LATERALLY SUPPORTED BEAMS

A beam is a structural member subjected to transverse loads (Loads ⊥ar to Longitudinal axis) is called a Beam.

→ When provided in buildings to support Roofs, they are Called "Joists"

→ A large beam supporting a no. of Joists is called a 'Girder'.

→ Beams which carry roof loads in Trusses are called 'Purlins'

→ I-sections are used as beam for light and Moderate loads

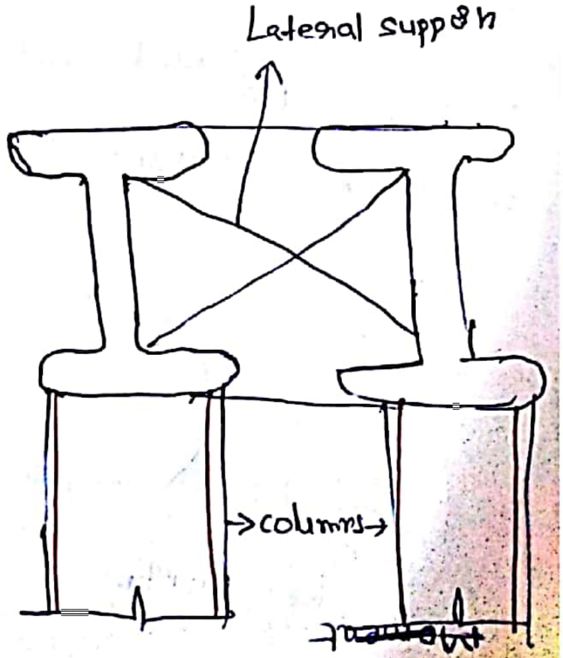
→ Heavy loads built up sections are used

→ Small loads Angle sections/ channel sections are used.

→ The compression flange of an I-section acts like a column and has a tendency to buckle sideways if the beam is not sufficiency stiff.

→ If the compression flange of the beam is restrained they are called "Laterally supported beams" (or) "Laterally Restrained beams"

→ These beams are supported by means of supports in Lateral direction.



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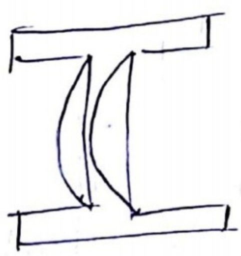
Classification of cross section (Table 2/18)

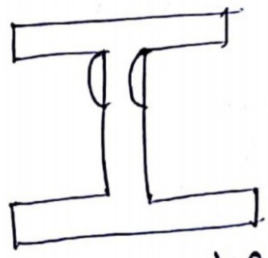
1) plastic - class I
2) compact - class II
3) Semicompact - class III

Design procedure

1) Based on loads and Bending moment values a trial section is Selected based on (Assuming) class I (or) plastic section properties Z_p values
2) Check the class of section (Table 2/18)
3) Design shear strength (V_d) is calculated and checked against Factored SF (V) (8.4/59)
(4) Based on $V \le 0.6 V_d$ (or) $V > 0.6 V_d$ section is classified as high and low shear case (8.2.1.3/53)
4) check for design moment (or) Bending strength (M_d) based on $V > 0.6 V_d$ (or) $V \le 0.6 V_d$ (8.2.1.2/53)
5) check for shear strength (8.4/59)
6) Check for deflection (based on Serviceability conditions by Elastic analysis) (Table 6/31)
7) check for web buckling (8.7.3.1/67)
8) check for web crippling (8.7.4/67)



web buckling



web crippling

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No.

1) A simply supported steel joist 4m effective span is laterally supported throughout. It carries a total UDL of 40kN inclusive of self wt. Design an appropriate section using Fe410 steel?

Sol Given data, $l_{eff}\,(KL) = 4m$

Total UDL $= \omega L = 40\,kN$.

Fe 410 steel $\Rightarrow$ $f_u = 410\,N/mm^2$; $f_y = 250\,N/mm^2$

Factored load $= 1.5 \times 40 = 60\,kN$

Factored moment $(M_{max}) = \frac{\omega l^2}{8} = \frac{\omega L(l)}{8} = \frac{60 \times 4}{8} = 30\,kNm$

Factored Shear Force $(V_{max}) = \frac{\omega L}{2} = \frac{60}{2} = 30\,kN$

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

(Table 5/30)

$= \frac{30 \times 10^6}{250} \times 1.10$

$= 132000\,mm^3$

This Z_p is based on BM only, the beam is subjected to shear force also hence while selecting the section increase the Z_p value by 30 to 50%.

