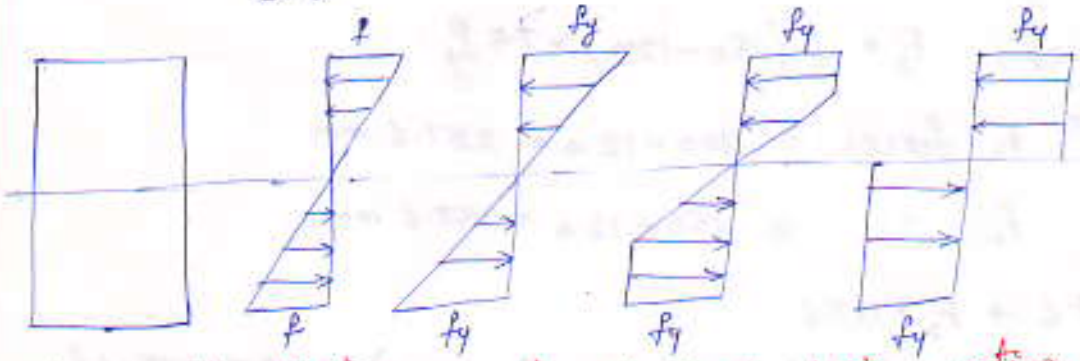
DESIGN OF BEAMS

Beam is a structural member with length considerably larger than c/s dimension subject to lateral loads which give rise to BM & SFs in the member. There are mainly two types of beams based on the lateral supports to compression flanges a) Laterally supported beams

b) " un " "

If the compression flanges are laterally supported by flooring; it is mainly subjected to bending and shear. If compression flange of beam is not supported laterally, the lateral buckling of compr " reduces the load carrying capacity of beams.

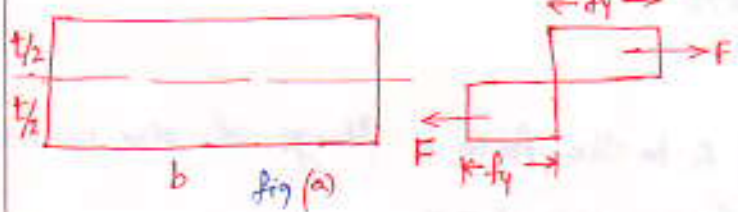


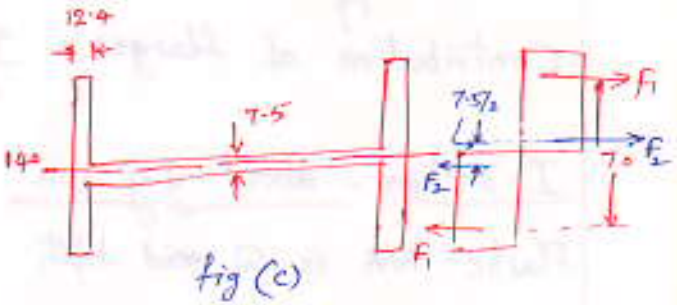
Prob Determine the plastic moment capacity & plastic section modulus of

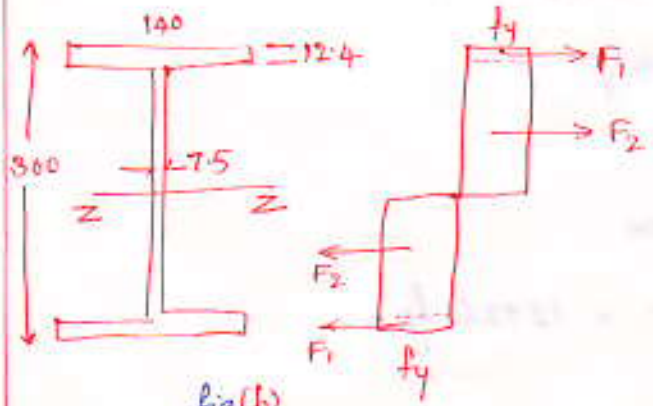
a) the rectangular section of size b x t about z-z axis (fig a)

b) I section about z-z axis (fig b)

c) " " " 4-4 axis (fig c)







Soln: a) Rectangular section

Due to symmetry plastic NA (axis of equal area) is at mid depth.

$\therefore A_t = A_c = \frac{b}{2} \times t \Rightarrow F = b/2\, t f_y$

The distance bet^n tensile & compressive forces = $t/2$

$\therefore M_p = F \times \frac{t}{2} = \frac{b}{2} \cdot t \cdot f_y \frac{t}{2} = \frac{1}{4} b t^2 f_y$

$\therefore Z_p = \frac{M_p}{f_y} = \frac{1}{4} b t^2$

b) I-Section - about z-z axis

Plastic NA is @ mid depth. When plastic hinge is formed, forces in flanges

$F_1 = 140 \times 12.4 \times f_y$

forces in the web; $F_2 = \frac{1}{2}(150 - 12.4) \times 7.5 f_y$

Distance bet^n F_1 forces = $300 - 12.4 = 287.6$ mm

" " F_2 " = $150 - 12.4 = 137.6$ mm

$\therefore M_p = F_1 \times 287.6 + F_2 \times 137.6$

$= (140 \times 12.4 f_y \times 287.6) + \left(\frac{1}{2} \times 137.6 \times 7.5 f_y \times 137.6\right) = 570275.6 f_y$

$\therefore Z_p \neq \frac{M_p}{f_y} = Z_p = 570.26 \times 10^3\ mm^3$

Contribution of flanges = $\frac{49274}{570276} \times 100 = 87.5\%$

c) I-section - about y-y axis

Plastic NA is @ mid depth. Let f_1 be the force in flange of size 140×12.4 & F_2 be the force in web of size $(300 - 2 \times 12.4)\ 7.5$

$f_1 = 140 \times 12.4 f_y \qquad F_2 = 275.2 \times 7.5 f_y$

Distance bet^n F_1 force = $140/2 = 70$ mm

" " F_2 " = $\frac{7.5}{2} = 3.75$ mm

$M_p = 140 \times 12.4 f_y \times 70 + 275.2 \times 7.5 f_y \times 3.75 = 129260 f_y$

$\therefore Z_p = \frac{M_p}{f_y} = 129260\ mm^3$

Contribution of flange = $\frac{121520}{129260} \times 100 = 94\%$

Classification of Cross Sections

i) Class 1 (Plastic) c/s
ii) Class 2 (compact) c/s
iii) Class 3 (semi compact) c/s
iv) Class 4 (slender) c/s

Refer IS 800 cl: 3.7.2, 3.7.4 Pg: 17

Bending strength of a laterally supported beam Pg: 52, 53, 54

If $\frac{d}{t_w} \leq 67\varepsilon$ $\left\{\varepsilon = \left(\frac{250}{f_y}\right)^{1/2}\right\}$; IS 800 considers 2 cases

i) $V \leq 0.6 V_d$
ii) $V > 0.6 V_d$

V = factored applied shear
V_d = design shear st^n

i) If $V \leq 0.6 V_d$; $M_d = \beta_b Z_p f_y \frac{1}{\gamma_{mo}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$ for S.S beam

$\leq 1.5 Z_e \frac{f_y}{\gamma_{mo}}$ for cantilever

$\beta_b = \frac{Z_e}{Z_p}$ for semi-compact sections

$= 1$ for plastic & compact "

ii) If $V > 0.6 V_d$; $M_d = M_{dv}$

a) for plastic or compact section $M_{dv} = M_d - \beta\left(M_d - M_{fd}\right) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$

$\beta = \left(\frac{2V}{V_d} - 1\right)^2$

M_d = plastic design moment of whole section
V = factored applied SF
V_d = design shear st^n
M_{fd} = plastic design st^n of area of c/s excluding shear area, considering partial safety factor γ_{mo}

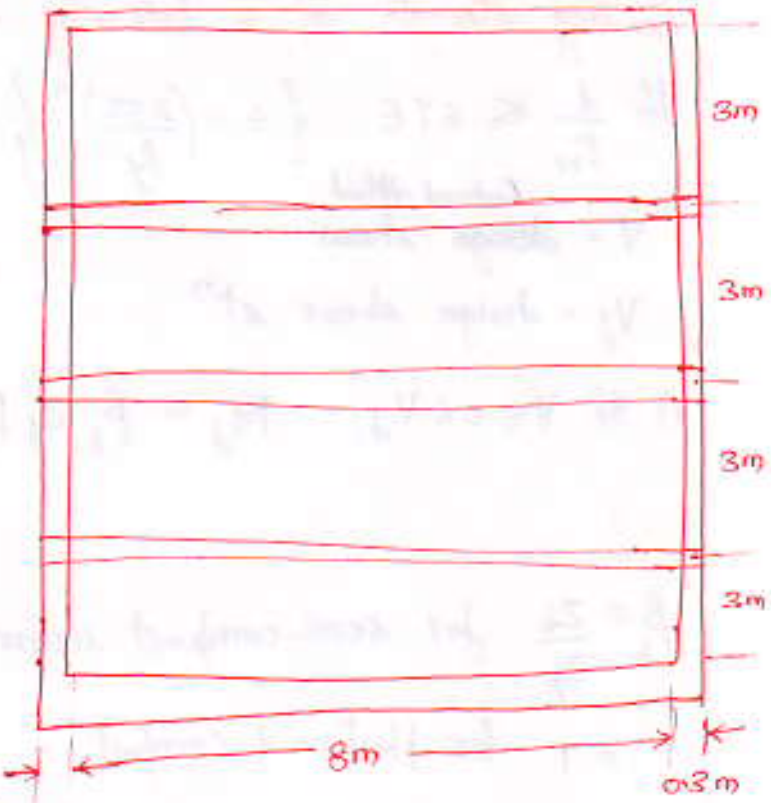
b) Semi-compact section

$$M_{dv} = \frac{Z_e f_y}{\gamma_{mo}}$$

Shear str of laterally supported beam

Refer clause 8.4 of IS 800; $V_d = \frac{A_v f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{mo}}$

