}} = \frac{0.7 \times 470.6}{49.6} = 6.64$

$\therefore f_{cd} = 227\ N/mm^2$ // [from IS-800 Table 9(c) Pg:-42]

⇒ On substituting,

we get,

$F_w = (150 + 270.9) \times 9.9 \times 227$

$= 945.8\ kN > 527.85\ kN$

(Hence Safe)

Step 11:-

Connection details:-

⇒ Horizontal force $= \dfrac{V \times (\Sigma a\bar{y})}{I_z}$

$= \dfrac{527.85 \times 10^3 \times \Sigma(a\bar{y})}{1.849638 \times 10^9}$

$\Sigma(a\bar{y}) = [5366 \times (312.54 - 24.4)] +$

$[(250 \times 16) \times (312.54 - 8.1 - \frac{16}{2})$

$$= (1546159.24) + [1185760]$$

$$= 2731919.24$$

$\therefore$ Horizontal force $= \dfrac{527.85 \times 10^3 \times 2731919.24}{1.849638 \times 10^9}$

$$= 779.63 \text{ kN}$$

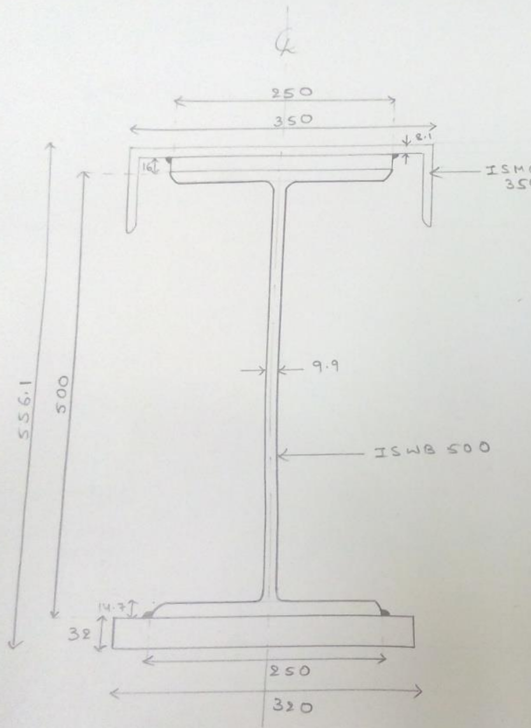
$\Rightarrow$ WKT,

Strength of weld $= \dfrac{0.7 \times S \times l \times f_u}{\sqrt{3}}$

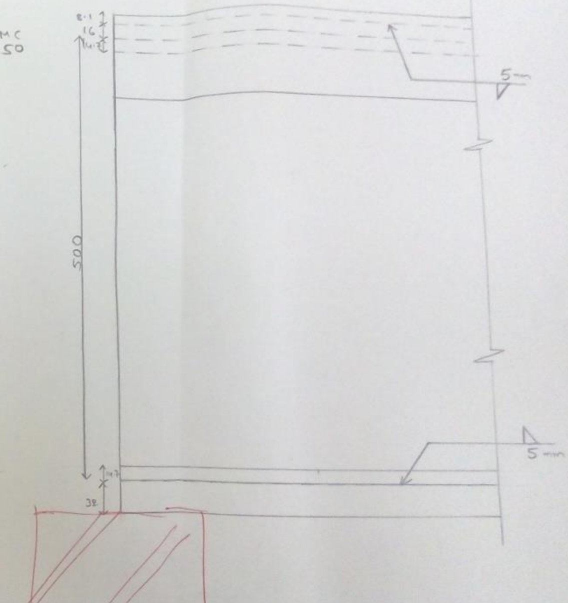
$$779.63 = \frac{0.7 \times S \times 1 \times 410}{\sqrt{3}}$$

