UNIT-1

TO

UNIT 3

DESIGN OF BEAMS

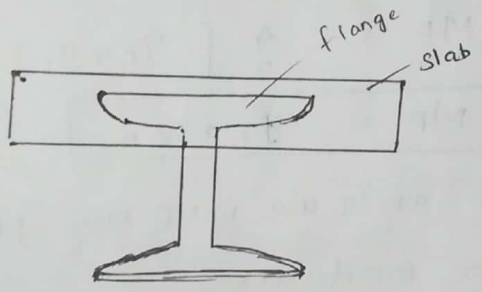
Beam is a structural Member with length considerably larger than cross sectional dimensions, subjected to lateral (or) transverse loads which give rise to BM. and SF in the member.

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

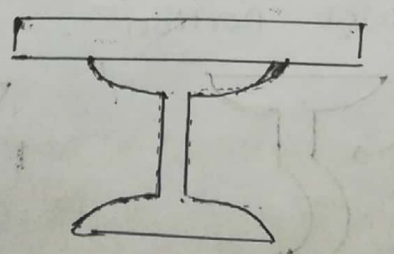
① Laterally supported - beams

② Laterally unsupported - beams.

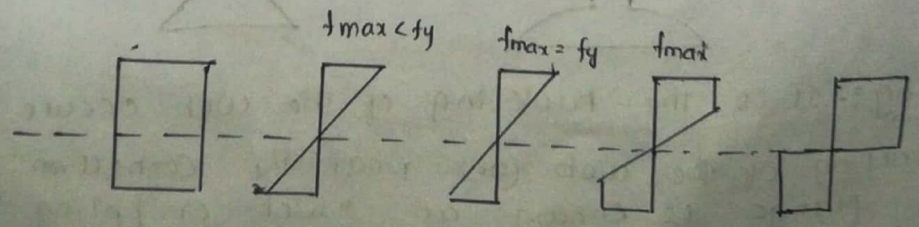
Laterally supported beam :- The beam under goes lateral buckling in addition to bending when subjected to bending about the major axis, since the moment of inertia about the minor axis is much less when comparative that about the major axis. The lateral buckling of the sections may be prevented by providing 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 :-



Scanned by CamScanner

Elastic behaviour: when the strain in the extreme fibres $\varepsilon_{max} < \varepsilon_y$ 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 $\varepsilon_{max} = \varepsilon_y$ 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 $\varepsilon_{max} > \varepsilon_y$ i.e $f_{max} > f_y$, there is no increase in the value of f_{max}. probably limited to "f_y", cross section becomes 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

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

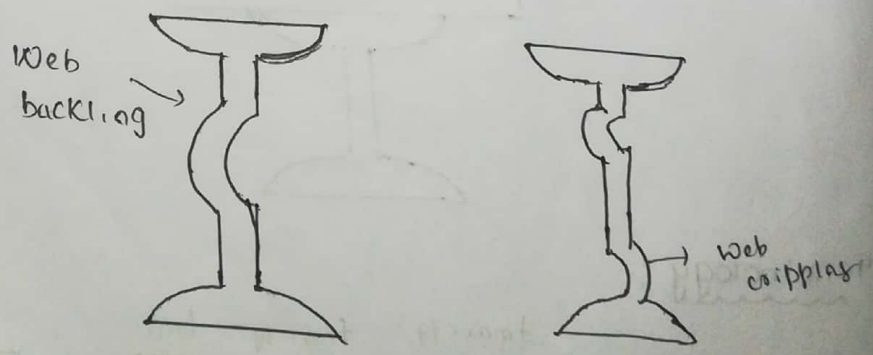
$$M_p = \frac{A}{2} [y_c + y_t]$$

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

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.

Scanned by CamScanner

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

(a) Shear capacity (b) Web buckling (c) Web crippling (d) Bending strength (e) 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"

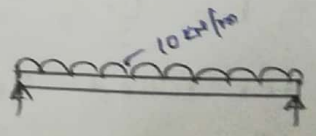
$$Z_p = \frac{M_p * \gamma_{m0}}{f_y}$$

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 Fe410. 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.

Scanned by CamScanner

Plastic section modulus.

