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Internal Assessment Test 3 – July 2023

Sub:	Design of Steel Structural Elements			SubCode:	18CV61		Branch:	Civil	
Date:	04.07.2023	Duration:	90 mins	Max Marks:	50	Sem/Sec:	VI	OBE	

	Provide neat sketches wherever necessary. IS800, SP 6 or steel table are permitted	MARKS	CO	RBT
1	What are lug angles? Explain design principle of lug angles.	[06]	CO6	L1
2	Design a built up column with two channels back to back to length 8m to carry an axial factored load of 1000 kN. The column is hinged at both ends. Design the column with single lacing with bolted connections	[14]	CO4	L3
3	Design a suitable unequal single angle section to carry a load of 150kN (Tension) assuming a single row of M20 bolts of grade 4.6 for the end connection. Assume Fe410 grade steel. The length of the member is 2.5m.	[12]	CO5	L3
4	Design a steel beam section for supporting roof of a hall for the following data and apply checks for shear, bending and deflection. Assume Fe415 grade steel clear span = 6.5m , End bearings = 150mm, c/c spacing of beams = 3m. Imposed load on beam = 10kN/m^2, Beam depth is restricted to 375mm. The compression flange of the beam is laterally supported throughout.	[18]	CO6	L3

1)

When the number of bolts are more the length of the joint increases. To decrease the length of the joint a small piece of angle is connected to outstanding leg called "Lug Angle".

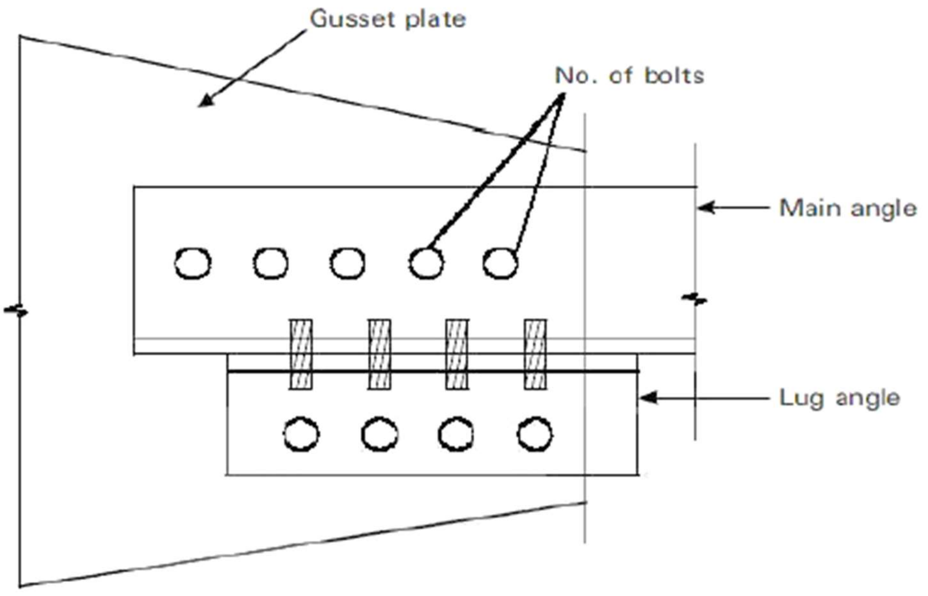


Fig. 4.31 : Connection of main angle with lug angle and gusset plate

IS : 800 - 2007, clause 10.12 specifications for lug angles are

1) Lug angles connecting outstanding leg of channel - shaped member shall, as far as possible, be disposed symmetrically with respect to the section of the member.
2) In the case of angle members, the lug angles and their connections to the gusset or other supporting member shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the member, and the attachment of the lug angle to the main angle shall be capable of developing a strength not less than 40 percent in excess of the force in the outstanding leg of the angle.
3) In the case of channel members and the like, the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10 percent in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20 percent in excess of that force.
4) In no case shall fewer than two bolts, rivets or equivalent welds be used for attaching the lug angle to the gusset or other supporting member.
5) The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the main member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.
6) Where lug angle are used to connect an angle member, the whole area of the member shall be taken as effective not withstanding the requirements of tension members.

