Bapatla Engineering College :: Bapatla

Civil Engineering Department

Scheme of Evaluation

¾ B.Tech Regular Degree Examinations

Fifth semester, February 2021

Subject: **Design of Steel Structures**

Code: **18 CE 504**

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18CE504

Hall Ticket Number

III/IV B.Tech (Regular) Degree Examination

February-2021 | Civil Engineering

Fifth Semester | Design of Steel Structures

Time: Three Hours | Maximum: 50 Marks

Answer ONE Question from each unit. | (5 x 10 = 50 Marks)

Note: 1. Using IS 800:2007 and Steel Tables is permitted.

2. Assume any necessary suitable data if required.

UNIT-I

1. A tie member of a roof trussconsists of 2 ISA 100X75X8mm. The angles are connected to either side of a 10mm gusset plates and the member is subjected to a working pull of 300KN. Design the welded connections are made in the workshop. CO1 (10 M)

or

2. Design a lap joint between the two plates each of width 120mm, if the thickness of one plate is 16mm and the other is 12mm. The joint has to transfer a design load of 160KN. The plates are of Fe 410 grade. Use bearing type bolts. CO1 (10 M)

UNIT-II

3. Design double angle tension member connected on each side of a 10mm thick gusset plate, to carry an axial factored load of 375KN. Use 20mm black bolts. Assume shop connection. CO2 (10 M)

or

4. A column 4m long has to support a factored load of 6000KN. The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates. CO2 (10 M)

UNIT-III

5. Design a slab base for a column ISHB300@577N/m carrying an axial factored load of 1000kN. UseM20 for concrete foundation. CO3 (10 M)

or

6. A column section ISHB350@710N/m with two plates 450mmX20mm carrying a factored load of 3600KN. The column is to be supported on concrete pedestal to be built with M20 concrete. CO3 (10 M)

UNIT-IV

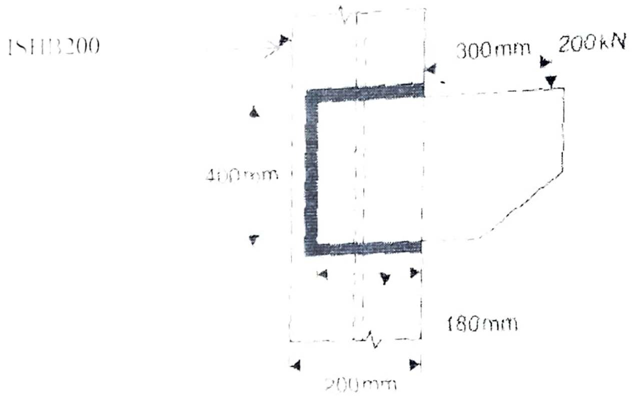
7. Design a simply supported beam of effective span 1.5m carrying a factored load of 360KN at mid span. Assume it is laterally supported beam. CO4 (10 M)

or

8. Design a laterally unsupported I beam with simply supported ends of effective span 6m subjected to a working load of 35KN/m inclusive of DL,LL &FF. CO4 (10 M)

UNIT-V

9. Design a connection for a bracket which is connected in the plane of the flange of a column by fillet weld. The width of the flange of the column is 200mm. CO5 (10 M)



or

10. Design a seat angle connection for a beam ISMB350@52.4Kg/m which transfers a factored end shear of 200kN to the flange of the column ISHB300@63kg/m.CO5 (10 M)

Q1. A tie member of a roof truss consists of 2 ISA 10075, 8 mm. The angles are connected to either side of a 10 mm gusset plates and the member is subjected to a working pull of 300 kN. Design the welded connection. Assume connections are made in the workshop.

Working Load = 300 kN

∴ factored Load = 300 × 1.5 = 450 kN

Thickness of weld :

(i) At the rounded toe of the angle section, size of weld should not exceed = 3/4 × thickness

$s = 3/4 \times 8 = 6\,mm$

(ii) At top (ref. fig (a)) the tickness should not exceed

$s = t - 1.5 = 8 - 1.5 = 6.5\,mm$

Hence provide s = 6 mm, weld.

Each angle carries a factored pull of $\frac{450}{2} = 225\,kN$.

Let L_w be the total length of the weld required.

Assuming normal weld, $t = 0.7 \times 6\,mm$

∴ Design strength of the weld $= L_w t \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25}$

$= L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$

Equating it to the factored load, we get

$$L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 225 \times 10^3$$

∴ $L_w = 283\,mm.$

centre of gravity of the angle section is at a distance 31 mm from top.

Let L_1 be the length of top weld and L_2 be the length of the lower weld. To make centre of gravity of weld to coincide with that of angle,

$$L_1 \times 31 = L_2 (100 - 31)$$

$$\therefore L_1 = \frac{69}{31} L_2$$

$$L_1 + L_2 = 283$$

i.e $$L_2 \left[\frac{69}{31} + 1 \right] = 283$$

or $$L_2 = 87 \text{ mm}$$

$$\therefore L_1 = 195 \text{ mm}$$

provide 6mm weld of $L_1 = 195$ mm and $L_2 = 87$ mm as shown in the fig (a)

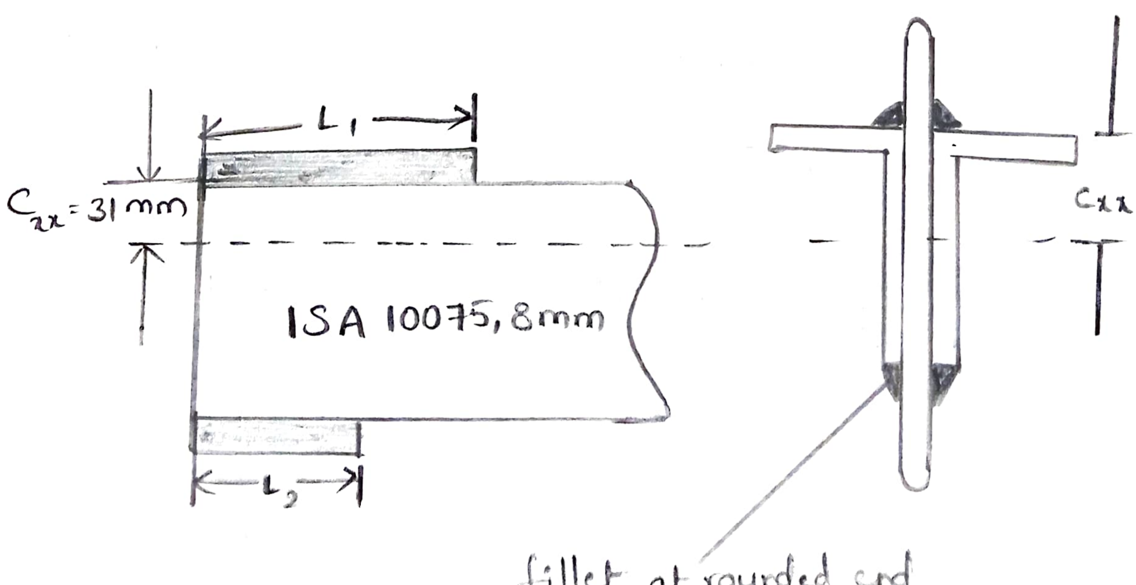


fig (a)