Prob: A roof a hall measuring 8m x 12m consists of 100 mm thick R.C slab supported on steel I-beams spaced 3m apart as shown in fig. F.F is 1.5 KN/m² & L.L = 1.5 KN/m². Design steel beam.



Soln Each beam has a clear span of 8m & 3m width of slab

Load per mt length of beam;

Wt of R.C slab = 0.1 x 1 x 3 x 25
= 7.5 KN/m

F.F load = 1.5 x 3 = 4.5 KN/m

Self wt (assumed) = 0.8 KN/m

Total DL = 12.8 KN/m

L.L = 1 x 3 x 1.5 = 4.5 KN/m

Factored DL = 1.5 x 12.8 = 19.2 KN/m

" LL = 1.5 x 4.5 = 6.75 KN/m

Total factored load = 25.95 KN/m

Eff span of S.S.B = d/c distance of supports.

Assuming width of support = 0.3 m

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

Design moment $M = \frac{wl^2}{8} = \frac{25.95 \times 8.3^2}{8} = 223.46$ KN.m

Design SF $V = \frac{wl}{2} = \frac{25.95 \times 8.3}{2} = 107.61$ KN

Section Mod req^d $Z_p = \frac{M}{f_y} \times \gamma_{mo} = \frac{223.46 \times 10^6}{250} \times 1.1 = 983224$ mm^3

Try ISMB 400 with $Z_p = 1175.2 \times 10^3$ mm^3.

$Z_p = \frac{A}{2}(y_1 + y_2)$

$y_1 = \frac{400 \times (140 \times 16)(192) + (184 \times 8.9 \times 92)}{(140 \times 16) + (184 \times 8.9)} = 149.76$

$Z_p = \frac{7850}{2}(149.76 + 149.76) = 1175015.9$ mm^3

fy: 140 of IS 800

Properties of section are :

Depth of section $h = 400$ mm

Width of flange $b = 140$ mm

Sectional area $A = 7846$ mm^2

Thickness of flange $t_f = 16.0$ mm

" " web $t_w = 8.9$ mm

M.I about z-z axis

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

Elastic section modulus

$Z_e = 1020 \times 10^3$ mm^4

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

$= 400 - 2(16 + 14) = 340$ mm

Section classification

$\epsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$; $b/t_f = \frac{140}{16} = 8.75 < 9.4\epsilon$ Pg: 17

$\frac{d}{t_w} = \frac{340}{8.9} = 38.2 < 84\epsilon$. Hence section is classified as plastic section. Weight of section = 0.604 KN/m; Assumed wt = 0.8 KN/m Difference is not much. So design is continued with moments & shears calculated

Check for shear stn

Design shear $V = 107.61$ KN.

Design shear stn of section Pg: 59

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times$ shear area. ($h t_w$)

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9$

$= 467128$ N > 107.61 KN

So section is adequate.

$0.6 V_d = 0.6 \times 467.128 = 280.277$ KN.

Check for design moment capacity.

Pg: 52, 53, 54

$\frac{d}{t_w} = 38.2$ which is less than 67ϵ since $\epsilon = 1$ & $V < 0.6 V_d$

So $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$ $\beta_b = 1.0$ since it is plastic section.

$\therefore M_d = 1.0 \times 1175.2 \times 10^3 \times \frac{250}{1.1} = 267.091 \times 10^6$ N·mm > 223.46 kN·m.

So adequate

Check for deflection

Total working load = $12.8 + 4.5 = 17.3$ kN/m $= 17.3$ N/mm

Max^m defl^n $\delta = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4} = 26.127$ mm

Permissible defl^n for a beam in building $= \frac{l_e}{300} = \frac{8300}{300} = 27.67$ mm > 26.12 mm

$\therefore$ Provide ISMB 400

Prob Design a SSB for eff span 1.5 m carrying factored concentrated load of 360 kN @ mid span.

Sol^n Max^m moment occurs @ mid span & is given by;

$M = \frac{WL}{4} = \frac{360 \times 1.5}{4} = 135$ kN·m

Z_p $M = f_y \frac{Z_p}{\gamma_{mo}} \Rightarrow Z_p = \frac{135 \times 10^6}{250} \times 1.1 = 594 \times 10^3$ mm^3

Select trial section as ISMB 300 with $Z_p = 651.7 \times 10^3$ mm^3

Sectional properties of ISMB 300; $h = 300$ mm; $b = 140$ mm $t_f = 12.4$ mm

depth of web $d = h - 2(t_f + r_1) = 300 - 2(12.4 + 14) = 247.2$ mm

$t_w = 7.5$ mm

$I_{zz} = 8603 \times 10^4$ mm^4

$Z_e = 573.6 \times 10^3$ mm^3

$Z_p = 651.7 \times 10^3$ mm^3

Self wt of beam $= 0.452$ kN/m

Factored weight $= 1.5 \times 0.452$

Additional factored moment due to self wt $= (1.5 \times 0.452)\frac{1.5^2}{8} = 0.19$ kNm

So total factored moment $M = 135 + 0.19 = 135.19$ kN·m

Factored SF due to self wt $= 1.5 \times 0.452 \times \frac{1.5}{2} = 0.508$ kN

Total factored SF on section $= \underset{\text{given load}}{\nearrow} \frac{360}{2} + 0.508 = 180.51$ kN

Section Classification

$\epsilon = \sqrt{\frac{250}{f_y}} = 1$ $\frac{b}{t_f} = \frac{140}{12.4} = 11.29 < 15.7\epsilon$; $\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84\epsilon$

So it is classified as semi-compact section.

Shear capacity of section

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5 = 295.235 \times 10^3$ N

Section is adequate to resist shear. $\therefore 0.6V_d = 0.6 \times 295.235 = 177.145$ KN

$V > 0.6V_d$

Moment capacity of section

Since $V > 0.6V_d$ & section belongs to class 3 (semi compact)

$M_d = M_{dv} = \frac{Z_e f_y}{\gamma_{mo}} = \frac{573.6 \times 10^3 \times 250}{1.1} = 130.36 \times 10^6$ N·mm $< M$

So section is not safe. Revise the section. Try ISMB 350.

Sectional properties of ISMB 350

$h = 350$ mm; $b = 140$ mm $t_f = 14.2$ mm; $t_w = 8.1$ mm; $d = 350 - 2(14.2+14) = 293.6$ mm

$Z_e = 778.9 \times 10^3$ mm^3 $Z_p = 889.6 \times 10^3$ mm^3 $I_{zz} = 13630.3 \times 10^4$ mm^4

Section classification

$\epsilon = 1$; $\frac{b}{t_f} = \frac{140}{14.2} = 9.86$ bet^n 9.4ϵ & 10.4ϵ; $\frac{d}{t_w} = \frac{293.6}{8.1} = 36.24$ bet^n 33.5ϵ and 42ϵ

$\therefore$ It belongs to class-3 (semi compact) section.

Shear capacity of section

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 350 \times 8.1 = 371.99 \times 10^3$ N > 180.508 KN

Section is adequate to resist shear. $0.6V_d = 0.6 \times 371.99 = 223.198$ KN

Moment capacity of section

Since $\frac{d}{t_w} < 67\epsilon$ and $V < 0.6V_d$

$\therefore M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{mo}} = \frac{778.9 \times 10^3}{889.6 \times 10^3} \times 889.6 \times 10^3 \times 250 \times \frac{1}{1.1} = 194.725 \times 10^6$ N·mm $> M$ (=135.19)

$\beta_b = \frac{Z_e}{Z_p}$ $\therefore$ section is adequate to resist moment.

Max^m defl^n corresponding to working load $\boxed{\delta = \frac{WL^3}{48EI}} = \frac{360 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 13630.3 \times 10^4} = 0.928$ mm $< \frac{1500}{300}$ (=5mm)

So section is adequate.

