LATERALLY UNSUPPORTED BEAMS

→ Laterally unsupported beams are susceptible for Lateral Torsional Buckling.

→ When a beam is not adequately supported against Lateral buckling, it is known as "Laterally unrestrained (unsupported) beam.

→ There is always the possible of Lateral buckling

→ Especially when the moment of inertia is about the minor axis.

→ The compression flange bends normal to the plane of Loading.

Built up beams

P/o

1) Design a Simply supported beam of effective span 8.5m carrying a load of 50KN/m. The depth of the beam should not exceed 500mm. The compression flange of the beam is Laterally unsupported. Assume stiff bearing length 80mm?

Sol Given data,

Effective span KL = 8.5m

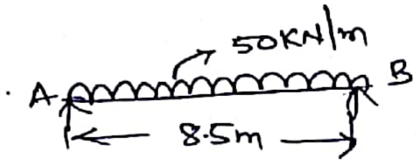
UDL = 50KN/m

Factored load = 1.5×50 = 75KN/m

Depth of the beam should not exceed 500mm ie; $d \not> 500$mm

Beam is Laterally unsupported

Stiff bearing length b_1 = 80mm



Step 1:

Maximum shear force $V = V_{max} = \frac{wl}{2} = \frac{50 \times 8.5}{2} = 212.5KN$

Maximum Bending moment $M = M_{max} = \frac{wl^2}{8} = \frac{50 \times 8.5^2}{8} = 451.56KN/m$

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$$Z_p(\text{required}) = \frac{M_p \gamma_{mo}}{f_y}$$

$$= \frac{451.56 \times 10^6}{250} \times 1.1$$

$$= 1986864 \text{ mm}^3$$

$$= 1986.86 \times 10^3 \text{mm}^3$$

As depth of beam should not exceed 500mm, Select a beam section ISMB 450 having $Z_p = 1533.4 \times 10^3 \text{mm}^3$ ($< Z_{p\,req}$)

Hence provide suitable plate over flanges

Z_p to be provided by plates $= 1986.86 \times 10^3 - 1533.4 \times 10^3$

$$= 453.46 \times 10^3 \text{mm}^3$$

Let 'Ap' be area of each cover plate required.

Assuming thickness of each cover plate as '20 mm'

The resisting force of each plate is '$A_p \times f_y$'.

Plastic moment of resistance of each plate (Zp) = Mp of each plate.

$$Z_p = (A_p \times f_y)\left(\frac{450}{2} + \frac{20}{2}\right)$$

$$Z_p \text{ of each plate} = \frac{M_p}{f_y} \times \gamma_{mo}$$

$$\Rightarrow \frac{453.46 \times 10^3}{2} = \frac{A_p \times \cancel{f_y}}{\cancel{f_y}}\left(\frac{450}{2} + \frac{20}{2}\right) \times 1.1$$

$$\Rightarrow 226.73 \times 10^3 = A_p(258.5)$$

$$A_p = 877.09 \text{ mm}^2$$

Minimum width of each plate (B) $= \frac{877.09}{20} = 43.85$mm

Assume B = 60mm,

Provide each plate of 60 x 20mm one on top flange and other on bottom flange

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From steel tables (ISMB 450)

$h = 450mm$; $b_f = 150mm$; $t_f = 17.4mm$; $t_w = 9.4mm$; $r_1 = 15mm$

Depth of Web $(d) = h - 2(t_f + r_1)$ $\qquad r_{min} = 30.1mm$

$= 450 - 2(17.4 + 15)$

$\boxed{d = 385.2mm}$

Step 2: classification of section

i) $\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$

ii) $b = \frac{b_f}{2} = \frac{150}{2} = 75mm$; $\frac{b}{t_f} = \frac{75}{17.4} = 4.31 < 9.4\varepsilon = 4.31 < 9.4(1)$

iii) $\frac{d}{t_w} = \frac{385.2}{9.4} = 40.97 < 84\varepsilon = 40.97 < 84(1)$

Hence section is plastic

Step 3: calculation of Design shear strength

As per cl. $\left(\frac{8.4}{59}\right)$ $V \le V_d$; But $V_d = \frac{V_n}{\gamma_{m0}}$

$V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ $\left(cl.\ \frac{8.4.1}{59}\right)$ $\qquad A_v = h t_w = 450 \times 9.4$

$V_d = \frac{A_v f_{yw}}{\sqrt{3}\gamma_{m0}} = \frac{450 \times 9.4 \times 250}{\sqrt{3}(1.1)} = 555.04 KN$

$\boxed{V_d = 555.04 KN}$

Step 4: check for Design Bending strength

As per cl. $\frac{8.2.2}{54}$, $M_d = \beta_b Z_p f_{bd}$ $\qquad \therefore \beta_b = 1.0$ for critical section

