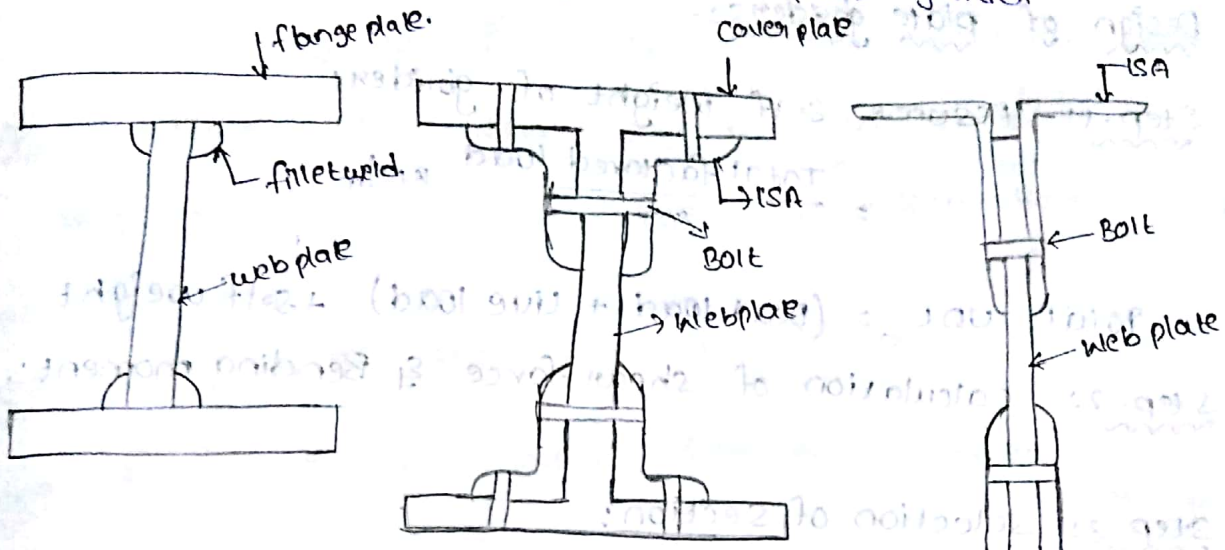
11/02/2020

UNIT-VI

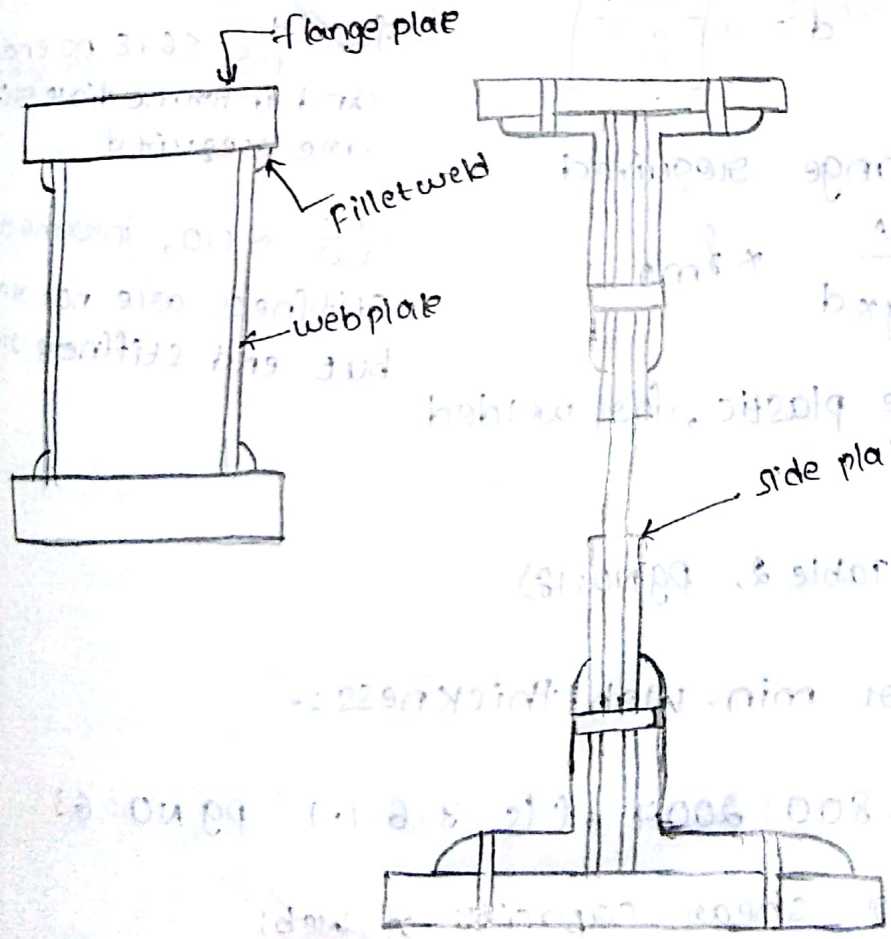
Design of plate Girders

A plate girder is an I-beam built up from plates using bolting & welding

Various cross-sections of the plate girder



(a) welded girder (b) bolted with cover plates (c) welded girder



(d) Box girder (e) girder with side plate.

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Gantry Girders:

Gantry Girders are laterally un-supported beams to carry heavy load from one place to another place in industrial buildings such as factories, workshops, lifting of heavy materials, equipment, etc. At the construction site & within the Building.

Design of plate girders:-

Step-1:- Assume self weight of girder:

$$= \frac{\text{Total factored load}}{200} \text{ kN/m}$$

Total UDL = (Dead load + Live load) + self weight.

Step-2:- Calculation of shear force & Bending moment:-

Step-3:- Selection of section:-

Determine the optimum depth of the member

$$d = \left[\frac{MK}{f_y}\right]^{0.33}$$

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

for $\frac{d}{t_w} \leq 67\varepsilon$ no end stiff and intermediate stiffner are required

$\frac{d}{t_w} \simeq 100$, intermediate stiffness are not require but end stiffness req^n.

Area of the flange required

$$A_f = \frac{M}{f_y \times d} \times \gamma_{m0}$$

for flange to be plastic, for welded section.

take $\frac{b}{t_f} = 8.4\varepsilon$ (Table 2, PG NO: 18)

Step-4: Check for min. web thickness:-

As per IS 800: 2007 Cls 8.6.1.1 PG NO: 63

Step-6: Check for shear capacity of web:

$$V_d = \frac{A_v \cdot f_{yw}}{\sqrt{3}\,\gamma_{m0}}$$

IS 800: 2007 Cls. 8.4.1 PG NO: 5

$V_n = V_p$

$V_p = A_v \cdot \tau$

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If $V_d < V_p$, Intermediate transverse stiffness are not required

If $V_d > V_p$, Intermediate transverse stiffness are required to improve the Buckling strength of slender web.

Step-5: Check for Moment capacity:-

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

$M_d > M$ [from IS 800-2007 Cls 8.2.12 (pg No: 53)]

Step-7: Check for anchor force in end pannel:-

This check is required when the intermediate stiffners are required.

Step8:- Design of end bearing stiffness:-

Step-9: Design of intermediate transvers stiffners are required

Step-10: Design of connection between web plate and flange.

Step-11: Design of connection b/w stiffners and web.

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① Design a welded plate girder for a simply supported bridge deck ~~Beam~~ beck a clear span 24 mt subjected to dead load of 20 KN/m (Excluding self weigh) live load 10 KN/m and two concentrated load 200 KN each at 6m from each end.

Assume that the top compression flange of plate girder is restrained laterally and prevented from rotating. use Fe415 grade of steel. design as an unstiffened plate girder with thick web.

Sol) Given data

assignment - 6

change values

live load = 10 KN/m

dead load = 20 KN/m

concentrated load = 200 KN

clear span = 24 mt

Step-1:

Total factored load = $1.5[(20 \times 24) + (10 \times 24)] + 1.5(200 + 200)$

= 1680 KN

Self weight = $\frac{\text{total factored load}}{200}$

= 8.4 KN/m

Total U.D.L = $(8.4 + 10 + 20) \times 1.5$ = ~~57.6~~ ~~912.6 KN~~ 38.4 KN/m

Total point load = 200 ⇒ factored point load = 1.5×200 = 300 KN (each)