So use ISMB 350 as beam.

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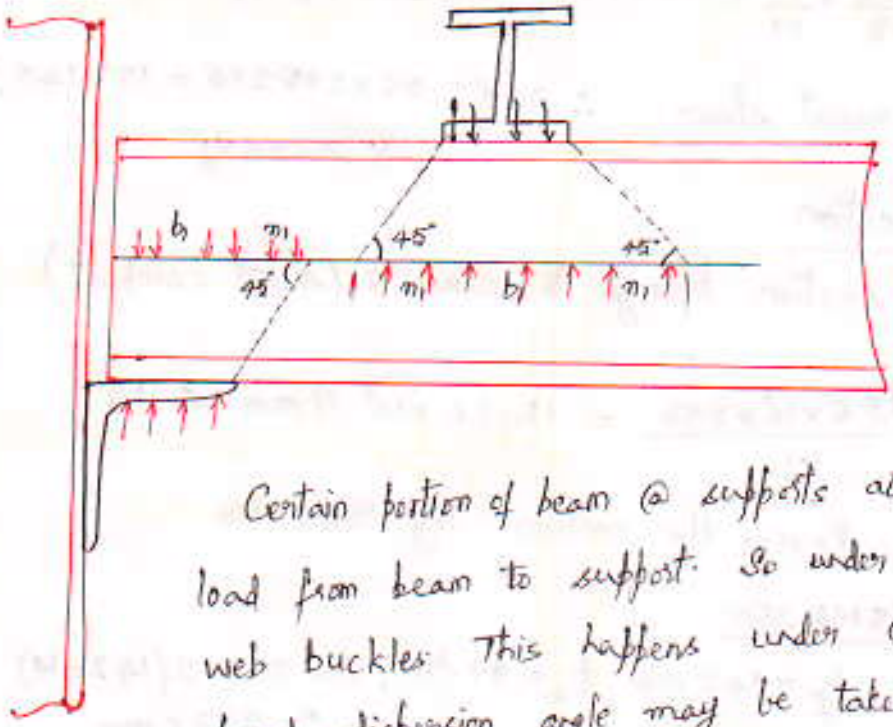
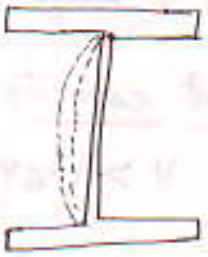
WEB BUCKLING STRENGTH





Certain portion of beam @ supports acts as column to transfer load from beam to support. So under this compressive force the web buckles. This happens under conc load on beam also. Load dispersion angle may be taken as 45° Hence there is need to check for web buckling.

As per IS-800; eff. web buckling st^n is found based on c/s on web $= (b_1 + n_1)\, t_w$

b_1 = width of stiff bearing on flange.

$n_1 = \frac{1}{2}h$; h is depth of section

$F_{cdw} = (b_1 + n_1)\, t_w f_c$

F_{cdw} = Web buckling st^n

f_c = allowable compr stress corresponding to assumed web column.

Eff. length $= 0.7d$

$r_y = \sqrt{\frac{I_y}{A}}$ of web $= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t_w^3}{(b_1+n_1)t_w}} = \frac{t_w}{2\sqrt{3}}$

$\lambda = \frac{LE}{r_y} = 0.7d \times \frac{2\sqrt{3}}{t_w} = 2.5\,\frac{d}{t_w}$

WEB CRIPPLING

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

In design; $F_w >$ load transferred by bearing.

b_1 = stiff bearing length

f_{yw} = yield stress of web

n_2: length obtained by dispersion thr the flange to web junction @ a slope 1:2.5 to plane of flange.

Prob Check the section of previous prob for web buckling & web crippling if stiff bearing is over a length $b_1 = 75$ mm.

Soln: Section selected ISMB 400

End reaction = End shear = 107.61 kN

Stiff bearings at ends = 75 mm

$t_w = 8.9$ mm $\quad t_f = 16.0$ mm

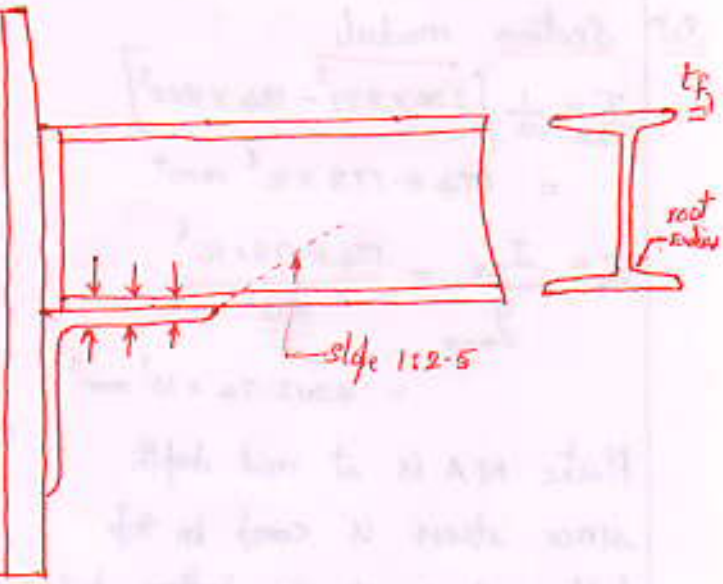
Radius at root = 14.0 mm

Depth of section $h = 400$ mm

$\therefore$ Depth of web $= 400 - 2(t_f + r_1)$

$= 400 - 2(16+14)$

$= 340$ mm



Check for web buckling:

$$\lambda = 2.5\frac{d}{t_w} = \frac{2.5 \times 340}{8.9} = 95.5$$

$$f_c = 121 - \frac{5.5}{10}(121-107) = 113.3 \text{ N/mm}^2$$

$$n_1 = \frac{400}{2} = 200 \text{ mm}$$

Web buckling resistance of section $\boxed{f_{cdw} = (b_1 + n_1)\, t_w \cdot f_c}$

$= (75+200)\, 8.9 \times 113.3$

$= 277.302 \times 10^3 \text{ N} > 107.61 \text{ kN}$

So safe against web buckling

Check for web crippling:

Flange thickness $t_f = 16.0$ mm

Radius @ root = 14.0 mm

$n_2 = 2.5 * (16+14) = 75$ mm

Strength of web against web crippling $\boxed{f_w = (b_1 + n_2)\, t_w \cdot f_{yw} \cdot \frac{1}{\gamma_{mo}}}$

$f_w = (75+75)\, 8.9 \times 250 \times \frac{1}{1.1} = 303.41 \times 10^3 \text{ N} >$ load transferred by bearing ($= 107.61$ kN)

So safe.

Prob Determine the udl carrying capacity of welded Plate girder shown in fig. when it is used as a cantilever beam of 4 m eff. span and check it for shear, defln, web crippling & buckling. Assume sftf bearing length as 100 mm

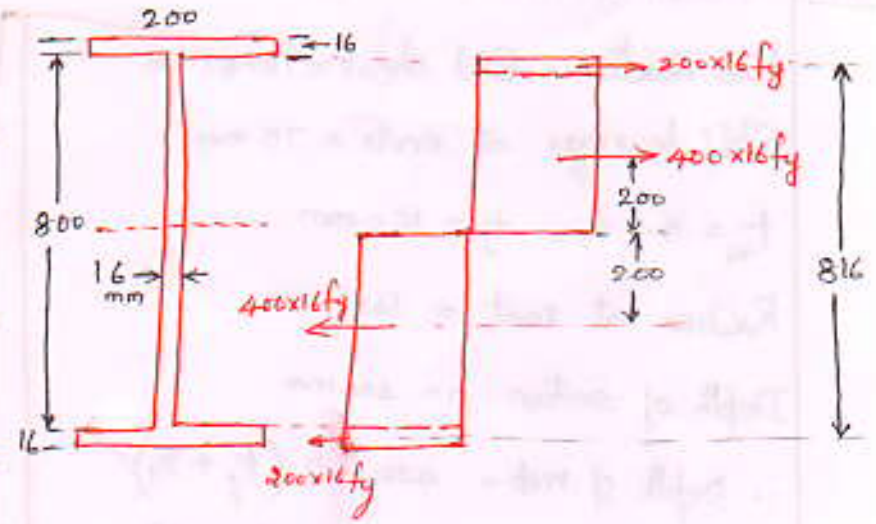
Soln Section moduli

$$I_{zz} = \frac{1}{12}\left[200 \times 832^3 - 184 \times 800^3\right]$$

$$= 1748.173 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{I_{zz}}{y_{max}} = \frac{1748.173 \times 10^6}{\frac{832}{2}}$$

$$= 4202.34 \times 10^3 \text{ mm}^3$$



Plastic NA is at mid depth since stress is comp in top half and tensile in bottom half.