Section and its properties

Try ISLB 200 @ 194.2 N/m having $Z_p = 184.34 \times 10^3\,mm^3$
$> 132 \times 10^3\,mm^3$

From steel tables, $h = 200\,mm$

$b_f = 100\,mm$ $\quad I_{zz} = 1696.6 \times 10^4\,mm^4$

$t_f = 7.3\,mm$ $\quad Z_e = 169.7 \times 10^3\,mm^3$

$t_w = 5.4\,mm$

$r_1 = 9.5\,mm$

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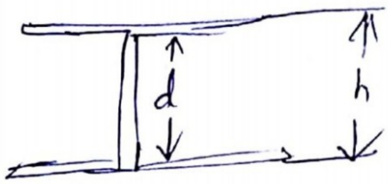
depth of web $(d) = h - 2(t_f + r_1)$



$= 200 - 2(7.3 + 9.5)$

$= 166.4\,mm$

<u>Step 2:</u> check for class of section (Table 2 / 18)

$b = b_f = \frac{100}{2} = 50\,mm$; Now $\frac{b}{t_f} = \frac{50}{7.3} = 6.85 < 9.4\varepsilon$

$= 6.84 < 9.4$

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

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

Hence section is plastic

<u>Step 3</u> calculation of Design Shear strength (V_d) $\left[\frac{8.4}{59}\right]$

$V \leq V_d$

$V_d = \frac{V_n}{\gamma_{m0}}$; $V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ (cl. $\frac{8.4.1}{59}$)

A_v = shear area

f_{yw} = yield strength of web.

$A_v = h t_w$ (cl. $\frac{8.4.1.1}{59}$)

$= 200 \times 5.4$

$\boxed{A_v = 1080\,mm^2}$

$V_n = \frac{1080 \times 250}{\sqrt{3}}$

$\boxed{V_n = 155.88\,KN}$

$V_d = \frac{155.88}{1.10}$

$\boxed{V_d = 141.17\,KN}$

<u>Check for Low (or) High shear</u> (cl. $\frac{8.2.1.2}{53}$)

If $V \leq 0.6 V_d \rightarrow$ Low shear case

$V > 0.6 V_d \rightarrow$ High shear case

$V = 30\,KN$; $V_d = 141.17\,KN \Rightarrow 0.6 V_d = 85.026\,KN$

$V < 0.6 V_d \rightarrow$ low shear case

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Step 4: check for design Bending strength for $V < 0.6V_d$ $\left(\frac{8.2.1.2}{53}\right)$

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} < 1.2 \frac{Z_e f_y}{\gamma_{m0}} \quad \text{(Simply supported)}$$

$$= \frac{1 \times 184.34 \times 10^3 \times 250}{1.1} < 1.2 \times 169.7 \times 10^3 \times \frac{250}{1.1} \quad (\beta_b = 1 \text{ for plastic section})$$

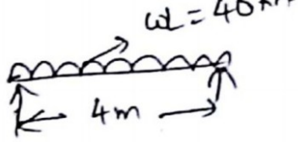
$$= 41.895 \times 10^6 \text{ Nmm} < 46.28 \times 10^6 \text{ Nmm} \quad \text{(safe)}$$

Step 5 check for Design Shear strength $\left(\frac{8.4}{59}\right)$

$$V \leq V_d$$

$$30 \leq 141.71 \text{ kN} \quad \text{(safe)}$$

Step 6 check for Deflection



Deflection is checked in Elastic state

$$(\delta_{cal}) = \frac{5}{384} \frac{wl^4}{EI}$$

$$= \frac{5}{384} \frac{(wL) l^3}{EI} = \frac{5}{384} \times \frac{(40 \times 10^3) \times (4 \times 10^3)^3}{2 \times 10^5 \times 1696.6 \times 10^4}$$

$$\boxed{(\delta_{cal}) = 9.82 \text{mm}}$$

-As per $\frac{\text{Table 6}}{31}$, for other buildings the vertical deflection for roof and floor elements not susceptible to cracking

$$\text{Maximum permissible Deflection} = \frac{\text{Span}}{300}$$

$$= \frac{4 \times 10^3}{300}$$

$$= 13.33 \text{mm} > 9.82 \text{mm}$$

Hence safe in Elastic design.