$Z_{p(provided)} = Z_{p\,(I\text{-section})} + Z_{p\,(Two\ plates)}$

$= 1533.4 \times 10^3 + 564000$

$= 2097.4 \times 10^3 mm^3.$

$Z_p = A(\bar{Y}_1 + \bar{Y}_2)$

$= 60 \times 20\left(\frac{450}{2} + \frac{20}{2}\right)$

$= 282000 mm^3$

$Z_p = 2 \times 282000$

$= 564000 mm^3.$

$I_{zz} = I_{zz}$ for I section $+ I_{zz}$ for plates

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$$I_{zz} = 30390\times10^4 + 2\left[\frac{60\times20^3}{12} + 60\times20\left(\frac{450}{2}+\frac{20}{2}\right)^2\right]$$

$$I_{zz} = 43652\times10^4\, mm^4$$

Calculation of Design Bending Stress (f_{bd})

i) slenderness ratio $\lambda = \frac{KL}{r_{min}} = \frac{8500}{30.1} = 282.39$

ii) $\frac{h}{t_f} = \frac{450}{17.4} = 25.86$

As per cl. 8.2.2/54, α_{LT}, Imperfection factor for Rolled steel section $\boxed{\alpha_{LT} = 0.21}$

from cl. 7.1.2.2/35, Table 7, Buckling class a, $\boxed{\alpha_{LT} = 0.21}$

Use Table 14/57, find $f_{cr,b}$

λ	h/t_f 25	25.86	30
280	74.7	X	64.1
282.39		O	
290	71.8	Y	61.5

$$X = 74.7 - \frac{(74.7-64.1)}{(30-25)}(25.86-25) = 72.87\, N/mm^2$$

$$Y = 71.8 - \frac{(71.8-61.5)}{(30-25)}(25.86-25) = 70.02\, N/mm^2$$

$$O = 72.87 - \frac{(72.87-70.02)}{(290-280)}(282.39-280) = 72.18\, N/mm^2$$

$\therefore$ critical stress in bending $f_{cr,b} = 72.18\, N/mm^2$

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Consider 13(a)/55,

$f_{cr,b}$	f_y
80	63.6
72.18	?
70	56.8

$$f_{bd} = 63.6 - \frac{(63.6 - 56.8)}{(80-70)}(80-72.18)$$

$$\boxed{f_{bd} = 58.28\ N/mm^2}$$

∴ Design Bending strength $M_d = \beta_b Z_p f_{bd}$

$= 1 \times 2097.4 \times 10^3 \times 58.28$

$= 122.23\ kNm < 451.56\ kNm$ (M_{max})

Hence Revise the section.

Step 5: Design of shear strength

As per cl. $\left(\frac{8.4}{59}\right)$ $V \leq V_d$

$\Rightarrow 212.5\ kN < 555.04\ kN$

Hence safe.

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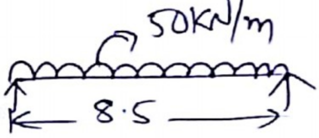
Step 6: check for Deflection

Deflection is checked in elastic condition,

Working load = $\frac{50}{1.5} = 33.33$ KN/m



$$(\delta_{cal}) = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384}\frac{(33.33)(8500)^4}{2\times10^5\times43652\times10^4} = 25.94\,mm$$

As per Table 6/31, Permissible deflection = $\frac{span}{300} = \frac{8500}{300} = 28.33$ mm

28.33 > 25.94 (mm)

(Safe)

Step 7: check for Web crippling

As per cl. $\left(\frac{8.7.4}{67}\right)$ $F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$

b_1 = stiff bearing length.

Assuming $b_1 = 100$ mm

$$n_2 = 2.5(t_f + r_1)$$
$$= 2.5(17.4 + 15)$$
$$\boxed{n_2 = 81\,mm}$$

$$F_w = (100+81) \times 9.4 \times \frac{250}{1.1}$$
$$= 386.68\,KN > 212.5\,KN \quad (Safe)$$

Step 8: check for Web buckling

As per cl. $\left(\frac{8.4.2.1}{59}\right)$ $\frac{d}{t_w} = \frac{285.2}{9.4} = 40.97 < 67\varepsilon$

$= 40.97 < 67(1)$

Hence check for Web buckling not required.

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2) Design a Laterally unsupported beam of effective span 4m, Factored bending moment 550KNm and Factored shear force 200 KN. Use Fe410 grade steel?

Sol Given data,

Step 1: Effective span = 4m = 4000mm

Factored Bending moment = M_{max} = 550 KNm

Factored shear Force = V = 200KN.

Plastic section Modulus required $Z_{p(req)} = \frac{M_p \gamma_{mo}}{f_y}$

$$Z_p = \frac{550\times10^6 (1.1)}{250}$$

$$= 2420\times10^3 mm^3$$

Increase Z_p value to 30 to 50%, $Z_p = 3146\times10^3 mm^3$

Try ISMB 600 @ 122.6 kg/m, having $Z_p = 3510.63\times10^3 > 3146\times10^3 mm^3$

From steel tables

$h = 600mm$; $b_f = 210mm$; $t_f = 20.8mm$; $t_w = 12mm$; $r_{min} = 41.2mm$

$r_1 = 20mm$

Depth of web (d) = $h - 2(t_f + r_1)$

$= 600 - 2(20.8+20)$

$= 518.4 mm$

Step 2: class of section

i) $\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$

ii) $b = b_f/2 = \frac{210}{2} = 105mm$; $\frac{b}{t_f} = \frac{105}{20.8} = 5.04 < 9.4\varepsilon$

iii) $\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 84\varepsilon$

class-I (or) plastic section

Hence section is plastic.