Note: In case the length available at the sides becomes insufficient, end fillet weld also may be provided. The length of the end fillet should be the same on either side of centroidal axis of the angle, so that neutral axis of the weld and the section coincide (fig (b))

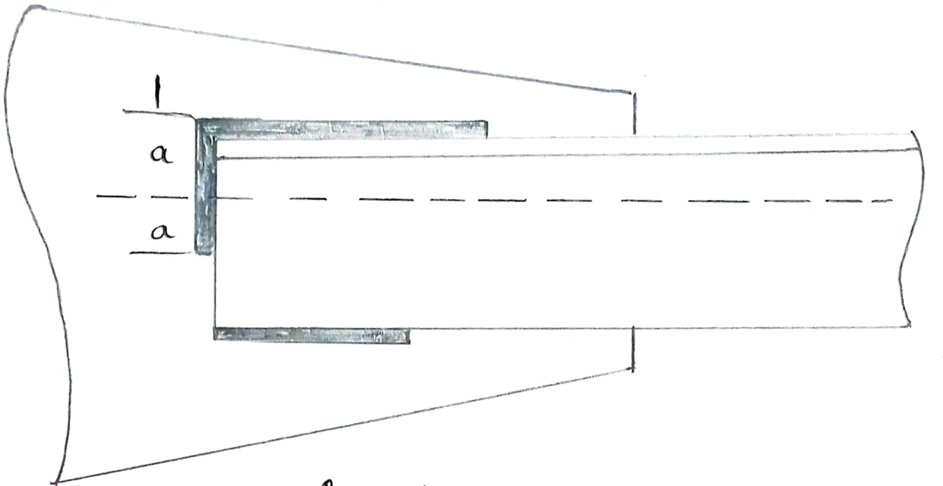


fig (b)

Q2. Design a lap joint between the two plates each of width 120 mm, if the tickness of one plate is 16mm and the other is 12mm. The joint has to transfer a design load of 160kN. The plates are of Fe 410 grade. Use bearing type bolts.

Using M16 bolts of grade 4.6,

$d = 16mm$ $\quad d_o = 18mm$ and $f_{ub} = 400N/mm^2$

strength of bolt :

Since it is lap joint bolt is in the single shear, the critical section being at the roof of bolt.

$\therefore$ Nominal strength of a bolt in shear $= \frac{f_{ub}}{\sqrt{3}}\left[1\times 0 + 0.78 \times \frac{\pi}{4} d^2\right]$

$= \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2$

$= 36218N.$

$\therefore$ Design shear strength $= \frac{36218}{1.25} = 28974\,N$.

Minimum edge distance to be provided $= 1.5 \times 18 = 27\,mm$

Minimum pitch to be provided $= 2.5 \times 16 = 40mm$.

providing $e = 30mm$, $p = 40mm$,

K_b is least of $\frac{30}{3 \times 18}$, $\frac{40}{3 \times 18} - 0.25$, $\frac{400}{410}$ and 1.0.

i.e., $k_b = 0.4907$

$\therefore$ Nominal bearing strength $= 2.5\,k_b\,d\,t\,f_u$

$= 2.5 \times 0.4907 \times 16 \times 12 \times 400$

$= 94222\,N$

$\therefore$ Design bearing strength $= \frac{94222}{1.25} = 75378\,N$

Design strength is lessor of Shearing strength and bearing strength.

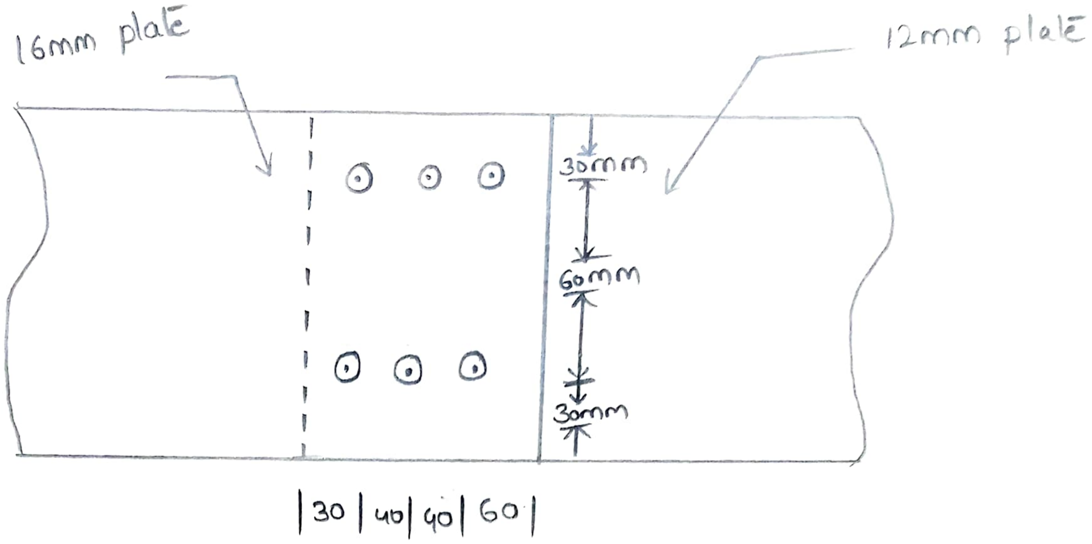
$\therefore$ Design strength of a M16 bolt $= 28974\,N$

$= 28.974\,kN$

Hence to transfer a design force of 160kN,

No. of bolts required $= \frac{160}{28.974} = 5.5$

$\therefore$ provide 6 bolts. They may be provided in two rows with a pitch of 40mm as shown in below figure



check for the strength of plate :

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_m} = \frac{0.9 \times (120 - 2\times 18) \times 12 \times 410}{1.25}$$

$$= 297562 \text{ N} = 297.562 \text{ kN} > 160 \text{ kN safe.}$$

Q3. Design a double angle tension member connected on each side of a 10 mm thick gusset plate, to carry an axial factored load of 375kN. Use 20mm black bolts. Assume shop connection.