$$M_p = (200 \times 16 \times 816 + 400 \times 16 \times 400) f_y \Rightarrow Z_p = \frac{M_p}{f_y} = 5171.199 \times 10^3 \text{ mm}^3$$

Section classification

$\epsilon = 1$; $\frac{b}{t_f} = \frac{200}{16} = 12.5$ (bet^n $9.4t$ & $13.6t$)

$\frac{d}{t_w} = \frac{800}{16} = 50 \quad < 84\epsilon$

It belongs to semicompact class of section

Trial section:

Assuming $V < 0.6 V_d$; $\boxed{M_d = \beta_b Z_p f_y \cdot \frac{1}{\gamma_{mo}} = \frac{Z_e}{Z_p} \cdot Z_p \cdot f_y \frac{1}{\gamma_{mo}} = Z_e \cdot f_y \cdot \frac{1}{\gamma_{mo}}}$

$$M_d = \frac{4202.338 \times 250}{1.1} = 955.0768 \times 10^6 \text{ N·mm} = 955.076 \text{ KN-m}$$

Let factored udl be w kN/m length Then $\boxed{M = \frac{WL^2}{2}} = \frac{W \times 4^2}{2} = 8W$

Equating to M_d ; $8W = 955.076 \Rightarrow W = 119.385$ kN/m

Shear $\boxed{V = WL} = 119.385 \times 4 = 477.538$ kN

Check for shear $\boxed{V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t}$ $= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 832 \times 16$

$= 1746.747 \times 10^3$ N $> V$ (477.54 kN)

$= 0.6 V_d = 1286.523$ kN $> V$

So calculated M_d is correct.

Check for defl^n

For cantilever beam $\delta = \frac{WL^4}{8EI_{zz}}$; W = Working load = $\frac{119.38 \times 10^3}{1.5 \times 1000} = 79.59$ N/mm

$$\delta = \frac{79.59 \times (4000)^4}{8 \times 2 \times 10^5 \times 1748.173 \times 10^6} = 7.28 \text{ mm} < \frac{4000}{300}$$ So safe.

Check for web buckling

Slen ratio $\lambda = 2.5 \frac{h}{t_w} = 2.5 \times \frac{816}{16} = 127.5$

From table-9.C of IS 800

$n_1 = \frac{832}{2} = 416$ mm

$f_c = 83.7 - \frac{7.5}{10}(83.7 - 74.3) = 76.65$ N/mm^2

$\therefore F_{cdw} = (b_1 + n_1) t_w f_c = (100 + 416) \times 16 \times 76.65 = 632.82 \times 10^3$ N $> V$ (OK)

check for web crippling

$F_w = (b_1 + 2.5 t_f) f_y \frac{1}{\gamma_{m0}} \cdot t_w = (100 + 2.5 \times 16) \times 250 \times \frac{1}{1.1} \times 16 = 509.09 \times 10^3$ N $> V$.

So we conclude that load carrying capacity (factored) is 119.38 KN/m.

Prob Design a simply supp beam of 10m eff. span carrying a total factored load of 60 KN/m. The depth of beam should not exceed 500 mm. The compression flange of beam is laterally supported by floor constn. Assume stiff end bearing is 75mm.

Soln $L = 10 \text{ m} = 10 \times 10^3$ mm; $W = 60$ KN/m

Trial section

Max^m BM $M = \frac{WL^2}{8} = \frac{60 \times 10^2}{8} = 750$ KNm

Z_p req^d $= \frac{M \cdot \gamma_{m0}}{f_y} = \frac{250 \times 10^6 \times 1.1}{250} = 3300 \times 10^3$ mm^3

Since depth is restricted to 500 mm; select ISMB 450 and suitable plates over flanges.

Z_p of ISMB 450 = 1553.4×10^3 mm^3

Z_p to be provided by cover plates = $(3300 - 1553.4) 10^3 = 1746.6 \times 10^3$ mm^3

If A_f is area of cover plate on each side, tensile force and comp force developed at the time of hinge formation = $A_f f_y$

If the distance betⁿ two plates is 'd'; plastic moment resisted = $A_f f_y d$

Z_p of plates = $\frac{A_f f_y d}{f_y} \times \frac{1}{\gamma_{mo}} = \boxed{\frac{A_f \cdot d}{1.1}}$ ($\frac{M}{f_y}\frac{1}{\gamma_{mo}}$)

$\therefore \frac{A_f \cdot d}{1.1} = 1746.6 \times 10^3$ Taking $d = 450 + t \simeq 450$ mm

$A_f = \frac{1.1 \times 1746.6 \times 10^3}{450} = 4269.5\ mm^2$

Provide 220×20 mm plates on either sides. +400

Check for shear

$\boxed{V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w} = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4 = 555.04 \times 10^3$ N

$0.6 V_d = 333.026$ KN $\Rightarrow$; $V = \frac{60 \times 10}{2} = 300$ KN

$> V$

So safe in shear

Section classification

Internal element of comp^{n} flange;

$\frac{b}{t_f} = \frac{150}{17.4} = 8.6 < 29.3\,\epsilon$

$\frac{d}{t_w} = \frac{450 - 2(17.4 + 15)}{9.4} = 40.9 < 84\,\epsilon$ Hence plastic section.

$\therefore M_d = 1.0 \frac{Z_p f_y}{\gamma_{mo}} = (Z_p \text{ of I section} + Z_p \text{ of plates}) \frac{f_y}{\gamma_{mo}}$

$= \left[1553.4 \times 10^3 + 220 \times 20 \times (450 + 20)\right] \frac{250}{1.1} = 823.045 \times 10^6$ N mm

$> M$ (OK)

Check for defl^{n}

Working load = $\frac{60}{1.5}$ KN/m = 40 N/mm

$\boxed{\delta = \frac{5 w l^4}{384 E I_{22}}} = \frac{5 \times 40 (10000)^4}{384 \times 2 \times 10^5 \times I_{22}}$ where I_{22} = I_{22} of ISMB 400 + I_{22} of due to plates.

$= 30390.8 \times 10^4 + 2 \times A_f \left(\frac{d}{2}\right)^2$

$= 30390.8 \times 10^4 + 2 \times 20 \times 220 \left(225+10\right)^2 = 789.88 \times 10^6 \text{ mm}^4$

$\delta = \dfrac{5 \times 40 \times (10000)^4}{384 \times 2 \times 10^5 \times 789.88 \times 10^6} = 32.97 \text{ mm}$

δ permissible, if elastic cladding is assumed.

From table-6 of IS-800; $\frac{L}{240} = \frac{10,000}{240} = 41.67 \text{ mm}$ So safe.

Check for web buckling.

$h = 450 \text{ mm}$ $\therefore \lambda = 2.5 \dfrac{h}{t_w} = 2.5 \times \dfrac{450}{9.4} = 119.68$

From table 9.c of IS 800; $f_{cd} = 94.6 - \frac{9.68}{10}(94.6 - 83.7) = 84.13 \text{ N/mm}^2$

$\therefore F_{cdw} = (b_1 + n_1) t_w f_{cd} = \left(75 + \frac{450}{2}\right) 9.4 \times 84.13 = 237.241 \times 10^3 \text{ N} < 300 \text{ KN}$

So needs increased effective bearing length b. Provide $b = 175 \text{ mm}$.

Then $F_{cdw} = \left(175 + \frac{450}{2}\right) 9.4 \times 84.13 = 316.94 \times 10^3 \text{ N} > 300 \text{ kN}$. So safe.

Check for web crippling.

$F_w = (b_1 + 2.5 t_f) f_y \cdot \dfrac{1}{\gamma_{mo}} \cdot t_w = \left(175 + 2.5(17.4+20)\right) 250 \times \frac{1}{1.1} \times 9.4$

$= 573.614 \times 10^3 \text{ N} > 300 \text{ KN}$. So safe.

Design of connection bet^n flange plates and flange.

Bolts/welds joining the plates and flange are to be designed for horizontal shear at that level.

Shear stress @ the level of plates & flanges $= \dfrac{F}{b I_{zz}} (a \bar{y})$

$= \dfrac{300 \times 10^3}{150 \times 789.884 \times 10^6} (220 \times 20 \times 225) = 2.506 \text{ N/mm}^2$

If bearing type bolts are used; f^n in single shear $= \dfrac{f_u}{\sqrt{3}} \times 0.78 \times \dfrac{\pi}{4} d^2 \times \dfrac{1}{1.25}$

Using 16 mm bolts; $= \dfrac{400}{\sqrt{3}} \times 0.78 \times \dfrac{\pi}{4}(16)^2 \times \dfrac{1}{1.25} = 28974 \text{ N}$.

Strength in bearing is more if min^m specified edge distances are provided.

There are two bolts in a pitch distance. (one on either side of web)

So SF per pitch length = sh. st^n of 2 bolts i.e $p \times (150 \times 2.506) = 2 \times 28974$

$p = 154.16 \text{ mm}$

Provide 20mm bolts @ 150 mm c/c

Prob: An ISMB 500 section is used as a beam over a span of 6 m, with simply supported ends. Determine the max^m factored udl that the beam can carry if the ends are restrained against torsion but compression flange is laterally unsupported.