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

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

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

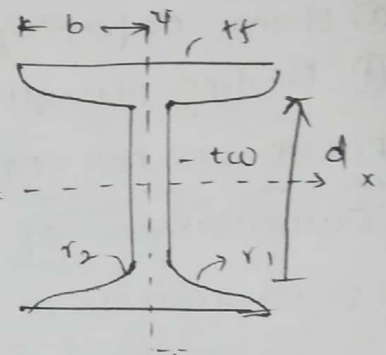
Selecting a trail section based on plastic section modulus (Pg-198)

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 bf = 100 mm

bf = 10 cm

Thickness of the flange (tf) = 7.3 mm = tf = 0.73 cm

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

r_1 = 9.5 mm = 0.95 cm (Root radius), r_2 = 3.0 mm = 0.3 mm (Radius toe).

Slope of the flange (α) = 91.5°

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

$d = 200 - 2[0.73 + 0.95]$

$d = 16.64$ 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

Scanned by CamScanner

Checks:-

(i) Check for shear Capacity :- $V \leq V_d$

where V = nominal shear force applied

V_d = design for shear strength (pg-89)

$$V_d = \frac{V_n}{\gamma_{m0}} \quad \text{and} \quad V_n = V_p$$

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

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

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

Hence the condition is safe for the section

(ii) check for design bending strength :- (pg-53, 8.2.1.2)

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

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

$M_d = 41.8\ KN-m. > 30\ KN-m.$

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)

$\frac{d}{t_w} > 67\varepsilon\sqrt{\frac{K_r}{5.35}}$ (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)

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

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

$n_2 = (t_f + r_1)\, 2.5$

$$n_2 = \frac{(9.3 + 9.5) * 2.5}{} = 42\ mm$$

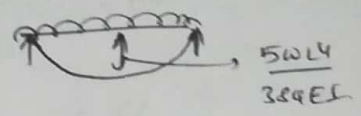
www.Jntufastupdates.com Scanned by CamScanner

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

$\therefore$ $F_w = 174.27$ KN > 30 KN

Hence the section is safe against web crippling.

check for deflection :-



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

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

$\delta = 9.827$ mm

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

Type-B Hence section is safe against deflection.

(*) Determine the design bending strength of ISLB 350 @ 456 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.

Sol:- Given data is

$F_u = 410$ N/mm², $f_y = 250$ N/mm²

Unsupported length (L) = 3 m

Step 1:- Section properties ISLB 350

Sectional Area (A) = 63.01 cm²

Depth of the section (h) = 350 mm

Width of the flange (b_f) = 165 mm

Thickness of the flange (t_f) = 11.4 mm

Thickness of the web (t_w) = 7.4 mm

$I_{xx} = 13158.3$ cm⁴ $= 13158.3 \times 10^4$ mm⁴.

$Z_e = 751.9 \times 10^3$ mm³

$Z_p = 851.1 \times 10^3$ mm³

Step 2:- classification of section. (pg.18, table-2)

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

Scanned by CamScanner

For web, $\frac{d}{t_w} = \frac{295.2}{7.4} = 39.9 < 84\epsilon$ | $\epsilon = \sqrt{\frac{250}{f_y}} = 1$

$[\because d = H - 2[t_f + r_1] = 295.2 \text{ mm}$

So Hence the Section is said to be plastic section

$\therefore \beta = 1$

Steps:- (a) For Laterally supported beam

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

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

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

RHS $\Rightarrow$ $\frac{1.2 * 851.9 \times 10^3}{1.1} = 205.1$ KN-m

$193.4 \leq 205.1$ KN-m

(b) For laterally unsupported beams:-

$M < M_d$

$M_d = \beta_b Z_p f_{bd}$

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

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

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

$\alpha_{LT} = 0.21$ 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.

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

Scanned by CamScanner

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

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

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

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

$$\boxed{\lambda_{LT} = 0.88 \le 0.9}$$

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

$$\phi_{LT} = 0.5[1+0.21(0.88-0.2)+0.89]$$

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

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

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

$\chi_{LT} = 0.74 \le 1.0$

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

$M_d = \beta_b Z_p f_{bd}$

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

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

(*) A Roof of a hall measuring 8m*12m consists of 100mm thick R.C 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 300mm

www.Jntufastupdates.com Scanned by CamScanner

① Design a S.S.B of span 4m carrying a reinforced concrete floor capable of providing laterally restrained to the 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 20 KN/m. If the compression flange is laterally unsupported and also check for deflection and shear draw to scale the (i) C/S (ii) L/S (iii) planning?

www.Jntufastupdates.com Scanned by CamScanner

Design procedure for Plate Girder :-

Step 1 :- Loading (Pg - 63)

Assume self weight of girder

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

Total UDL,

$$W = Udl\,(DL + L.L) + \text{Self weight of girder}.$$

Step 2 :- S.F and B.M calculation,

Step 3 :- Determination of optimum depth of web (or) Section selection

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

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

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

∴ Area of flange required

$$A_f = \frac{M}{fy * d} \cdot \gamma_{m0}$$

For flange to be plastic. for welded section

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

Step 4 :- check for minimum web thickness requirement

(As per IS : 800 - 2007, cl- 8.6.1.1, Pg-63)

Step 5 :- check for moment capacity.