Area required from the consideration of yielding

$$= \frac{1.1 \times 375 \times 1000}{250} = 1650 \text{ mm}^2.$$

Try 2 ISA 7550, 8mm thick which has gross

area = $2 \times 938 = 1876 \text{ mm}^2$.

strength of 20mm black bolts:

(a) In double shear $= \left[\frac{\pi}{4} \times 20^2 + 70.8 \times \frac{\pi}{4} \times 20^2\right] \times \frac{400}{\sqrt{3}} \frac{1}{1.25}$

$= 103314$ N.

(b) strength in bearing :

Taking $e = 40mm$, $p = 60\ mm$,

K_b is smaller of $\frac{40}{3\times22}$, $\frac{60}{3\times22} - 0.25$, $\frac{400}{410}$, 1.0

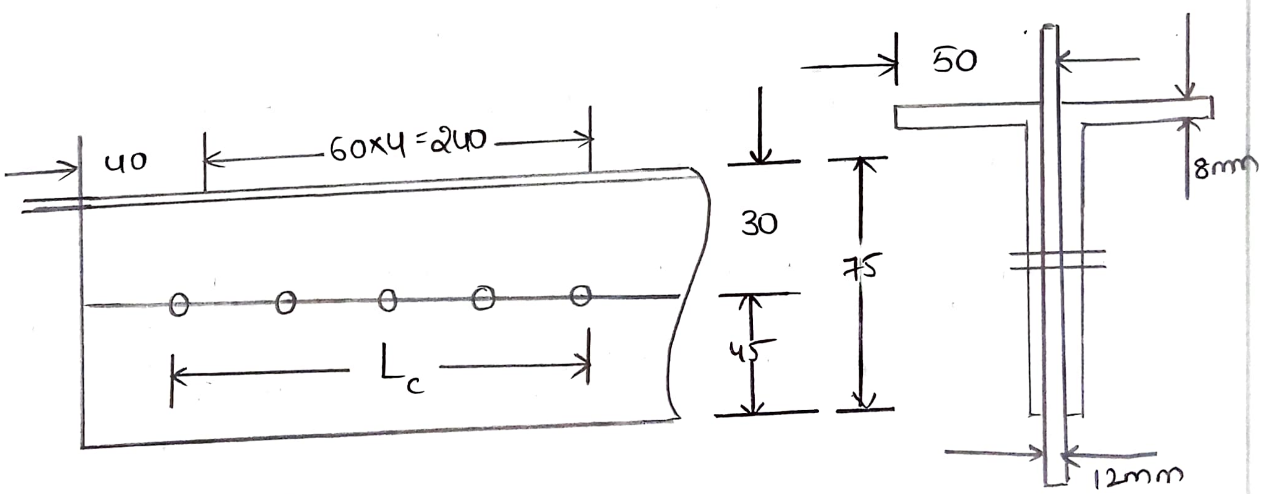
i.e., $K_b = 0.606$

$\therefore V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 20 \times 8 \times 400 = 77568N$

$\therefore$ Bolt value $= 77568N$.

Number of bolts required $= \frac{375000}{77568} = 4.83$

provide 5 bolts in a row as shown in below figure



$w = 50mm$

$b_s = 50 + 35 - 8 = 77\ mm$

$L_c = 60\times4 = 240mm$

checking the design :

(a) strength against yielding $= \frac{A_g f_y}{\gamma_{m_0}} = \frac{1876 \times 250}{1.1}$

$= 426364N > 375 \times 1000$

(b) Strength of plate in rupture :

Area of ~~plate in rupture~~ :
connected leg :

$$A_{nc} = 2\left[75 - 22 - \frac{8}{2}\right] \times 8 = 784\,mm^2$$

Area of outstanding leg,

$$A_{go} = 2 \times \left[50 - \frac{8}{2}\right] \times 8 = 736\,mm^2$$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{50}{8} \times \frac{250}{410} \times \frac{77}{240}$$

$$= 1.307$$

$$\therefore\; T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 410 \times 784}{1.25} + 1.307 \times \frac{736 \times 250}{1.1}$$

$$= 450062 > 375000\,N$$

(c) Strength against block shear failure :

per angle :

$$A_{vg} = (40 + 60 \times 4) \times 8 = 2240\,mm^2$$

$$A_{vn} = (40 + 60 \times 4 - 4.5 \times 22) \times 8 = 1448\,mm^2$$

$$A_{tg} = (75 - 35) \times 8 = 320\,mm^2$$

$$A_{tn} = (75 - 35 - 0.5 \times 22) \times 8 = 232\,mm^2$$

Strength against block failure of each angle is the smaller of the following two values:

(i) $= \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{mo}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{ml}}$

$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$

$= 362410$ N

(ii) $= \frac{0.9 A_{vn} f_u}{\sqrt{3} \times \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$

$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$

$= 319515$ N

∴ strength of two angles against block failure

$= 2 \times 319515 > 375000$

Hence use 2 ISA 7550, 8mm with 5 bolts of 20mm diameter.

Q. A column 4m long has to support a factored load of 6000 kN. The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates.

Assuming $f_{cd} = 200\,N/mm^2$

Area required $= \frac{6000 \times 10^3}{200} = 30000\,mm^2$

using ISHB 450 @ 907 N/m,

Area provided = 11789 mm², width of flange = 250 mm.

∴ Area to be provided ~~by~~ plates = 30000 − 11789 = 18211 mm²

Selecting 20mm plates, breadth required 'b' is obtained from

$$2b \times 20 = 18211$$
$$b = 455.3$$

provide 20mm × 500 mm plate

check for overhang:

$$\text{overhang} \quad \frac{500-250}{20} = 12.5 < 12t \text{ (clause 10.2.3.2 in IS 800)}$$

Total area provided

$$A_e = 11789 + 500 \times 20 \times 2$$
$$= 31789 \text{ mm}^2$$

for ISHB 450 @ 907 N/m

$$I_{zz} = 40349.9 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 3045 \times 10^4 \text{ mm}^4$$

∴ for the section selected,

$$I_{zz} = 40349.9 \times 10^4 + 2 \times 500 \times 20 (225+10)^2$$
$$= 1507.994 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 3045 \times 10^4 + 2 \times \frac{1}{12} \times 20 \times 500^3$$
$$= 447.1167 \times 10^6 \text{ mm}^4$$

$$\therefore r = r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{447.1167 \times 10^6}{31789}} = 118.6 \text{ mm}$$

Effective length $KL = 0.8L = 0.8 \times 4000 = 3200$ mm.