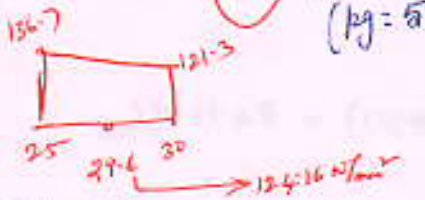
Soln For ISMB 500, $h = 500$ mm; width of flange $b = 180$ mm; $t_f = 17.2$ mm; $t_w = 10.2$ mm

$r_{yy} = 35.2$ mm

Eff. length for torsional buckling = 6 m

$$\therefore \frac{KL}{r} = \frac{6 \times 10^3}{35.2} = 170.45, \qquad \frac{h}{t_f} = \frac{500}{17.2} = 29.06$$

From table 14 of IS-800; (pg: 57)



$\frac{h}{t}$ →	25	29.6	30
$\frac{KL}{r}$ ↓			
170	136.7	X	121.3
170.45	....	O	....
180	127.1	Y	112.2

To obtain the value for $\frac{h}{t_f} = 29.6$ & $\frac{KL}{r} = 170.45$, double linear interpolation is req^d.

Find values of X and Y corresponding to $\frac{h}{t} = 29.6$

For $\frac{KL}{r} = 170$; $\frac{h}{t_f} = 29.6$; $f_{cb} = 136.7 - \frac{4.6}{5}(136.7 - 121.3) = 124.16$ N/mm^2

For $\frac{KL}{r} = 180$; $\frac{h}{t_f} = 29.6$; $f_{cb} = 127.1 - \frac{4.6}{5}(127.1 - 112.2) = 114.57$ N/mm^2

$\therefore$ the value of f_{cb} @ $\frac{h}{t_f} = 29.6$ and $\frac{KL}{r} = 170.45$

To get the value @ O; $f_{cb} = 124.16 - \frac{0.45}{10}(124.16 - 114.57) = 123.5$ N/mm^2

From table 13-a in IS-800; $f_{bd} = 77.3$ N/mm^2 for $f_{cb} = 100$

$f_{bd} = 106.8$ N/mm^2 for $f_{cb} = 150$

so for $f_{cb} = 123.5$ N/mm^2; $f_{bd} = 91.165$ N/mm^2

Section classification

$$\epsilon = \sqrt{\frac{250}{250}} = 1; \qquad \frac{b}{t_f} = \frac{180}{17.2} = 10.46 < 10.5\epsilon$$

$$\therefore \frac{d_1}{t_w} = \frac{431.6}{10.2} \leq 105t$$

$$d_1 = h - 2(t_f + R) = 500 - 2(17.2 + 17) = 431.6 \text{ mm};$$

So it belongs to class-2 (compact) category.

$\therefore M_d = \beta_b \cdot Z_p \cdot f_{bd}$; $\beta_b = 1$; $Z_p = 2074.7 \times 10^3\ mm^3$ $f_{bd} = 91.165\ N/mm^2$

$\therefore M_d = 1 \times 2074.7 \times 10^3 \times 91.165 = 189.14 \times 10^6\ N.mm$

If udl 'w' in KN/m, then $\frac{wL^2}{8} = M_d \Rightarrow \frac{w \times 6^2}{8} = 189.14 \Rightarrow w = 42.03\ KN/m$

Self wt = $86.9 \times 9.81 = 852\ N/m$ = $0.852\ KN/m$

$\therefore$ Factored self wt = $1.5 \times 0.852 = 1.278\ KN/m$

Superimposed load that beam can carry = $42.03 - 1.278 = 40.752\ KN/m$

Prob: A SS steel joist of 4 m eff. span is laterally supported thr out. It carries a total udl of 40 kN, inclusive of self wt. Design an appropriate section using steel of grade Fe410.

Sol: For Fe410 grade; $f_y = 250\ N/mm^2$; $\gamma_{mo} = 1.1$; $\gamma_f = 1.5$ (par. saf. fac for load)

Total ud service load = 40 kN

Total factored load = $1.5 \times 40 = 60$ kN

Max^m BM $M = \frac{(wl)l}{8} = \frac{60 \times 4}{8} = 30$ kN m

Max^m SF $V = \frac{wl}{2} = \frac{60}{2} = 30$ kN

Plastic section Mod req^d $Z_p = \frac{M \cdot \gamma_{mo}}{f_y} = \frac{30 \times 10^6 \times 1.1}{250} = 132 \times 10^3\ mm^3$

Let us try ISLB 200 @ 194.2 N/m

$h = 200$ mm; $b_f = 100$ mm; $t_f = 7.3$ mm $t_w = 5.4$ mm

$R_1 = 9.5$ mm

Depth of web $d = h - 2(t_f + R_1)$

$= 200 - 2(7.3 + 9.5) = 166.4$ mm

M.I $I_z = 1696.6 \times 10^4\ mm^4$

$Z_{p_z} = 184.34 \times 10^3\ mm^3$ $Z_{ez} = 169.7 \times 10^3\ mm^3$

Section Classification

$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

Outstand of flange $b = \frac{b_f}{2} = \frac{100}{2} = 50$ mm

$\frac{b}{t_f} = \frac{50}{7.3} = 6.849 < 9.4$

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

So, section is plastic.

Check for shear capacity

Design SF = Max^m SF $V = 30$ KN

Design shear st^n of section $V_d = \frac{f_y}{\sqrt{3}\,\gamma_{m0}} h \cdot t_w$

$V_d = \frac{250}{\sqrt{3} \times 1.1} \times 200 \times 5.4 \times 10^{-3}$

$= 141.713$ KN > 30 KN (OK)

Check for high / low shear

$0.6 V_d = 0.6 \times 141.713 = 85.02$ KN

The case is of low shear since $V < 0.6 V_d$

Check for design bending st^n

$M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_{m0}} = 1.0 \times 184.34 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 41.895$ KNm

$\leq 1.2 Z_{ez} \frac{f_y}{\gamma_{m0}} = 1.2 \times 169.7 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 46.28$ KN·m (OK)

So design bending st^n of section $= 41.895$ KNm

Check for $defl^n$

Permissible $defl^n$ $\delta = l/300 = \frac{4 \times 10^3}{300} = 13.13$ mm

Max^m $\delta = \frac{5}{384} \frac{wl^4}{EI} = \frac{5}{384} \times \frac{40 \times 10^3 \times (4 \times 10^3)^3}{2 \times 10^5 \times 1696.6 \times 10^4} = 9.82$ mm < 13.13 mm

Prob Design a laterally supported beam of eff span 6 m, subjected to max^m BM of 150 kNm & max^m SF of 210 kN. Use Fe410.

Sol^n $f_y = 250\ N/mm^2$ $f_u = 410\ N/mm^2$ $\gamma_{mo} = 1.1$; $M = 150$ kNm $V = 210$ kN

Plastic sec. Mod $Z_{pz\ req} = \dfrac{M\gamma_{mo}}{f_y} = \dfrac{150\times10^6 \times 1.1}{250} = 660\times10^3\ mm^3$

Try ISLB 325 @ 422.81 N/m

$h = 325$ mm; $b_f = 165$ mm; $t_f = 9.8$ mm; $t_w = 7.0$ mm $R_1 = 16$ mm

$d = h - 2(t_f + R_1) = 325 - 2(9.8+16) = 273.4$ mm

$I_z = 9874.6\times10^4\ mm^4$ $Z_{pz} = 687.76\times10^3\ mm^3$ $Z_{ez} = 607.7\times10^3\ mm^3$

Section classification

$\epsilon = \sqrt{\dfrac{250}{f_y}} = 1$

Outstand of flange $b = \dfrac{b_f}{2} = \dfrac{165}{2} = 82.5$ mm

$\dfrac{b}{t_f} = \dfrac{82.5}{9.8} = 8.41 < 9.4$ So section is plastic.

