IT

CONTAINS

UNIT-1

TO

UNIT 3

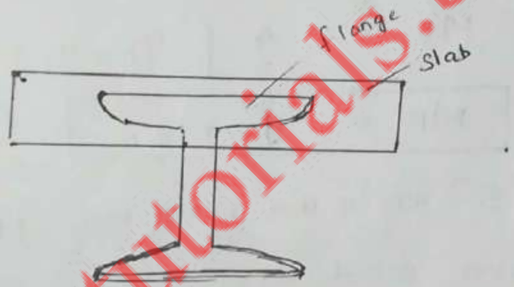
DESIGN OF BEAMS

Beam is a structural Member with length considerably larger than cross sectional dimensions, subjected to lateral (or) transverse loads which give rise to BM. and SF in the member.

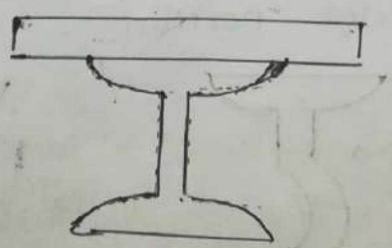
Depending on the lateral restrained provided for the compression flange of the beam, to avoid lateral buckling, the beams are classified into

1. Laterally supported - beams
2. Laterally unsupported - beams.

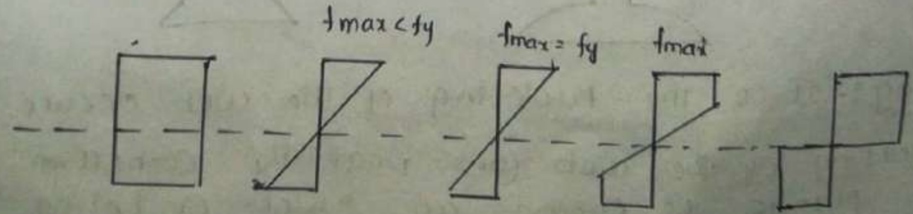
Laterally supported beam :- The beam under goes lateral buckling in addition to bending when subjected to bending about the major axis, since the moment of inertia about the minor axis is much less when comparative that about the major axis. The lateral buckling of the sections may be provented by providing continuous lateral support to the compression flange by fixing into the slab. as shown in figure.



Laterally unsupported beam :- When compression flange of the beam it is not support in the lateral direction (or) just resting of the slab on the beam as shown in figure there is always the possibility of lateral buckling.



Terminology :-



Elastic behaviour when the strain in the extreme fibres, $\epsilon_{max} < \epsilon_y$ i.e, $f_{max} < f_y$, the behaviour of the beam is elastic, the stress and strain varie linearly varie across the depth of the section.

$$\therefore M = f_{max} * Z_e$$

Z_e = Elastic section modulus.

Elastic plastic behaviour :- when $\epsilon_{max} = \epsilon_y$ i.e $f_{max} = f_y$ the yeilding of the extreme fibres takes place. when the inner fibres of the beam remains elastic.

$$\therefore M_y = f_y * Z_e.$$

when $\epsilon_{max} > \epsilon_y$ i.e $f_{max} > f_y$, there is no increase in the value of f_{max} probably limited to "f_y", cross section become plastic. at this stage for that deformation is possible at a constant moment and it is called "formation of plastic hinge" corresponding to plastic moment

$$\therefore M_p = f_y[A_c y_c + A_t y_t]$$

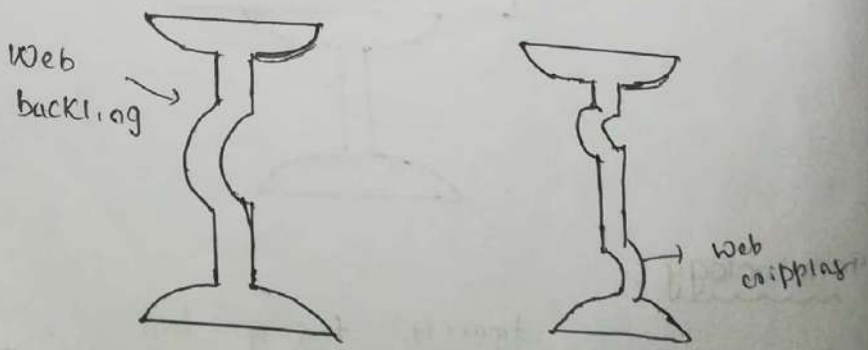
$$M_p = \frac{A}{2}[y_c + y_t]$$

$$\boxed{M_p = f_y * Z_p.}$$

Shape factor (S) :- It is the ratio b/w plastic section modulus to elastic section modulus.

$$\text{Shape factor (S)} = \frac{Z_p}{Z_e}$$

Web buckling :- When the applied load on the beam section is more than the design strength, the web portion of the section behaviour like a compression member, buckling will occur at the web portion



Web crippling :- It is the buckling of the web occurs just at the starting of the web (or) Near the connection of web and flange is known as "web crippling". It occurs due to the reaction force of the beam against the applied load.

Classification of Sections :- (Depends on mp & zp).
(cl. 3.7, pg-17)

IS 800 : 2007

		Plastic hinge	ductility
class 1	plastic	✓	✓
class 2	compact	✓	×
class 3	Semi compact	×	✓
class 4	Slender	×	×

Design of laterally supported beams :- (Pg-52, cl- 8.2.1 IS 800-2007)

Design of this beam consist of selecting a section on the basis of plastic section modulus and checking it for
ⓐ shear capacity ⓑ web buckling ⓒ web crippling
ⓓ Bending strength ⓔ Deflection.

PROCEDURE FOR DESIGN :-

Step 1 :- Calculation of factored load

Step 2 :- Calculation of max. BM and shear force.

Step 3 :- Selection of a trail section based on "Z_p"

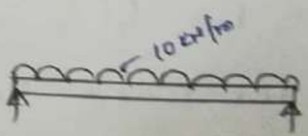
$$Z_p = \frac{M_p * \gamma_{m0}}{f_y}$$

where γ_{m0} = partial safety factor

Step 4 :- trail section is checked for ① shear ② bending strength ③ crippling ④ buckling ⑤ deflection

EG 1 :- A SS steel joist (beam) of 4m effective length is laterally supported throughout it carries a total UDL of 40 KN including self weight. Design an appropriate section using steel of grade Fe 410. Assume any required if required.

Sol :- Given data is.



total load = 40 KN

UDL (w) = $\frac{40}{4}$ = 10 KN/m

(w) factored load = $1.5 * 10$ = 15 KN/m

Max. Bending moment (M_{max}) = $\frac{Wl^2}{8} = \frac{15 \times 4^2}{8}$

M_{max} = 30 KN-m

Max. shear force (V) = $\frac{WL}{2} = \frac{15 \times 4}{2}$.

(V) = 30 KN.

plastic section modulus.

$$Z_p = \frac{M_p \times \gamma_{m0}}{f_y} = \frac{30 \times 10^6 \times 1.1}{250}$$

$\therefore$ $Z_p = 132 * 10^3 \ mm^3$

(or) $Z_p = 132 \ cm^3$.