$$S = 4.70 \approx 5 \text{ mm}$$

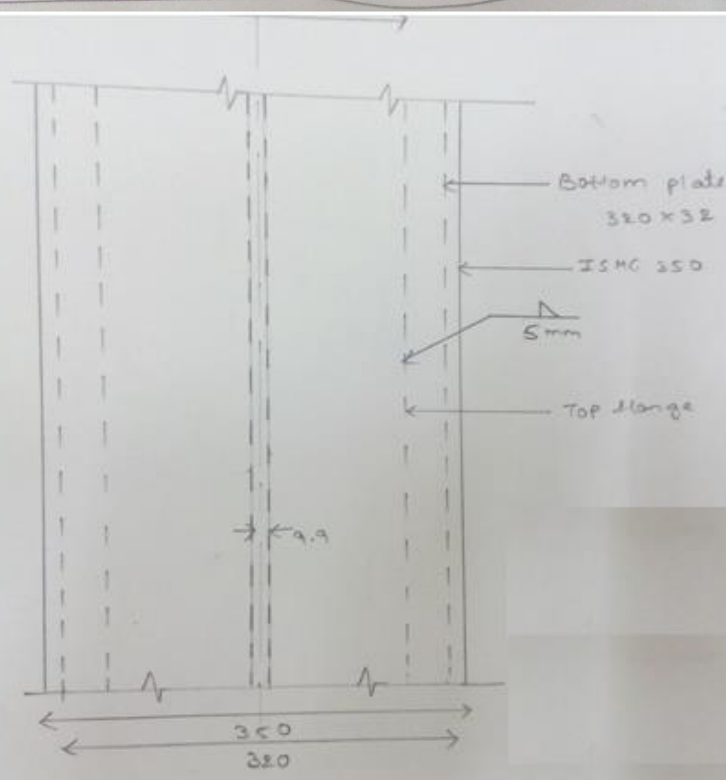
$\therefore$ Provide welding of 5mm where ever requ



SECTION THROUGH GANTRY

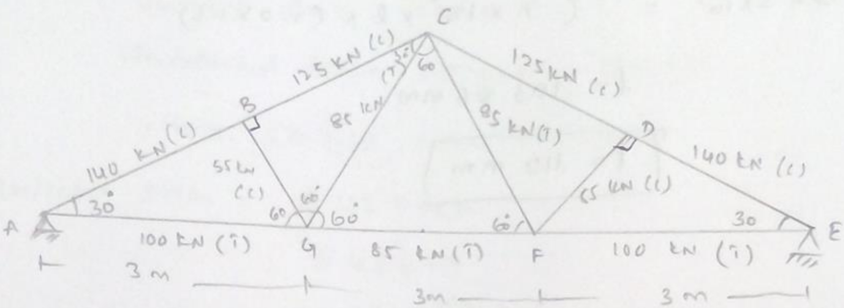


SIDE ELEVATION



PLAN

Q.4



Design of top chord member [AB, BC, CD, DE]

max^n compressive force = 140 kN

factored compressive force = 140 × 1.5 = 210 kN

$P = A \times f_{cd}$

Assume $f_{cd} = 100\ N/mm^2$

$210 \times 10^3 = A \times 100$

$A = \frac{210 \times 10^3}{100} = 1909\ mm^2$

from steel table Pg 10 back to back equal angle

2 ISA 90 × 90 × 6 mm

$A = 2094\ mm^2$

$r_{min} = 2.77\ cm = 27.7\ mm$

Slenderness ratio $= \frac{kl}{r_{min}} = \frac{0.85 \times 2600}{27.7} = 79.78$

AB = 3 cos 30°
= 2.598
= 2.6 m

BG = 3 sin 30°
= 1.5 m

BC = 3 cos 30°

$CG = \frac{1.5}{\sin 30} = 3m$

70 → 152
80 → 136
79.78 → ? 136.35 N/mm^2

$f_{cd} = 136.35\ N/mm^2$

$P = A \times f_{cd}$

$= 2094 \times 136.35 = 285.51\ kN$

285.51 > 210 kN hence safe

Design of connection

assume Dia of bolt = 20mm Class 4.6

shear strength of bolt

from IS 800 : 2007 Pg - 75

Cla 10.3.3

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

$$V_{nsb} = \frac{f_u}{\sqrt{3}} \left(n_n A_{sb} + n_s A_{sb} \right)$$ ($n_s = 0$)