$\dfrac{d}{t_w} = \dfrac{273.4}{7.0} = 39.05 < 84$

Check for design capacity of the section

$M_{dz} = Z_{pz}\dfrac{f_y}{\gamma_{mo}} = 126.86\times10^3 \times \dfrac{250}{1.1}\times10^{-6} = 28.83$ kNm

$\leq 1.2 Z_{ez}\dfrac{f_y}{\gamma_{mo}} = 1.2\times111.9\times10^3\times\dfrac{250}{1.1}\times10^{-6} = 30.51$ kNm (ok)

~~$M_{dz} = 28.83$ kN.m > 17.579 kNm~~ (ok)

Check for shear capacity

Design shear $V = 210$ kN

Design sh. st^n of the section $V_d = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} \cdot h\, t_w$

$= \frac{250}{\sqrt{3} \times 1.1} \times 325 \times 7 \times 10^{-3}$

$= 298.5$ kN > 210 kN (OK)

Check for high/low shear

$0.6\, V_d = 0.6 \times 298.5 = 179.1$ kN < 210 kN

This case is of high shear $\because V > 0.6 V_d$

Check for design bending st^n

Design bending st^n $M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2 Z_{ez} \frac{f_y}{\gamma_{mo}}$

$\beta = \left(2\frac{V}{V_d} - 1\right)^2 = \left(2 \times \frac{210}{298.5} - 1\right)^2 = 0.1656$

Plastic moment of whole section disregarding high SF but considering the web buckling effect $\Big\}$ $M_d = Z_{pz} \frac{f_y}{\gamma_{mo}}$

$= 687.76 \times 10^3 \times \frac{250}{1.1} \times 10^{-6}$

$= 156.30$ kN

Section modulus of flange $Z_{fd} = Z_{pz} - A_w y_w$

$= 687.76 \times 10^3 - (325 \times 7) \times \frac{325}{4}$

$= 502.916 \times 10^3$ mm^3

Plastic design strength of area of c/s excluding the shear area

$M_{fd} = Z_{fd} \cdot \frac{f_y}{\gamma_{mo}} = 502.916 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 114.29$ kN·m

$M_{dv} = 156.3 - 0.1656\,(156.3 - 114.29) = 149.34$ kNm < 150 kN·m. Hence the section is unsafe.

Let us try ISLB 350 @ 495 kN/m

$h = 350$ mm; $b_f = 165$ mm; $t_f = 11.4$ mm; $t_w = 7.4$ mm $R_1 = 16$ mm

$d = h - 2(t_f + R_1) = 295.2$ mm

$I_z = 13158.3 \times 10^4$ mm^4 $\quad Z_{pz} = 851.11 \times 10^3$ mm^3 $\quad Z_{ez} = 751.9 \times 10^3$ mm^3

Section classification

Outstand of flange element $b = \frac{bf}{2} = \frac{165}{2} = 82.5$ mm

$b/t_f = \frac{82.5}{11.4} = 7.23 < 9.4$ So section is plastic.

$d/t_w = \frac{295.2}{7.4} = 39.9 < 84$

Check for shear capacity

Design SF $V = 210$ KN

Design shear strth of section $V_d = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} h t_w$

$= \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 7.4 \times 10^{-3}$

$= 340$ KN > 210 KN. (OK)

Check for high/low shear.

$0.6 V_d = 0.6 \times 340 = 204$ KN.

$\because V > 0.6 V_d$ i.e $210 > 204$, it is a high shear case.

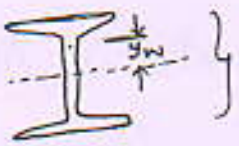
Check for design bending strength

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

$\beta = \left(2 \frac{V}{V_d} - 1\right)^2 = \left(2 \times \frac{210}{340} - 1\right)^2 = 0.554$

$M_d = Z_{pz} \frac{f_y}{\gamma_{mo}} = 851.11 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 193.43$ KN.m

$Z_{fd} = Z_{pz} - A_w y_w$ $\quad \left\{ y_w = \frac{h}{4} \right.$



$= 851.11 \times 10^3 - (350 \times 7.4) \frac{350}{4} = 624.485 \times 10^3$ mm^3

$M_{fd} = Z_{fd} \frac{f_y}{\gamma_{mo}} = 624.485 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 141.93$ KN m

$M_{dv} = 193.43 - 0.0554\,(193.43 - 141.93) = 190.57$ kNm > 150 kNm

$\leq 1.2\, Z_e \frac{f_y}{\gamma_{mo}} = 1.2 \times 751.9 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 205.06$ kNm (OK)

Check for web buckling @ support

Web buckling check is not req^d when $\frac{d}{t_w} < 67\varepsilon$. Since shear is high the check has been applied

Assume a stiff bearing length $b = 100$ mm

$A_b = B \cdot t_w = (b + n)\, t_w = (100 + 175)\,7.4$ $\qquad n = \frac{h}{2} = \frac{350}{2} = 175$ mm

$= 2035$ mm^2

Eff length of web $KL = 0.7d = 0.7 \times 295.2 = 206.64$ mm

I_{eff} of web $= \frac{bt_w^3}{12} = \frac{100 \times 7.4^3}{12} = 3376.86$ mm^4

A_{eff} of web $= bt_w = 100 \times 7.4 = 740$ mm^2

$r = \sqrt{\frac{3376.86}{740}} = 2.136$ mm

Sl. ratio $\lambda = \frac{KL}{r} = \frac{206.64}{2.136} = 96.7415$

For $\lambda = 96.7415$, $f_y = 250$ N/mm^2, buckling curve c; the design compr. stress $f_{cd} = 111.55$ N/mm^2

Capacity of section $= A_b f_{cd}$

$= 2035 \times 111.55 \times 10^{-3}$

$= 227$ kN > 210 kN (OK).

Check for web bearing:

$$F_w = (b + n_1)\, t_w \frac{f_{yw}}{\gamma_{m0}}$$

$n_1 = 2.5\,(t_f + R_1)$ (assuming dispersion @ 1:2.5)

$= 2.5\,(11.4 + 16) = 68.5$ mm

Stiff bearing length assumed; $b = 100$ mm

$$F_w = (100 + 68.5) \times 7.4 \times \frac{250}{1.1} \times 10^{-3} = 283.38 \text{ KN} > 210 \text{ KN} \quad (\text{ok})$$

Prob Design a laterally unsupported beam for the following data;
Eff span : 4 m ; Max^m BM = 550 KNm, Max^m SF = 200 kN Use Fe410 grade

Sol^n $f_y = 250$ N/mm^2;

Shear Mod $G = 76.923 \times 10^3$ N/mm^2

$\gamma_{m0} = 1.1$; $L_{LT} = KL = 4000$ mm;

Imperfection factor $\alpha_{LT} = 0.21$ (for rolled section)

$= 0.49$ (for welded ")

Plastic section modulus

$$Z_{pz\,req} = 1.3\, M \frac{\gamma_{m0}}{f_y} = 1.3 \times 550 \times 10^6 \times \frac{1.1}{250} = 3146 \times 10^3 \text{ mm}^3$$

Plas sec mod req^d is increased by 30% to account for lateral torsional buckling.

Try ISMB 600 @ 1202.71 N/m

$h = 600$ mm $\quad t_w = 12$ mm $\quad I_z = 91813 \times 10^4$ mm^4

$b_f = 210$ mm $\quad R_1 = 20$ mm $\quad I_y = 265) \times 10^4$ mm^4

$t_f = 20.8$ mm $\quad d = h - 2(t_f + R_1)$ $\quad Z_{pz} = 3510.63 \times 10^3$ mm^3

$= 518.4$ mm $\quad Z_{pez} = 3060.4 \times 10^3$ mm^3

Section classification $\epsilon = \sqrt{\frac{f_y}{250}} = \sqrt{\frac{250}{250}} = 1$

Outstand of flange $b = \frac{b_f}{2} = \frac{210}{2} = 105$ mm

$\frac{b}{t_f} = \frac{105}{20.8} = 5.04 < 9.4$

So section is plastic

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

Check for design bending strength

Lateral - torsional buckling moment $M_{cr} = \sqrt{\frac{\pi^2 E I_y}{L_{LT}^2}\left(G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right)}$

$E = 2 \times 10^5$ N/mm^2 $\quad L_{LT} = KL = 4000$ mm $\quad$ pg:54

$G = 76.923 \times 10^3$ N/mm^2 $\quad I_t = \frac{\sum b_i t_i^3}{3}$

$= \frac{2 \times 210 \times 20.8^3}{3} + \frac{(600 - 20.8)\,12^3}{3}$

$= 1.59 \times 10^6$ mm^4

$I_w = (1 - \beta_f)\,\beta_f\, I_y\, h_{fy}^2$ $\quad$ (pg:129)

$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$ $\quad (\because I_{fc} = I_{ft})$

$h_f = 600 - 20.8 = 579.2$ mm

$I_w = (1 - 0.5) \times 0.5 \times 2651 \times 10^4 \times 579.2^2 = 2.22 \times 10^{12}$ mm^4

$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 2651 \times 10^4}{4000^2} \times \left(76.923 \times 10^3 \times 1.59 \times 10^6 + \frac{\pi^2 \times 2 \times 10^5 \times 2.22 \times 10^{12}}{4000^2}\right)}$

$= 1138.3$ kNm

Non dimensional slenderness ratio

$\lambda_{LT} = \sqrt{\frac{\beta_b Z_{p2} f_y}{M_{cr}}} = \sqrt{\frac{1.0 \times 3510.63 \times 10^3 \times 250}{1138.3 \times 10^6}} = 0.878 > 0.4$

Since λ_{LT} is greater than 0.4, the effect of lateral torsional buckling has to be considered.