Selecting a trail section based on pelastic section modulu

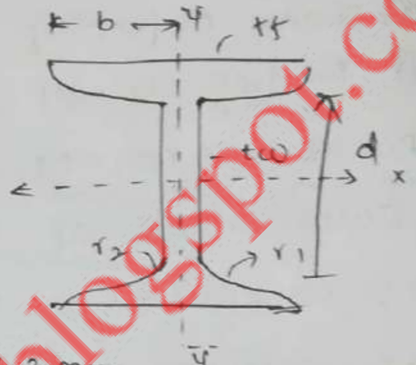
(Pg-198)

ISLB 200 @ 19.8 Kg/m →

$Z_e = 169.7 \ cm^3$

$\therefore Z_p = 184.34 \ cm^3$

Properties of section :-



Sectional Area = 25.27 cm^2

Depth of the section (h) = 200 mm

h = 20 cm

width of the flange b_f = 100 mm

b_f = 10 cm

Thickness of the flange (t_f) = 7.3 mm.

t_f = 0.73 cm

Thickness of the web (t_w) = 5.4 mm = 0.54 cm

r_1 = 9.5 mm = 0.95 cm , r_2 = 3.0 mm = 0.3 mm.

Root Radius   Radius toe.

Slope of the flange (ϕ) = 91.5°

$\therefore$ depth of the web (d) = $H - 2(t_f + r_1)$

$d = 20 - 2[0.73 + 0.95]$

$\boxed{d = 16.64 \ cm}$

Step 4 :-

Classification of section ((Page-18) (table-2))

$b = \frac{b_f}{2} = \frac{10}{2} = 5 \ cm$, $d = 16.64 \ cm$

For rolled steel section :

for flange, $\frac{b}{t_f} = \frac{50}{7.3} = 6.849 < 9.4\varepsilon = 9.4(1)$

$\therefore \frac{b}{t_f} = 9.4$.

For web $\frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 84\varepsilon$

$\therefore$ Section is classified as plastic section.

$\therefore$ The section ISLB 200 is plastic section

Checks:-

(i) Check for shear capacity :- $V \le V_d$

where V = nominal shear force applied

V_d = design for shear strength (Pg-59)

$$V_d = \frac{V_n}{\gamma_{mo}} \quad \text{and} \quad V_n = V_p$$

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

$A_v = h\,t_w = 200 * 0.54 = 10.8\ cm^2$

$$V_d = \frac{10.8 * 250}{\sqrt{3} \times 1.1} = 1417.0\ KN \ge 30\ KN$$

Hence the condition is safe for the section

(ii) Check for design bending strength :- (Pg-53, 8.2.1.2)

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

$$M_d = \frac{1 \times 184.34 \times 250 \times 10^3}{1.1}$$

$M_d = 41.8\ KN\text{-}m > 30\ KN\text{-}m$.

Hence section is safe against bending

(iii) Check for web buckling :- (Pg-18, Note (below the table))

$\frac{d}{t_w} > 67\varepsilon$ (for a web without stiffness) → (Pg-18)

$\frac{d}{t_w} > 67\varepsilon \sqrt{\frac{K_v}{5.35}}$ (for web with stiffness) (Page-67)

$\therefore \frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 67\varepsilon$.

Hence section is safe against web buckling

(iv) Check for web crippling / bearing :- (Pg-67, 8.7.4)

$$F_w = \frac{(b_1 + n_2)\, t_w * f_{yw}}{\gamma_{mo}}$$

n_2 = length obtained by dispersion through the flange to web junction 1:2.5 for plane of the flange

b_1 = stiff bearing length = 100 mm

~~Fcw =~~ ~~100+42~~ $n_2 = [t_f + r_1]\, 2.5$

$n_2 = [7.3 + 9.5] * 2.5 = 42\ mm$

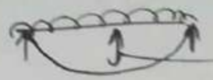
~~Fcw~~,

$F_w = \frac{(100+42) * 5.4 * 250}{1.1}$

$\therefore F_w = 174.27 \text{ KN} > 30 \text{ KN}$

Hence the Section is Safe against web crippling.

Check for deflection :-



, $\frac{5wL^4}{384EI}$

$$\delta = \frac{5wL^4}{384EI}$$

$$\delta = \frac{5 * 10 \times 10^3 \times [4 \times 10^3]^4}{384 \times 2 \times 10^5 \times 1696.6 \times 10^4}$$

$$\delta = 9.827 \text{ mm}$$

$$\delta_{allowble} = \frac{L}{300} = \frac{4 \times 10^3}{300} = 13.33 > 9.82.$$

Hence section is Safe against deflection

Type-II

(*) Determine the design bending strength of ISLB 350 @ 486 N/m considering the beam to be (a) Laterally Supported (b) Laterally Unsupported. The design shear force V is less than the design Shear Strength. The Unsupported length of the beam is 3m. assume Steel of grade Fe 410. (b)

Sol:-

Given data is

$F_u = 410 \text{ N/mm}^2$, $f_y = 250 \text{ N/mm}^2$

Unsupported length $(L) = 3 \text{ m}$

Step 1:- Section properties ISLB 350

Sectional Area $(A) = 63.01 \text{ cm}^2$

Depth of the Section $(h) = 350 \text{ mm}$

Width of the Flange $(b_f) = 165 \text{ mm}$

Thickness of the flange $(t_f) = 11.4 \text{ mm}$

Thickness of the web $(t_w) = 7.4 \text{ mm}$

$I_{xx} = 13158.3 \text{ cm}^4 = 13158.3 \times 10^4 \text{ mm}^4.$

$Z_e = 751.9 \times 10^3 \text{ mm}^3$

$Z_p = 851.1 \times 10^3 \text{ mm}^3$

Step 2:- Classification of Section. (pg.18, table-2)

For flange, $\frac{b}{t_f} = \frac{(165/2)}{11.4} = 7.23 < 9.4\varepsilon$

For web, $\frac{d}{t_w} = \frac{295.2}{...}$ = ... $\epsilon = \sqrt{\frac{250}{f_y}}$

$[\because d = H - 2[t_f + r_1] = 295.2\ mm$ $\epsilon = 1$

So Hence the Section is Said to be plastic Section

$\therefore \beta = 1$

Step 3: (a) For Laterally Supported beam

$M < M_d$; $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \leq \frac{1.2 Z_e f_y}{\gamma_{m0}}$

$$M_d = \frac{1 \times 851.1 \times 10^3 \times 250}{1.1}$$

$$\boxed{M_d = 193.4\ kN\text{-}m}$$

RHS $\Rightarrow$ $\frac{1.2 \times 751.9 \times 10^3}{1.1} = 205.1\ kN\text{-}m$

$193.4 \leq 205.1\ kN\text{-}m$

(b) For laterally Unsupported beams:-

$M < M_d$

$M_d = \beta_b Z_p f_{bd}$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}$$

$$\chi_{LT} = \frac{1}{\{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1.0$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

$\alpha_{LT} = 0.21$ for rolled steel section

$\alpha_{LT} = 0.49$ for Welded steel section.

a) Non dimensional Slenderness ratio. λ_{LT}

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{\frac{1.2 Z_e f_y}{M_{cr}}}$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}}$$

Elastic lateral torsional buckling moment.

$$M_{cr} = \sqrt{\left\{\left[\frac{\pi^2 E I_y}{(L_{LT})^2}\right]\left[G I_t + \frac{\pi^2 E I_\omega}{(L_{LT})^2}\right]\right\}} = \beta_b Z_p f_{cr,b}$$

$$M_{cr} = \frac{\pi^2 * 2*10^5 * 631.9*10^4 * 331.4}{2*3000}\left[1+\frac{1}{20}\left[\frac{3000/31.7}{331.4/11.4}\right]^2\right]$$

$\therefore M_{cr} = 267.8$ KN-m

$$\lambda_{LT} = \sqrt{\frac{B_b Z_p f_y}{M_{cr}}} \leq \sqrt{\frac{1.2 Z_e f_y}{M_{cr}}}$$

$$\lambda_{LT} = \sqrt{\frac{1*851.1\times10^3*250}{267.8\times10^6}} \leq \sqrt{\frac{1.2*751.9\times10^3}{267.8*10^6}}$$

$$\boxed{\lambda_{LT} = 0.89 \leq 0.9}$$

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}^2\right]$$

$$\phi_{LT} = 0.5\left[1+0.21(0.89-0.2)+0.89^2\right]$$

$$\boxed{\phi_{LT} = 0.96}$$

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT}+\left[\phi_{LT}^2-\lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

$$\chi_{LT} = \frac{1}{0.96+(0.96^2-0.9^2)^{0.5}} \leq 1.0$$

$\chi_{LT} = 0.74 \leq 1.0$.