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

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

Z_p = plastic section modulus of flanges only

M_d should be greater than factored moment

www.Jntufastupdates.com Scanned by CamScanner

Step6 :- Check for shear capacity of web.

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

Step7 :- Check for shear buckling of web using simple post-critical method (Cl-8.4.2.2(a))

$$V_a = V_{cr}$$

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

⇒ 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)

Step8 :- Check for anchor forces in the end panel (Cl-8.5.3)

Step9 :- Design of end bearing stiffener.

Step10 :- Design of connection b/w web plate and flange.

Step10 :- Design of intermediate transverse stiffners, if required

Step12 :- 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 slab 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 :-

Step1 :- Load Analysis

Total factored load on girder

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

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

Scanned by CamScanner

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

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

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

$= 38.4$ KN/m

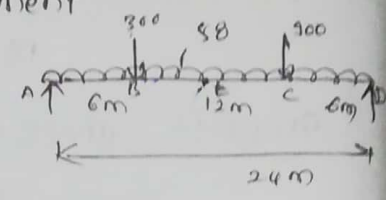
$\therefore$ Factored UDL $= 1.5 \times 38.4 = 57.6$ KN/m

(W) $= 58$ KN/m

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

$= 300$ KN (each)

Step 2 :- Shear force and Bending moment



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

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

$R_A = R_B = 996$ 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

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

$K = \dfrac{d}{t_w} > 200$ intermediate stiffness are required.

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

Assume thickness of web plate 12 mm (thick web)

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

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

check for shear buckling of web is required

Scanned by CamScanner

Assume that B.M is resisted by flanges only.

Area of flange

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

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

$$A_f = 18781.71 \ mm^2$$

For compression flange with welded section.

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

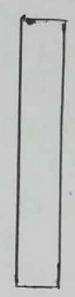
$\therefore$ $b = 8.4 t_f$ (table-2, P-18)

$\therefore$ Area of one flange

$$A_f = b_f * t_f$$

$$A_f = 2b * t_f$$

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



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

$$t_f = 33.43 \ mm$$

Provide $t_f = \underline{40 \ mm}$

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

Provide $b_f = 500 \ mm$

$\therefore$ Size of the flange $\underline{500 \times 40 \ mm}$

Step 4:- minimum web thickness requirements

* Serviceability requirement :- (IS-800-2007, cl-8.6.1 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

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

There fore minimum thickness of the web portion is $\underline{12 mm}$ is sufficient.

www.Jntufastupdates.com Scanned by CamScanner

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

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

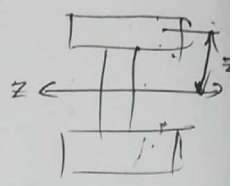
($\beta_b = 1$ for plastic section

Z_p = plastic section modulus

moment of flanges Z-Z axes.

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

$$Z_p = 28.80 \times 10^6 \text{ mm}^3$$



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

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

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

Section is adequate for moment carrying capacity

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

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

$$V_d = \frac{(1400 * 12) * 250}{\sqrt{3} * 1.10} = 2.204 * 10^6 \text{ N}$$

$\therefore V_d = 2204$ KN

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

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"

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

Scanned by CamScanner

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

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

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

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

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

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

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

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

$$= 16800 \times 71.08$$

$$V_{cr} = 1194 \ kN > 99.6 \ kN$$ (~~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

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

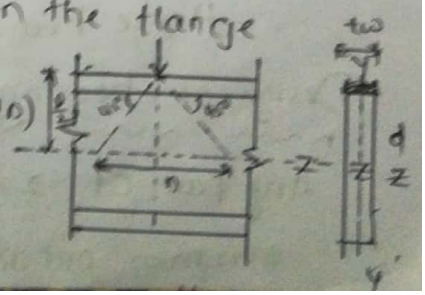
b_1 = width of stiff bearing on the flange

$b_1 = 0$

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

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

$n_1 = 1480 \ mm$



Scanned by CamScanner

$t_w = 12mm$, $\frac{KL}{r_y} = ?$

f_{cd} will be given from (7.2.1 - Cl, TN9C pg-42)

$KL = 0.7d$ (pg-66)