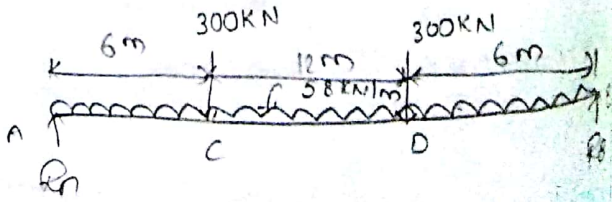
factored U.D.L = 1.5×38.4 = 58 KN/m

Step-2: Calculation of shear force and bending moment

$R_A = R_B = \frac{\text{Total load}}{2}$

$= \frac{(58 \times 24) + 2(300)}{2}$

= 996 KN



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S.F @ B 0 and 996 KN

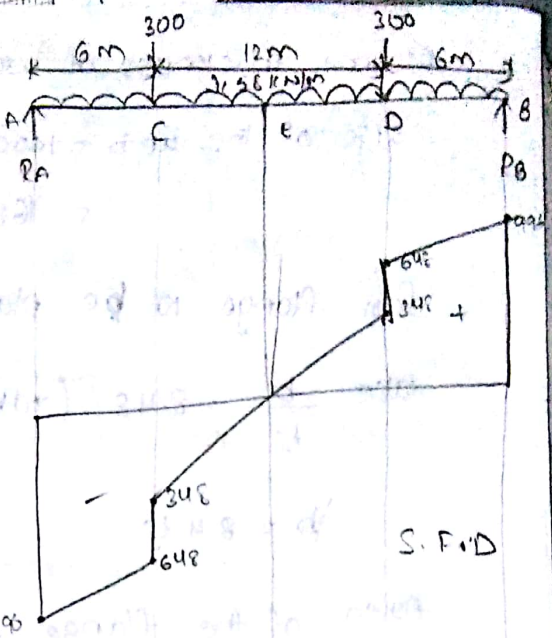
S.F @ D 996 − 58×6 = 648 KN

648 − 300 = 348 KN

S.F @ C 996 − 58×6 = 648 KN

648 − 300 = 348 KN

S.F @ A 0, & 996 KN



B.M Max @ shear force 0

BM "E" is 996×12 − 58×12×6 − 300×6

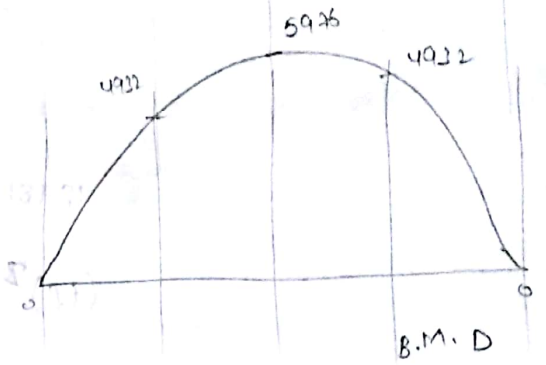
= 5976 km

B.M 'C' is = 996×6 − 58×3×6

= 4932

B.M 'D' is = 996×6 − 58×3×6

= 4932



Max. shear force V = 996 KN

Max. B. Moment M_{max} = 5976 KN-m

Step-3: Selection of section:

Determine the optimum depth of the member

$$d = \left[\frac{MK}{f_y}\right]^{0.33}$$

If $K = \frac{d}{t_w} > 200$ intermediate transverse stiffners are required

$$d = \left[\frac{5976 \times 150 \times 10^6}{250}\right]^{0.33}$$

$$K = \frac{d}{t_w} = 150$$

Assume.

$d = 1422.34 \cong$ **d = 1400 mm** ∴ Thickness (t_w) = 12mm

Area of flange required

$$A_f = \frac{M}{f_y \times d} \times \gamma_{m0}$$

$$= \frac{5976 \times 10^6}{250 \times 1400} \times 1.10 \Rightarrow \boxed{A_f = 18781.71\ mm^2}$$

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Assume thickness of web = 12 mm

size of the web = 1400 × 12 mm

= 16800 mm

for flange to be plastic, for welded section.

take $\frac{b}{t_f} = 8.4\varepsilon$ (Table No: 2)

$b = 8.4\, t_f$ $\varepsilon = 1$

Area of the flange $A_f = b_f \times t_f$

$= 2b \times t_f$

$= 2(8.4\, t_f) \times t_f$

$18781.71 = 2 \times 8.4 (t_f)^2$

$(t_f)^2 = \frac{18781.71}{2 \times 8.4}$

$(t_f)^2 = (1117.95)^{1/2}$

$\boxed{t_f = 33.37 mm}$

$\boxed{t_f = 40 mm}$

$\therefore b = 8.4(40.00)$

$\boxed{b = 336 mm}$

Sub. in area of the flange $A_f = b_f \times t_f$

$18781.71 = b_f \times 40$

$b_f = 469.5 mm$

$\boxed{b_f = 500 mm}$

$\therefore$ Size of the flange = 500 × 40

= 20,000

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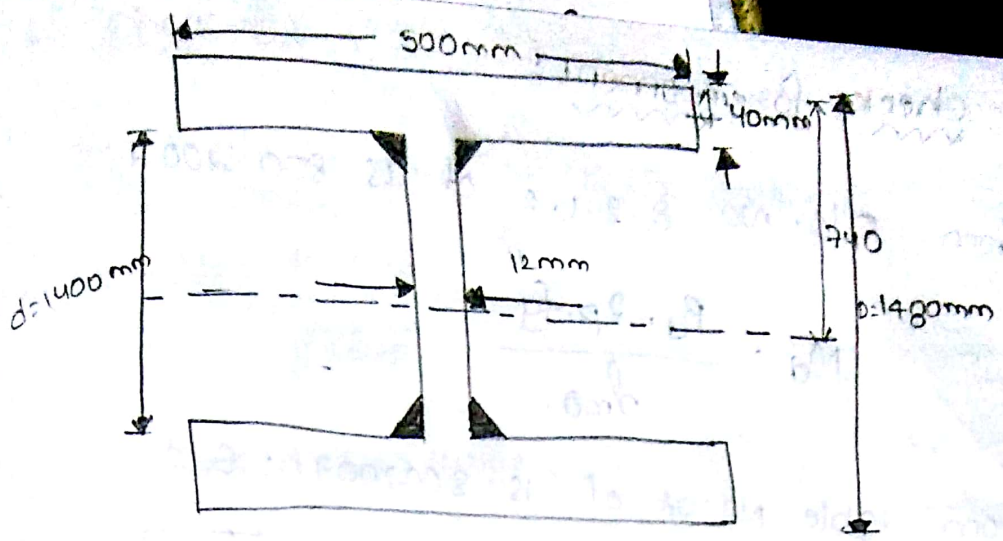

Step-4: Check for minimum thickness web:-

As per IS 800:2007 of Cls: 8.6.1.1

(a) when transverse stiffeners are not provided,

* $\frac{d}{t_w} \leq 200\varepsilon$ $\boxed{\because \varepsilon = 1}$

$\frac{d}{t_w} = \frac{1400}{12} = 116.67 \leq 200\varepsilon$.

$\therefore \frac{d}{t_w} \leq 200\varepsilon$

$\therefore$ So, web connected to flanges along both longitudinal edges

* $\frac{d}{t_w} \leq 90\varepsilon$

(b) compression flange buckling requirement:

Cls. 8.6.1.2 of IS 800:2007

* $\frac{d}{t_w} = \frac{1400}{12} = 116.67 \leq 345\varepsilon_1^2$

$\therefore$ The minimum thickness of the web is sufficient. i.e., 12mm

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Step-5: check for moment:

from Cls. NO: 8.2.1.2 of IS 800:2007

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

from table NO:02 of IS: 800-2007 ; $\beta = 1$

$$\frac{b}{t_f} = \frac{250}{40} = 6.25 \leq 8.4\varepsilon \qquad \boxed{\varepsilon = 1}$$

$$6.25 \leq 8.4$$

∴ Then the section is plastic.

Z_p = plastic section modulus

$$Z_p = \frac{I}{\bar{y}} = \frac{\frac{bd^3}{12} \times 2 + a(\bar{y}-y)^2}{\bar{y}} = \frac{2\left[\frac{500 \times (40)^3}{12}\right] + (500 \times 40)(740-20)^2}{740}$$

$$= \frac{5333333.34 + 1.0368 \times 10^{10}}{740}$$

$$\boxed{Z_p = 28.020 \times 10^6 \, mm^3}$$

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

$$= \frac{(1) \cdot (28.802 \times 10^6) \times 250}{1.10}$$

$$M_d = 6368.018 \times 10^6 \, mm$$

$$\boxed{M_d = 6368.018 \, kN \cdot m}$$

$$M_d > M_{max}$$

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Step-6:- Check for shear capacity:-

from Cls. NO: 8.4.1 of IS 800:2007

$$V_d = \frac{A_v \cdot f_{yw}}{\sqrt{3}\,\gamma_{m0}}$$

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

$$V_n = V_p$$

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

$$A_v = d \times t_w = 1400 \times 12$$

$$V_d = \frac{1400 \times 12 \times 250}{\sqrt{3} \times 1.10}$$

$$V_d = 2424.84\ kN$$

$$V_d = \boxed{V_d = 2204.42\ kN}$$

$$0.6\,V_d = 1322\ kN > 996\ kN$$

$$V_d > V$$

∴ So, section is adiquate for shear capacity.

Step-7: Check for shear buckling of web using simple post critical method

from Cl. NO: 8.4.2.2 of IS 800:2007

$$\boxed{V_n = V_{cr}}$$

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

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

where τ_b = shear stress corresponding to web buckling.