$\therefore \frac{KL}{r} = \frac{3200}{118.6} = 26.98$

$t_f = t_f$ of I section + 20 $= 13.7 + 20 = 31.7 < 40$ mm

It belongs to buckling class c for buckling about y-y Axis

$\therefore$ from table

$f_{cd} = 224 - \frac{6.98}{10}(224 - 211)$

$= 214.9 \text{ N/mm}^2$

$\therefore P_d = A_e f_{cd} = 31789 \times 214.9$

$= 6831456$ N

$= 6831.456$ kN > factored load

Hence safe.

Q5 Design a slab base for a column ISHB 300@ 577 N/m carrying an axial factored load of 1000 kN. M20 concrete is used for the foundation. Provide welded connection between column and base plate.

Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 20 = 9 \text{ N/mm}^2$

factored load $P_u = 1000$ kN.

$\therefore$ Area of base plate required $= \frac{1000 \times 10^3}{9}$

$= 111111 \text{ mm}^2$

provide 360 x 310 size plate.

Area provided 360 x 310 = 111 600 mm².

$$\text{pressure} = \frac{1000 \times 10^3}{111600} = 8.96 \ N/mm^2$$

projections are

$$a = \frac{360 - 300}{2} = 30mm$$

$$b = \frac{310 - 250}{2} = 30mm$$

$$\therefore t_s = \left[\frac{2.5 \times 8.96\,(30^2 - 0.3 \times 30^2) \times 1.1}{250}\right]^{0.5}$$

$$= 7.88 \ mm.$$

Thickness of flange of ISHB 300 @ 577 N/m is 10.6mm

provide 12mm thick plate.

Connecting 360 x 310 x 12 mm plate to concrete foundation:

use 4 bolts of 20mm diameter 300mm long to anchor the plate.

welds: properly machined column is to be connected to base plate using fillet weld.

Total length available for welding ref fig (a)

$$= 2(250 + 250 - 7.6 + 300 - 2 \times 10.6) = 1542.4 \ mm$$

$$\text{strength of weld} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37 \ N/mm^2$$

let 's' be the size of weld. Then effective area of weld $= 0.7 s L_e$

Where L_e is effective length.

$\therefore$ the design condition is $0.7 s L_e \times 189.37 = 1000 \times 10^3$

$$sL_e = 7543.8$$

using 6mm weld, $L_e = 1257$ mm.

After deducting for end return of the twice the size of the weld at each end.

Available effective length $= 1542.4 - 2 \times 6 \times$ No. of returns

$= 1542.4 - 2 \times 6 \times 12$

$= 1398.4 > 1257$ mm

Hence 6mm weld is adequate.