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

$\frac{KL}{r_y} = \frac{0.7d}{t_w/\sqrt{12}} = 2.43\ d/t_w$

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

By extrapolation

$240 \rightarrow 26.2$
$250 \rightarrow 24.3$
$10 - 1.9$

$240 \rightarrow 26.2$
$283.5 \rightarrow ?$
$43.5 \rightarrow ?$

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

$x = 8.265$

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

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

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

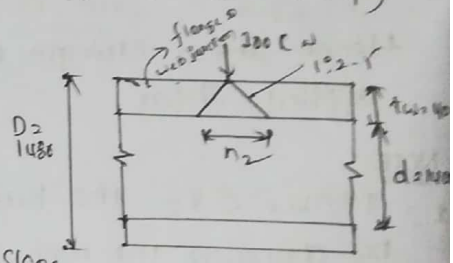
0°

∴ web is Safe against buckling below 300 KN

∴ No load carrying stiffer required

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

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



assume $b_1 = 0$.

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

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

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

$\boxed{F_w = 545.45\ KN}$

① At the location of 300 KN point load

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

② At Supports.

at support reactions $= 996\ KN$

$F_w = 545.45\ KN < 996\ KN$

Hence bearing stiffners are required at the supports

Design of bearing stiffners at supports:-

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

Maximum outstand of stiffner (Cl-8.7.1.2] (pg-65)

Scanned by CamScanner

$= 20\ tq\ \varepsilon$

$= 20 \times 12 \times 1$

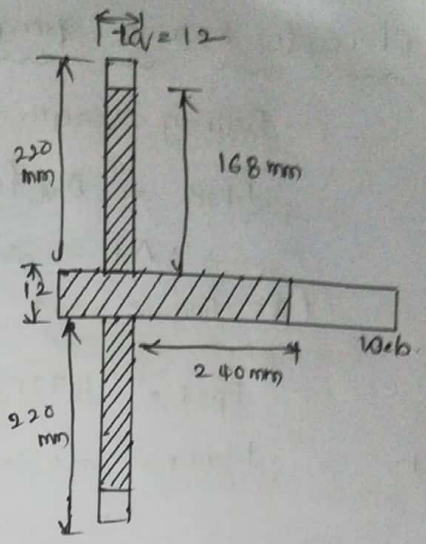
$= 240\ mm$

$\therefore\ 220 < 240\ mm$

Hence Stand of core Section

$14\ tq\ \varepsilon = 14 * 12 * 1$

$= 168\ mm < 220\ mm$



$\therefore$ 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

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

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

Hence the stiffners is safe to bearing

Scanned by CamScanner

Check for bearing strength of 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)

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

Hence. Safe in bearing.

Step 11 :- Weld connecting web plate and flange

Assume fillet weld on each side of web

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

$\therefore V = 996 \ KN$

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

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

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

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

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

$\therefore q_w = 0.305 \ KN/mm$

Strength of weld for mm

$P = t_w . t_t . f_{wd}$

$P = 1 * 4.2 \times 189$

$P = 793.8 \ N/mm$

$\therefore P = 0.794 \ KN/mm$

(L_w = 1 mm least)

$t_w = 12 mm$

minimum size of weld

provide s = 6mm

$t_f = 0.7s = 0.7 \times 6$

$t_f = 4.2 mm$

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

Scanned by CamScanner

For 1 mm effective length

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

$$\frac{KL}{r_y} = 2.043 \frac{d}{t_w} = 2.43 * \frac{1400}{12} = 283.5$$ (IS 800-2007 Table 9(c) P-42)

∴ $f_{cd} = 26.2 - 8.265 = 17.94$ N/mm²

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

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

(half as calculated in step 8) | Chamfer length = 15 mm

∴ Net shear force = 996 − 159.31 = 836.69 KN

∴ length of weld = $\frac{1400 - 2 * 15}{} = 1370$ mm

Scanned by CamScanner

$\therefore q_2 = \frac{836.69}{1370} = 0.61$ 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