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right]$$ pg:54

Since the section is rolled; $\alpha_{LT} = 0.21$

$$\phi_{LT} = 0.5\left[1 + 0.21(0.878 - 0.2) + 0.878^2\right] = 0.9566$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \left(\phi_{LT}^2 + \lambda_{LT}^2\right)^{0.5}} = \frac{1}{0.9566 + \left(0.9566^2 - 0.878^2\right)} = 0.7483$$

Design bending ~~capacity~~ compressive stress

$$M_d \quad f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{mo}} = 0.7483 \times \frac{250}{1.1} = 170.06 \ N/mm^2$$

Design bending capacity;

$$M_d = \beta_b Z_{pz} f_{bd} = 1 \times 3510.63 \times 10^3 \times 170.06 \times 10^{-6} = 597 \ kNm > 550 \ kNm$$

($\beta_b = 1$; $\because$ section is plastic)

Check for shear capacity

Design SF $V = 200$ kN.

Design shear st^n of section $V_d = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} h t_w$

$$= \frac{250}{\sqrt{3} \times 1.1} \times 600 \times 12 \times 10^{-3}$$

$$= 944.75 \ kN > 200 \ kN$$

Check for web buckling

Web buckling check is not req^d in general when $\frac{d}{t_w} < 67\epsilon$

$d/t_w = 43.2 < 67 \times 1 = 67$. However let us apply the check to show that it is not req^d

Assume a rigid bearing $b = 100$ mm

C/s area of web for buckling $A_b = B \cdot t_w$

$= (b + n)\, t_w$ $\quad n = \frac{600}{2} = 300$ mm

$= (100 + 300)\, 12$

$= 4800\ mm^2$

Eff length of web $= 0.7\, d$ pg: 58

$= 0.7 \times 518.4 = 362.88$ mm

I_{eff} of web $= \frac{100 \times 12^3}{12} = 14400\ mm^2$

A_{eff} of web $= 100 \times 12 = 1200\ mm^2$

Radius of gyration $r = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{14400}{1200}} = 3.46$

Slenderness ratio $\lambda = \frac{KL}{r} = \frac{362.88}{3.46} = 104.88$

For $\lambda = 104.88$; $f_y = 250\ N/mm^2$; buckling curve c; design compr stress $f_{cd} = 100.95\ N/mm^2$

Capacity of section $= A_b f_{cd} = 4800 \times 100.95 \times 10^{-3}$

$= 484.56$ KN > 200 KN.

Check for web bearing

$F_w = (b + n_1)\, t_w \frac{f_{yw}}{\gamma_{mo}}$

$b = 100$ mm

$n_1 = 2.5\,(t_f + R_1) = 2.5\,(20.8 + 20) = 102$ mm

$F_w = (100 + 102) \times 12 \times \frac{250}{1.1} \times 10^{-3} = 550.90$ KN > 200 KN. (OK)

——— x ——— x ———

Max^m permissible stresses in rivets

Type of rivet	Axial tension σ_{tf} N/mm^2	Shear τ_{vf} N/mm^2	Bearing σ_{bf} N/mm^2
Power driven rivets	100	100	300
Hand " "	80	80	250

St^n of rivet in shear $P_s = \tau_{vf} \cdot \frac{\pi}{4} d^2$

St^n of rivet in bearing $P_b = \sigma_{bf} \cdot d \cdot t$

Rivet value = Min^m of P_s & P_b

No. of rivets = $\frac{\text{Force}}{\text{RV}}$

Prob Design a steel beam section for supporting roof of a big hall for the following data & apply usual checks. Assume Fe410 grade steel.

Compound beams

Clear span : 6.5 m

End bearings : 150 mm

C/c spacing of beams : 3m

Imposed load on beam : 10 KN/m^2

DL (inclusive of self wt) : 4 KN/m^2

Restriction on beam depth = 375 mm

The compr flange of beam is laterally supported thr out.

Soln: Factored DL = 1.5 x 4 = 6 KN/m^2

Factored imposed load = 1.5 x 10 = 15 KN/m^2

C/c spacing of beams = 3 m

So udl = 6 x 3 + 15 x 3 = 63 KN/m

Eff. span l = clear span + 2 x $\frac{\text{width of end bearing}}{2}$

= 6.5 + 2 x $\frac{0.15}{2}$ = 6650 m

Max^m BM $M = \frac{wl^2}{8} = \frac{63 \times 6.65^2}{8} = 348.25$ kNm

Max^m SF $V = \frac{wl}{2} = \frac{63 \times 665}{2} = 209.475$ kN

Plastic section modulus req^d $Z_{pz\,req} = \frac{M \gamma_{mo}}{f_y}$

$= 348.25 \times 10^6 \times \frac{1.1}{250} = 1532.3 \times 10^3$ mm^3

Since the depth of beam is restricted to 375 mm;

Try ISWB 350 @ 558.19 N/m ;

$Z_{pz} = 995.49 \times 10^3$ mm^3

Balance sec mod is provided by additional cover plates.

Sec. mod. req^d by plates $Z_a = Z_{pz\,reqd} - Z_{pz}$

$Z_a = 1532.3 \times 10^3 - 995.49 \times 10^3 = 536.81 \times 10^3$ mm^3.

Area of plate; $A_a = \frac{Z_a}{h} = \frac{536.81 \times 10^3}{350} = 1533.74$ mm^2

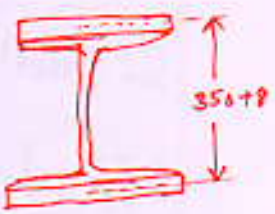
Taking width of flange cover plate equal to width of flange, thickness of flange cover plate $t_{fp} = \frac{A_a}{b_f}$

$t_{fp} = \frac{1533.74}{200} = 7.668 \not< 6$ mm

Provide 8 mm thick cover plate on each flange

$Z_{p\,provi} = 995.49 \times 10^3 + \left(\frac{200}{2} \times 8\right) \times (350 + 8)$ $\left(\frac{A}{2}(y_1 + y_2)\right)$

$= 1568.29 \times 10^3$ mm^3



Total depth of beam section $= 350 + 2(8) = 366$ mm

< 375 mm (OK)

Properties of ISHB 350 @ 558.19 N/m.

$h = 350$ mm

$b_f = 200$ mm

$t_f = 11.4$ mm

$t_w = 8.0$ mm

$R_1 = 12$ mm

$d = h - 2(t_f + R_1)$

$= 303.2$ mm

$I_z = 15521.7 \times 10^4$ mm^4

$Z_{pz} = 995.49 \times 10^3$ mm^3

$Z_{ez} = 887 \times 10^3$ mm^3

Section classification

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

Outstand of flange $b = \frac{b_f}{2} = \frac{200}{2} = 100$ mm

$\frac{b}{t_f} = \frac{100}{11.4} = 8.77 < 9.4$ $\therefore$ section is plastic.

$\frac{d}{t_w} = \frac{303.2}{8.0} = 37.9 < 84$

$\frac{d}{t_w} < 67\epsilon$; so shear buckling check of web is not req^d.

Check for shear capacity

Design SF $V = 209.475$ kN

Design sh. st^n of section $V_d = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} \cdot h \cdot t_w$

$= \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 8 \times 10^{-3}$

$= 367.40$ kN $>$ 209.475 kN (Ok)

Check for high/low shear

$0.6\,V_d = 0.6 \times 367.4 = 220.44$ kN $>$ 209.475 kN

It is a low shear case $\because V < 0.6\,V_d$.

Check for design bending strength :

($\beta_b = 1$; ∵ it is plastic section)

$$M_d = \beta_b \cdot Z_{p2} \frac{f_y}{\gamma_{mo}} = 1.0 \times 1568.29 \times 10^3 \times \frac{250}{1.1} \times 10^{-6}$$

$$= 356.43 \text{ kNm} > 348.25 \text{ kNm}$$

M_d should also be $\leq 1.2 \frac{Z_e}{\gamma_{mo}}$

$$I_{prov} = 15521.7 \times 10^4 + 2\left[\frac{200 \times 8^3}{12} + (200 \times 8)\left(\frac{350+8}{2}\right)^2\right] \quad *$$

$$= 25776.52 \times 10^4 \text{ mm}^4$$

$$Z_e = \frac{25776.52 \times 10^4}{(175+8)} = 1408.55 \times 10^3 \text{ mm}^3$$

$$356.43 \leq 1.2 \times 1408.55 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 384.15 \text{ kNm} \quad \text{(ok)}$$

Check for defl^n

Service udl $= \frac{63}{\gamma_f} = \frac{63}{1.5} = 42 \text{ kN/m}$

& Permissible defl^n $\delta = \frac{L}{300} = \frac{6.5 \times 10^3}{300} = 21.66 \text{ mm}$

$$I_z = 23126.58 \times 10^4 \text{ mm}^4$$

Max^m defl^n $\delta_{max} = \frac{5}{384} \frac{wl^4}{EI} = \frac{5}{384} \times \frac{42 \times 6500^4}{2 \times 10^5 \times 23126.58 \times 10^4} = 21.10 \text{ mm}$

$< 21.66 \text{ mm}$

Check for web bearing.