1) $\lambda_w \le 0.8$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}}$$

2) $0.8 \le \lambda_w < 1.2$

$$\tau_b = \frac{[1 - 0.8(\lambda_w - 0.8)]\,f_{yw}}{\sqrt{3}}$$

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St 3) when $\lambda_w \geq 1.2$ $\tau_b = \dfrac{f_{yw}}{\sqrt{3}\,\lambda_w^2}$

where

λ_w = non-dimentional web slenderness ratio

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,c}}}$$

where $\tau_{cr,c}$: The elastic critical shear stress web

$$\tau_{cr,c} = \frac{K_v \pi^2 E}{12(1-\mu^2)[d/t_w]^2}$$

$$= \frac{5.35 \times \pi^2 \times 2\times10^5}{12(1-(0.3)^2)\left[\frac{1400}{12}\right]^2}$$

$$= \frac{10560476.71}{10.92 \times 13611.112}$$

$$\boxed{\tau_{cr,c} = 71.050}\ N/mm^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\times\tau_{cr}}}$$

$$= \sqrt{\frac{250}{\sqrt{3}\times 71.05}}$$

$$\boxed{\lambda_w = 1.425}$$

$\tau_b = ?$

(i) $\lambda_w \leq 0.5$

$1.425 \leq 0.5$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}} = \frac{250}{\sqrt{3}}$$

$$\boxed{\tau_b = 144.33}$$

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2) $0.8 \le \lambda_w < 1.2$

$0.8 < 1.42 < 1.2$

$$\tau_b = \frac{[1-0.8(\lambda_w - 0.8)]f_{yw}}{\sqrt{3}}$$

$$= \frac{[1-0.8(1.42-0.8)]250}{\sqrt{3}}$$

$$\boxed{\tau_b = 72.74\ N/mm^2}$$

3) $\lambda_w \ge 1.2$ $\quad \tau_b = \dfrac{f_{yw}}{\sqrt{3}\,\lambda_w^2}$

$$= \frac{250}{\sqrt{3}\,(1.425)^2}$$

$$\boxed{\tau_b = 71.080\ N/mm^2}$$

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

$= 1400 \times 12 \times 71.080$

$$\boxed{V_{cr} = 1194\,KN > V}$$

$$\boxed{\therefore V_n = 1194\,KN}$$

$$\boxed{\therefore V_n > V}$$

So, section is adiquate for shear buckling of web using simple post critical method.

$\because$ Hence shear strength of web is greater than the applied shear

Note: If the V_{cr} is less than V, the buckling strength is required to be improved by changing the web

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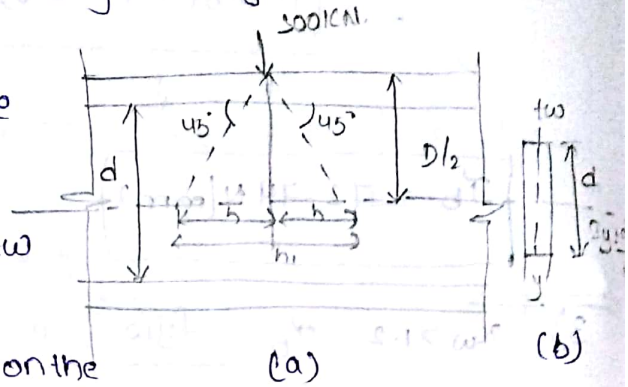
As the compression flange is restrained, there is no need to check for lateral buckling.

Step 8: Check for designed load carrying stiffness:-

Load carrying web stiffness should be provided where compression force applied through a flange by load & reaction exceeds the buckling strength.



Buckling strength of the stiffness web:

$F_{cdw} = (b_1 + n_1) f_{cd} \times t_w$

b_1 = width of stiff bearing on the flange

= 0 (assume)

$n_1 = 2 \times \frac{D}{2}$

$= D$

$\boxed{n_1 = 1480 \text{ mm}}$

$t_w = 12 \text{ mm}$

f_{cdw} = from table 9(c) of IS800:2007

Buckling strength of web about axis parallel to web is calculated using curve 'c'.

$$\frac{Kl}{r} = \frac{Kd}{r}$$

where $Kd = Kl = 0.7 \times 1400 = 980$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{d t_w^3}{12}}{d t_w}} = \sqrt{\frac{d t_w^3}{12 d t_w}} = \frac{\sqrt{(t_w)^2}}{\sqrt{12}}$$

$\boxed{r = 3.46}$

$$\frac{Kl}{r} = 2.43 \times \frac{d}{}$$

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$= 2.43 \times \frac{1400}{12} = 283.5$

by extrapolation, $f_y = 250$

240 → 26.2

250 → 24.2

10 1.9

240 − 283.5 = 43.5 → ? ⇒ 8.265.

240 → 26.2 − 8.265 = 17.94 N/mm²

$$\boxed{f_{cd} = 17.94\ N/mm^2}$$

$$f_{wd} = (b_1 + n_1) t_w \times f_{cd}$$

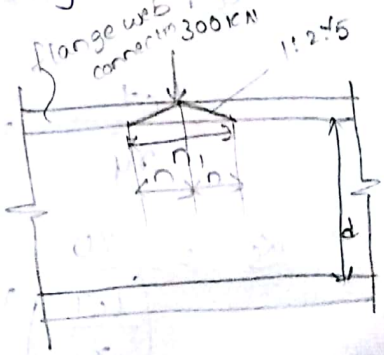
$$= (0 + 1480) \times 12 \times 17.94$$

$$\boxed{f_{wd} = 318.614\ KN} > 300KN$$

∴ web is safe against below 300KN load so, there is no need of stiffners.

Step-9: Design of load bearing stiffners:-

bearing stiffners should provided for web where force applied through a flange by load & reactions exceeds the local capacity of web.



$b_1 = 0$

$n_2 = (40 \times 2.5) = 200mm$

$$f_{wd} = \frac{(b_1 + n_1) t_w f_{yw}}{\gamma_{m0}}$$

$$= \frac{200 \times 12 \times 250}{1.10}$$

$$\boxed{f_{wd} = 545.45KN}$$

(7) At the location of 300KN point load $f_w = 545.45K > 300KN$. Hence no bearing stiffners below point load is required.

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(II) At support, $f_w = 545.45 < 996$ KN. Hence bearing stiffners are required at support.

Step-10: Check for torsional resistance provided by end stiffness:-

$I_s \geq 0.34 \alpha_s D^3 T_{cf}$ | Cls. S no. 8.9.9

$r_y = \sqrt{\frac{I_{yy}}{A}}$ | page - 68

r_y = M I of entire section of grider yy

$$I_{yy} = 2\left(\frac{t_f b_f^3}{12}\right) + \left(\frac{d \times t_w^3}{12}\right)$$

$$I_{yy} = 2\left(\frac{40 \times (500)^3}{12}\right) + \left(\frac{1400 \times (12)^3}{12}\right)$$

$$= 833.53 \times 10^6 \text{ mm}^4$$

$$A = 2\left[(40 \times 500) + (1400 \times 12)\right]$$

$$= 56800 \text{ mm}^2$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{838.53 \times 10^6}{5600}}$$

$$= 121.14 \text{ KN-m}$$

l_{cT} = 1 × effective span = 1 × 24000

= 24000 mm

$$\frac{l_{cT}}{r_y} = \frac{24000}{121.14} = 198.11 > 100$$

$$\alpha_s = \frac{30}{\left(\frac{l_{cT}}{r_y}\right)^2} = \frac{30}{(198.11)^2} = 7.64 \times 10^{-4}$$

$$= 0.34 \times 7.64 \times 10^{-4} \times 14803 \times 40$$

$$= 33.70 \times 10^6 \text{ mm}^4$$

$$I_s = \frac{12(2 \times 220 + 12)^3}{12}$$

Where I_s = M.I of stiff at entire line of web.

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Step-11: Weld connecting web & flange:-

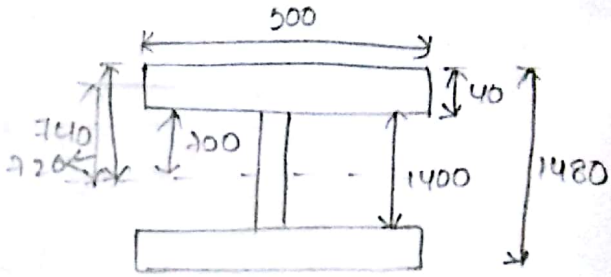
$$q_w = \frac{V \cdot A_f \cdot \bar{y}}{2 \times I_{zz}}$$

q_w = Horizontal shear force per mm

V = max. shear force = 996 kN

A_f = 500 × 40 = 20,000 mm²

$\bar{y} = 700 + \frac{40}{2} = 720$ mm



I_{zz} = net moment of inertia

$$= \frac{500(1480)^3}{12} - 2\left[\frac{(500-12) \times (1400)^3}{12}\right]$$ [By Box method]

$$I_{zz} = 1.3507 \times 10^{11} - 2\left[1.115 \times 10^{11}\right]$$

$$\boxed{I_{zz} = 2.35 \times 10^{10} \text{ mm}^4}$$

$$q_w = \frac{996 \times 20{,}000 \times 720}{2 \times 2.35 \times 10^{10}}$$

$$\boxed{q_3 = 0.305 \text{ N/mm}}$$

Design strength of the welded connection

$$P_{dw} = \frac{f_y}{\sqrt{3}\gamma_{mw}} \times l_w \times t_e$$

Assume

Size of the weld = 6mm

Throat thickness (t_e) = 0.7 × 6 = 4.2mm

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$P_{dw} = 410$

assume $\gamma_{mw} = 1.25$ (Shop weld)

$f_u = 410\ N/mm^2$; $L_w = 1$

$$P_{dw} = \frac{410}{\sqrt{3} \times 1.25} (1) \times 4.2$$

$$\boxed{P_{dw} = 795.35\ N/mm}$$

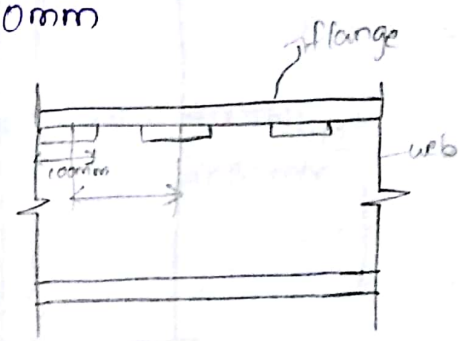
$$\boxed{P_{dw} = 0.795\ KN/mm}$$

∴ The value of welded strength is higher than the shear force. Hence the section is safe.