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Step 7 Check for Web crippling (or) Web bearing

As per cl. 8.7.4 / 67, Design Capacity of web in bearing (f_w)

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

b_1 = stiff bearing length (8.7.1.3)

Assuming $b_1 = 70mm$ ($< b_f = 100mm$)

n_2 = Length obtained by dispersion @ slope of 1:2.5

$$n_2 = 2.5(t_f + r)$$

$$= 2.5(7.3 + 9.5)$$

$$\boxed{n_2 = 42}$$

$$f_w = (70+42) \times \frac{5.4 \times 250}{1.1}$$

$$= 137.45 KN > 30 KN \quad \text{(Safe)}$$

Step 8: check for Web buckling

As per cl. 8.4.2.1 / 59, $\frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 67\varepsilon$

$$= 30.81 < 67$$

Hence check for web buckling not required.

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2) Design a Laterally supported beam of Effective span 6m, Factored moment 150 kNm and Factored shear force 210 kN. Take Fe410 steel?

Sol Given data, Effective span = 6m

Factored moment (M) = 150 kNm = 150×10^6 Nmm

Factored shear force (V) = 210 kN

Step 1 Fe410 steel $\Rightarrow f_y = 250\ N/mm^2 = f_{yw}$

Plastic section Modulus (Z_p) required $= \frac{M_p\ \gamma_{m0}}{f_y}$

$= \frac{150 \times 10^6 (1.1)}{250}$

$= 660 \times 10^3\ mm^3$

To accomodate shear force, the section should have more than 30 to 50% Z_p value.

Section and Its properties:

Try ISLB 350 @ 495 N/m having $Z_p = 851.11 \times 10^3\ mm^3 > 660 \times 10^3$

From steel tables, $h = 350mm$; $b_f = 165mm$; $t_f = 11.4\ mm$

$t_w = 7.4mm$; $r_1 = 16mm$; $I_z = 13158.3 \times 10^4\ mm^4$

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

$= 350 - 2(11.4 + 16)$

$\boxed{d = 295.2mm}$

Step 2: class of section

From IS 800: 2007, pg No. 18 / Table 2,

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

b) $\frac{b}{t_f} = \frac{70}{16} = 4.38 < 9.4\varepsilon = 4.38 < 9.4$

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c) $\frac{d}{t_w} = \frac{338}{8.9} = 37.98 < 84\varepsilon = 37.98 < 84$

$\therefore$ Hence the given section is belongs to Class I. Hence it is plastic section.

Step 3: Calculation of Design Shear Strength (V_d)

As per cl. $\left(\frac{8.4}{59}\right)$, $V \leq V_d$; $V_d = \frac{V_n}{\gamma_{mo}}$

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

But $A_v = h t_w = 350 \times 7.4 = 2590 \text{mm}^2$

$$V_n = \frac{2590 \times 250}{\sqrt{3}} = 373.83 \text{KN}$$

$$\therefore V_d = \frac{V_n}{\gamma_{mo}} = \frac{373.83 \times 10^3}{1.1} = 339.84 \text{KN}$$

$$\therefore \boxed{V_d = 339.84 \text{ KN}}$$

Check for Low shear (or) High shear:

$V \leq 0.6 V_d \rightarrow$ Low shear case
$V > 0.6 V_d \rightarrow$ High shear case
} cl. $\left(8.2.1.2/53\right)$

$$0.6 V_d = 0.6 \times 339.84 = 203.91 \text{ KN}$$

$V > 0.6 V_d$

(210KN) > (203.91 KN), Hence it is High shear case

Step 4: Check for Design Bending Strength

As per cl. 8.2.1.3/53, when V exceeds $0.6V_d$, $M_d = M_{dv}$

As per cl. 9.2.2(a)/70, $M_{dv} = M_d - \beta(M_d - M_{fd}) \leq \frac{1.2 Z_e f_y}{\gamma_{mo}}$