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Step 3: calculation of Design shear strength (V_d)

As per cl. $\left(\frac{8.4}{59}\right)$, $V_d = \frac{V_n}{\gamma_{mo}}$; $V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ $\left(cl.\ \frac{8.4.1}{59}\right)$

$A_v = h t_w = 600 \times 12$

$V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{600 \times 12 \times 250}{\sqrt{3}\,(1.1)} = 947.36\,kN.$

$\boxed{V_d = 947.36\,kN}$

Step 4: check for Design Bending strength

As per cl. $\left(\frac{8.2.2}{54}\right)$ $M_d = \beta_b Z_p f_{bd}$ $\beta_b = 1.0$ for critical section

calculation of Design bending stress (f_{bd})

i) slenderness ratio $\lambda = \frac{KL}{r_{min}} = \frac{4000}{41.2} = 97.08$

ii) $\frac{h}{t_f} = \frac{600}{20.8} = 28.84$

From cl. 8.2.2/54, Imperfection factor for rolled section $\boxed{\alpha_{LT} = 0.21}$ → class 'a'.
Table 7/35

Use Table 14/57, Find $f_{cr,b}$

λ	h/t_f		
	25	28.84	30
90	344.2	x	322.9
97.08		0	
100	291.4	y	257.7

$$x = 344.2 - \frac{(344.2 - 322.9)}{(30-25)}(28.84-25) = 336.12\,N/mm^2$$

$$y = 291.4 - \frac{(291.4 - 257.7)}{(30-25)}(28.84-25) = 278.59\,N/mm^2$$

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$$\sigma = 336.12 - \frac{(336.12 - 278.59)}{(100-90)}(97.08-90) = 295.41\ N/mm^2$$

$\therefore$ critical stress in bending $f_{cr,b} = 295.41\ N/mm^2$

Consider 13(a)/55, to calculate f_{bd}

$f_{cr,b}$	f_y (N/mm²)
300	163.6
295.41	?
250	152.3

$$f_{bd} = 163.6 - \frac{(163.6-152.3)}{(300-250)}(300-295.41)$$

$$\boxed{f_{bd} = 162.56\ N/mm^2}$$

$\therefore$ Design Bending Strength $M_d = \beta_b Z_p f_{bd}$

$= 1 \times 3510.63 \times 10^3 \times 162.56$

$= 570.68\ KNm > 550\ KNm$ (M_{max})

Hence safe

Step 5: Design of shear strength

As per cl. (8.4 / 59), $V \leq V_d$

$\Rightarrow 200KN \leq 947.36KN$

Hence safe

Step 6: check for Deflection

Since Load is not given, check for Deflection is not required.

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Step 7: check for web crippling

As per cl. $\left(\frac{8.7.4}{67}\right)$, $F_w = (b_1 + n_2)\, t_w \frac{f_{yw}}{\gamma_{m0}}$

b_1 = stiff bearing length

Assuming $b_1 = 100mm < (b_f = 210mm)$

$n_2 = 2.5(t_f + r_1)$

$= 2.5(20.8 + 20)$

$= 102mm$

$\therefore F_w = (100 + 102) \times 12 \times \frac{250}{1.1}$

$F_w = 550.90\ KN \ngtr 200KN$

(Safe)

Step 8: check for web buckling

As per cl. $\left(\frac{8.4.2.1}{59}\right)$, $\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 67\varepsilon$

Hence check for web buckling not required.

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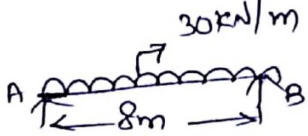
3) Design a Laterally unsupported beam of effective span 8m Carrying a Factored udl of 30KN/m. Use Fe410grade steel

Sol Given data, Effective span = 8m = 8000mm

Factored udl = 30KN/m

Fe410grade steel = $f_u = 410 N/mm^2$

= $f_y = 250 N/mm^2$



Plastic section Modulus (Z_p) required = $\frac{M_p \gamma_{mo}}{f_y}$

Step 1

Factored shear Force = $\frac{wl}{2} = \frac{8000 \times 30}{2} = 120KN$

Factored Bending moment = $\frac{wl^2}{8} = \frac{30 \times 8000^2}{8} = 240KNm$

$$Z_p = \frac{240 \times 10^6}{250} \times 1.1$$

$$\boxed{Z_p = 1056 \times 10^3 mm^3}$$

Increase Z_p value to 30 to 50%. $Z_p = 1372.8 \times 10^3 mm^3$

Try ISMB 450 @ 72.4kg/m, having $Z_p = 1533.36 \times 10^3 mm^3$.