Assume stiff bearing of 100 mm.

Bearing st^n $F_w = A_e \frac{f_{yw}}{\gamma_{mo}} = (b + n_1) t_w \cdot \frac{f_{yw}}{\gamma_{mo}}$

$n_1 = 2.5(t_f + R_1) = 2.5(11.4 + 12) = 58.5 \text{ mm}$

$$F_w = (100 + 58.5) \times 8 \times \frac{250}{1.1} \times 10^{-3} = 288.18 \text{ kN} > 209.475 \text{ kN}$$

⟵ × ⟵ × ⟶

DS-16

Prob: Design a built-up column with 4 angles. The column is 12 m long & supports a factored axial comp^{r} load of 700kN. The ends of the column are held in position and restrained against rotation. Design a suitable connecting system. Use Fe410

Soln: Assume design comp^{r} stress for buckling class c; $f_{cd} = 168\ N/mm^2$ (pg: 42)

C/s area req^{d} $= \frac{700 \times 10^3}{168} = 4166.7\ mm^2$

Try 4 no. of ISA 90x90x6

Properties of 1-ISA 90x90x6; $A = 1047\ mm^2$; $C_{xx} = C_{yy} = 24.2\ mm$;

$r_z = r_y = 27.7\ mm$; $I_z = I_y = 80.1 \times 10^4\ mm^4$

Area provided $= 4 \times 1047 = 4188\ mm^2$

Spacing of angles

~~For desig~~ For $f_{cd} = 168\ N/mm^2$; $f_y = 250\ N/mm^2$; Buckling curve: c; eff. sl. ratio $\lambda = 60$.

For laced columns; eff. sl. ratio $= 1.05 \times 60 = 63$

$l = KL = 0.65 \times 12 \times 10^3 = 7800\ mm$

$r = \frac{l}{\lambda} = \frac{7800}{63} = 123.8\ mm$

M.I of section req^{d} $I = Ar^2$

$= 4188 \times 123.8^2 = 6418.7 \times 10^4\ mm^4$

M.I req^{d} = M.I provided

$6418.7 \times 10^4 = 4 \times 80.1 \times 10^4 + 4188\ \bar{y}^2 \Rightarrow \bar{y} = 120.67\ mm$

Spacing of angles $S = 2(120.67 + 24.2)$

$= 289.74 \simeq 290\ mm$.

Design compr st^n $P_d = f_{cd} A_e$

$= 168 \times 4188$

$= 703.5 \text{ KN} > 700 \text{ KN}$ (OK)

Connecting system

Provide double lacing system with lacing flats inclined @ 45°. Bolts are provided @ centre of leg of the angle.

Spacing of lacing bars $a_1 = (290 - 45 - 45) \cot 45°$

$= 200 \text{ mm}$

$\frac{a_1}{r_y} = \frac{200}{27.7} = 7.22 < 50$

Also it should be $< 0.7 \times 63 (= 44.1)$

$V_t = \frac{2.5}{100} \times 700 \times 10^3 = 17.5 \times 10^3 \text{ N}$

Transverse shear in each plane $= \frac{V_t}{N}$

$= \frac{17.5 \times 10^3}{2} = 8.75 \times 10^3 \text{ N}$

As double lacing is provided;

Compr. force in lacing bar $= \frac{1}{2} \times \frac{V_t}{N} \operatorname{cosec} \theta$

$= \frac{1}{2} \times 8.75 \times 10^3 \times \operatorname{cosec} 45°$

$= 6.19 \times 10^3 \text{ N}$

Spacing of lacing flat

Let us provide 20 mm dia bolts of grade 4.6

For 20 mm dia bolts (3d)

Min^m width of flat $= 3 \times 20 = 60 \text{ mm}$

Thickness of lacing flat $= \frac{1}{60}(290-45-45)\,\text{cosec}\,45^\circ$

$= 4.714 \text{ mm} \simeq 8 \text{ mm}$

Provide 60 ISF 8 mm flat section

Min^m radius of gyration $r = \frac{t}{\sqrt{12}} = \frac{8}{\sqrt{12}} = 2.31$ mm

Slen ratio $\frac{l_1}{r} = \frac{0.7(290-45-45)\,\text{cosec}\,45^\circ}{2.31} = 85.7 < 145$ (OK)

For $\frac{l_1}{r} = 85.7$; $f_y = 250 \text{ N/mm}^2$; Buck class c; $f_{cd} = 127.45 \text{ N/mm}^2$

Design comp^r st^n $P_d = f_{cd} A_e = 127.45 \times 60 \times 8 \times 10^{-3}$

$= 61.176 \text{ KN} > 6.19 \text{ KN}$ (OK)

Tensile st^n of lacing flat is min^m of

i) $0.9(B-d_h)\,t\,\frac{f_u}{\gamma_{m1}} = 0.9(60-22) \times 8 \times \frac{410}{1.25} \times 10^{-3} = 89.74 \text{ KN}$ (rupture)

(or) ii) $\frac{A_g f_u}{\gamma_{m0}} = (60 \times 8) \times \frac{250}{1.1} \times 10^{-3} = 109.9 \text{ KN}$ (Gross section yielding)

So tensile st^n of lacing flat is 89.74 KN.

Connection

Dia of bolt hole $d_h = 20+2 = 22$ mm

St^n of 20 mm dia bolt in double shear $= 2 A_{nb} \times \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}}$

$= 2 \times 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 90.52 \text{ KN}$

St^n of 20 mm dia bolt in bearing (on 6 mm plate)

$= 2.5\, k_b\, d\, t\, f_u / \gamma_{mb}$

$= 2.5 \times 1.0 \times 20 \times 6 \times \frac{400}{1.25} \times 10^{-3} = 96.0$ kN

So s^{n} of bolt = 90.52 kN

No. of bolts $= \frac{2 \times 6.19 \times \cot 45^{\circ}}{90.52} = 0.136 \simeq 1$

Provide 1 bolt of 20 mm dia of grade 4.6 @ ends of flat.

Tie Plate

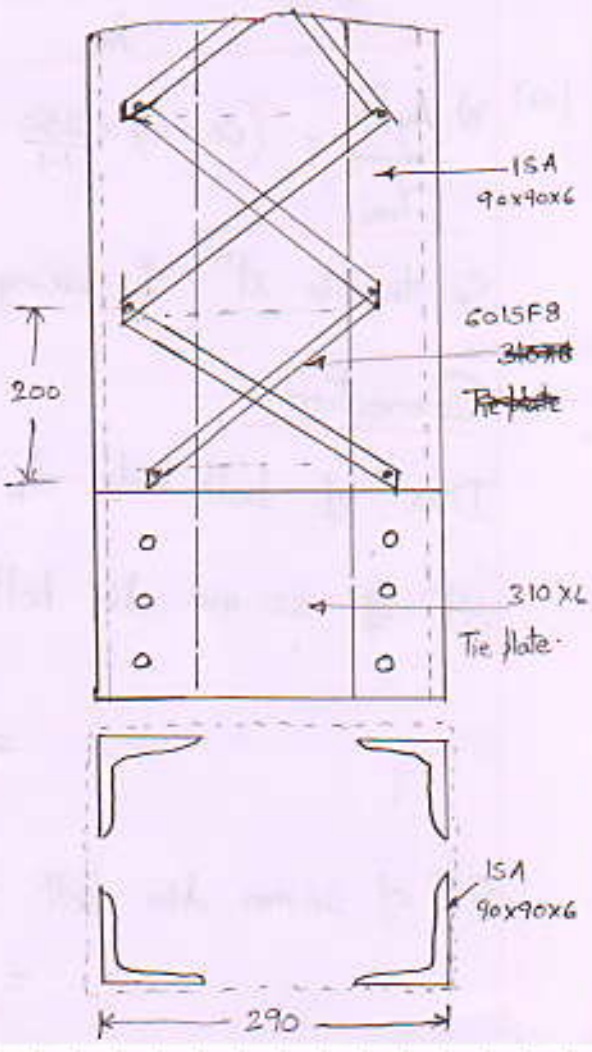
These are provided @ each end of built-up column

Eff. depth of tie plate = $290 - 2 \times 24.2 = 241.6$ mm $> 2 \times 90$ mm

Overall depth of the tie plate = $241.6 + 2 \times 1.5 \times 22 = 307.6 \simeq 310$ mm

Length of tie plate = 290 mm

Thickness of tie plate $= \frac{1}{50} \times (290 - 45 - 45)$

$= 4.00 \simeq 6$ mm

Provide $290 \times 310 \times 6$ mm tie plate & connect it with bolts as shown in fig.