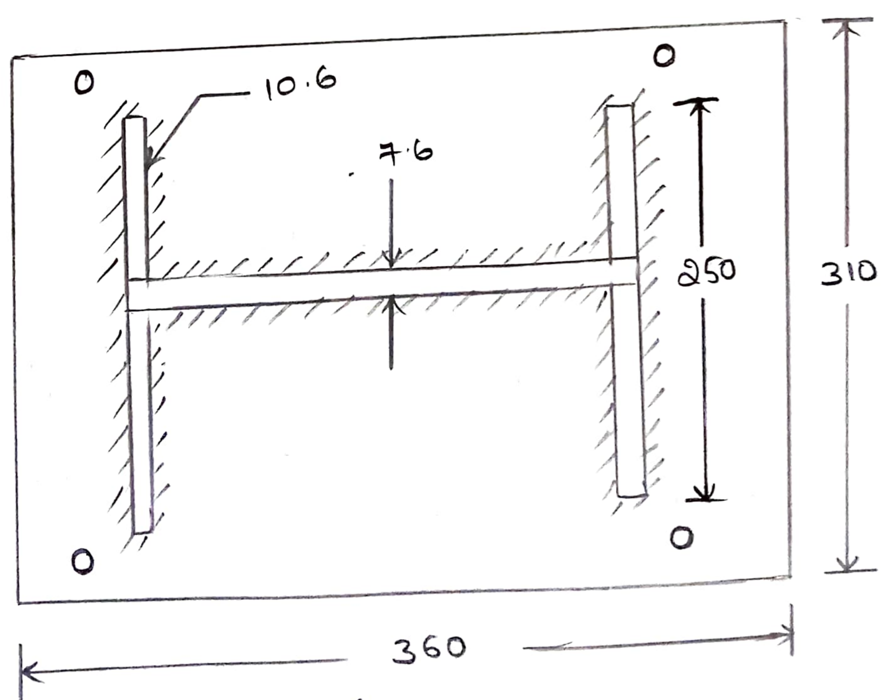


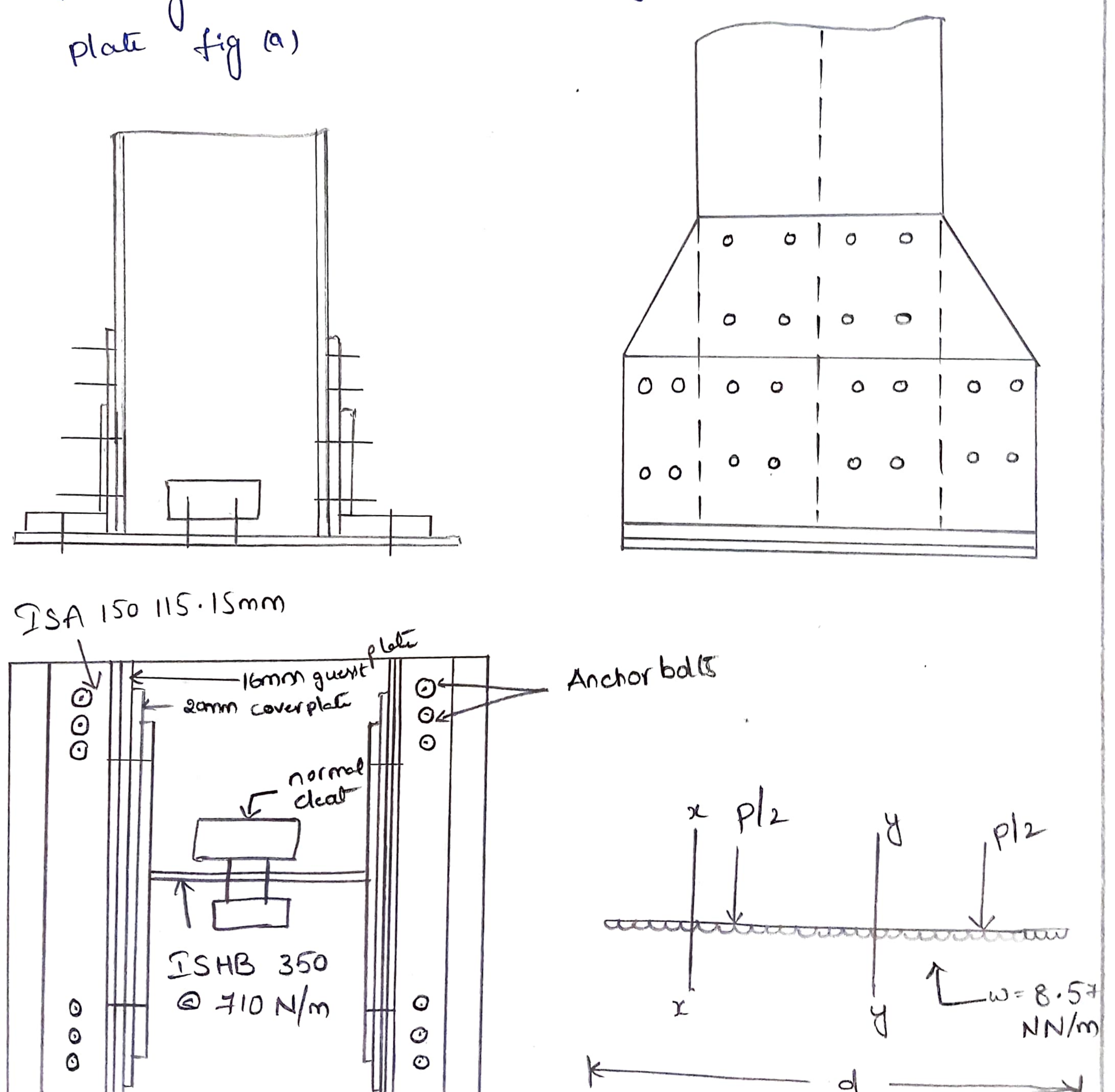
figure (a)

96. Design a gusseted base for a column ISHB 350 @ 710 N/m with two plates 450 mm × 20 mm carrying a factored load of 3600 kN. The column is to be supported on concrete pedestal to be built with M20 concrete.

$$f_{ck} = 20\ \text{N/mm}^2.$$

$$A = \frac{P_u}{0.45 f_{ck}} = \frac{3600 \times 10^3}{0.45 \times 20} = 400000\ \text{mm}^2$$

selecting ISA 150115, 15mm angle and 16mm thick gusset plate fig (a)



$d = 350 + 2 \times 20 + 2 \times 16 + 2 \times 115 = 652\text{mm}$

fig (a)

Minimum width required $= 350 + 2 \times 20 + 2\times 16 + 2\times 115$

$= 652mm.$

Use 700mm wide plate.

$\therefore$ length of base plate $= \dfrac{400000}{700} = 571mm$

provide $700 \times 600mm$ plate.

pressure under base plate $= \dfrac{3600 \times 10^3}{700 \times 600} = 8.57\ N/mm^2$

$$a = \frac{700 - (350 + 20\times 2 + 16\times 2 + 2\times 15)}{2}$$

$= 124mm$

BM at section X–X per mm width

$$= 8.57 \times \frac{124^2}{2} = 65886\ N\text{-}mm$$

At section Y–Y, bending moment [Note: per mm width $P = 8.57 \times 350$ N]

$$M_{yy} = 8.57 \times \frac{350^2}{2} - \frac{700}{2} \times 8.57 \times \left[\frac{350}{2} + 20 + \frac{16+15}{2}\right]$$

$= 106482\ N\text{-}mm$

$\therefore$ Design moment $= 106482\ N\text{-}mm$

Bending strength $= \dfrac{f_y}{\gamma_{mo}} = \dfrac{250}{1.1} = 227.27\ N/mm^2$

Equating moment of resistance to bending moment we get,

$$1.2 \times \frac{1}{6} \times 1 \times t^2 \times 227.27 = 106482$$

$\therefore t = 48.4\ mm$

$\therefore$ Use, 56mm base plate of size 700×600 mm.

Assuming ends of columns are faced for complete bearing, the connection between gusset plate and column will be designed for 50 percent of axial load.

Design ~~load~~ ~~on each~~ splice load $= 0.5 \times 3600 = 1800\,kN.$

Load on each splice $= \frac{1800}{2} = 900\,kN.$

Using 24mm shop bolts,

Strength of bolt in single shear $= 0.78 \times \frac{\pi}{4} \times 24^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25}$

$= 65192\,N.$

Strength in bearing is higher.

$\therefore$ Bolt value $= 65192\,N$

$\therefore$ No. of bolts required $= \frac{900 \times 10^3}{65192} = 13.8$

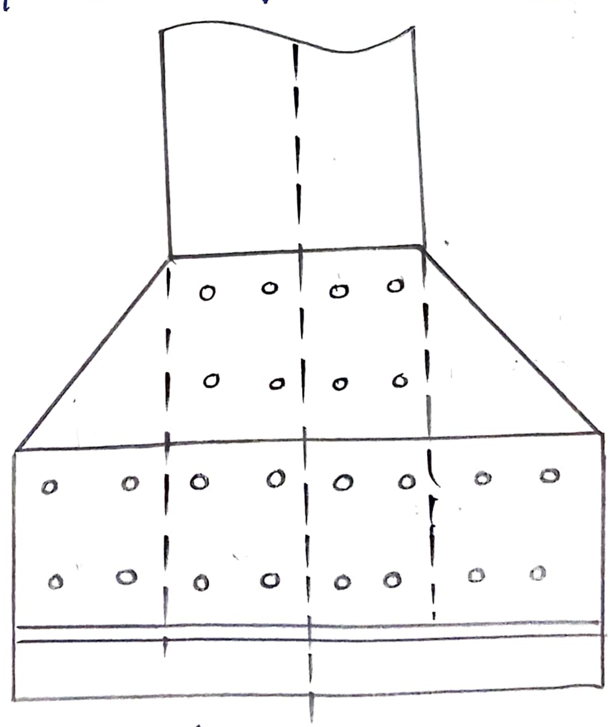


fig (b)

Provide 16 bolts as shown in figure(b), for connecting column to gusset plate. Use another 8 bolts to connect cleat angle to gusset plate.

Q7. Design a simply supported beam of effective span 1.5 m carrying a factored concentrated load of 360 kN at mid span.