From steel tables

$h = 450mm$; $b_f = 150mm$; $t_f = 17.4mm$; $t_w = 9.4mm$; $r_1 = 15mm$

$I_{xx} = 30390.8 \times 10^4 mm^4$; $r_{min} = 30.1mm$;

Depth of web (d) = $h - 2(t_f + r_1)$

$= 450 - 2(17.4 + 15)$

$= 385.2mm$

Step 2: class of section

i) $\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$

ii) $b = b_f/2 = \frac{150}{2} = 75mm$; $\frac{b}{t_f} = \frac{75}{17.4} = 4.31 < 9.4\varepsilon$

iii) $d/t_w = 385.2/9.4 = 40.97 < 84\varepsilon$

Hence section is plastic

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Step 3: calculation of Design shear strength (V_d)

From cl. 8.4/59, $V_d = \frac{V_n}{\gamma_{mo}}$; $V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ (cl. $\frac{8.4.1}{59}$)

$A_v = h t_w$

$\boxed{A_v = 450 \times 9.4}$

$\therefore \; V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{450 \times 9.4 \times 250}{\sqrt{3}(1.10)}$

$\boxed{V_d = 555.04 \text{ kN}}$

Step 4: check for Design Bending Strength (M_d)

As per cl. $\frac{8.2.2}{54}$, $M_d = \beta_b Z_p f_{bd}$ $\quad \therefore \beta_b = 1.0$ for plastic section

To Find Design Bending stress (f_{bd})

Slenderness ratio $\lambda = \frac{KL}{r_{min}} = \frac{8000}{30.1} = 265.78$

$\frac{h}{t_f} = \frac{450}{17.4} = 25.86$

From IS 800 : 2007, fig 57/table 14, critical ~~stress in bending~~

$f_{cr,b} = ?$

$\lambda = \frac{KL}{r_{min}}$ $\qquad \frac{h}{t_f}$

25 $\quad$ 25.86 $\quad$ 30

To find Imperfection Factor (α_{LT})

The imperfection factor α_{LT} depends upon buckling class

Buckling class $= \frac{h}{b_f} = \frac{450}{150} = 3 > 1.2$

$t_f = 17.4\text{mm} \leq 40\text{mm}$

Hence it belongs to class 'a' buckling about z-z axis

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For class 'a' buckling the imperfection factor $\alpha_{LT} = 0.21$

From cl. 7.1.2.2/IS. Table 7, $\alpha_{LT} = 0.21$

Use Table 14/ST, to find $f_{cr,b}$

λ	h/t_f		
	25	25.86	30
260	81.3	X	70.1
265.78		O	.
270	77.9	Y	67.0

$$X = 81.3 - \frac{(81.3 - 70.1)}{(30-25)}(25.86 - 25) = 79.37 \text{ N/mm}^2$$

$$Y = 77.9 - \frac{(77.9 - 67)}{(30-25)}(25.86 - 25) = 76.02 \text{ N/mm}^2$$

$$O = 79.37 - \frac{(79.37 - 76.02)}{(270-260)}(265.78 - 260) = 77.43 \text{ N/mm}^2$$

$\therefore$ critical stress in bending $\therefore f_{cr,b} = 77.43$ N/mm²

For imperfection factor $\alpha_{LT} = 0.21$, design ~~bending~~ critical stress $f_{cr,b}$

Consider Table ~~13(a)~~ ~~of~~

$f_{cr,b}$	f_y
80	63.6
77.43	δ
70	56.8

$$f_{bd} = 63.6 - \frac{(63.6 - 56.8)}{(80-70)}(80 - 77.43)$$

$$= 61.85 \text{ N/mm}^2$$

$\therefore$ Design Bending strength $M_d = \beta_b Z_p f_{bd}$

$= 1 \times 1533.36 \times 10^3 \times 61.85$

$= 94.83$ kNm $< M_{max}$

(Revise section)

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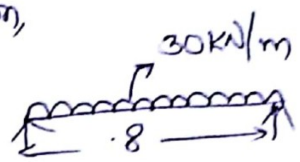
Step5: Design of shear strength

As per cl. (8.4/59), $V \le V_d \Rightarrow 120 \le 555.04\,(kN)$ Hence safe

Step6: check for Deflection

Deflection is checked in elastic condition,



Working load $= \frac{30}{1.5} = 20\,kN/m$

$$(\delta_{cal}) = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384}\frac{(20)(8000)^4}{2\times10^5 \times 30390.8\times10^4} = 17.54\,mm$$

As per Table6/31, Permissible deflection $= \frac{span}{300} = \frac{8000}{30} = 26.67\,mm$

$26.67 > 17.54\,mm$ Hence safe.

Step7: check for Web crippling

As per cl. $\left(\frac{8.7.4}{67}\right)$, $F_w = (b_1 + n_2)\,t_w \frac{f_{yw}}{\gamma_{m0}}$

b_1 = stiff bearing length.

Assuming $b_1 = 100mm$; $n_2 = 2.5(t_f + r_1)$

$= 2.5(17.4 + 15)$

$= 81mm$

$F_w = (100 + 81) \times 9.4 \times \frac{250}{1.1}$

$F_w = 386.68\,kN > 120\,kN$ (safe)

Step8: check for Web buckling

As per. cl, $\left(\frac{8.4.2.1}{59}\right)$, $\frac{d}{t_w} = \frac{385.2}{9.4} = 40.97 < 67\varepsilon$

$= 40.97 < 67(1)$

Hence web buckling not required.