$n_n = 2$

$$= \frac{400}{\sqrt{3}} \left[2 \times 0.78 \times \frac{\pi}{4} \times 20^2 \right]$$

$$\frac{113.18 \text{ kN}}{1.25} = 90.54 \text{ kN}$$

Bearing strength of bolt

IS 800 : 2007 Pg - 75

Clas 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$V_{npb} = 2.5 \, k_b \, t \, f_u \times d$$

$$k_b = \frac{e}{3d_0}, \quad \frac{p}{3d_0} - 0.25, \quad \frac{f_{ub}}{f_u}, \quad 1$$

$e = 1.7 d_0 = 37.4 \approx 40 \text{ mm}$

$p = 2.5 d = 50 \text{ mm}$

$$\frac{40}{3 \times 22} = 0.60 \; ; \quad \frac{50}{3 \times 22} - 0.25 = 0.5 \; ; \quad \frac{400}{410} = 0.97 \; ; \; 1$$

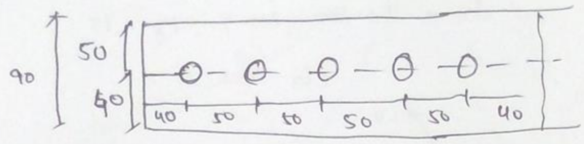
$$\boxed{k_b = 0.5}$$

$$V_{npb} = 2.5 \times 0.5 \times 6 \times 410 \times 20 = 61.5 \text{ kN}$$

$$V_{dpb} = \frac{61.5}{1.25} = 49.2 \text{ kN}$$

Hence provide bolt value = 49.2 kN

no. of bolts = $\frac{210}{49.2}$ = 4.26 kN = 5 no



Design of bottom chord member

max^n tension force = 100 kN

factored tension force = 100 × 1.5 = 150 kN

from IS 800 : 2007 Pg 32 Cl 6.2

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

$$150 \times 10^3 = \frac{A_g \times 250}{1.1} = 660 \text{ mm}^2$$

increase by 30 %.

$$1.3 \times 660 = 858 \text{ mm}^2$$

Hence provide from ISA steel table

2 ISA 50 × 50 × 6 mm

A = 11.36 cm^2 = 1136 mm^2

Design connection

Shear strength of bolt — IS 800 : 2007 Pg 75

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times \left[2 \times 0.78 \times \frac{\pi}{4} \times 12^2\right]$$

32.59 kN

Bearing strength of bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$V_{npb} = 2.5 \times d \times k_b \times t \times f_u$$

$k_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}; 1$

$e = 1.7 \times 14 = 23.8 \approx 25 mm$

$p = 2.5 \times 1.4 = 35$ mm

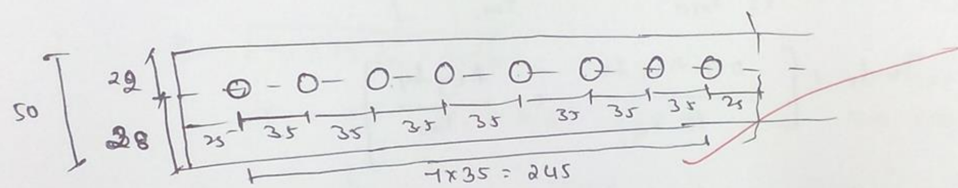
$k_b = 0.46$

$V_{npb} = 2.5 \times 0.46 \times 12 \times 6 \times 410 = 33.94$ kN

$V_{dpb} = \frac{33.94}{1.25} = 27.15$ kN

Provide bolt value 27.15 kN

$\text{no. of bolts} = \frac{210 \times 10^3}{27.15} = 7.73 \approx 8$ no



Check for rapture   Pg: 30 Clause 6.3.3 IS 800:2007

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

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u \gamma_{m0}}{f_y}\right) \ge 0.7$$

$w = 50$

$t = 6$

$b_s = w + w_1 - t$

$= 50 + 28 - 6 = 72$ mm

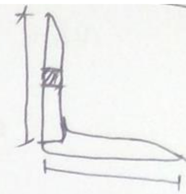
$L_c = 245$

$$= 1.4 - 0.076\left(\frac{50}{6}\right)\left(\frac{250}{410}\right)\left(\frac{72}{245}\right) \le \frac{410 \times 1.1}{250}$$