Maximum moment occurs at mid span and is given by

$$M = \frac{WL}{4} = \frac{360 \times 1.5}{4} = 135 \text{kN-m} = 135 \times 10^6 \text{ N-mm}$$

$\therefore Z_p$ required is obtained from the relation $f_y \frac{Z_p}{\gamma_{mo}} = M$

$$Z_p = \frac{135 \times 10^6}{250} \times 1.1 = 594.0 \times 10^3 \text{ mm}^3$$

Select trial section as ISMB 300 which has $Z_p = 651.731 \times 10^3 \text{ mm}^3$.

The sectional properties of ISMB 300 are

overall depth $h = 300$mm.

width of flange $b = 140$mm

Thickness of flange $t_f = 12.4$mm

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

$= 300 - 2(12.4 + 14)$

$= 247.2$mm

$\therefore h_f$ = Centre to centre distance of flanges

$= 300 - \frac{12.4}{2} = 293.8$mm

Thickness of web $t_w = 7.5$mm

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

$Z_e = 573.6 \times 10^3 \text{ mm}^4$

$Z_p = 651.73 \times 10^3 \text{ mm}^4$

Self weight of beam $= 0.4336$ kN/m.

$\therefore$ factored weight $= 1.5 \times 0.4336$ kN/m

$\therefore$ additional factored moment due to self at

$$= 1.5 \times 0.4336 \times \frac{1.5^2}{8} = 0.183 \text{ kN-m}$$

$\therefore$ total factored moment

$$M = 135 + 0.183 = 135.183 \text{ kN-m}.$$

Factored shear force due to self weight $= 1.5 \times 0.4336 \times \frac{1.5^2}{8}$

$$= 0.488 \text{ kN}$$

$\therefore$ Total factored shear force on section $= \frac{360}{2} + 0.488 = 180.488$ kN

Section classification:

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1, \text{ overhang } b = \frac{140}{2}$$

$$\frac{b}{t_f} = \frac{140/2}{12.4} = 5.64 < 9.4\epsilon$$

$$\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84\epsilon$$

It is classified as plastic section : class 1

Shear capacity of the Section :

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5$$

$$= 295.235 \times 10^3 \text{ N} = 295.235 \text{ kN}$$

Section is Adequate to resist shear

$\therefore \quad 0.6 V_d = 0.6 \times 295.235 = 177.145 \text{ kN}$

$\therefore \quad V > 0.6 V_d$

Moment capacity of the Section:

Since $V > 0.6 V_d$ and section belongs to plastic catigory,

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$$

$$M_d = Z_p f_y \frac{1}{\gamma_{mo}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$$

Now $Z_p f_y \times \frac{1}{\gamma_{mo}} = 651.7 \times 10^3 \times 250 \times \frac{1}{1.1} = 148.114 \times 10^6 \text{ N-mm}$

$1.2 Z_e f_y \times \frac{1}{\gamma_{mo}} = 1.2 \times 573.57 \times 10^3 \times 250 \times \frac{1}{1.1} = 156.428 \times 10^6 \text{ N-mm}$

$\therefore \quad M_d = 148.120 \times 10^6$

$$\beta = \left[\frac{2V}{V_d} - 1\right]^2 = \left(\frac{2 \times 180.488}{295.235} - 1\right)^2 = 0.05$$

Since it is double symmetric section, f_{crb} may be obtained from table 14 of the code

$$\frac{KL}{r} = \frac{1500}{28.4} = 52.8 \text{ and } \frac{h_f}{t_f} = \frac{293.8}{12.4} = 23.69$$

Referring to table 14, by double interpolation we get $f_{crb} = 886.2$

from table 13(a),

$$f_{cd} = 204.5 - \frac{88.2}{100}(206.8 - 204.5)$$

$$= 204.77$$

$\therefore \quad M_{fd} = f_{cd} \times A = 204.77 \times 5626 = 1.15204 \times 10^6 \text{ N-mm}$

$\therefore M_{dv} = f_{cd} \times$

$\therefore M_{dv} = 148.114 \times 10^6 - 0.05(148.114 \times 10^6 - 1.15204 - 10^6)$

$= 140.77 \times 10^6$ N-mm $= 140.77$ kN-m > 135.190 kN-m.

Maximum deflection corresponding to working load

$$\delta = \frac{WL^3}{48EI} = \frac{360 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 8603 \times 10^4}$$

$$= 1.68 \text{ mm} < \frac{1500}{300}.$$

Hence Section is adequate.

Use ISMB 300 as beam.

Q8. Design a laterally unsupported I beam with simply supported ends of effective span 6m subjected to a working load of 35kN/m Assume that full torsional and warping restraints are provided at the supports and the load acts on the upper flange which will have destabilization effect.

The factored load $= 1.5 \times 35 = 52.5$ kN/m.

The maximum shear force, $V = \frac{52.5 \times 6}{2} = 157.5$ kN

The maximum bending moment, $M = \frac{52.5 \times 6^2}{8} = 236.25$ kNm

The design of a laterally unsupported beam involves trail and error procedure.

Let $f_{bd} = 120$ N/mm²

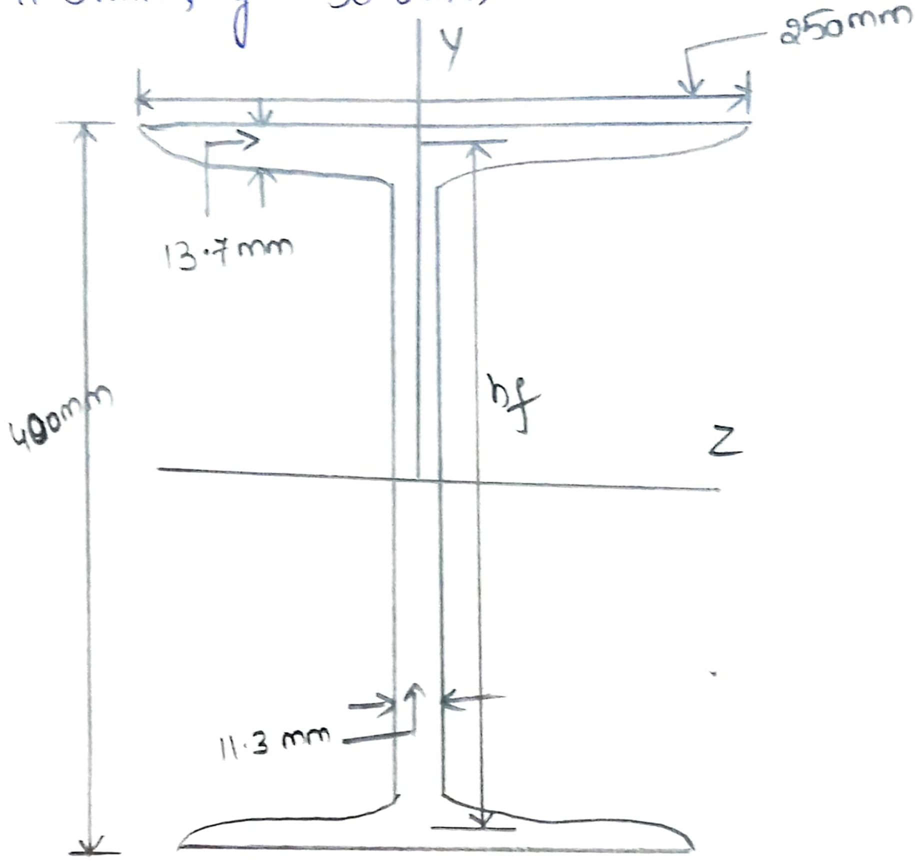
The plastic section modulus to be provided $= \frac{236.25 \times 10^6}{120}$