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Design procedure

1) Maximum wheel load.
2) Maximum Bending moment in the gantry girder due to Live loads
3) Maximum shear Force is computed
4) Lateral forces (or) surge forces on the girder and the maximum Bending moment and shear force due to these are calculated
5) Selection of Trial section. A trial section is to be selected for which the following guide lines are useful:
   i) $Z_{pz} = (1.4 \text{ to } 1.5) \dfrac{M_p}{f_y}$
   ii) Economical depth of gantry girder is not less than $\frac{1}{12}$ of the span
   iii) Compression flange width should be $\frac{1}{40}$ to $\frac{1}{30}$ of the span.

   The properties of section such as Z_{ez}, Z_{ey}, Z_{pz} and Z_{py} are calculated
6) Classification of section
7) Check for design Moment capacity
8) Check for Local Moment capacity of girder
9) Check for buckling resistance in bending
10) Check for Biaxial bending.
11) Check for Design shear strength
12) Check for Web buckling
13) Check for Web crippling
14) Check for Deflection
15) Design of welded connections.

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P/Q)

1) Design a gantry girder to be used in an Industrial building carrying a manually operated overhead travelling Crane, for the following data:

Crane capacity : 200KN

Self wt of the crane girder excluding trolley : 200KN

Self wt of the trolley, electric motor hook etc : 40KN

Approximate minimum approach of the crane hook to the gantry girder : 1.20m

Wheel base : 3.5m

c/c Distance between gantry rails : 16m

c/c Distance between columns (Span of gantry girder) : 8m

Self weight of rail section : 300N/m

Diameter of crane wheels : 150mm

Steel is of grade Fe 410. Design also the field welded connection if required.

Sol: Given data,

Fe410 grade of steel ⇒ $f_u = 410 N/mm^2$

$f_y = 250 N/mm^2$

For Manually operated overhead Travelling crane

Lateral Loads = 5% of Maximum static wheel load

Longitudinal Loads = 5% of Weight of crab and weight lifted

Partial safety factors $\gamma_{mo} = 1.10$

$\gamma_{mw} = 1.50$

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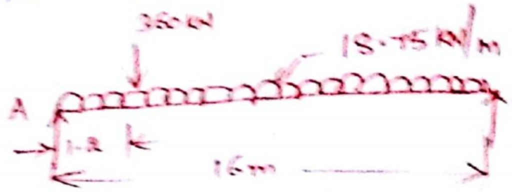
Step 1: Maximum Wheel load

Maximum concentrated load on crane = 200 + 40 = 240 KN

Maximum factored load on



Crane = 1.5 × 240

= 360 KN

The crane will carry the self weight as a udl = $\frac{200}{16}$

= 12.5 KN/m

Factored uniform load = 1.5 × 12.5 = 18.75 KN/m

For maximum reaction on the gantry girder the loads are placed on the crane girder in fig above

Taking moments about B

$$R_A \times 16 - 360(16 - 1.2) - 18.75 \times 16 \times \frac{16}{2}$$

$$\boxed{R_A = 483\text{ KN}}$$

Total Load $R_A + R_B = 360 + (18.75 \times 16)$

$\Rightarrow \quad R_B = 360 + (18.75 \times 16) - 483$

$$\boxed{R_B = 177\text{ KN}}$$

The reaction from the crane girder is distributed equally on the two wheels at the end of crane girder

∴ Maximum wheel load on each wheel of crane = $\frac{483}{2}$ = 241.5 KN

Step 2: Maximum Bending moment

Assume self weight of gantry girder = 2 KN/m = 2000 N/m

Total Dead load, ω = 2000 + 300 = 2300 N/m = 2.3 KN/m

Factored Dead load = 1.5 × 2.3 = 3.45 KN/m

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The position of one wheel load from the mid point of span

$$= \frac{\text{Wheelbase}}{4}$$

$$= \frac{3.5}{4} = 0.875m$$

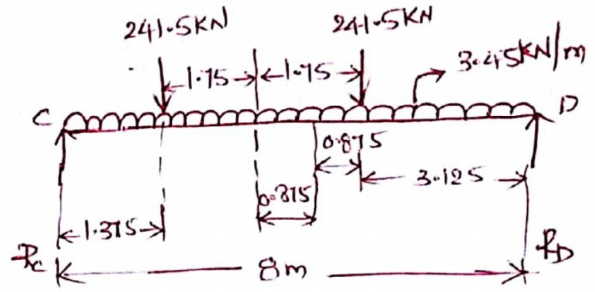
Bending moment due to Live load only.

Taking moment about D,

$\Rightarrow R_C \times 8 - 241.5(8 - 1.375)$

$\quad - 241.5(3.125)$

$\boxed{R_C = 294.33KN}$



Taking moment about C,

$\Rightarrow R_D \times 8 - 241.5(1.375) - 241.5(4.875)$

$\boxed{R_D = 188.67KN}$

Maximum bending moment due to live load $= 188.67 \times 3.125$

$= 589.6 KNm$

Bending moment due to Impact $= 0.1 \times 589.6 = 58.96KNm$

∴ Total bending moment due to live and impact loads

$= 589.6 + 58.96$

$= 648.56 KNm$

Bending moment due to Dead load $= \frac{wl^2}{8} = \frac{3.45 \times 8^2}{8} = 27.6 KNm$