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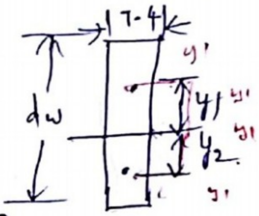
As per cl. $\frac{8.2.1.2}{53}$, $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} = \frac{1 \times 851.11 \times 10^3 \times 250}{1.1}$

$$M_d = 193.43 \times 10^6 \text{ Nmm}$$

From $\frac{9.2.2(a)}{70}$, $\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 210 \times 10^3}{339.84 \times 10^3} - 1\right)^2 \rightarrow \boxed{\beta = 0.0556}$

M_{fd} = plastic design strength of the area of c/s excluding shear area considering γ_{m0}



$$M_{fd} = \frac{Z_{fd} \times f_y}{\gamma_{m0}}$$

Z_{fd} = plastic section Modulus excluding shear Area

$= Z_p - Z_{p\,web}$

$= 851.11 \times 10^3 - 161.21 \times 10^3$

$$\boxed{Z_{fd} = 689.895 \times 10^3 \text{ mm}^3}$$

$y_1 = y_2$

$Z_{p(web)} = \frac{A}{2}(y_1 + y_2)$

$= \frac{A}{2}(2y_1)$

$= Ay_1$

$= A\left(\frac{d_w}{4}\right)$

$= (295.2 \times 7.4)\left(\frac{295.2}{4}\right)$

$$\boxed{Z_{p(web)} = 161.21 \times 10^3 \text{ mm}^3}$$

Now,

$$M_{fd} = \frac{689.895 \times 10^3 \times 250}{1.1}$$

$$\boxed{M_{fd} = 156.79 \times 10^6 \text{ Nmm}}$$

$\therefore$ $M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e \frac{f_y}{\gamma_{m0}}$

$= 193.43 \times 10^6 - 0.0556\left(193.43 \times 10^6 - 156.79 \times 10^6\right)$

$\leq 1.2 \times 751.9 \times 10^3 \times \frac{250}{1.1}$

$= 191.39 \times 10^6 \text{ Nmm} \leq 205.06 \times 10^6 \text{ Nmm}$

Hence safe

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Step 5: Check for Design Shear strength:

As per cl. 8.4/59, $V \le V_d$ here $210KN \le 339.84 KN$

Hence safe

Step 6: Check for Deflection:

As the type of load on the beam is not given, deflection cannot be checked.

Step 7: check for Web crippling (or) Web bearing:

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

b_1 = stiff bearing length ($< b_f = 165mm$)

See (8.7.1.3/65)

Assume $b_1 = 100mm$

$n_2 = 2.5(t_f + r)$

$= 2.5(11.4 + 16)$

$n_2 = 68.5$

$\therefore F_w = (100 + 68.5)\, 7.4 \times \frac{250}{1.1}$

$\boxed{F_w = 283.386 KN} > V\ (210KN)$

Hence safe

Step 8: Check for Web buckling:

As per cl. $\frac{8.4.2.1}{59}$, $\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 67\varepsilon$

$= 39.89 < 67.$

Hence check for web buckling is Not required

$\therefore$ (web is safe against buckling)

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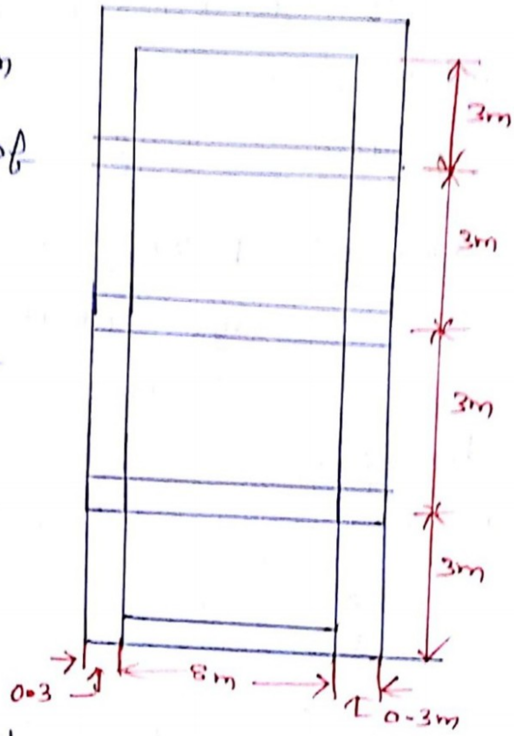
3) A roof of a hall measuring 8m×12m consists of 100mm thick R.C slab supported on steel I beams spaced 3m apart. The finishing load may be taken as 1.5 kN/m² and Live load as 1.5 kN/m². Design the steel beam?