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.74\times250}{1.1} = 168.18 \text{ N/mm}^2$$

$M_d = B_b Z_p f_{bd}$

$M_d = 1\times851.1\times10^3\times168.18$

H.W laterally unsupported

$$\boxed{M_d = 143.41 \text{ KN-m}}$$

(*) Design a laterally unsupported beam for following data effective span 4m. max. bending moment 550 KN-m, max. shear force 200 KN, grade of steel Fe410

Sol:-

(*) A Roof of a hall measuring 8m*12m consists of 100mm thick RCC slab supported on steel I-beam spaced at 3m apart, the finishing load may be taken as 1.5 KN/m². and live load as 1.5 KN/m². design the steel beam, clear span is 8m and end bearings 200mm

① Design a S.S.B of span 4m carrying a reinforced concrete floor [illegible] top compression flange. The UDL is made up of 20 KN/m imposed load and 20 KN/m dead load. assume Fe 410 grade of steel?

② Design a beam of 5m effective span, carrying a Uniform load of 20KN/m. If the compression flange is laterally un supported and also check for deflection and shear draw to scale the (i) C/S (ii) L/S (iii) planning?

Design procedure for Plate Girder:-

Step1:- Loading (Pg - 63)

Assume self weight of girder

$$= \frac{\text{Total factor load}}{200} \quad (KN/m)$$

Total UDL,

w = udl (DL + L.L) + self weight of girder

Step2:- S.F and B.M calculation,

Step3:- Determination of optimum depth of web (or) Section selection

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

$K = \frac{d}{t_w}$ (For $\frac{d}{t_w} \le 67\varepsilon$ - no end stiffer & intermediate transverse stiffness are required)

(for $\frac{d}{t_w} \simeq 100$ intermediate transverse stiffer not req & end stiffers required)

∴ Area of flange required

$$A_f = \frac{M}{f_y * d} \cdot \gamma_{mo}$$

For flange to be plastic. for welded section

take $\frac{b}{t_f} \simeq 8.4\varepsilon$ (Table-2, code pg-18)

Step4:- check for minimum web thickness requirement (As per IS: 800-2007, cl- 8.6.1.1, Pg-63)

Step5:- check for moment capacity.

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

$B_b = 1$ for plastic and semi compact section,

Z_p = plastic section modulus of flanges only

M_d should be greater than factored moment

Step 6 :- Check for shear capacity of web.

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

Step 7 :- Check for shear buckling of web using simple post - critical method (Cl - 8.4.2.2 (a))

$$V_d = V_{cr}.$$

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

⇒ If $V_{cr} > V$ (Intermediate transverse stiffners are not required

⇒ If $V_{cr} > V$ (Intermediate transverse stiffners are required to improve the buckling strength of slender web)

Step 8 :- Check for Anchor forces in the end panel (Cl - 8.5.3)

Step 9 :- Design of end bearing stiffener

Step 10 :- Design of connection b/w web plate and flange.

Step 10 :- Design of intermediate transverse stiffners, if required

Step 12 :- Design of connection b/w stiffners and web.

Problem :-

(*) Design a welded plate girder for a simply supported bridge deck beam with clear span 24 m, subjected to D. L of 20 KN/m (excluding self weight). L.L 10 KN/m and two concentrated loads 200KN each at 6m from each end, Assume that the top compression flange of plate girder is restrained laterally and prevented from rotating. Use Fe - 415 grade of steel. Design as an Unstiffened plate girder with thick web.

Sol :-

Step 1 :- load Analysis

Total factored load on girder

$$= 1.5(20+10) \times 24 + 1.5 \times 2 \times 200$$

$$= 1680 \text{ KN}.$$

$\therefore$ Self weight of girder $= \frac{\text{Total factored load}}{200}$

$= \frac{1680}{200} \approx 8.4$ KN/m.

$\therefore$ Total UDL $= 20 + 10 + 8.4$

$= 38.4$ KN/m

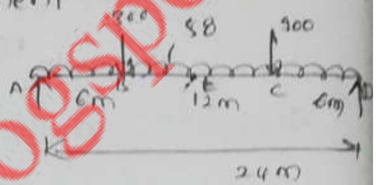
$\therefore$ Factored UDL $= 1.5 \times 38.4 = 57.6$ KN/m

$(W) = 58$ KN/m

$\therefore$ Factored concentrated loads $= 1.5 \times 200$

$= 300$ KN (each)

Step 2 :- Shear force and Bending moment



Each Reaction $= \frac{\text{Total load}}{2}$

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

$R_A = R_B = 996$ KN

Maximum SF $V_z = 996$ KN.

Maximum B.M at E.

$M_E = (996 \times 12) - (58 \times 12 \times 6) - (300 \times 6)$

$\therefore M_E = 5976$ KN-m

$\therefore M_C = M_D = 996 \times 6 - 58 \times 6 \times 3 = 4932$ KN-m

Step 3 :- Section selection.

Optimum depth of the girder,

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

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

Provide $d = 1400$ mm

$K = \frac{d}{t_w}$ ~~200~~

$K = \frac{d}{t_w} > 200$

intermediate stiffners are required.

To avoid transverse stiffness assume $K = \frac{d}{t_w} = 150$

Assume thickness of web plate 12 mm (thick web)

$\therefore$ Size of the web plate is $\Rightarrow$ 1400 x 12 mm

$\frac{d}{t_w} = \frac{1400}{12} = 116.66 > 67\varepsilon$ (cl-8.4)

check for shear buckling of web is required

Assume that B.M is resisted by flanges only.

Area of flange

$$A_f = \frac{M}{f_y} * \frac{\gamma_{mo}}{d}$$

$$A_f = \frac{5976*10^6}{250} * \frac{1.10}{1420}$$

$$A_f = 18781.71 \text{ mm}^2$$

For compression flange with welded section.

$$\frac{b}{t_f} = 8.4\,\varepsilon \quad (\text{For plastic section})$$

$\therefore$ $b = 8.4\,t_f$ (table-2, P-18)

$\therefore$ Area of one flange

$$A_f = b_f * t_f$$

$$A_f = 2b * t_f$$

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

$\therefore$ $2(8.4\,t_f)\,t_f = 18781.71$

$$t_f = 33.43 \text{ mm}$$

Provide $t_f = 40$ mm

$$b_f = \frac{18781.71}{40} \simeq 469.54 \text{ mm}$$

Provide $b_f = 500$ mm

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

Step 4: minimum web thickness requirements

* Serviceability requirement :- (IS-800-2007, cl-8.6.1.1, P-63)

when transverse stiffners are not provided.

$$\frac{d}{t_w} = \frac{1400}{12} = 116.67 < 200\,\varepsilon \quad (\text{Hence ok})$$

* compression flange buckling requirement :

· When transverse Stiffners are not provided

$$\frac{d}{t_w} = \frac{1400}{12} = 116.67 < 345\,\varepsilon_f^2 \quad (\text{Hence ok})$$

Therefore minimum thickness of the web portion is 12 mm is sufficient.