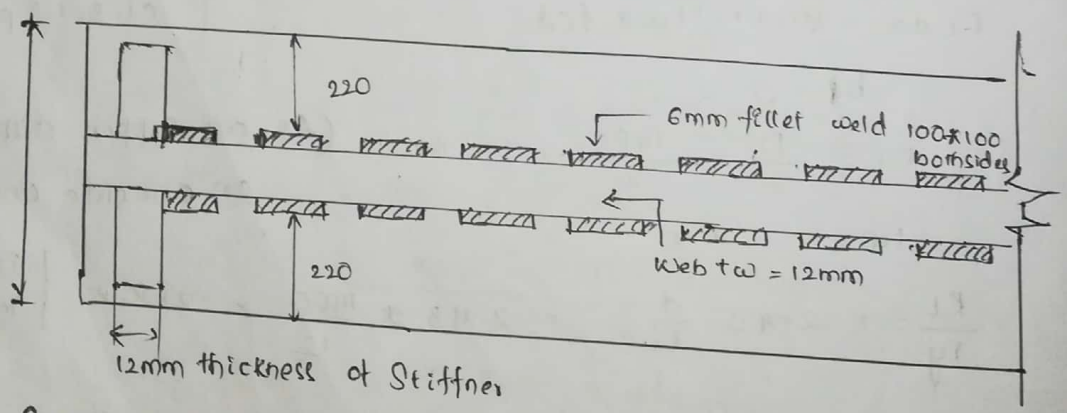
(say 200mm)

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

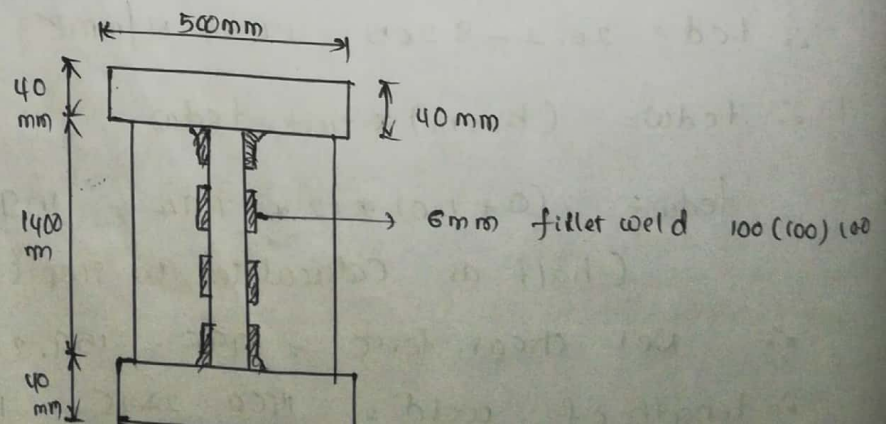
Clear spacing $\nmid 12t = 12 * 12 = 144$ mm

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

Plan of plate girder:-



Section:-



Scanned by CamScanner

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

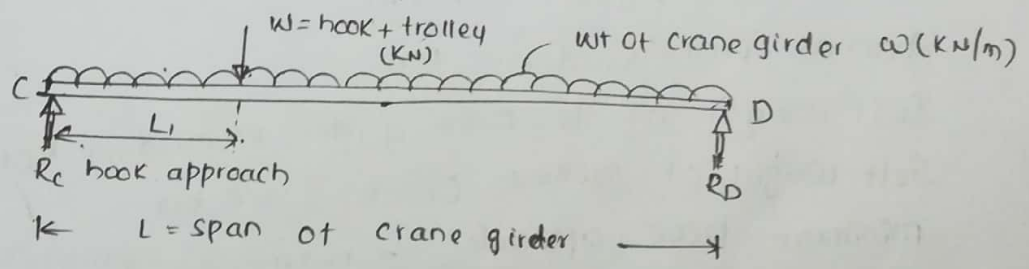
Loads acting on gantry girder :-

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

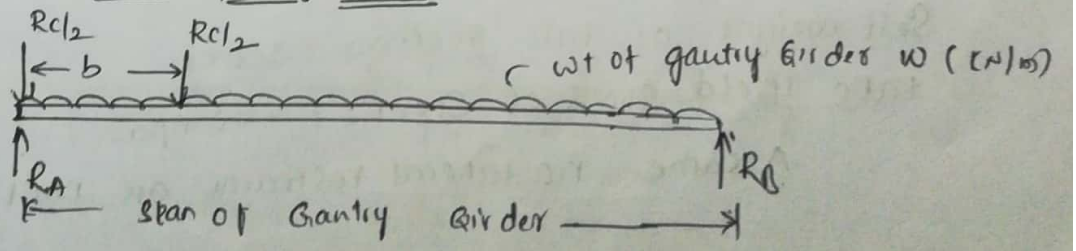
② Verticle Forces
- → Wheel loads
- → crab weight + hook load
- → weight of crane girder