∴ $0.795 > 0.305$

Assume length of the weld = 100mm

c/c spacing of the weld =



= Ratio of the strength × 100

$$= \frac{0.795}{0.305} \times 100$$

$= 260$

But, clear spacing cls NO: 10.5.5.2 of IS 800:2007

$\not> 12t = 12 \times 12 = 144mm$

since $16mm > 144mm$

Provide clear spacing 100mm length of weld is 100mm size of the weld is 6mm on both side of the web at top and bottom of the section.

Step-12:- connecting End stiffner & web:-

Provide clear spacing 100mm length of weld is 100mm size of the weld is 6mm on both side of the web at top and bottom of the section.

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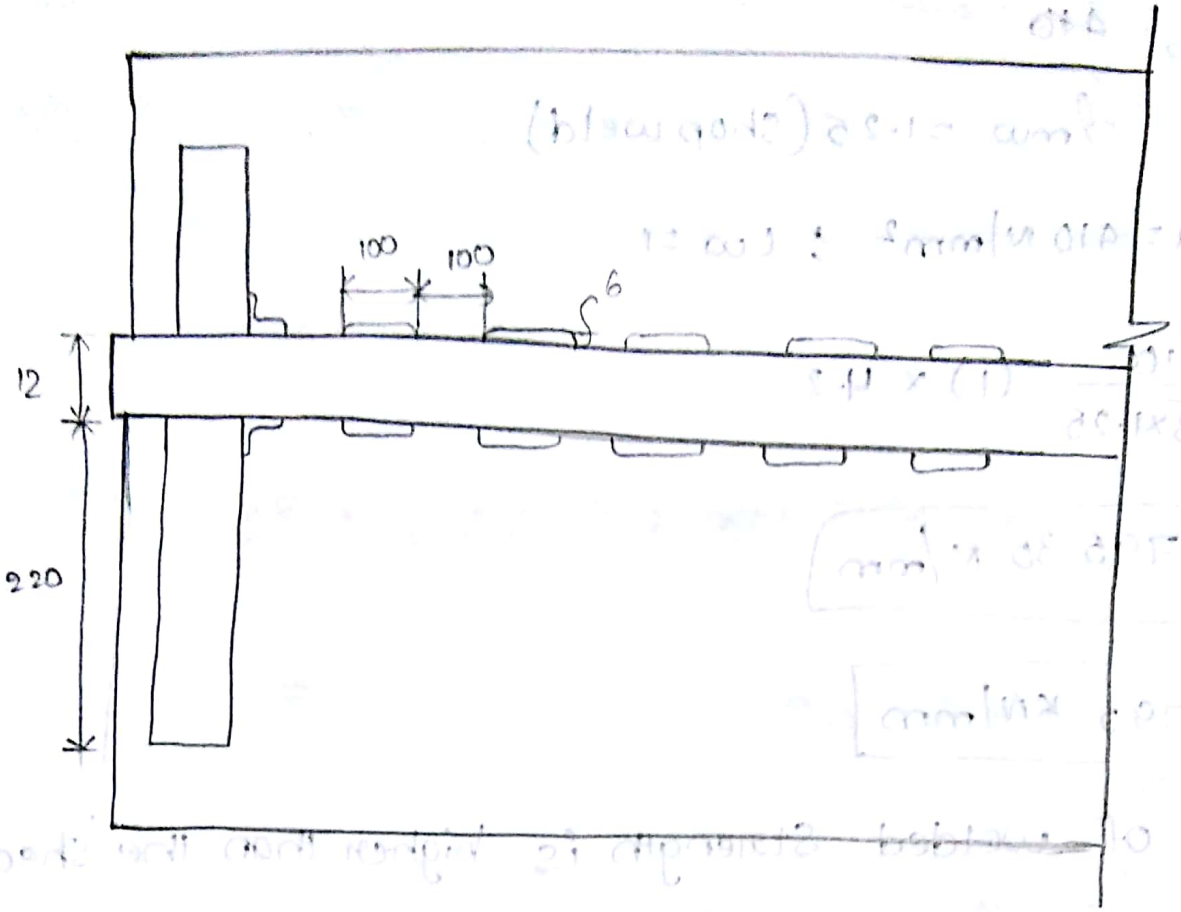

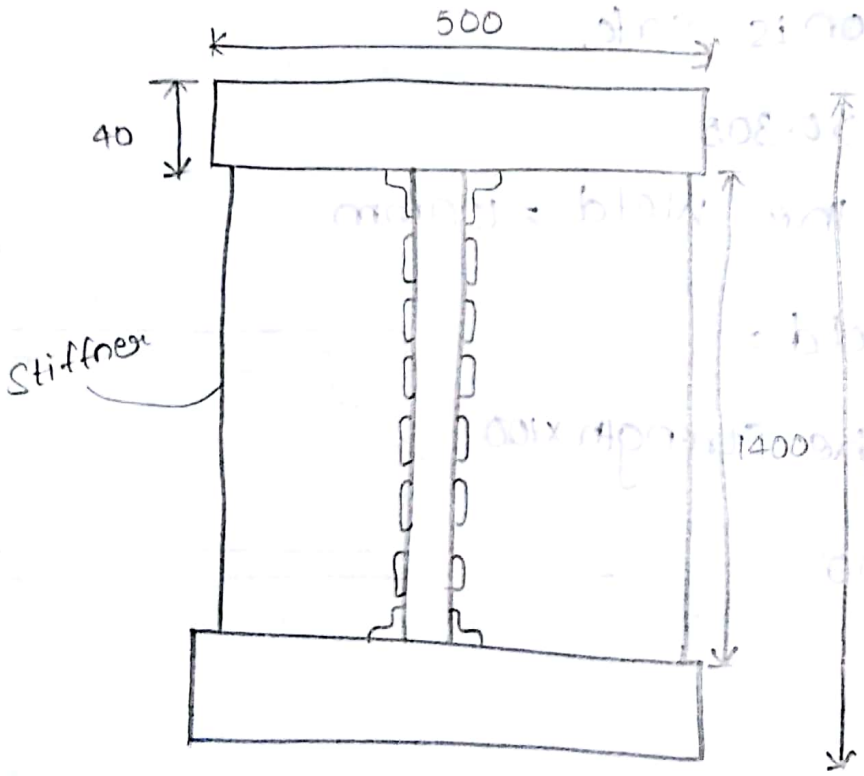

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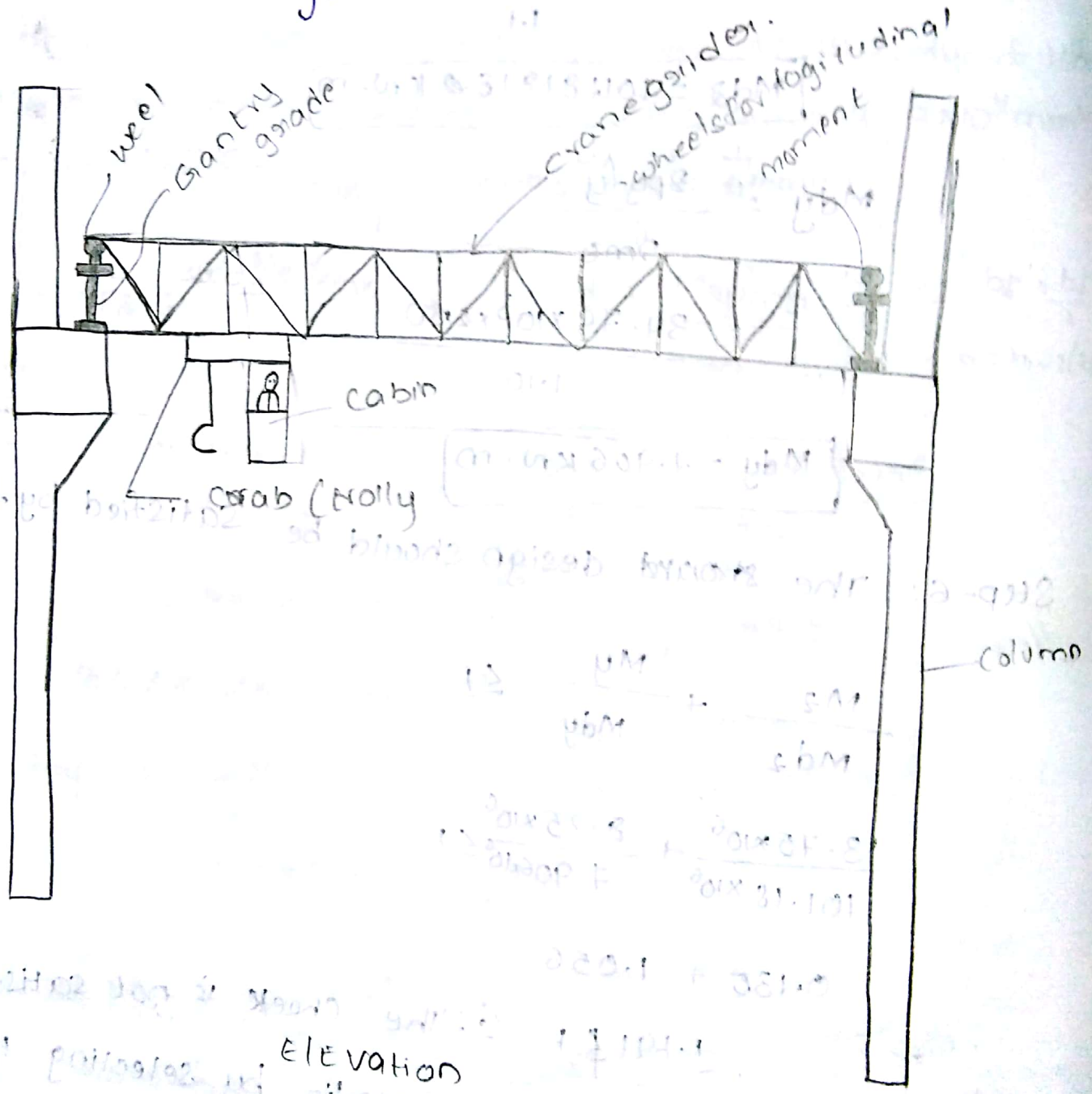
GANTRY GIRDER