Step 5:- check for moment capacity of flanges

Section classification for flange,

$$\frac{b}{t_f} = \frac{250}{40} = 6.25 < 8.4\,\varepsilon$$

flanges are plastic

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

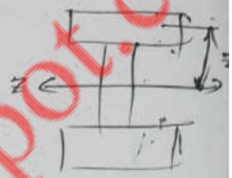
($\beta_b = 1$ for plastic section

Z_p = plastic section modulus

moment of flanges ZZ axes)

$$Z_p = 2 \times [(500 \times 40) \times 720]$$

$$Z_p = 28.80 \times 10^6 \ mm^3$$



$$M_d = \frac{1 \times 28.80 \times 10^6 \times 250}{1.10}$$

$$M_d = 6545.45 \times 10^6 \ N.mm$$

$$M_d = 6545.45 \ KN.m > 5976 \ KN.m$$ (Hence ok.)

Section is adequate for moment carrying capacity

Step 6:- check for shear capacity of web (cl 8.4) (Pg-59)

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

$$V_d = \frac{(400 * 12) * 250}{\sqrt{3} * 1.10} = 2.204 * 10^6 \ N$$

$\therefore V_d = 2204$ KN

check:- $0.6\, V_d = 0.6 \times 2204 = 1322$ KN > 996 KN

Hence OK.

Step 7:- check for shear buckling of web (using simple post critical method)

Nominal shear strength

$$V_n = V_{cr}$$

V_{cr} = Shear force corresponding web buckling

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

τ_b - depends upon "λ_w"

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

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

$\because$ (pg-60) $K_v = 5.35$, $\mu = 0.3$ (pg-12)

$$\tau_{cr} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1400}{12}\right)^2}$$

$$\therefore \boxed{\tau_{cr} = 71.04\ N/mm^2}$$

$$\lambda_w = \sqrt{\frac{250}{\sqrt{3} \times 71.04}} = 1.425 > 1.2$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.425^2}$$

$$\tau_b = 71.08\ N/mm^2$$

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

$$= 16800 \times 71.08$$

$V_{cr} = 1194\ kN > $ [illegible] kN (~~Hence~~ ok).

Hence Shear strength of the web is greater than the applied shear.

NOTE :-

* If $V_{cr} < V_z$ the buckling strength is required to be improved by changing the web section (or) should be design using tension field action
* As the compression flange is restrained, there is no need to check for lateral torsional buckling

Step 8 :- Design of load carrying stiffners (Cl-8.7.3 Pg-67)

load carrying web stiffners should be provided where compressive forces applied through a flange by loads (or) reactions, exceeds the buckling strength. f_{cdw} of the unstiffened web.

buckling of unstiffened web

$$f_{cdw} = (b_1 + n_1)\, t_w * f_{cd}$$

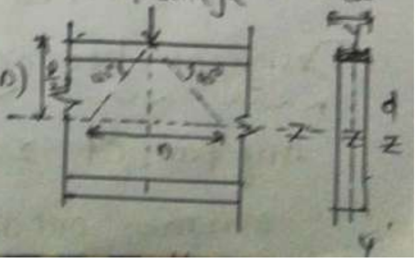
b_1 = width of stiff bearing on the flange

$b_1 = 0$

$n_1 = 2 \times B/2$ (For 45° load dispersion)

$$n_1 = 2 \times \frac{1480}{2}$$

$n_1 = 1480$ mm



$t_\omega = 12\,mm \qquad \frac{KL}{r_y} = ?$

f_{cd} will be given from (T 8.2.1 − Cl, TN9C pg − 42)

$K_L = 0.7d \qquad$ (pg - 66)

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{d t_\omega^2}{12 \times d \times t_\omega}} = \frac{t_\omega}{\sqrt{12}}$$

$$\frac{KL}{r_y} = \frac{0.7d}{t_\omega/\sqrt{12}} = 2.43\, d/t_\omega$$

$$\frac{KL}{r_y} = 2.43 * \frac{1400}{12} = \underline{283.5}$$

By extrapolation

$240 \rightarrow 26.2$	$240 \rightarrow 26.2$
$250 \rightarrow 24.3$	$283.5 \rightarrow ?$
$10 - 1.9$	$43.5 \rightarrow ?$

$\frac{10}{1.9} = \frac{43.5}{x}$

$x = 8.265$

$f_{cd} = 26.2 - 8.265 = 17.94\ N/mm^2$

$f_{cdw} = (b_1 + n_1)\, t_\omega * f_{cd}$

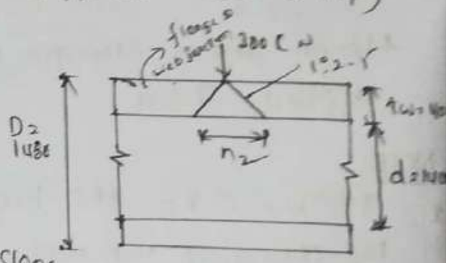
$f_{cdw} = (0 + 1480) * 12 * 17.94 = 318.64\ KN < 7300\ KN$

∴ web is safe against buckling below 300 KN

∴ No load carrying stiffer required

Step 9 :- Design of load bearing stiffners (Cl - 8.7.4)

$$F_\omega = \frac{(b_1 + n_2)\, t_\omega\, f_{y\omega}}{\gamma_{m0}}$$



assume $b_1 = 0$

$n_2 = 2 \times (40 \times 2.5)$

$n_2 = 200\ mm$ (for slope 1:2.5)

$F_\omega = (0 + 200) * 12 * \frac{250}{1.1}$

$\boxed{F_\omega = 545.45\ KN}$

① At the location of 300 KN point load

$F_\omega = 545.45\ KN > 300\ KN$ (Hence No bearing Stiffners below point load is required)

② At Supports.

at support reaction = 996 KN

$F_\omega = 545.45\ KN < 996\ KN$

Hence bearing stiffners are required at the supports

Design of bearing stiffners at supports :-

Try pair of 220 × 12 mm mild steel plate (Assume)

Maximum out stand of stiffner [Cl - 8.7.1.2] (pg - 65)

$= 20\, t_q\, \varepsilon$

$= 20 \times 12 \times 1$

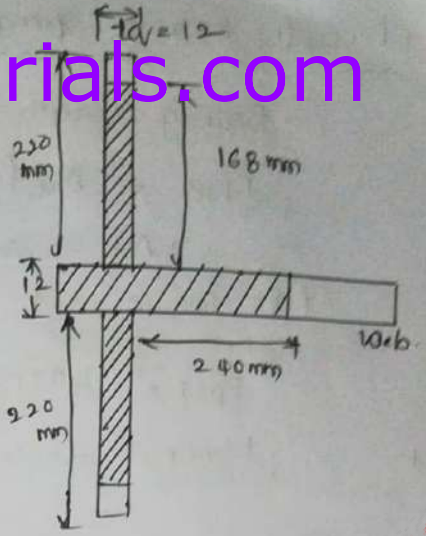
$= 240\ mm$

$\therefore 220 < 240\ mm$

Hence stand of core section

$14\, t_q\, \varepsilon = 14 * 12 * 1$

$= 168\ mm < 220\ mm$



$\therefore$ Use out stand $= 168\ mm$

Check for buckling :-

Effective Area $= 2 \times (168 \times 12) + (240 \times 12)$

$\therefore eff(A) = 6912\ mm$

$$I_{yy} = \frac{12 \times (2 \times 168 + 12)^3}{12} + \frac{240 \times 12^3}{12}$$

$\therefore I_{yy} = 42.17 \times 10^6\ mm^4$

$$r_y = \sqrt{\frac{I_{yy}}{A}}$$

$$r_y = \sqrt{\frac{42.17 \times 10^6}{6912}}$$

$\boxed{r_y = 78.10\ mm}$ (cl- 8.7.1.5 - pg. 66)

$\therefore \frac{KL}{r} = \frac{0.7d}{r} = \frac{0.7 \times 1400}{78.10} = 12.55$

(From IS 800 - 2007, Table 9(c). P-42)

$f_{cd} = 226.24\ N/mm^2$

Buckling resistance

$F_{qd} =$ effective area $* f_{cd} = 6912 * 226.24$

$F_{qd} = 1563770\ N$ (or) $1563.77\ KN > 996\ KN$ (OK)

Hence the stiffeners is safe to bearing

Check for bearing strength at end Stiffners :-

Bearing strength = Fpsd

$F_{psd} = A_q f_{yq} / (0.8\,\gamma_{mo}) \geq F_x$.