Sol Given data, From the given figure,



Each beam has a clear span of 8m and takes care of 3m width of slab.

Hence load per metre length of beam is as follows.

a) Weight of R.C. slab

$= 0.1 \times 1 \times 3 \times 25$

$= 7.5 \, kN/m$

b) Finish load $= 1.5 \times 3 \times 1 = 4.5 \, kN/m$

c) Live load $= 3 \times 1.5 \times 1 = 4.5 \, kN/m$

d) Assume self wt of the steel section $= 0.8 \, kN/m$

$\therefore$ Total load $= 7.5 + 4.5 + 4.5 + 0.8 = 17.3 \, kN/m$

Total factored load $= 1.5 \times 17.3 = 25.95 \, kN/m$

Effective span $= \frac{0.3}{2} + 8 + \frac{0.3}{2} = 8.3m.$

Maximum factored shear force $= \frac{wl}{2} = \frac{25.95 \times 8.3}{2} = 107.69 \, kN$

Maximum factored Bending moment $= \frac{wl^2}{8} = \frac{25.95 \times (8.3)^2}{8} = 223.46 \, kN m.$

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

$$= \frac{223.46 \times 10^6 (1.1)}{250}$$

$$= 983224 \, mm^3$$

Select a section ISMB400 having $Z_p = 1176.163 \times 10^3 mm^3 > 983.22 \times 10^3 mm^3$

Section and properties

$h = 400$; $b_f = 140mm$; $A = 7845.58 mm^2$; $t_f = 16mm$; $t_w = 8.9mm$; $r_1 = 14mm$

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

$= 400 - 2(16+14)$

$\boxed{d = 340 \, mm}$

$Z_e = 1022.7 \times 10^3 mm^3$

$I_{zz} = 20458.4 \times 10^4 mm^4$

Step2: class of section:

From IS:800:2007, Table 2/18,

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

b) $\frac{b}{t_f} = \frac{(b_f/2)}{t_f} = \frac{(140/2)}{16} = \frac{70}{16} = 4.38 < 9.4\varepsilon = 4.38 < 9.4$

c) $\frac{d}{t_w} = \frac{340}{8.9} = 38.20 < 84\varepsilon = 38.20 < 84$

Hence section is plastic.

Step3: calculation of Design shear strength (V_d)

As per cl. $\left(\frac{8.4}{59}\right)$, $V \leq V_d$; $V_d = \frac{V_n}{\gamma_{mo}}$

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

But $A_v = ht_w = 400 \times 8.9$

$\cancel{V_d} = \frac{\cancel{V_n}}{\sqrt{3}\cancel{\gamma_{mo}}}$ $V_d = \frac{A_v f_{yw}}{\sqrt{3}\gamma_{mo}} = \frac{400 \times 8.9 \times 250}{\sqrt{3}(1.1)}$

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$$\boxed{V_d = 467.128\,kN}$$

Check for low shear/High shear case

$0.6 V_d = 0.6(467.128) = 280.277\,kN$

$V \le 0.6 V_d$ ⟶ Hence it is "Low shear case".

$(107.69) \le (\cancel{467.128}\ 280.277\,kN)$

Step 4: check for Design Bending Strength

As per cl. $\left(\frac{8.2.1.2}{53}\right)$, $M_d = \dfrac{\beta_b Z_p f_y}{\gamma_{mo}} < 1.2\, Z_e \dfrac{f_y}{\gamma_{mo}}$

$M_d = 1 \times \dfrac{1176.163 \times 10^3 \times 250}{1.1} < 1.2 \times 1022.7 \times 10^3 \times \dfrac{250}{1.1}$

$M_d = 267.31 \times 10^6 < 278.91 \times 10^6$ (safe)

Step 5 check for Design shear strength

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

$107.69 \le 467.128\,kN$ (safe)

Step 6: check for Deflection

Total working load = ~~[illegible]~~ 17.3 KN/m = 17.3 N/mm.