gantry girders are laterally unsupported beams to carry heavy load from one place to another place in industrial buildings such as factories, work shops. lifting of heavy materials equipments etc. at construction site are with in the building.



Elevation

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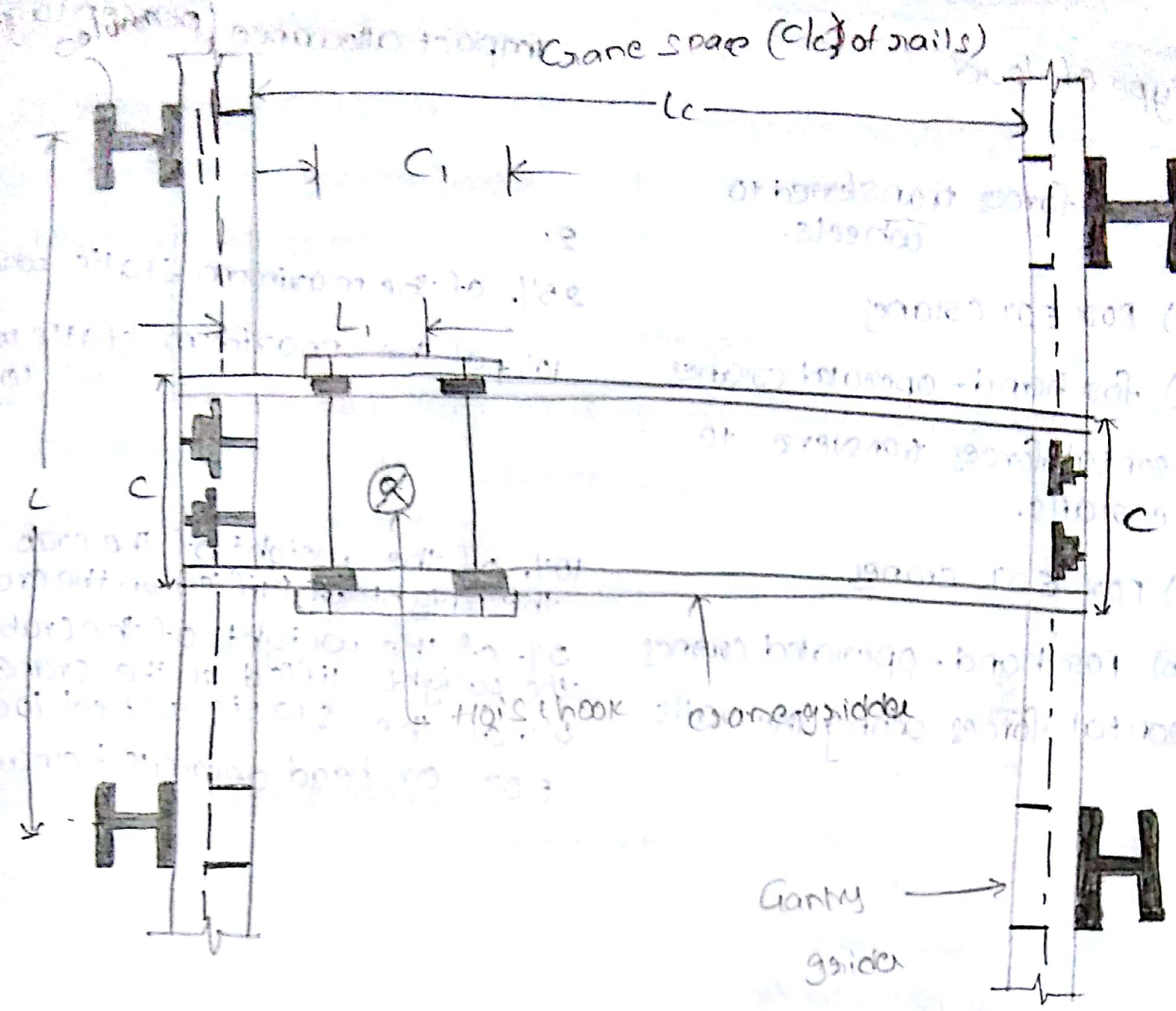