$A_f = 2(220-15) * 12 = 4920\ mm^2$

(IS 800 : 2007 . pg. 66) (cl - 8.7.5.2)

$F_{psd} = (4920 \times 250) / (0.8 \times 1.1) = 1397727\ N > F_x = 996\ K$

Hence. Safe in bearing.

Step II :- Weld connecting web plate and flange

Assume fillet weld on each side of web

$q_w = \dfrac{V \cdot A_f \cdot \bar{y}}{2 \times I_z}$ → q_w = horizontal shear force per mm length of weld.

$\therefore V = 996\ KN$

$A_f = 500 \times 40 = 20{,}000\ mm^2$

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

$I_z = \dfrac{500 \times 1480^3}{12} - \dfrac{(500 * 12) \times 1400^3}{12}$

$\therefore I_z = 2.35 \times 10^{10}\ mm^4$

$q_w = \dfrac{996 \times 10^3 \times 20{,}000 \times 720}{2 \times 2.35 \times 10^{10}} = 305.15\ N/mm$

$\therefore q_w = 0.305\ KN/mm$

Strength of weld for mm

$P = L_w \cdot t_t \cdot f_{wd}$

$P = 1 * 4.2 \times 189$

$P = 793.8\ N/mm$

$\therefore P = 0.794\ KN/mm$

($L_w = 1$ mm) least

$t_w = 12$ mm

minimum size of weld

provide s = 6mm

$t_t = 0.7s = 0.7 \times 6$

$t_t = 4.2$ mm

$f_{wd} = 189\ N/mm^2$

c/c spacing of weld $= \frac{P}{q_w} = \frac{0.794}{0.305} = 2.60$

Assume effecting length of weld = 100 mm

c/c spacing of weld = 2.6*100 = 260 mm

clear spacing = 260 - 100 = 160 mm

but clear spacing $\ngtr$ 12t = 12 x 12 = 144 mm

Since 160 mm > 144 mm. (pg-79 cl-10.5.5.2)

provide clear spacing 100 mm and effective length of weld 100 mm at each side of the web.

Step 12 :- Weld connecting end stiffner and web.

$$q_1 = \frac{t_w^2}{5 b_s}$$ (IS: 800-2007 cl-8.7.2.6 - pg-67)

where t_w = web thickness in mm.

b_s = Outstand width of the Stiffener in mm

$$q_1 = \frac{(12)^2}{(5*220)} = 0.13 \text{ KN/mm}$$

∴ Net SF = V_z - SF resisted by web

SF. resisted by web:

$F_{cdw} = (b_1+n_1) t_w * f_{cd}$. (IS: 800-2007 cl-8.7.3, p-67)

$b_1 = 0$

$n_1 = \frac{D}{2} = \frac{1480}{2} = 740$ mm (At end support dispersion on one side only)

t_w = 12 mm

$\frac{KL}{r_y} = 2.43 \frac{d}{t_w} = 2.43 * \frac{1400}{12} = 283.5$ | IS 800-2007 Table 9(c) P-42

∴ f_{cd} = 26.2 - 8.265 = 17.94 N/mm²

∴ $f_{cdw} = (b_1+n_1) * t_w * f_{cd}$

f_{cdw} = (0 + 740) * 12 * 17.94 = 159.31 KN

(half as Calculated in step 8) | chamfer length = 15 mm

∴ Net shear force = 996 - 159.31 = 836.69 KN

∴ length of weld = 1400 - 2*15 = 1370 mm

$\therefore q_2 = \frac{836.69}{1360} = 0.61$ KN/mm

$\therefore q_w = q_1 + q_2 = 0.13 + 0.61 = 0.74$ KN/mm

$\therefore$ Force on each side of weld.

$= \frac{0.74}{2} = 0.37$ KN/mm

Strength of the weld per mm (6mm size)

$P = 0.794$ KN/mm

0.794 KN/mm > 0.37 KN/mm (Hence ok)

Consider intermittent fillet weld

For 1 mm effective length.

c/c spacing of weld $= \frac{P}{q_w} = \frac{0.794}{0.37} = 2.14$

Assume effective length of weld = 100mm

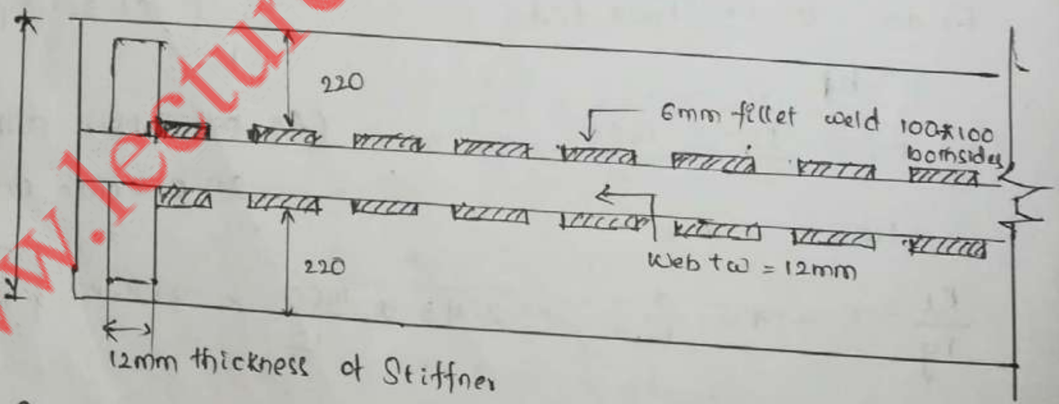
$\therefore$ c/c spacing of weld $= 2.14 * 100 = 214$mm (say 200mm)

$\therefore$ Clear spacing $= 200 - 100 = 100$mm

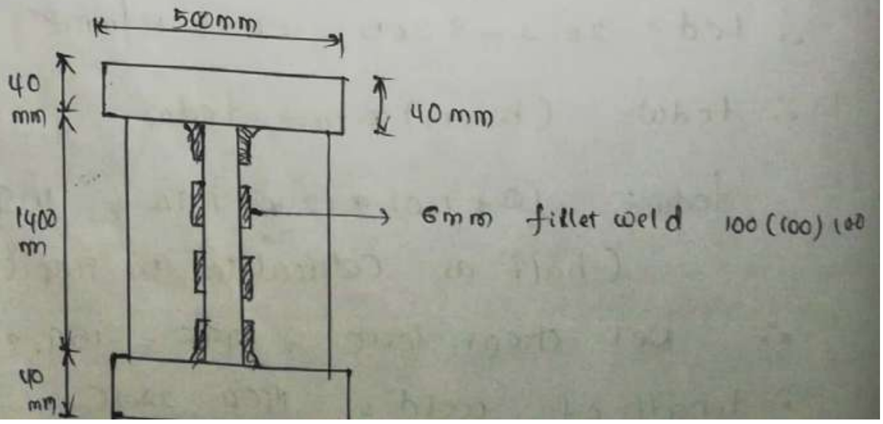
Clear spacing $\ngtr 12t = 12 * 12 = 144$ mm

$\therefore$ provide 6mm fillet weld, 100mm clear spacing for 100mm on each side of the stiffner.