∴ Maximum Bending moment $= 648.56 + 27.6 = 676.16KNm$

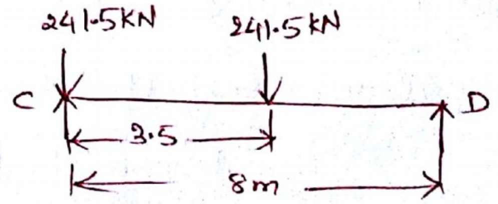
$= 676.16 \times 10^6 Nmm$

Step 3: Maximum shear force

Taking moment about D

$R_C \times 8 - 241.5 \times 8 - 241.5 \times 4.5$

$\Rightarrow \boxed{R_C = 377.34KN}$



Note: [It consists of Maximum shear due to moving wheel loads and self wt. of the girder]

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Hence maximum shear force due to wheel loads $= 377.34$ KN

Step 4: Lateral Forces.

Lateral forces transverse to the rails = 5% of weight of crab and weight lifted

$= 0.05 \times 240$

$= 12$ KN

Factored Lateral force $= 1.5 \times 12 = 18$ KN

Lateral Force on each wheel $= \frac{18}{2} = 9$ KN.

Maximum horizontal reaction due to the Lateral force by Proportion at C,

$$= \frac{\text{Lateral force} \times \text{Reaction @C due to Vertical load}}{\text{Max. Wheel load due to vertical load}}$$

$$= \frac{9 \times 294.33}{241.5}$$

$= 10.97$ KN

Horizontal reaction due to Lateral force @ D $= 18 - 10.97$

$= 7.03$ KN

Maximum bending moment due to lateral load by proportion

$$= \frac{9}{241.5} \times 589.6$$

$= 21.97$ KNm

Maximum shear force due to lateral load by proportion

$$= \frac{377.34}{241.5} \times 9$$

$= 14.06$ KN

Step 5: Selection of Trial Section

A trial section is selected based on

i) $Z_{px} = 1.4 \frac{M_p}{f_y} = 1.4 \times \frac{676.16 \times 10^6}{250} = 3786.5 \times 10^3 \, mm^3$

ii) Economical depth of section $= \frac{L}{12} = \frac{8 \times 10^3}{12} = 666.66 \simeq 600$ mm

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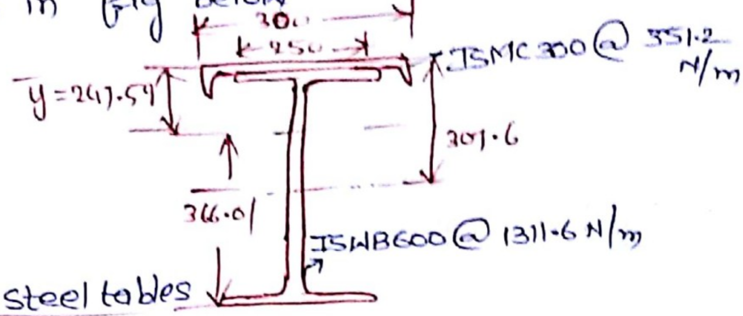
iii) Width of Flange $= \frac{L}{30} = \frac{8\times10^3}{30} = 266.66 \simeq 300mm$

∴ Let us try ISWB 600 @ 1311.6 N/m with ISMC 300 @ 351.2 N/m on its top flange as shown in fig below



Properties of section from steel tables

Property	I-Section ISWB600	Channel section ISMC 300
Area (A)	17,038 mm^2	4564 mm^2
Thickness of Flange (t_f)	21.3mm	13.6mm
Thickness of web (t_w)	11.2mm	7.6mm
Width of Flange (b_f)	250mm	90mm
Moment of Inertia (I_z)	$106198.5\times10^4 mm^4$	$6362.6\times10^4 mm^4$
(I_y)	$4702.5\times10^4 mm^4$	$310.8\times10^4 mm^4$
Depth of section (h)	600mm	300mm
Radius of root (r_1)	17mm	–
C_{yy}	–	23.6mm

Moment of Inertia of gantry girder

The distance of NA of built up section from the extreme fiber of compression flange

$$\bar{Y} = \frac{\Sigma AY}{\Sigma A}$$

$$= \frac{17038\times(300+7.6) + 4564\times23.6}{4564+17,038}$$

$$= 247.59mm$$

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Gross Moment of Inertia of the built up section

$$I_z(gross) = I_{z(beam)} + I_z(channel)$$

$$I_z(gross) = \left[106{,}198.5\times10^4 + 17{,}038(307.6-247.59)^2\right]$$

$$+$$

$$\left[310.8\times10^4 + 4564(247.59-23.6)^2\right]$$

$$= 135{,}543\times10^4 mm^4$$

$$I_y(gross) = I_y(beam) + I_y(channel)$$

$$= 4702.5\times10^4 + 6362.6\times10^4$$

$$= 11{,}065.1\times10^4 mm^4$$

$$Z_{ez} = \frac{I_z}{y} = \frac{135{,}543\times10^4}{(600+7.6-247.59)} = 3764.98\times10^3 mm^3$$

from figure, equal area axis,

$$4564 + 250\times21.3 + \bar{Y}_1\times11.2 = 250\times21.3 + (600-2\times21.3-\bar{Y}_1)\times11.2$$