$= 1968.7$ cm³

from Appendix A, HB 450 @ 92.5 kg/m may be tried.

Its properties are $Z_{pz} = 2030.95 cm^3$, $b_f = 250mm$, $t_f = 13.7\ mm$,

$t_w = 11.3mm$, $r_y = 50.8mm$



The length for the lateral torsional buckling,

$$L_{LT} = 0.85 \times 6{,}000 = 5{,}100\,mm$$

$$h_f = 450 - 13.7 = 436.3\,mm$$

$$f_{cr,b} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(5{,}100/50.8)^2}\left[1 + \frac{1}{20}\left(\frac{5{,}100/50.8}{436.3/13.7}\right)^2\right]^{0.5} = 263.8\,MPa$$

$$\lambda_{LT} = \sqrt{\frac{250}{263.8}} = 0.97$$

$$\phi_{LT} = 0.5\left[1 + 0.21(0.97 - 0.2) + 0.97^2\right] = 1.05$$

$$\chi_{LT} = \frac{1}{1.05 + \sqrt{(1.05^2 - 0.97^2)}} = 0.69$$

$$f_{bd} = 0.69 \times 250/1.1 = 156.5\,MPa$$

$$\frac{b}{t_f} = \frac{125}{13.7} = 9.1 < 9.4\varepsilon \quad \text{where } \varepsilon = 1.0$$

Hence the section is plastic and $\beta_b = 1.0$.

$M_d = 1.0 \times 2{,}030.95 \times 10^3 \times 156.5 = 317.9\ kNm >> 236.25\ kNm$ not satisfactory. HB 400 @ 82.2 kg/m may be tried. Its properties are

$Z_{pz} = 1{,}626.36\ cm^3$, $b = 250mm$, $t_f = 12.7mm$, $t_w = 10.6mm$, $r_y = 51.6mm$

$h_f = 400 - 12.7 = 387.3mm$

$$f_{cr,b} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(5{,}100/51.6)^2}\left[1 + \frac{1}{20}\left[\frac{5{,}100/51.6}{387.3/10.6}\right]^2\right]^{0.5} = 259.8\ MPa$$

$$\lambda_{LT} = \sqrt{\frac{250}{259.8}} = 0.981$$

$$\phi_{LT} = 0.5\left[1 + 0.21(0.981 - 0.2) + 0.981^2\right] = 1.063$$

$$\chi_{LT} = \frac{1}{1.063 + \sqrt{(1.063^2 - 0.981^2)}} = 0.68$$

$$f_{bd} = 0.68 \times 250/1.1 = 154.3\ MPa$$

$$b/t_f = 125/13.7 = 9.84 > 9.4\varepsilon$$

$$d/t_w = \frac{400 - 2 \times 12.7}{10.6} = 35.3 < 84\varepsilon \quad \text{where } \varepsilon = 1.0$$

The section is compact and Hence $\beta_b = 1.0$

$$M_d = 1.0 \times 1626.36 \times 10^3 \times 154.3 = 250.9\ kNm > 236.25\ kNm$$

$$V_d = V_n/\gamma_{mo} = \frac{A_v f_y}{\sqrt{3}\gamma_{mo}} = \frac{400 \times 10.6 \times 250}{\sqrt{3} \times 1.1} = 556\ KN > 157.5$$

check for deflection

from appendix A, I_z of HB400 @ 82.2 kg/m is $28{,}800\ cm^4$

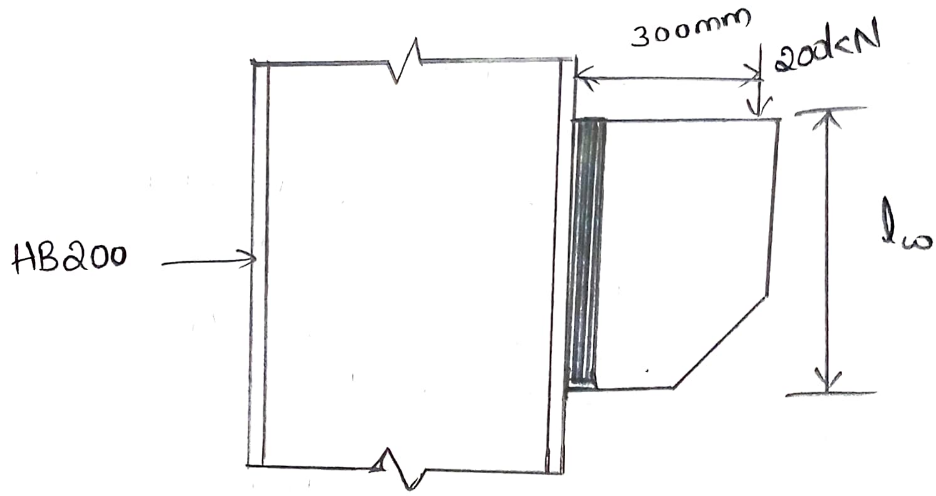
$$\delta_{max} = K\frac{WL^3}{EI} = \frac{5}{384}\frac{(35 \times 6000) \times 6000^3}{2 \times 10^5 \times 28{,}800 \times 10^4} = 10.25mm < \frac{6000}{240} = 25mm.$$

Q9. Design a connection for a bracket using the butt weld to carry a factored vertical load of 200kN acting at 300mm from the face of the column. The grade of the steel is E250.