Plan of plate girder:-



Section:-



Gantry Girder :-

Gantry girders are laterally unsupported beams to carry heavy loads from one place to another, place in industrial buildings such as factories, workshops, lifting of heavy materials, equipment etc at the construction sites or with in the building

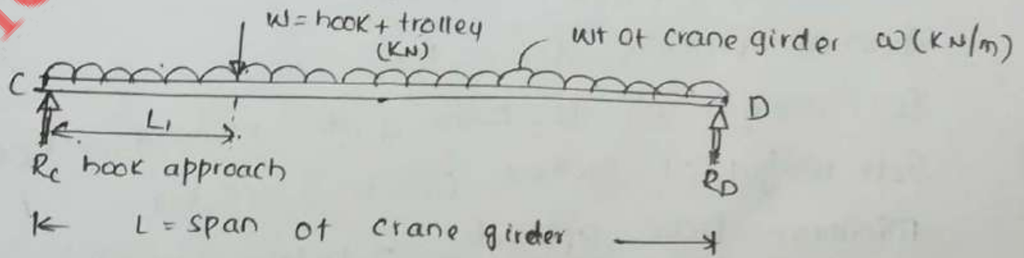
Loads acting on gantry girder :-

① Horizontal forces → Surge forces ($\perp^{er}$ to the rails), Braking forces ($\parallel^{er}$ to rails)

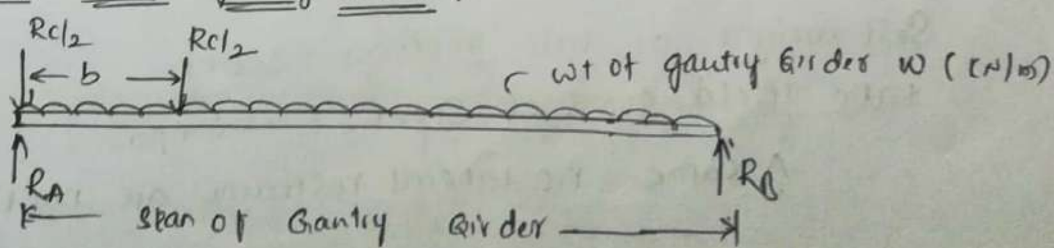
② Verticle forces → Wheel loads, crab weight + hook load, weight of crane girder

Type of load	Impact allowence (%)
Verticle forces transferred to wheels	
ⓐ For EOT cranes	25% of the maximum static wheel load
ⓑ For hand-operated cranes	10% of max. static wheel load.
Horizontal forces transverse to the rails.	
ⓐ For EOT cranes	10% of weight of the crab and weight of lifted crane
ⓑ For hand operated cranes	5% of weight of the crab and weight of lifted cranes
ⓒ Horizontal forces along the rails	5% of static wheel loads for EOT or hand operated

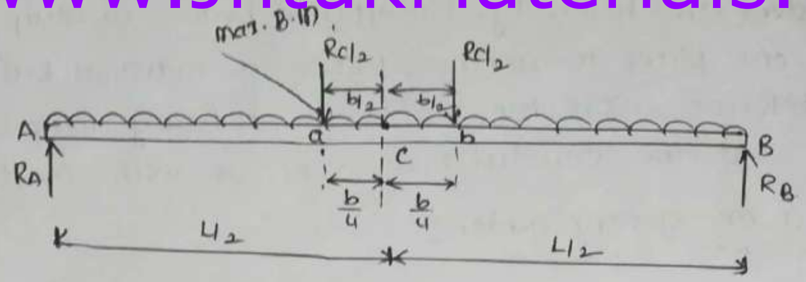
load Analysis on crane Girder :-



load Analysis on gantry girder :-



Max BM calculation :-



Design procedure :-

Step 1 :- Maximum wheel load

Step 2 :- Maximum B.M in gantry girder

Step 3 :- Maximum shear force

Step 4 :- Section selection for gantry girder.

Step 5 :- Calculate I_{zz}, I_{yy} and Z_p of the trail section selected

Step 6 :- check for moment capacity

Step 7 :- check for shear capacity

Step 8 :-

⊛ Design a gantry girder for the following data

Crane capacity = 200 KN.

Span of gantry girder = 7.5 m

Span of Crane Girder = 15 m

Self weight of the crane girder excluding trolley = 200 KN

Self weight of trolley (crab) = 40 KN.

Minimum hook aproach = 1.2 m

wheel base of crane = 3.5 m

Self weight of rail section = 300 KN

take yield stress of steel = 250 mpa.

Assume no lateral restrains are not required.

Step 1: maximum wheel load

= crane capacity + wt of trolley (crab)

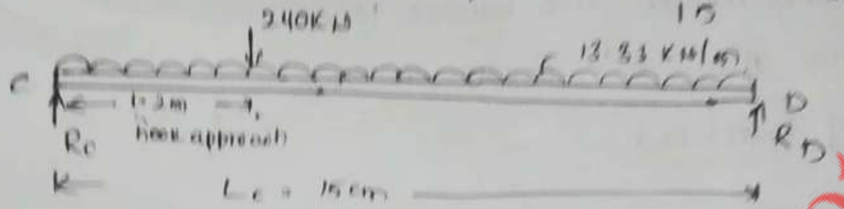
= 200 + 40

= 240 KN.

minimum hook approach = 1.2 m

Total weight of crane girder = 200 KN.

Weight of crane girder per m = $\frac{200}{15}$ = 13.33 KN/m



Reactions:-

$R_C + R_D = 240 + 13.33 (15) = 439.95$ KN

Taking moment @ C

$R_D \times 15 = 240 \times 1.2 + 13.33 \times 15 \times 7.5$

$R_D = 119.18$ KN

$\therefore$ R_C = Total load $- R_D$

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

$R_C = 320.77$ KN (maximum reaction)

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, Cl- 6.1, P- 15

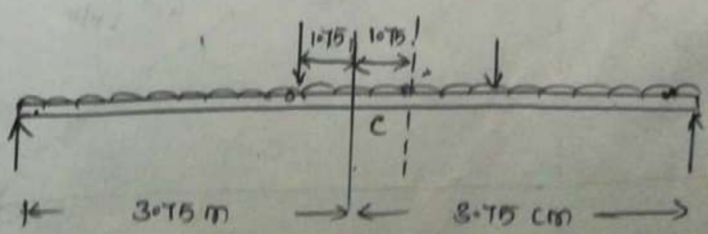
$\therefore$ Design load on each wheel

= 1.25 × 160.40 = 200.50 KN.

$\therefore$ Factored load on each wheel

= 1.5 × 200.50 = 300.75 KN.

Step 2:- Maximum B.M in the Gantry girder (In vertical plane)



Max BM will occur below wheel load 'a' when centre of the sp... is at equal distance from the ... wheel load and the c... the loads (R)

Assume Self weight of gantry girder = 1.6 KN/m

Weight of rail section = 0.3 KN/m

Total load = 1.9 KN/m.

$\therefore$ Factored UDL = $1.5 \times 1.9 = 2.85$ KN/m

"a" and "b" are two wheel loads.

Maximum B.M will occur at "a".