$(\delta_{cal}) = \dfrac{5}{384} \dfrac{wl^4}{EI} = \dfrac{5}{384} \times \dfrac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4} = 26.127\,mm$

As per Table 6 / 31 For other buildings the vertical deflection for roof and floor, elements not susceptible to cracking.

Max. permissible Deflection $= \dfrac{span}{300} = \dfrac{8300}{300} = 27.67\,mm > 26.127\,(mm)$

(Safe)

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Step 7: check for web crippling

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

b_1 = stiff bearing length.

Assuming $b_1 = 100mm$ ($< b_f = 140mm$)

n_2 = Length obtained by dispersion @ slope of 1:2.5

$n_2 = 2.5(t_f + r_1)$

$= 2.5(16+14)$

$\boxed{n_2 = 75mm}$

$\therefore F_w = (100+75)\, 8.9 \times \frac{250}{1.1}$

$\boxed{F_w = 353.97 kN > 107.69 kN}$ Hence safe

Step 8: check for web buckling

As per cl. $\left(\frac{8.4.2.1}{59}\right)$, $\frac{d}{t_w} = \frac{340}{8.9} = 38.20 < 67\varepsilon$

$= 38.20 < 67(1)$

Hence check for web buckling not required.

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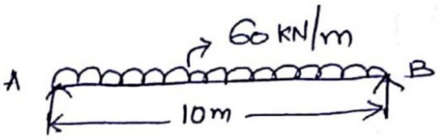
BUILT UP BEAMS

1) Design a simply supported beam of 10m effective span carrying a total factored load 60 kN/m. The depth of the beam should be less than 500mm. Compression Flange of the beam is Laterally supported by floor construction?

Sol Given data,



Effective span = 10m

Factored load = 60 kN/m

Depth of beam < 500mm

Step 1:

Maximum Shear Force $V = V_{max} = \frac{wl}{2} = \frac{60\times10}{2} = 300\,kN$

Maximum Bending moment $M = M_{max} = \frac{wl^2}{8} = \frac{60\times10^2}{8} = 750\,kNm$

$$Z_{p(required)} = \frac{M}{f_y}\gamma_{m0}$$

$$= \frac{750\times10^6}{250}\times1.1 = 3300\times10^3\,mm^3$$

As depth of the beam should be less than 500mm, Select a beam Section ISMB450 having $Z_p = 1533.4\times10^3\,mm^3$ ($< Z_p$ required). Hence Provide Suitable plates over flanges

Z_p to be provided by plates $= 3300\times10^3 - 1533.4\times10^3$

$$= 1766.6\times10^3\,mm^3$$

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

Assuming thickness of each cover plate as 20mm.

The resisting force of each plate is $(A_p\times f_y)$

Plastic Moment of resistance of each plate (Z_p) = M_p of each plate.

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

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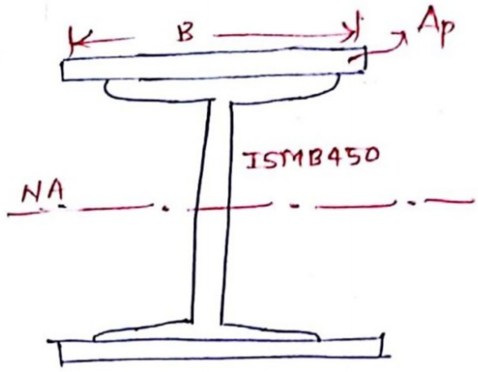
Z_p of each plate $= \frac{M_p}{f_y} \times \gamma_{mo}$



$$\Rightarrow \frac{1766.6 \times 10^3}{2} = \frac{A_p \times \cancel{f_y}\left(\frac{450}{2}+\frac{20}{2}\right)\times 1.1}{\cancel{f_y}}$$

$$\Rightarrow \quad A_p = 3417.02 \, mm^2$$

Minimum width of each plate $(B) = \frac{3417.02}{20} = 170.85 \, mm$

Assume $B = 220 \, mm$ ∴ provide each cover plate of $220 \times 20 \, mm$.
One on top flange and other on bottom flange

From steel tables (ISMB 450):

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

~~Step2~~ Depth of web $(d) = h - 2(t_f + r_1)$

$$= 450 - 2(17.4 + 15)$$

$$\boxed{d = 385.2 \, mm}$$

Step 2: classification of section:

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

$$= 4.31 < 9.4(1)$$

$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}}$

$\boxed{\varepsilon = 1}$

ii) $\frac{d}{t_w} = \frac{385.2}{9.4} = 40.97 < 84\varepsilon = 40.97 < 84.$

Hence section is plastic

Step 3: Calculation of Design Shear strength:

From cl. 8.4/59, $V \le V_d$

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

But $V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ $\left(cl. \frac{8.4.1}{59}\right)$

$$\Rightarrow A_v = h \cdot t_w$$

$$= 450 \times 9.4$$

$$\boxed{A_v = 4230 \, mm^2}$$

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

$$= \frac{4230 \times 250}{\sqrt{3}}$$

$$V_n = V_p = 610.54\ KN$$

$$\therefore V_d = \frac{V_n}{\gamma_{m0}} = \frac{610.54}{1.1} = 555.04\ KN. \therefore \boxed{V_d = 555.04\ KN}$$

Check for Low/High shear

$0.6V_d = 0.6 \times 555.04 = 333.02\ KN$

$(V) = 300KN < \overset{0.6}{(V_d)}\ 333.02\ KN$. Hence it is "Low shear case".

Step 4: Check for Design Bending Strength: (M_d)

As per cl. 8.2.1.2 / 53, $V < 0.6V_d$, $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} < 1.2 \frac{Z_e f_y}{\gamma_{m0}}$

$Z_{p\,(provided)} = Z_{p\,(I\,Section)} + Z_{p\,(Two\ plates)}$

$= 1533.4 \times 10^3 + 2068 \times 10^3\ mm^3$

$$\boxed{Z_p = 3601.4 \times 10^3\ mm^3}$$

$\left[Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)\right.$

$= \frac{220 \times 20}{2}\left(\frac{450}{2} + \frac{20}{2} + \frac{450}{2} + \frac{20}{2}\right)$

$= 1034 \times 10^3\ mm^3$

$Z_p = 2 \times 1034 \times 10^3$

$\left.= 2068 \times 10^3\ mm^3\right]$

$$M_d = \frac{1 \times 3601.4 \times 10^3 \times 250}{1.1} < \frac{1.2 \times 3225.19 \times 10^3 \times 250}{1.1}$$

$$\boxed{M_d = 818.5 \times 10^6 < 879.6 \times 10^6\ Nmm}$$

$Z_e = \frac{I_{zz}}{y_{max.}} = \frac{79017 \times 10^4}{\left(\frac{450}{2} + 20\right)} = 3225.19 \times 10^3\ mm^3$

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

$= 30390 \times 10^4 + 2\left[\frac{220 \times 20^3}{12} + (220 \times 20)\left(\frac{450}{2} + 10\right)^2\right]$

$= 30390 \times 10^4 + 486.27 \times 10^6$

$$\boxed{I_{zz} = 79017 \times 10^4\ mm^4}$$

Hence safe.

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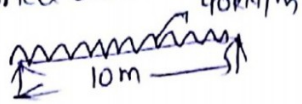
Step 5: check for Design shear strength

As per cl. 8.4/59, $V \le V_d$ (300 kN $\le$ 555.04 kN) Hence safe

Step 6: check for Deflection:

As per cl. 8. Deflection is checked in Elastic condition.



$\therefore$ Working load $= \frac{60}{1.5} = 40\,kN/m$

$$\delta_{cal} = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384}\frac{40 \times (10 \times 10^3)^4}{2 \times 10^5 \times 790.17 \times 10^6}$$

$\delta_{cal} = 32.95\,mm$

As per Table 6/31, Permissible deflection $= \frac{span}{300} = \frac{l}{300} = \frac{10 \times 10^3}{300} = 33.33\,mm$

$\therefore$ 33.33 > 32.95 mm Hence safe

Step 7: Check for Web crippling (or) Web bearing:

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

b_1 = stiff bearing length. Assume $b_1 = 100\,mm$; $n_2 = 2.5(t_f + r_1)$

$= 2.5(17.4 + 15)$

$\boxed{n_2 = 81\,mm}$

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

$= 386.68\,kN > 300\,kN$ Hence safe

Step 8: check for Web buckling

$\frac{d}{t_w} = \frac{385.2}{9.4} = 40.97 < 67\varepsilon$. Hence web buckling is not required.

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