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 :-

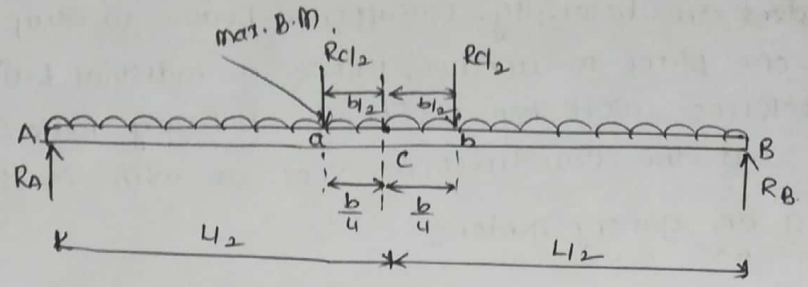


load Analysis on gantry Girder :-



Scanned by CamScanner

Max BM calculation :-



Design procedure :-

Step1 :- Maximum wheel load

Step2 :- Maximum B.M in gantry girder

Step3 :- Maximum shear force

Step4 :- Section selection for gantry girder.

Step5 :- Calculate Izz, Iyy and Zp of the trail section selected

Step6 :- check for moment capacity

Step7 :- check for shear capacity

Step8 :-

⊛ Design a gantry girder for the following data

Crane capacity = 200 KN.

Span of gantry girder = 7.5 m

Span of Crane Girder = 15m

Self weight of the crane girder excluding trolley = 200KN

Self weight of trolley (crab) = 40 KN.

Minimum hook aproach = 1.2m

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.

Scanned by CamScanner

Step 1 :- maximum wheel load

= Crane capacity + wt of trolley (crab)

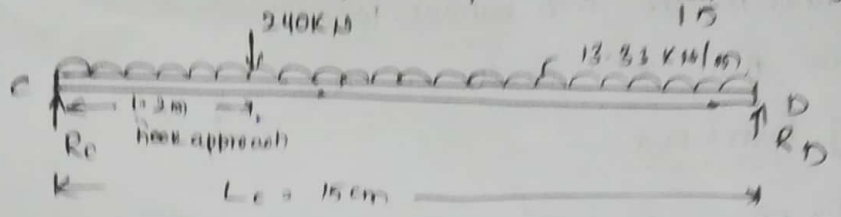
= 200 + 40

= 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 × 1.2 + 13.33 × 15 × 7.5

R_D = 119.18 KN

∴ R_C = Total load − R_D

R_C = (240 + 13.33 × 15) − 119.18

R_C = 320.77 KN (maximum reaction)

At the each end of crane girder, there are two wheels

∴ 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

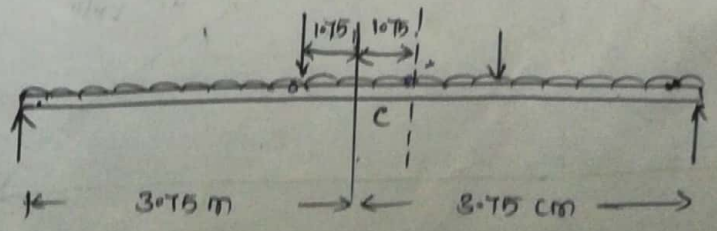
∴ Design load on each wheel

= 1.25 × 160.40 = 200.50 KN.

∴ Factored load on each wheel

= 1.5 × 200.50 = 300.75 KN.

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



Scanned by CamScanner

Max. BM will occur below wheel load 'a' when centre of the sp[illegible] "b" is at equal distance from the wheel load and the C[illegible] 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.

∴ Factored UdL = 1.5 × 1.9 = 2.85 KN/m

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

Maximum B.M will occur at "a".

Taking Reactions:-

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

$$R_A + R_B = 681.65 \text{ KN}$$

Taking moment @ A

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

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

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

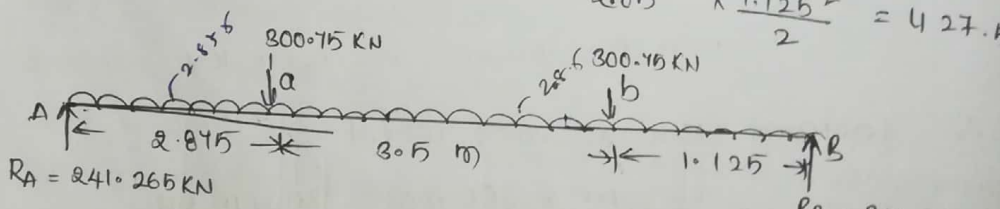
maximum bending moment will occur at "a"