Taking Reactions:-

$$R_A + R_B = (2 \times 300.75) + \left(2.85 \times 7.5 \times \frac{7.5}{2}\right)$$

$$R_A + R_B = 68\text{9}.65 \text{ KN}$$

Taking moment @ A

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

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

$$\boxed{R_A = 241.265 \text{ KN.}}$$

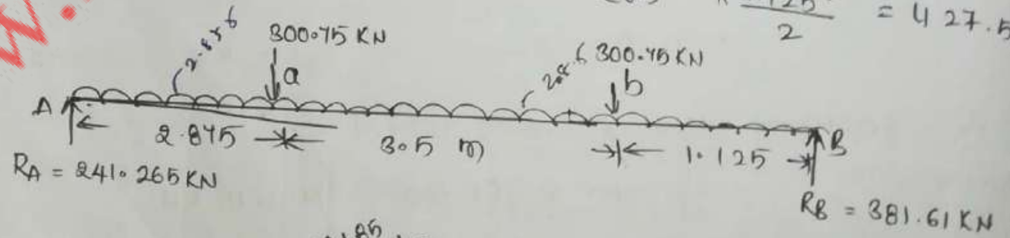
Maximum bending moment will occur at "a"

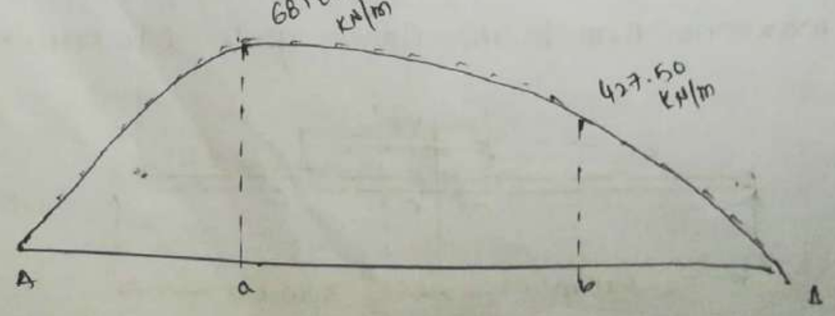
$$M_a = R_A \times 2.875 - 2.85 \times \frac{2.875^2}{2}$$

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

Moment about

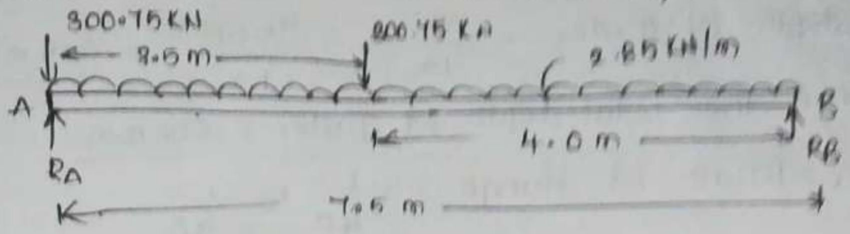
$$M_b = 381.61 \times 1.125 - 2.85 \times \frac{1.125^2}{2} = 427.50 \text{ KN}$$





Step 3:- Maximum shear force in gantry girder (in vertical plane)

maximum shear force at A will occur when one of the wheel load is at A (i.e on the coloumn)



Taking moment @ B

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

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

$\therefore$ maximum SF = 471.83 KN.

Horizontal force (surge load) transverse to the rail:

Total Surge load = 10 % of (hook 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

$= 1.5 \times 6 = 9$ KN

Horizontal B.M due to surge load:-

Horizontal bending moment (M_y)

by proportion.

300.75 KN $\rightarrow$ M = 681.85 KN.m

9 KN $\rightarrow$?

$$M_y \rightarrow 681.85 \times \frac{9}{300.75}$$

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

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

On one gantry girder Wheels ...

$\therefore$ factored braking load = 1.5 × 16.04 = 24.06 kN

Step 4 :- Section Selection for Gantry girder

$\therefore$ Economic depth of girder $= \frac{L}{12} = \frac{7500}{12} = 625$ mm

Choose the total depth of girder = 600 mm

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

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

$= 187.5$ to 250 mm

$\therefore Z_p$ required $= 1.4 \frac{M_u}{f_y}$

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

$= 3.82 \times 10^6 \text{ mm}^3 = 3820 \text{ cm}^3$

Select ISMB 600 with a channel ISMC 300

properties are listed below, from IS : 800-2007 and H.B

ISMB 600 @ 1.23 kN/m

$Z_p = 3510.63 \text{ cm}^3$

$A = 15621 \text{ mm}^2$

$t_f = 20.8$ mm

$t_w = 12.0$ mm

$b_f = 210$ mm

$R = 20$ mm

$I_{zz} = 91813 \times 10^4 \text{ mm}^4$

$I_{yy} = 2651 \times 10^4 \text{ mm}^4$

ISMC 300 @ 0.358 kN/m

$A = 4564 \text{ mm}^2$

$t_f = 13.6$ mm

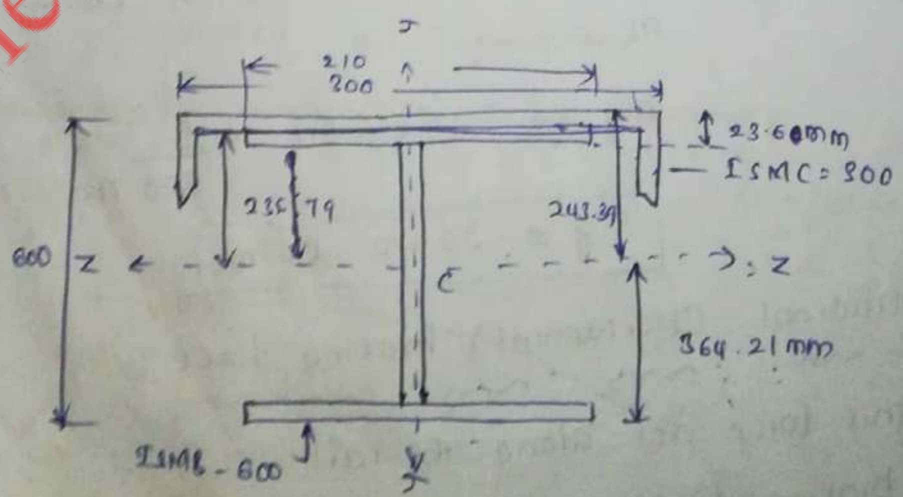
$t_w = 7.6$ mm

$b_f = 90$ mm

$I_{zz} = 6362 \times 10^4 \text{ mm}^4$

$I_{yy} = 310.8 \times 10^4 \text{ mm}^4$

$I_{yy} =$



Step 5:- Calculation of I_{zz} [illegible] properties of Built up Sections:-

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2} = \frac{15621 \times 300 + 4564 \times 584}{15621 + 4564}$$

$$\bar{y} = \frac{7351676}{20185} = 364.21 \text{ mm (from bottom)}$$

I_{zz} of built up section

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

$$I_{zz} = (I_g + Ah^2) + (I_g + Ah^2)$$

$$I_{zz} = 91813 \times 10^4 + 15621 \times (64.21)^2 + 314 \times 10^4 + 4564 \times (243.39 - 23.6)^2$$

$$I_{zz} = 982.53 \times 10^6 + 223.58 \times 10^6$$

$$\therefore I_{zz} = 1206.11 \times 10^6 \text{ mm}^4$$

$$\therefore Z_{ez} = \frac{I_{zz}}{y_{max}} = \frac{1206.11 \times 10^6}{364.21} = 3.31 \times 10^6 \text{ mm}^3$$

$\therefore$ I_{yy} of built up section

$$I_{yy} = I_{yy} \text{ of I section} + I_{zz} \text{ of channel section}$$

$$= 2651 \times 10^4 + 6362.5 \times 10^4$$

$$= 9013.5 \times 10^4 \text{ mm}^4$$

I_y of tension flange about y-axis

$$I_{tf} = 20.8 \times \frac{210^3}{12} = 16.05 \times 10^6 \text{ mm}^4$$

I_y of comp flange about y-axis

$$I_{cf} = 20.8 \times \frac{210^3}{12} + 6362.6 \times 10^4 = 63.626 \times 10^6$$

$$I_{cf} + I_{tf} = 63.626 \times 10^6 + 16.05 \times 10^6$$

$$I_{yy} = 79.676 \times 10^6 \text{ mm}^4$$

Z_y of top flange along

$$= \frac{79.676 \times 10^6}{150}$$

$$= 531173 \text{ mm}^3$$

Calculation of plastic section modulus :-

The plastic N.A divides the area into two equal ar...