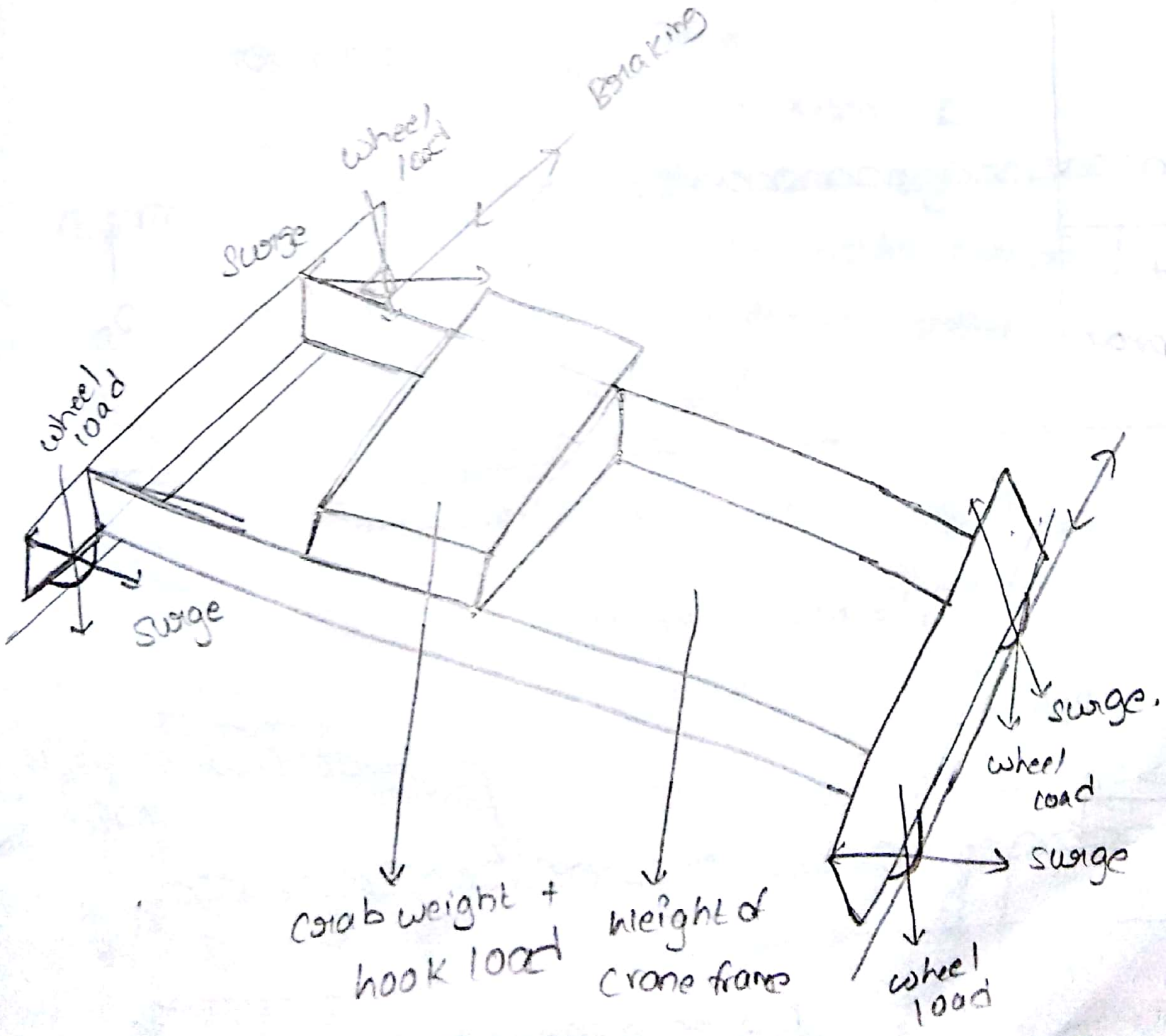

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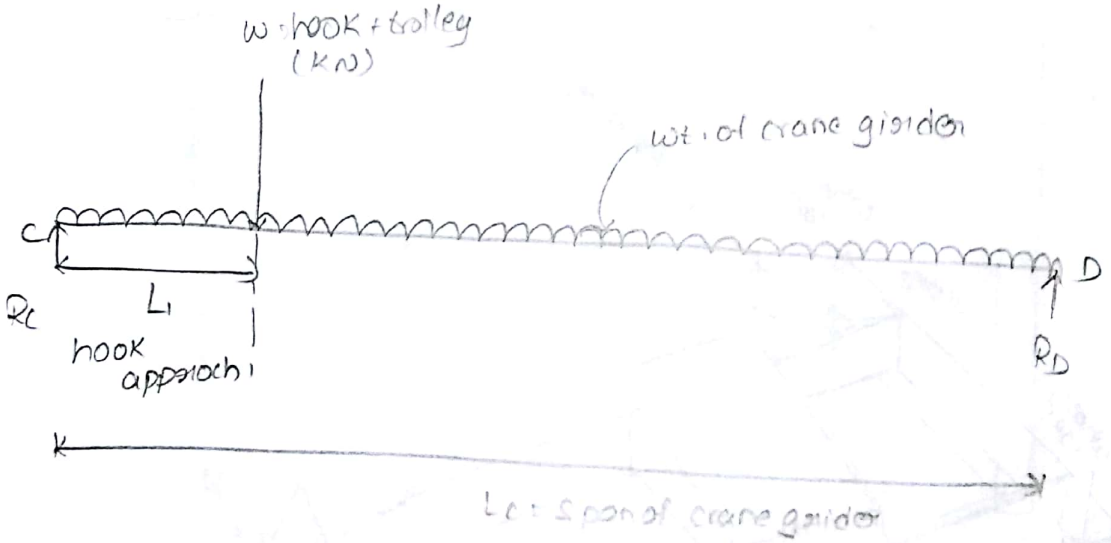
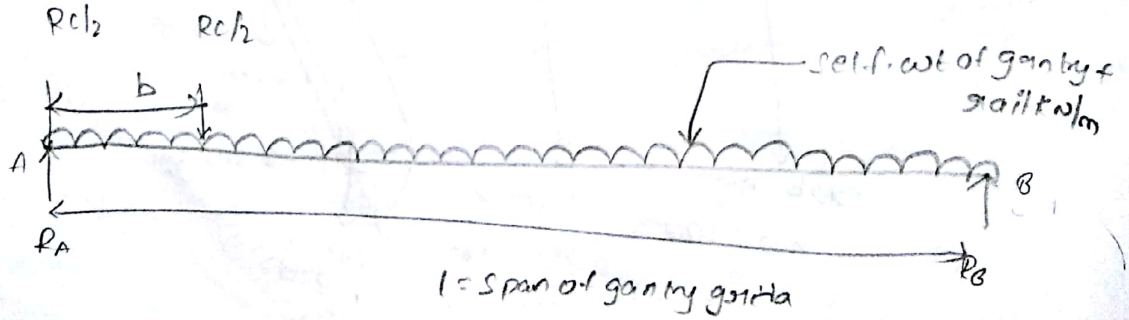
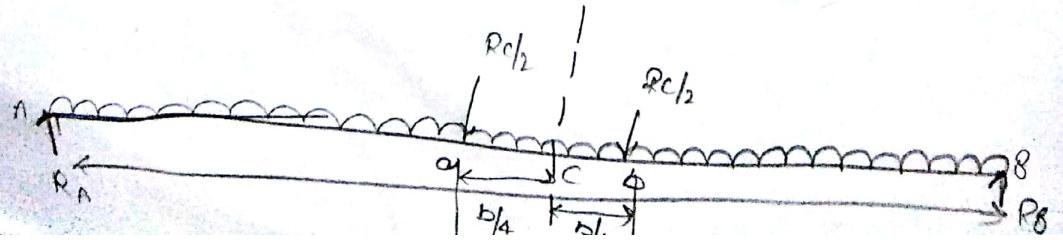
Type of load	Impact allowance (percentage)
Vertical forces transferred to wheels.	
(a) For EOT Cranes	25% of the maximum static wheel load
(b) for hand-operated cranes	10% of the maximum static wheel load.
Horizontal forces transverse to the rails.	
(a) For EOT cranes	10% of the weight of the crab and the weight of lifted on the crane.
(b) For hand-operated cranes	5% of the weight of the crab and the weight lifted on the crane
Horizontal forces along the rails	5% of the static wheel loads for EOT or head operated cranes



L_1 = hook approach.





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Procedure:

1. Maximum wheel load.
2. Max. B.M in gantry girder
3. Max. shear force
4. Section selection for gantry girder.
5. calculate I_{zz}, I_{yy} and z_p of the trial section selected.
6. Check for moment capacity
7. Check for shear capacity
8. Check for buckling resistance.
9. Check for local Buckling
10. weld design
11. Check for deflection.

Sol Given data.

Crane capacity = 200kN

span of gantry girder = 7.5m = L

Span of crane girder = 15m = L_c

self weight of the crane girder excluding trolley = 200kN

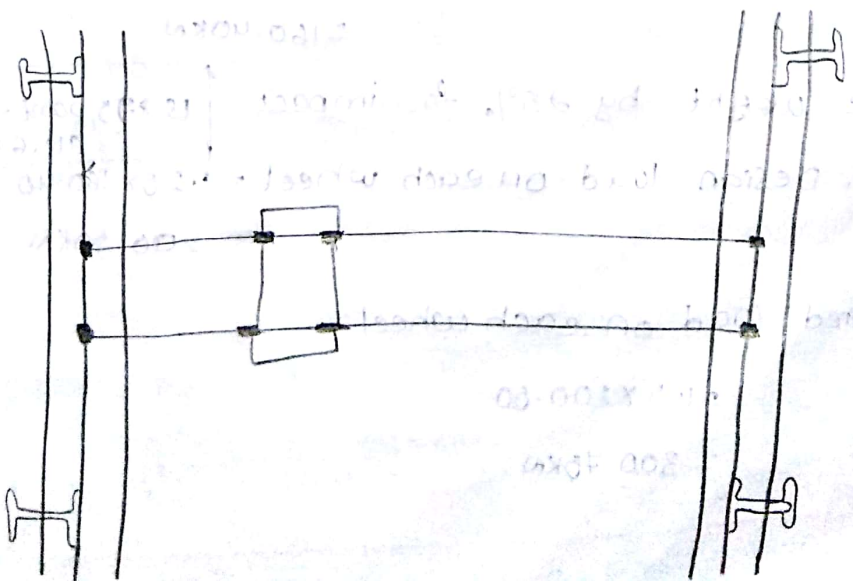
self weight of trolley (crab) = 40kN

Minimum hook approach = 1.2m = L_1

wheel base of crane = 3.5m = c

self weight of rail section = 300N = 3kN

Take yield stress of steel = 250MPa.



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(1) Maximum wheel load:

Crane capacity + wt. of trolley (crab)