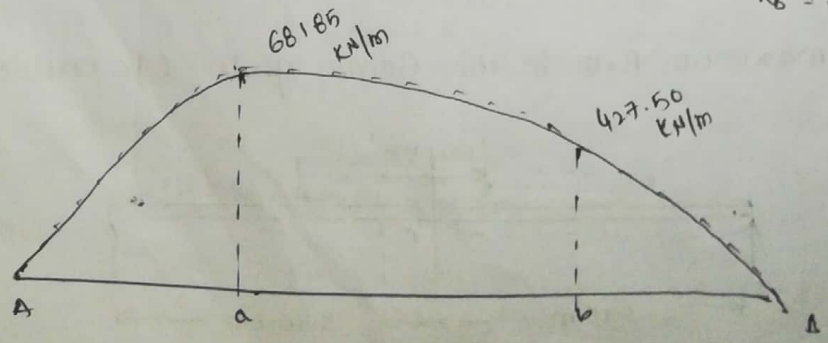
$$M_a = R_A \times 2.875 - 2.85 \times \frac{2.875^2}{2}$$

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

moment about

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

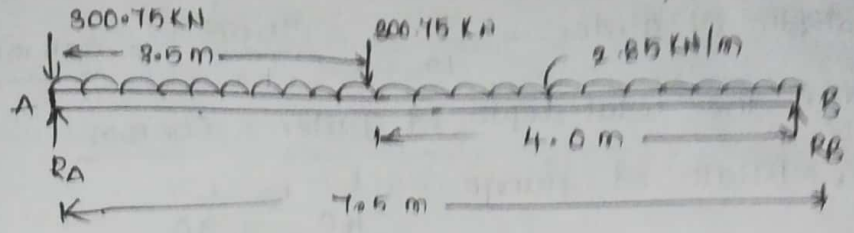




Scanned by CamScanner

steps :- 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

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

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

∴ maximum SF = 471.83 KN.

Horizontal force (surge load) transverse to the rail :-

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

= 0.10 × (200 + 40)

= 24 KN.

this load is shared by 4 wheels

∴ Surge load per wheel = $\frac{24}{4}$ = 6 KN

∴ Factored surge load on each wheel

= 1.5 × 6 = 9 KN

Horizontal B.M due to surge load :-

Horizontal bending moment (M_y)

by proportion.

300.75 KN → M = 681.85 KN.m

9 KN → ?

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

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

Longitudinal (Horizontal) braking force :-

This force act along the rail

braking force = 5% of static wheel load

= (2 × 160.40) × 0.05

= 16.04 KN

on one gantry girder wheel

Scanned by CamScanner

∴ Factored braking load = 1.5 × 16.04 = 24.06 kN

Step 4: Section Selection for Gantry girder

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

Choose the total depth of girder = 600 mm

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

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

= 187.5 to 250 mm

∴ $Z_{p\,required} = 1.4 \frac{M_u}{f_y}$

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

$= 3.82 \times 10^6$ mm³ = 3820 cm³

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 cm³

A = 15621 mm²

t_f = 20.8 mm

t_w = 12.0 mm

b_f = 210 mm

R = 20 mm

I_{zz} = 91813 × 10⁴ mm⁴

I_{yy} = 2651 × 10⁴ mm⁴

ISMC 300 @ 0.358 kN/m

A = 4564 mm²

t_f = 13.6 mm

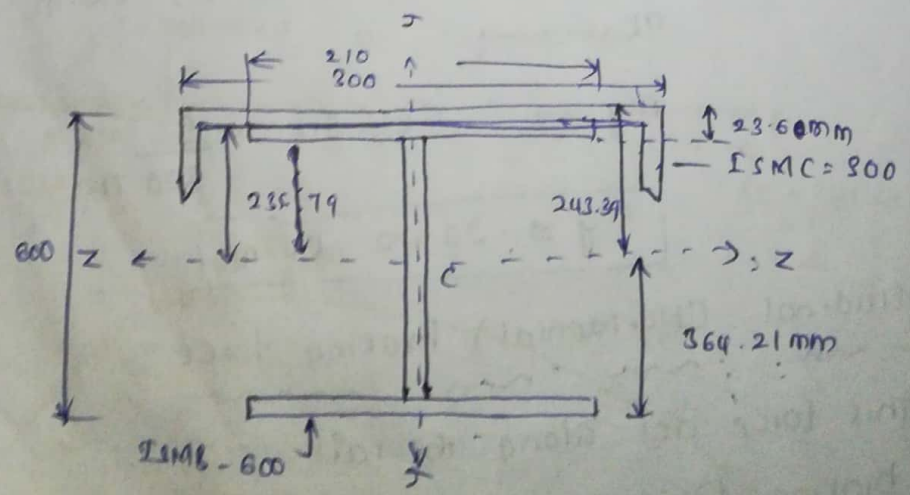
t_w = 7.6 mm

b_f = 90 mm

I_{zz} = 6362.5 × 10⁴ mm⁴

I_{yy} = 310.8 × 10⁴ mm⁴

I_{yy} =



Scanned by CamScanner

Step 5 :- Calculation of I_{zz}, I_{yy}, Z_e of the built section

Properties of combined section :-

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2} = \frac{15621 * 300 + 4564 * 564}{15621 + 4564}$$