Let

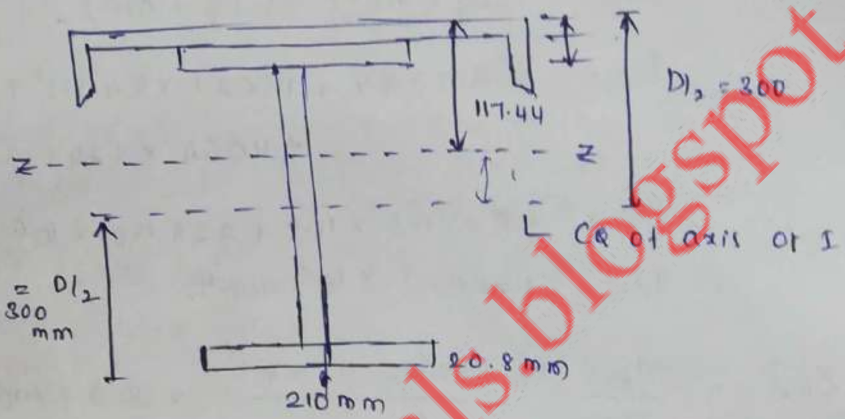
A_s = Area of I-section

A_c = Area of c-section

$A = A_s + A_c$.

$$\therefore \frac{A_s}{2} + d_p \cdot t = \frac{A_s}{2} - d_p \cdot t + A_c$$

$$d_p = \frac{A_c}{2t} = \frac{2564}{2 \times 12} = 190.16 \text{ mm}$$



plastic section modulus below the equal area axis

$$\Sigma A\bar{y}\,(Z_{P_1}) = 20.8 \times 210 \times \left(490.16 - \frac{20.8}{2}\right) + (490.16 - 20.8) \times 12 \times \left[\frac{490.16 - 20...}{2}\right]$$

$$Z_{P_1} = 3.417 \times 10^6 \text{ mm}^3$$

plastic section modulus above the equal area axis

$$\Sigma A\bar{y}\,(Z_{P_2}) = 4564 \times (117.44 - 28.6) + 210 \times 20.8 \times (117.44 - 7.6...) + 89.04 \times 12 \times \frac{89.04}{2}$$

$$Z_{P_2} = 910.21 \times 10^3 \text{ mm}^3$$

$$\therefore Z_{Pz} = 3.417 \times 10^6 + 910.21 \times 10^3$$

$$= 4.327 \times 10^6 \text{ mm}^3 > 3.82 \times 10^6 \text{ mm}^3 \text{ (required)}$$

Hence Section modulus is OK.

Step 6 :- check for moment capacity (Cl-8.2.1.2, P-53)

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{mo}} \leq \frac{1.2 Z_e f_y}{\gamma_{mo}}$$

Md should be greater than Mu.

$$Md = \frac{1 \times 4.327 \times 10^6 \times 250}{1.1}$$

$$Md = 981.8 \text{ KN-m} > 681.8 \text{ KN-m}$$

Hence section is safe for moment capacity

Step 7:- Check for shear capacity (Cl-8.4.1, P-59)

Maximum shear force (SF) = V_z

$$\text{Shear capacity } (V_d) = \frac{A_v * f_{yw}}{\sqrt{3}\, \gamma_{m0}}$$

$$A_v = h * t_w = 607.6 * 12$$

Area of shear $A_v = 7291.2 \text{ mm}^2$

$$V_d = \frac{7291.2 * 250}{\sqrt{3} \times 1.1}$$

$$V_d = 956 \text{ KN}$$

$$= 0.6 * 956 = 574 \text{ KN} > 471.83 \text{ KN}$$

Step 8:- Check for buckling resistance (Cl 8.2.2, P-54)

When gantry girder is laterally not supported, the design bending strength of beam is given by

$$Md = \beta_b Z_p f_{bd}$$

$$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$$

Elastic torsional buckling moment

$$M_{cr} = \sqrt{\left\{ \left[\frac{\pi^2 E I_y}{L_{LT}}\right] \left[G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2} \right] \right\}} = \beta_b Z_p f_{cr,b}$$

$$M_{cr} = \left[\frac{\pi^2 \times 2 \times 10^5 \times 9013.676 \times 10^4 \times 583.4}{2 \times (1.00 * 7500)^2}\right] \left[1 + \frac{1}{20}\left[\frac{\frac{1.00 * 7500}{66.828}}{\frac{583.4}{20.4}}\right]^2\right]^{0.5}$$

r_y = I_{yy} of I section + I_{zz} of channel sec

~~r_y = 4 (12 × 14.8) = 15.93 cm²~~

~~r_y = 0.15.93 mm²~~ $\sqrt{\frac{I}{A}} = \sqrt{\frac{9013.6 \times 10^4}{(15621 + 4564)}} = 66.82 \text{ mm}^3$

$$\boxed{M_{cr} = 1410 \text{ KN-m}}$$

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} = \sqrt{\frac{1 \times 4.327 \times 10^6 \times 250}{1410 \times 10^6}}$$

$$\lambda_{LT} = 0.857 \le 0.93$$

$\phi_{LT} = 0.5[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}^2]$

$\phi_{LT} = 0.5[1+0.21(0.85-0.02)+(0.85)^2]$

$\boxed{\phi_{LT} = 0.86}$

$$\chi_{LT} = \frac{1}{\{\phi_{LT}+[\phi^2_{LT}-\lambda^2_{LT}]^{0.5}\}} \leq 1.0$$

$$\chi_{LT} = \frac{1}{\{0.86+(0.86^2-0.85^2)^{0.5}\}} \leq 1.0$$

$\chi_{LT} = 1.0 \leq 1.0$

Step 9:- check for local buckling.

At points of concentrated load (wheel load (or) reaction) the web of the girder must be checked for local buckling and, if necessary, load carrying stiffeners must be providing to prevent local buckling of web

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

$$\frac{KL}{r} = 2.43 \frac{d}{t_w}$$

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Buckling resistance must be more than maximum factored wheel load.

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Step 10 :- weld design

Required shear capacity of weld

$$q_w = \frac{V_A \bar{y}}{I_z}$$

Weld is required to connect channel to the top flange of the I-beam capacity of 1 mm long weld.

$$P = l_w \cdot t_f \cdot f_{wd}$$

If $P > q_w$ (Safe)

$t_f = 0.7S$

$f_{wd} = 189\ N/mm^2$

for shop welding

$l_w = 1\ mm$

Step 11 :- check for deflection (at working loads)

Maximum deflection at centre for two wheel loads

$$\delta_c = \frac{WL^3}{6EI}\left[\frac{3a}{4L} - \frac{a^3}{L^3}\right]$$

Permissible deflection :-

For EOT crane upto 500 kN capacity

$$\delta_{per} = \frac{L}{750}$$

If $\delta_c < \delta_{per}$ (Safe).