$$\boxed{\bar{Y}_1 = 74.95mm}$$

plastic section Modulus of the section above equal area axis,

$$Z_{pz_1} = 300\times7.6\times\left(74.95+21.3+\frac{7.6}{2}\right)$$

$$+\left[2(90-7.6)\times13.6\times\left(74.95+21.3-\frac{90-7.6}{2}\right)\right]$$

$$+250\times21.3\times\left(74.95+\frac{21.3}{2}\right) + 74.95\times11.2\times\frac{74.95}{2}$$

$$Z_{pz_1} = 838.77\times10^3 mm^3$$

plastic section Modulus of the section below equal area axis,

$$Z_{pz_2} = 250\times21.3\times\left(600-21.3-74.95-\frac{21.3}{2}\right) + \frac{(600-2\times21.3-74.95)\times11.2}{2}$$

$$Z_{pz_2} = 3929.20\times10^3 mm^3$$

$$\therefore Z_{pz} = 838.77\times10^3 + 3929.20\times10^3 = 4767.97\times10^3 mm^3$$

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Plastic section modulus of compression flange about y-axis,

$$Z_{pfy} = \frac{250\times21.3\times250}{4} + \frac{2\times(300-13.6\times2)^2\times7.6}{8} + 2\left(13.6\times90\times\frac{300-13.6}{2}\right)$$

$$= 824.76\times10^3\,mm^3$$

Step 6: classification of section

I) $b = \frac{b_f}{2} \rightarrow$ outstand of flange I section, $b = \frac{b_f}{2} = \frac{250}{2} = 125\,mm$

II) $\frac{b}{t_f} = \frac{125}{21.3} = 5.86 < 8.4\varepsilon$

III) $\frac{d}{t_w} = ?$ $\quad d = h - 2(t_f + r_1)$

$= 600 - 2(21.3+17)$

$\boxed{d = 523.4\,mm}$

III) $\frac{d}{t_w} = \frac{523.4}{11.2} = 46.73 < 84\varepsilon = 46.73 < 84$

Hence section is plastic

Step 7: check for Moment capacity

Local moment capacity:

$$M_{dz} = \beta_b Z_{pz} \frac{f_y}{\gamma_{mo}} \le 1.2 Z_e \frac{f_y}{\gamma_{mo}}$$

$$= 1\times4767.97\times10^3\times\frac{250}{1.10} \le 1.2\times3764.98\times10^3\times\frac{250}{1.10}$$

$$= (1083.63\,kNm) \qquad (1026.81\,kNm)$$

∴ Hence Moment capacity of section $\boxed{M_{dz} = 1026.81\,kNm}$

Moment capacity of compression flange about y-axis

$$M_{dy,f} = \beta_b Z_{pyf} \frac{f_y}{\gamma_{mo}} \le 1.2 Z_{ey,f} \frac{f_y}{\gamma_{mo}}$$

$$= 1\times824.76\times10^3\times\frac{250}{1.1} \le 1.2\times580.92\times10^3\times\frac{250}{1.1}$$

$$= (187.44\,kNm) \qquad (158.43\,kNm)$$

∴ Hence, Moment capacity of flange $\boxed{M_{dy,f} = 158.43\,kNm}$

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8) Combined check for local moment capacity

$$\frac{M_z}{M_{dz}} + \frac{M_{y,f}}{M_{dy,f}} \leq 1.0$$

$$\frac{676.16}{1026.81} + \frac{21.97}{158.43} \leq 1.0$$

$0.79 \leq 1.0$ (safe)

9) check for buckling resistance in bending.

Assuming Gantry girder is laterally unsupported

As per cl. 8.2.2.1 / 54, $M_{dz} = \beta_b Z_{pz} f_{bd}$

Effective length $L_{LT} = 8 \times 10^3$ mm.

Radius of gyration $r_{min} = \sqrt{\frac{I_y}{A}}$

$I_{yy} = 4702.5 \times 10^4 + 6362.6 \times 10^4 = 11065.1 \times 10^4 \text{ mm}^4$

$r_{min} = \sqrt{\frac{11065.1 \times 10^4}{(17038 + 4564)}} = 71.57$ mm

c/c distance between flanges (h_f) $= 600 - \frac{21.3}{2} - \frac{21.3}{2}$

$= 578.7$ mm

i) $\frac{h_f}{t_f} = \frac{578.7}{21.3} = 27.17$

ii) $\lambda = \frac{KL}{r} = \frac{8 \times 10^3}{71.57} = 111.77$

From Table 14/57, $\frac{Kl}{r} = 111.77$ & $\frac{h_f}{t_f} = 27.17$

For class 'a' buckling, the imperfection factor $\alpha_{LT} = 0.21$

From cl. 7.1.2.2/35, Table 7, $\alpha_{LT} = 0.21$

Use Table 14/57, To find $f_{cr,b}$

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λ	h/t_f		
	25	27.17	30
110	251.8	X	232.1
111.77		0	
120	221.2	Y	202.4