$= 200 + 40$

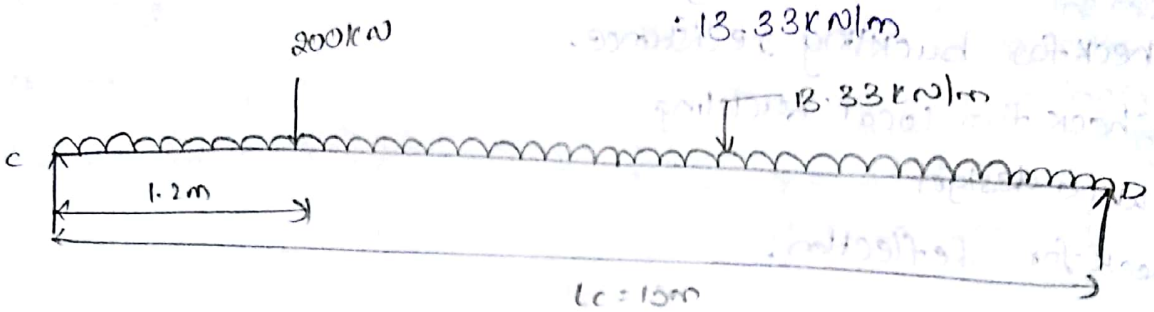
$= 240\ kN$

Minimum hook approach $= 1.2\ m$

Total weight of crane girder $= 200\ kN$

$\therefore$ weight of crane girder per m $= \frac{200}{15}$

$= 13.33\ kN/m$



Taking moment @ C,

$R_D \times 15 = 200 \times 1.2 + 13.33 \times 15 \times \frac{15}{2}$

$\boxed{R_D = 119.18\ kN}$

R_C = Total load $- R_D$

$= (240 + 13.33 \times 15) - 119.18$

$= 320.77\ kN$

At the each end of crane girder there are two wheels

$\therefore$ load on each wheel $= \frac{320.77}{2}$

$= 160.40\ kN$

Increase weight by 25% for impact

IS 875, part-II, Cls. 6.1, P.15

$\therefore$ Design load on each wheel $= 1.25 \times 160.40$

$= 200.50\ kN$

$\therefore$ factored load on each wheel

$= 1.5 \times 200.50$

$= 300.75\ kN$

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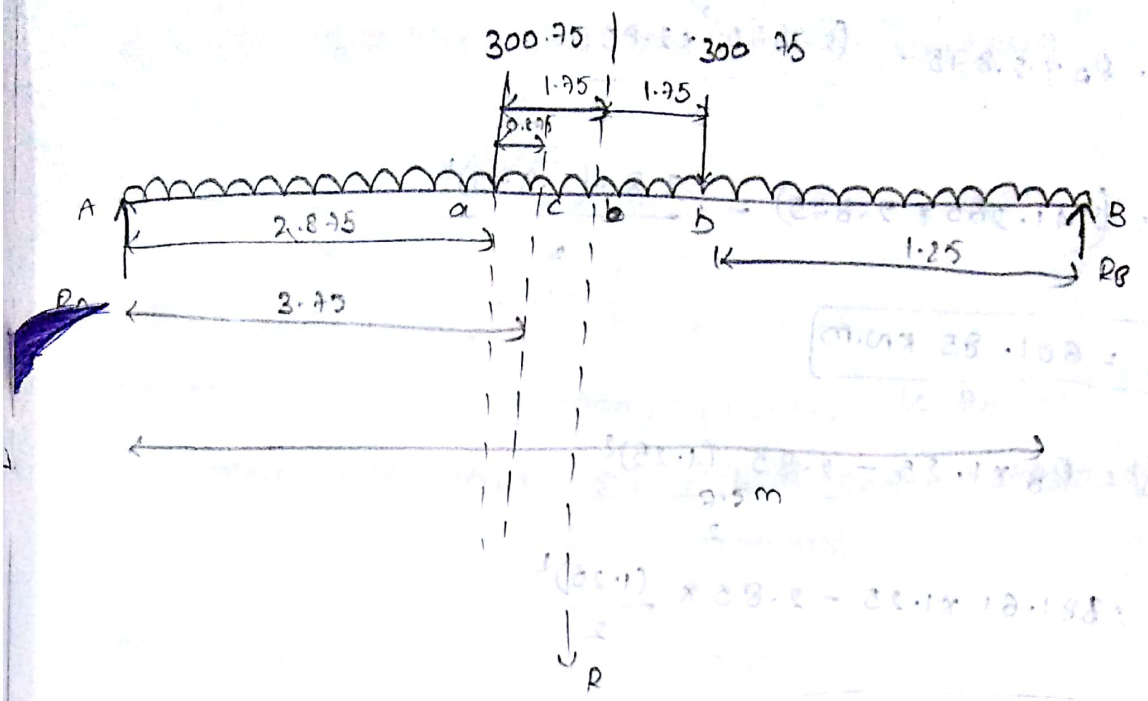
(2) Maximum B.M in the Gantry girder.



Max. B.M will occur below wheel load 'a' when centre of the span (c) is at equal distance from the wheel load and the C.G of the loads (R)

Assume self weight of gantry girder = 1.6 kN/m

Weight of rail section = 0.3 kN/m

1.9 kN/m

∴ factored load = 1.5 × 1.9

= 2.85 kN/m

'a' and 'b' are two wheel loads

Maximum B.M will occur at 'a'.

Taking moment @A,

$$R_B \times 7.5 = (300.75 \times 2.875) + (300.75 \times 6.375) + 2.85 \times \frac{(7.5)^2}{2}$$

$$\boxed{R_B = 381.61 \text{ kN}}$$

$$R_A = \text{Total load} - R_B$$

$$R_A = (300.75 + 300.75) + 2.85(7.5) - 381.61$$

$$\boxed{R_A = 241.265 \text{ kN}}$$

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Max. B.M will occur at 'a'

$$M_a = R_A \times 2.875 = \frac{(2.875)^2 \times 2.85}{2}$$

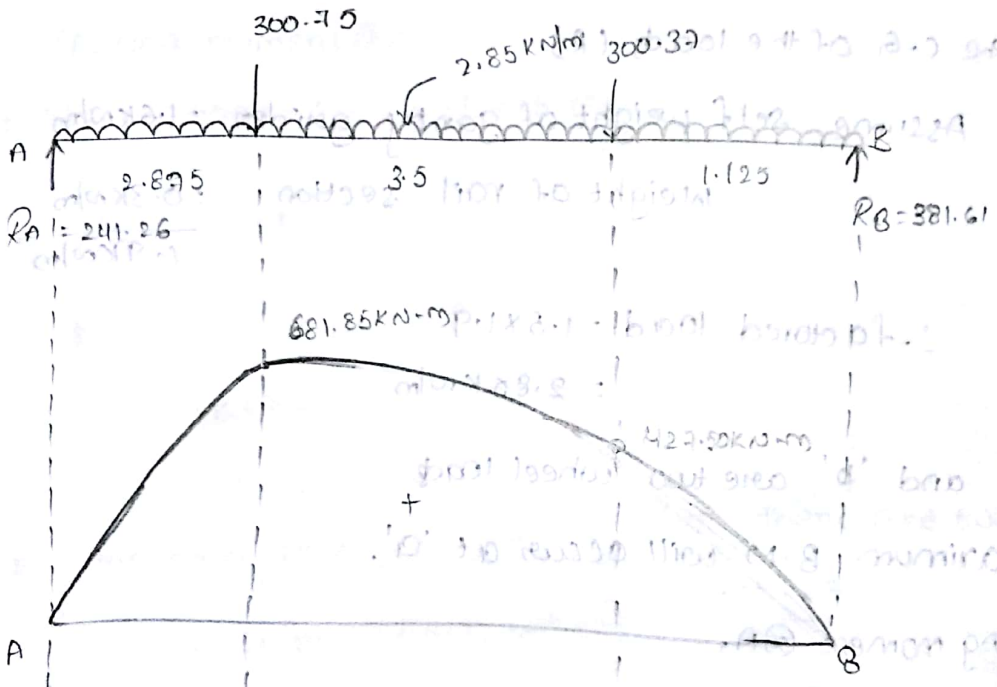
$$= (241.265 \times 2.875) - \frac{2.85(2.875)^2}{2}$$

$$\boxed{M_a = 681.85 \text{ KN.m}}$$

$$M_b = R_B \times 1.25 - \frac{2.85\ (1.25)^2}{2}$$

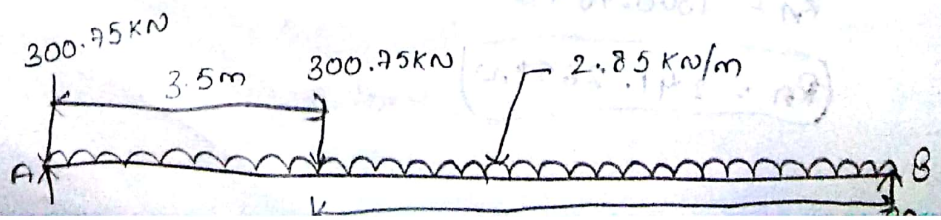
$$= 381.61 \times 1.25 - 2.85 \times \frac{(1.25)^2}{2}$$

$$\boxed{M_B = 427.50 \text{ KN.m}}$$



(3) Max. shear force in gantry girder (in vertical plane) :-

Max. S.F at A will occur when one of the wheel load is at A (i.e., on the column)



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Taking moment at B

$$R_A \times 7.5 = (300.75 \times 7.5) + (300.75 \times 4) + 2.85 \times \frac{(7.5)^2}{2}$$

$$\boxed{\therefore R_A = 471.83\,KN}$$

$$\boxed{Max.S.F = 471.83\,KN}$$

- Horizontal force (surge load) transverse to the rail

Total surge load = 10% of (~~load~~ load + crab load)

$= 0.10 \times (200+40)$

$= 24\,KN$

this load is shared by 4 wheels

$\therefore$ surge load per wheel $= \frac{24}{4}$

$= 6\,KN$

$\therefore$ factored surge load on each wheel $= 1.5 \times 6$

$= 90\,KN$

- Horizontal B.M due to surge load

horizontal bending moment (M_y)

by proportion.

$300.75\,KN \longrightarrow M = 681.85\,KN\text{-}m$

$9\,KN \longrightarrow ?$

$$\therefore M_y = \frac{681.85 \times 9}{300.75}$$

$$\boxed{M_y = 20.40\,KN\text{-}m}$$

- Longitudinal (horizontal) braking force

This force act along the rail.

braking force = 5% of static wheel load

$= (2 \times 160.40) \times 0.05$

$= 16.04\,KN$

$\therefore$ factored braking load $= 1.5 \times 16.04$

$= 24.06\,KN$

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(4) section selection for gantry girder.

Economic depth of girder $= \frac{L}{12}$

$= \frac{7500}{12}$

$= 625mm$

choice total depth of girder = 600mm.