$1.28 \le 1.8 \ge 0.7$

$\beta = 1.28$

$A_{nc} = (50 - \frac{6}{2} - 14)\,6 = 198\ mm^2$



$A_{go} = (50 - \frac{6}{2})\,6 = 282\ mm^2$

$$T_{dn} = \frac{0.9 \times 198 \times 410}{1.25} + \frac{1.28 \times 282 \times 250}{1.1}$$

$140.48\ kN \times 2$

$280.96\ kN > 150\ kN$

Hence safe

Check for block shear — Pg 33 Cla 6.4.1 IS 800 : 2007

$$T_{bd_1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9\,A_{tn} f_u}{\gamma_{m1}}\right]$$

$$T_{bd_2} = \left[\frac{0.9\,A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

$A_{vg} = (7 \times 35 + 25) \times 6 = 1620\ mm^2$

$A_{vn} = (7 \times 35 + 25 - (7.5 \times 14 \times 6)) = 990\ mm^2$

$A_{tn} = 168 - (0.5 \times 14 \times 6) = 126\ mm^2$

$A_{tg} = 28 \times 6 = 168\ mm^2$

$$T_{bd_1} = \left[\frac{1620 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}\right] = 249.76\ kN$$

$= 499.52\ kN > 150\ kN$

$$T_{bd_2} = \frac{0.9 \times 990 \times 410}{\sqrt{3} \times 1.25} + \frac{168 \times 250}{1.1}$$

$413.38\ kN > 150\ kN$

Hence safe.

Design of inner compressive member

max^m compressive force => 55 kN

Factored forced = 55 × 1.5 = 82.5 kN

Assume f_{cd} = 30 to 120 N/mm^2

$$P = A \times f_{cd}$$

$$A = 82.5 \times 10^3 / 50$$

$$= 1650 \ mm^2$$

from steel table pg-4, Rolled steel equal angle

ISA 100 × 100 × 8

A = 1539 mm^2 $\quad r_{vv}$ = 1.95 cm = 19.5 cm

$$\lambda = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

Pg 48 Clas 7.5.12

$k_1 = 0.7 \quad k_2 = 0.6 \quad k_3 = 5$ IS 800 : 2007

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{1500}{19.5}}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} \quad \frac{76.92}{88.85} = 0.865$$

$$\lambda_\phi = \frac{\frac{b_1 + b_2}{2t}}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{100 + 100}{2 \times 8}}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = \frac{12.5}{88.85} = 0.140$$

$$\lambda_c = \sqrt{0.7 + (0.865^2 \times 0.6) + (5 \times 0.140^2)}$$

$$\lambda_c = 1.11$$

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

from pg 34 clas 7.1.

IS 800 : 2007

$$\phi = 0.5 [1 + \alpha (\lambda^2 - 0.2) + \lambda^2)$$

$\alpha = 0.49$

$\phi = 0.5 \cdot \left[1 + 0.49(1.11 - 0.2) + 1.11^2\right]$

$\phi = 1.339$

$$f_{cd} = \frac{\frac{250}{1.1}}{1.339 + \left[1.339^2 - 1.11^2\right]^{0.5}} = \frac{227.27}{2.08} = 108.85 \text{ N/mm}^2$$

$P = 1539 \times 108.85 = 167.52 \text{ kN} > 82.5 \text{ kN}$

Hence safe

Design of connection

$$\text{shear strength of bolt} = \frac{400}{\sqrt{3} \times 1.25}\left[1 \times 0.78 \times \frac{\pi}{4} \times 20^2\right]$$

45.27 kN

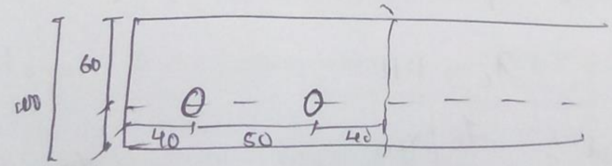
$$\text{bearing strength of bolt} = \frac{2.5 \times k_b \times d \times t \times f_u}{1.25}$$

$k_b = 0.5$

$$= \frac{2.5 \times 0.5 \times 20 \times 8 \times 410}{1.25} = 65.60 \text{ kN}$$

Hence provide bolt value $= 45.27$ kN

$$\text{no. of bolts} = \frac{82.5}{45.27} = 1.82 \approx 2 \text{ no}$$



Design of inner tension member

max^m tension force = 85 kN

factored tension force = 85 × 1.5 = 127.5 kN

pg - 32 IS 800 : 2007

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

$$A_g = \frac{127.5 \times 1.1}{250} \times 10^3 = 561 \text{ mm}^2$$

Increase area by 30 %.