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

I_{zz} of built up section

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

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

$$I_{zz} = 9813 \times 10^4 + 15621 \times (64.21)^2 + 310.8 \times 10^4 + 4564 \times (243.39 - 23.6)^2$$

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

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

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

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

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

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

$$= 9013.6 \times 10^4 \text{ mm}^4$$

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

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

$$= 53\,1173 \text{ mm}^3.$$

Scanned by CamScanner

Caluculation of plastic Section modulus :-

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

Let

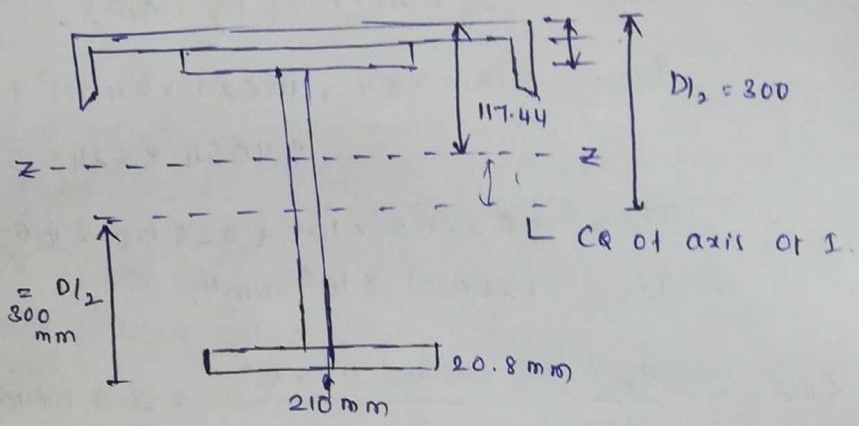
A_s = Area of I-section

A_c = Area of c-section

$A = A_s + A_c$.

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

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



Plastic section modulus below the equal area axis

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

$$Z_{P1} = 3.417 \times 10^6 \text{ mm}^3$$

Plastic section modulus above the equal area axis

$$\Sigma A\bar{y} \; (Z_{P_2}) = 4564 \times (117.44 - 28.67) + 210 \times 20.8 \times (117.44 - 7.6 \ldots$$

$$+ 89.04 \times 12 \times \frac{89.04}{2}$$

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

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

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

Hence Section modulus is OK.

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

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

M_d should be greater than M_u.

Scanned by CamScanner

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

$M_d = 981.8$ KN-m > 681.8 KN-m,

Hence section is safe for moment capacity

Step7:- check for shear capacity (Cl-8.4.1, P-59)

maximum shear force (SF) = V_z

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

$A_v = h * t_w = 607.6 * 12$

Area of shear $A_v = 7291.2$ mm²

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

$V_d = 956$ KN.

$= 0.6 * 956 = 574$ KN > 471.83 KN

Step8 :- 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

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

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

Ealastic torsional buckling moment

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

$$M_{cr} = \sqrt{\left[\frac{\pi^2 \times 2 \times 10^5 \times 9013.6 \times 10^4 \times 583}{2 \times (1.00 * 7500)^2}\right]\left[1 + \frac{1}{20}\left[\frac{1.00 * 7500}{66.82}\right]^2 \frac{583.4}{20.4}\right]}$$

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

~~r_y = 0.115 9.8 mm²~~

$$\sqrt{\frac{I}{A}} = \sqrt{\frac{9013.6 \times 10^4}{(15621 + 4564)}} = 66.82 \text{ mm}^2$$

$M_{cr} = 1470$ KN-m

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} = \sqrt{\frac{1 \times 4.327 \times 10^6 \times 250}{1470 \times 10^6}}$$

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

Scanned by CamScanner

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

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

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

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

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

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

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

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

IS : 800-2007 Cl. 8.7.3.1, pg. 67

Buckling resistance must be more than maximum factored wheel load.

www.Jntufastupdates.com Scanned by CamScanner

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.

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

If $P > q_w$ (Safe)

$t_f = 0.75$

$f_{wd} = 189$ N/mm² 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).

Scanned by CamScanner