$$X = 251.8 - \frac{(251.8-232.1)}{(30-25)}(27.17-25)$$

$$= 243.25 \text{ N/mm}^2$$

$$Y = 221.2 - \frac{(221.2-202.4)}{(30-25)}(27.17-25)$$

$$= 213.04 \text{ N/mm}^2$$

$$0 = 243.25 - \frac{(243.25-213.04)}{(120-110)}(111.77-110)$$

$$= 237.90 \text{ N/mm}^2$$

$\therefore$ critical stress in bending $f_{cr,b} = 237.90 \text{ N/mm}^2$

for imperfection factor $\alpha_{LT} = 0.21$, $f_{cr,b}$. 139/55 page

$f_{cr,b}$	f_y
250	152.3
237.90	?
200	134.1

$$f_{bd} = 152.3 - \frac{(152.3-134.1)}{(250-200)}(250-237.90)$$

$$= 147.944 \text{ N/mm}^2$$

$\therefore$ Design Bending strength (or) Buckling resistance $M_{dz} = \beta_b Z_{pz} f_{bd}$

$$= 1 \times 4767.977 \times 10^3 \times 147.94$$

$$= 705.37 \text{ KNm} > 676.16 \text{ KNm}$$

(Safe)

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$M_{dy} = Z_{yt} \times \frac{f_y}{\gamma_{mo}}$

Z_{yt} = section modulus for top flange only

$= \frac{I_{yt}}{y_{max}}$

$= \frac{\frac{1}{2} I_y \text{ for I-section} + I_z \text{ of channel section}}{y_{max}}$

$= \frac{\frac{1}{2} \times 4702.5 \times 10^4 + 6362.6 \times 10^4}{\left(\frac{300}{2}\right)}$

$Z_{yt} = 581 \times 10^3 \, mm^3$

$\therefore M_{dy} = 581 \times 10^3 \times \frac{250}{1.1} = 132.04 \times 10^6 \, Nmm$

Step 10: Check for Bi-axial bending

$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1 \Rightarrow \frac{676.16}{837.20} + \frac{21.97}{132.02} = 0.974 < 1.0$

Hence safe

Step 11: Check for Design shear strength

As per cl. 8.4.1/59, $V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

$A_v = h t_w = 600 \times 11.2$

$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{600 \times 11.2 \times 250}{\sqrt{3}(1.1)}$

$\boxed{V_d = 881.77 \, KN}$

Maximum shear force due to wheel load = 377.34 KN

Impact load = $0.1 \times 377.34 = 37.34$ KN (10% of wheel load)

Design shear force = $377.34 + 37.73 = 415.07$ KN

$\therefore V \leq V_d$

(415KN ≤ 881.77 KN)

(Safe)

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Check for Low/High shear case:

$0.6V_d = 0.6(881.77) = 529\ KN$

$V \leq 0.6V_d$

$(415.07 \leq 529\ KN)$ (Safe)

Step 12: Check for Web crippling

As per cl. 8.7.4/67, $F_w = (b_1 + n_2) t_w \dfrac{f_{yw}}{\gamma_{m0}}$

b_1 = stiff bearing length = 150mm (Diameter of wheel)

$n_2 = 2.5(t_f + r_1) = 2.5(21.3+17) = 95.75$

$F_w = (150+95.75) 11.2 \times \dfrac{250}{1.1}$

$F_w = 625.54\ KN > (241.5\ KN)$ (Safe)

Step 13: Check for Web buckling

As per cl. 8.4.2.1/59, $\dfrac{d}{t_w} = \dfrac{523.4}{11.2} = 46.73 < 67\varepsilon$

$= 46.73 < 67$

Hence check for Web buckling not required.

Step 14: Check for Deflection

$$\delta = \frac{WL^3 \times \left(\frac{3a}{4L} - \frac{a^3}{L^3}\right)}{6EI}$$

Where W = Maximum Static wheel load $= \dfrac{241.5}{1.5} = 161\ KN$

$a = \dfrac{L-c}{2} = \dfrac{8\times10^3 - 3.5\times10^3}{2} = 2.25\times10^3\ mm$

Vertical deflection $(\delta) = \dfrac{161\times10^3 \times (8\times10^3)^3 \times \left[\dfrac{3\times2.25\times10^3}{4\times8\times10^3} - \dfrac{(2.25\times10^3)^3}{(8\times10^3)^3}\right]}{6\times2\times10^5\times135{,}543\times10^4}$

$(\delta_{cal}) = 9.56mm$

As, per Limits, permissible max. deflection $= \dfrac{L}{500} = \dfrac{8000}{500} = 16mm$

$16mm > 9.56mm$ (Safe)

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Step 15: Design of Welded Connection

Required shear capacity of the weld, $q = \frac{VA\bar{y}}{I_z}$

$A\bar{y}$ = Moment of Top flange the channel from NA

$\bar{y} = 247.59 - 7.6 = 239.9\,mm$

$$\therefore q = \frac{377.34 \times 10^3 \times 4564 \times 239.9}{135{,}543 \times 10^4}$$

$$\boxed{q = 304.82\,N/mm}$$

Strength of weld$/_{mm}$ $\Rightarrow l_w \times t_t \times f_{wd} = 304.82$

$$\Rightarrow 1 \times 0.7 \times S \times \frac{410}{\sqrt{3}(1.25)} = 304.82$$

$$S = 3.04 \simeq 3mm.$$

∴ Provide 3mm size fillet weld.

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