Let the thickness of the bracket, $t = 12mm$

Minimum depth of the bracket is given by

$$l_w = \sqrt{\frac{6 \times 200 \times 10^3 \times 300}{12 \times (250/1.1)}} = 363 mm$$



A bracket of depth of 500mm may be tried.

$$q = \frac{P}{l_w t} = \frac{200 \times 10^3}{500 \times 12} = 33 MPa$$

$$f_b = \frac{6Pe}{t l_w^2} = \frac{6 \times 200 \times 10^3 \times 300}{12 \times 500^2} = 120 MPa$$

$$f_e = \sqrt{120^2 + 3 \times 33^2} = 133 MPa << (250/1.1 = 227 MPa)$$

The depth of the bracket may be reduced to 400mm

$$q = \frac{200 \times 10^3}{400 \times 12} = 42 MPa$$

$$f_b = \frac{6 \times 200 \times 10^3 \times 300}{12 \times 400^2} = 187.5 MPa$$

$$f_e = \sqrt{187.5^2 + 42^2} = 192\,MPa < 227\,MPa$$

Hence, 400mm deep and 12mm thick bracket may be provided which may be welded to the column using a full penetration butt weld.

Q10 Design a seat angle connection for a beam MB350@52.4kg/m which transfers a factored end shear of 200kN to the flange of the column HB 300@63kg/m. $f_y = 250\,MPa$ and $f_u = 410\,MPa$

2∠100 100 may be used as seat angles on either side of the seat angle of the beam. The height of the angle may be 200mm (0.57 times depth of beam). An erection clearance (g) of 10mm may be assumed. Shear transferred by each angle = $V/2 = 200/2$
= 100kN

From Appendix A, for MB 350, $R_1 = 14mm$, $t_f = 14.2mm$ and $t_w = 8.1mm$. $n_2 = 2.5(R_1 + t_f) = 2.5(14 + 14.2) = 70.5mm$

$$b_1 = \frac{200 \times 10^3 \times 1.1}{8.1 \times 250} - 70.5 = 38\,mm$$

The minimum width of the horizantal leg of a seat angle

$= g + b_1 = 10 + 38 = 48mm$

The bending moment in the horizantal leg of a seat angle,

$$M = V\left[\frac{b_1}{2} - t\right]$$

$$= 200 \times 10^3 \left[\frac{38}{2} - t\right]$$

$$= 200 \times 10^3 (19 - t)$$

Since the width of the flange of the beam MB 350 is 140mm, the length of the seat angle may be 180mm which is less than the flange width of the column HB 300, i.e, 250mm.

The ultimate moment capacity of the horizantal leg of the seat angle $M = 1.2\left[\frac{bt^2}{6}\right]\left[\frac{f_y}{\gamma_{mo}}\right] = 1.2 \times \frac{180 \times t^2}{6} \times \frac{250}{1.1}$

$$\therefore 1.2 \times \frac{180 \times t^2}{6} \times \frac{250}{1.1} = 200 \times 10^3 (19 - t)$$

or $t = 12.6$mm

$\angle$ 130 130 x 16 may be provided as the seat angle.

The bending moment acting on the fillet welds =

$$V(g + b_1/2) = 200 \times 10^3 \left[10 + \frac{38}{2}\right]$$

$$= 5.8 \text{kNm}$$

$$q_b = \frac{3V_e}{t_t l_w^2} = \frac{3 \times 5.8 \times 10^6}{0.7s \times 130^2} = 1474/s$$

$$q_s = \frac{V}{2t_t l_w} = \frac{200 \times 10^3}{2 \times 0.7s \times 130} = 1099/s$$

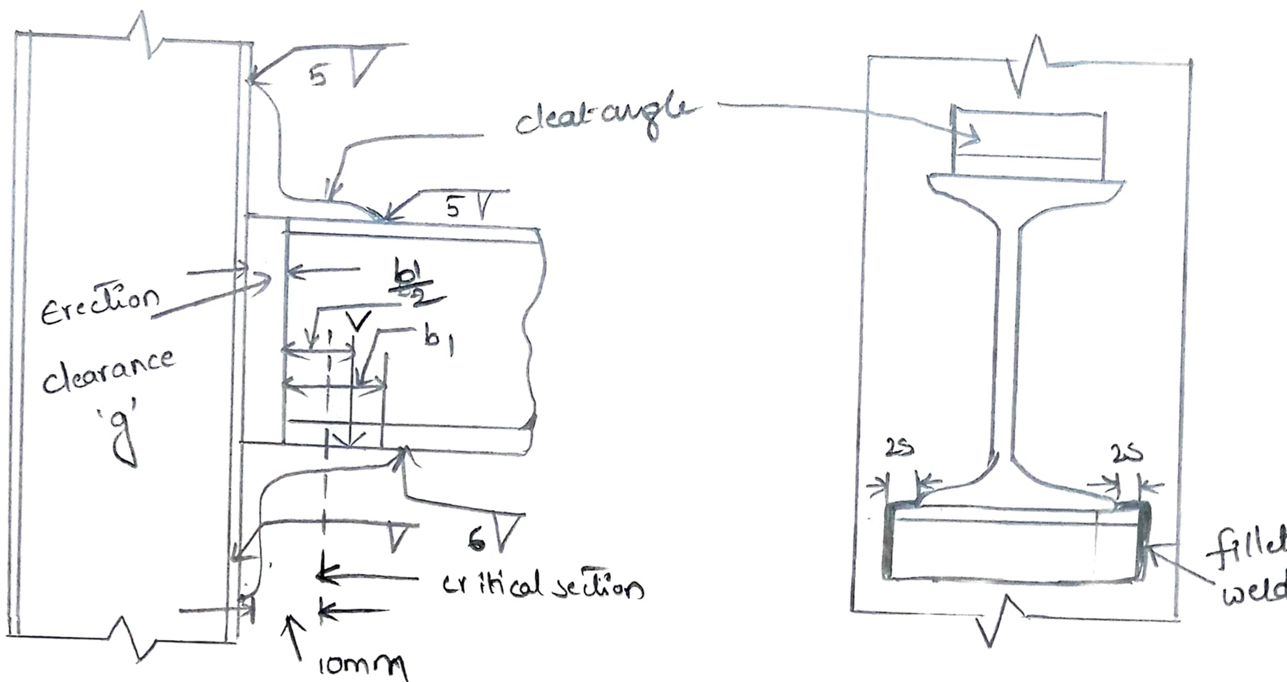
$$q = \frac{1}{s}\sqrt{1474^2 + 1099^2} = 1839/s$$

$f_{wd} = 189.4$ MPa

$1839/s = 189.4$

or $s = 9.7$mm

fillet welds of the size 10mm ($\angle$ 3/4th of the angle thickness) may be provided which should be returned for the length of 2 x 10 = 20 mm at the top of the vertical legs of the seat angle as shown in below figure



Seat-angle welded to the flange of a column