1.3 × 561 = 729.3

Provide ISA 50 × 50 × 6 mm (minimum)

Design connection

adopt 12 mm dia bolt

$$\text{shear capacity of bolt} = \frac{f_u}{\sqrt{3} \times 1.25}(n_n A_{nb})$$

$$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times \frac{\pi}{4} \times 0.78 \times 12^2\right)$$

$$= 16.29 \text{ kN}$$

Bearing Capacity of bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$V_{npb} = 2.5 \times 0.46 \times 12 \times 6 \times 410 = 33948$$

$$V_{dpb} = \frac{33948}{1.25} = 27.15 \text{ kN}$$

$$\text{no. of bolts} = \frac{127.5 \text{ kN}}{16.29} = 7.82 = 8 \text{ no}$$

Check for rapture.

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m_1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m_1}}\right) \geq 0.7$$

$w = 50$

$t = 6$

$b_s = 50 + 28 - 6 = 72 \text{ mm}$

$L_c = 245$

$$\beta = 1.4 - 0.076\left(\frac{50}{6}\right)\left(\frac{400}{410}\right)\left(\frac{72}{245}\right) \leq \frac{410 \times 1.25}{250} \geq 0.7$$

$\beta = 1.28$

$A_{nc} = (50 - \frac{6}{2} - 14)\, 6 = 198 \text{ mm}^2$

$A_{go} = (50 - \frac{6}{2})\, 6 = 282 \text{ mm}^2$

$$T_{dn} = \frac{0.9 \times 198 \times 410}{1.25} + \frac{1.28 \times 282 \times 250}{1.1}$$

140.48×2

$= 280.96 \text{ kN} > 127.5 \text{ kN}$

Hence safe.

Check for block shear

$$T_{bd_1} = \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$T_{bd_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m_1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$A_{vg} = 1620 \text{ mm}^2$

$A_{vn} = 990 \text{ mm}^2$

$A_{tn} = 126 \text{ mm}^2$

$A_{tg} = 168 \text{ mm}^2$

$T_{bd_1} = 499.52$ kN > 127.5 kN

$T_{bd_2} = 413.38$ kN > 127.5 kN

Hence safe

Bearing Plate design

support force = 160 kN

factored force = 160 × 1.5 = 240 kN

from IS 456 : 2000 Cla 34.4

Permissible stress = $0.45 \times f_{ck}$

$= 0.45 \times 20 = 9\ N/mm^2$

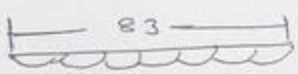
Area of plate = $\frac{240 \times 10^3}{9} = 26666.66$

consider square plate = $\sqrt{26666.66}$

= 163.29 ≈ 200 mm

Size of adopt = 200 mm × 200 mm

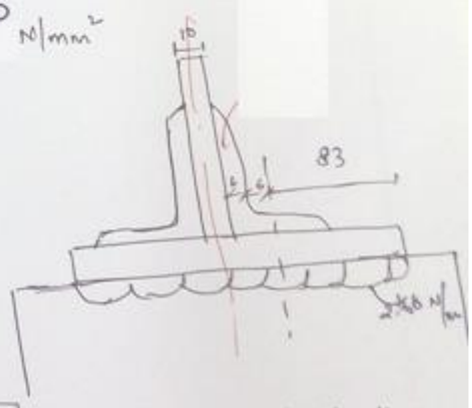
Bearing pressure = $\frac{240 \times 10^3}{200 \times 200} = 6\ N/mm^2$



$6 \times 83 \times \frac{83}{2} = 20.66$ kN

$= 250 \times 1 \times \frac{(6+t)^2}{6}$

$t = 20$ mm



$100 - 6 - 6 - \frac{10}{2} = 83$