Width of flange $= \frac{L}{40}$ to $\frac{L}{30}$

$= \frac{7500}{40}$ to $\frac{7500}{30}$

$= 187.5$ to $250mm$

$Z_p(req) = 1.4 \times \frac{M_u}{f_y}$ | $K = 1.4$

$= 1.4 \times \frac{681.85 \times 10^6}{250}$

$Z_p(req) = 3820 cm^2$

Select ISMB 600 with a channel ISMC 300

ISMB 600 @ 1.23 KN/m	ISMC 300 @ 0.358 KN/m
$Z_p = 3510.63 cm^3$	$A = 4504 mm^2$
$A = 15621 mm^2$	$t_f = 13.6 mm$
$t_f = 20.8 mm$	$t_w = 2.6 mm$
$t_w = 12.0 mm$	$b_f = 90 mm$
$b_f = 210 mm$	$I_{zz} = 6362.6 \times 10^4 mm^4$
$r = 20 mm$	$I_{yy} = 310.8 \times 10^4 mm^4$
$I_{zz} = 91.813 \times 10^4 mm^4$	$Z_p = 496.71$
$I_{yy} = 26.3 \times 10^4 mm^4$	$Z_e = 424.2 cm^3$

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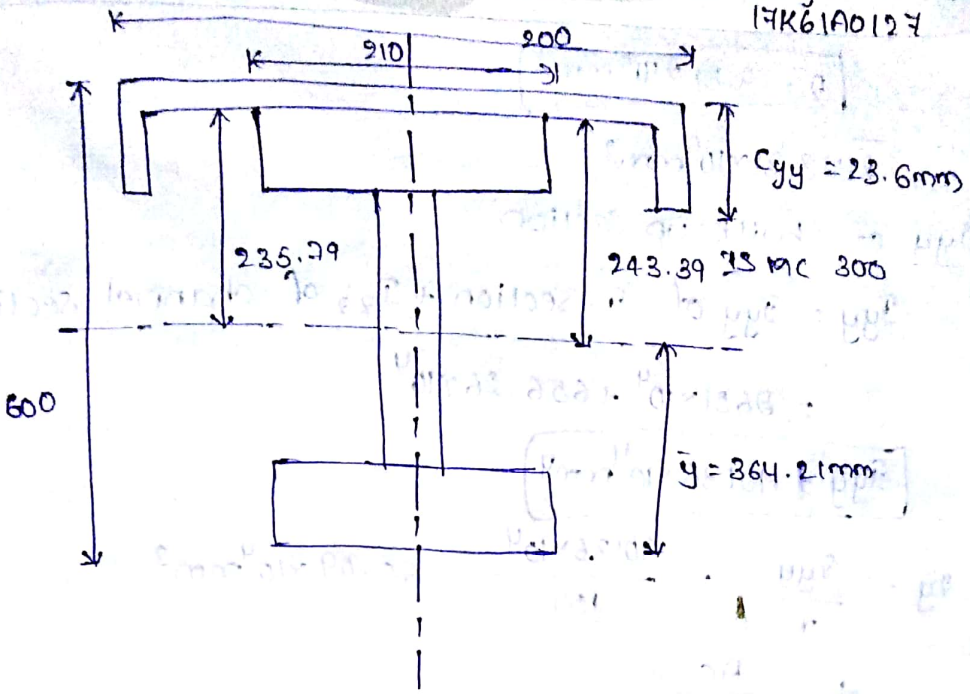

⑥ calculating I_{zz}, I_{yy}, Z_p and of the trial section

→ properties of combined section

$$\bar{y} = \frac{a_1\bar{y} + a_2\bar{y}_2}{a_1 + a_2}$$

$$= \frac{15621 \times 300 + 4562 \times 584}{15621 + 4564}$$

$$= \frac{7351.676}{2018}$$

$$= 364.21\text{mm (from bottom)}$$

$y_2 = 600 + 7.6 - 13.6$
$= 584\text{mm}$
$c_y = 25.6\text{mm}$

I_{zz} of build up section

$$I_{zz} = I_{zz_1} + I_{zz_2}$$

$$= (I_y + ah^2) + (I_y + ah^2)$$

$$= [91813 \times 10^4 + 15621 \times (64.2)^2] + [3108 \times 10^4]$$

$$= 14564 \times (243.3 + 23.6)^2$$

$$= 98253 \times 10^6 + 223.5 \times 10^6$$

$$\boxed{I_{zz} = 1906.11 \times 10^6}$$

Elastic modulus:

$$Z = \frac{I_{zz}}{y_{max}} = \frac{1906.11 \times 10^6}{364.21}$$

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$$\boxed{Z = 3.31 \times 10^6 \, mm^3}$$

$$Z = 3.31 \times 10^6 \, mm^3$$

I_{yy} of built up section

I_{yy} = I_{yy} of I-section + I_{zz} of channel section

$$= 2651 \times 10^4 + 656.26 \times 10^4$$

$$\boxed{I_{yy} = 90136 \times 10^4 \, mm^4}$$

$$Z_y = \frac{I_{yy}}{x} = \frac{90136 \times 10^4}{150} = 60.09 \times 10^4 \, mm^3$$

$$d_p = \frac{A_C}{2t}$$

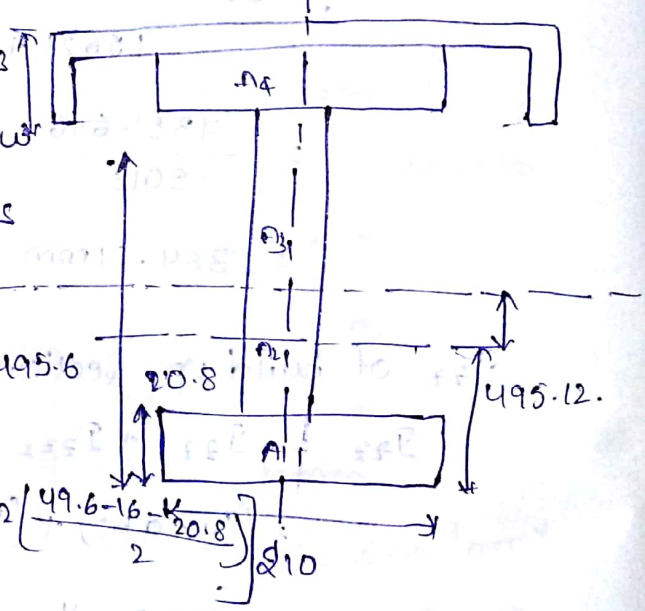
where A_C = area of channel section

t = thickness of section

$$d_p = \frac{456.4}{2 \times 12}$$

$$\boxed{d_p = 190.16 \, mm}$$

$$y_1 = 107.44.76 \times \frac{10.8}{2}$$



The plastic modulus below the equal area axis

$$\Sigma Ay = A_1\bar{y}_1 + A_2\bar{y}_2$$

$$= \left[(210 \times 20.8)\left(490.16 - \frac{20.6}{2}\right)\right]$$

$$+ \left[(490.16 - 20.8) \times 12 \left(\frac{49.6 - 16 - 20.8}{2}\right)\right]$$

$$\boxed{\Sigma Ay = 3.417 \times 10^6 \, mm^3}$$

The plastic modulus above the equal area axis

$$\Sigma A\bar{y} = A_3\bar{y}_3 + A_4\bar{y}_4 + A_5\bar{y}_5$$

$$= \left[(4564)(117.44 - 23.6) + (210 \times 20.8)\left(117.44 - 76 - \frac{20.8}{2}\right)\right.$$

$$\left. + (117.44 - 7.6 - 20.8)(12)\left(\frac{117.44 - 7.6 - 20.8}{2}\right)\right]$$

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$\boxed{\Sigma A\bar{y} = 0.910 \times 10^6 mm^3}$

$Z_p = 3.417 \times 10^6 + 0.910 \times 10^6$

Z_p (calculated) $= 4.327 \times 10^6 mm^3$

$4.327 \times 10^6 > 3.82 \times 10^6 mm^3$

Hence OK

The dimension details are satisfied.

⑥ check for moment capacity: (cls. 3.8.2.12)

$$M_d = \frac{\beta_p Z_p \cdot f_y}{\gamma_{m0}} < \frac{1.2 Z f_y}{\gamma_{m0}} \Rightarrow (1) \frac{4.32 \times 10^6 \times 250}{1.10} < \frac{1.2 \times 25 \times 4}{1.10}$$

M_d should be greater than M_u $\quad 981.18 \times 10^6 < 1178.18 \times 10^6$

⑦ check for shear capacity: (cls. 4.1) Pg.43

$$\text{Max } S.F = L/2 = \frac{7500}{2} = 3750$$

$$\text{Shear capacity } (V_d) = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} = \frac{15621 \times 250}{\sqrt{3} \times 1.10}$$

$\boxed{V_d = 2050 \text{ kN}}$

⑧ check for buckling:- (cls. 8.2.2 pg.54)

$$M_d = \beta_b \cdot Z_p \cdot f_{bd} = (1) \times 4.327 \times 410 = 1774.07$$

M_d should more than M_0

⑨ check for local buckling: (cls. 8.2.3.1)

buckling resistance $= (b_1 + n_1) t_w f_{cd}$

$$\frac{KL}{r} = 2.43 \, d/t_w \Rightarrow \frac{1.4 \times 7500}{20} = 2.43 \frac{d}{10.8}$$

buckling resistance must be more than ~~525~~ $d = 4494.0934$

⑩ weild design:

required shear capacity of weld

$$q_w = \frac{V \cdot A \cdot \bar{V}}{I_z}$$

$P = l_w \times t_t \times f_{wd}$

If $P > q_w$ safe.

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(11) check for deflection: (CLS - 5.6.1)

$$\delta_c = \frac{wL^3}{6EI}\left[\frac{3a}{SL^2} - \frac{a^3}{L^3}\right]$$

$$S = \frac{L}{750} = \frac{7500}{750} = 10$$

$$\delta_c = \frac{200(7500)^3}{6\times 2\times 10^5 \times 26.3\times 10^4}\left[\frac{3\times 2018}{10(7500)^2} - \frac{(2018)^3}{(7500)^3}\right]$$

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