2) Similar solution

Given data

$$\text{Length of column} = 9 \text{ m}$$

$$\text{End condition} = \text{Fixed at both ends}$$

$$\therefore \quad K = 0.65$$

$$\text{Axial load on column} = 1200 \text{ kN}$$

$$= 1200 \times 10^3 \text{ N}$$

Assuming approximate design compressive stress = 125 N/mm²

$$\text{Steel grade} ; f_y = 250 \text{ N/mm}^2$$

(i) Design of column

$$\text{Area of column required} = \frac{1200 \times 10^3}{125}$$

$$= 9600 \text{ mm}^2$$

$$\text{For one section} = \frac{9600}{2}$$

$$= 4800 \text{ mm}^2$$

Referring to steel table,

Select ISMC 350 @ 413 N/m

Properties are

$$A = 5366 \text{ mm}^2 > 4800 \text{ mm}^2 \quad \therefore \text{OK}$$

$$h = 350 \text{ mm}, \qquad b_w = 100 \text{ mm}$$

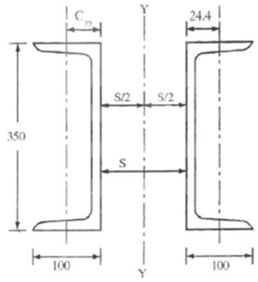
$$t_f = 13.5 \text{ mm}, \qquad t_w = 8.1 \text{ mm}$$

$$I_{xx} = I_{zz} = 100.88 \times 10^3 \text{ mm}^4$$

$$I_{yy} = 4.306 \times 10^6 \text{ mm}^4$$

$$r_{xx} = r_{zz} = 136.60 \text{ mm}$$

$$r_{yy} = 28.3 \text{ mm}, \; c_{yy} = 24.4 \text{ mm}$$



(ii) Safety of section adopted

$$\lambda = \frac{L}{r} = \frac{KL}{r_{zz}} = \frac{0.65 \times 9000}{136.60}$$

$$= 42.825$$

The $\frac{L}{r}$ or λ of the built-up column should be taken as $1.05\left(\frac{L}{r_{zz}}\right)$

$$\lambda = 1.05 \times 42.825 = 44.96 \equiv 45$$

$$\therefore \quad \text{For } \lambda = 45 \text{ and } f_y = 250 \text{ N/mm}^2$$

Referring to Table 9(c) of the IS : 800-2007

For built-up section - buckling class '*c*'

$$\therefore \quad f_{cd} = 190.5 \text{ N/mm}^2$$

∴ Load carrying capacity

$$P_d = A_e \times f_{cd}$$
$$= 2 \times 5366 \times 190.5$$
$$= 2044.46 \times 10^3 \text{ N}$$

or $\qquad P_d = 2044.46 \text{ kN} > 1200 \text{ kN}$

∴ Hence, column is safe

To fine the spacing '*S*' between the two channels

We have

$$2I_{zz} = 2\left[I_{yy} + A\left(\frac{S}{2} + c_{yy}\right)^2\right]$$

$$\cancel{2} \times 100.08 \times 10^6 = \cancel{2}\left[4.306 \times 10^6 + 5366\left(\frac{S}{2} + 24.4\right)^2\right]$$

$$95.774 \times 10^6 = 5366\left(\frac{S}{2} + 24.4\right)^2$$

or $\qquad \left(\frac{S}{2} + 24.4\right)^2 = 17848.30$

or $\qquad \frac{S}{2} + 24.4 = 133.59$

or $\qquad S = 218.39 \text{ mm}$

Let the distance between the backs of two channels be $S = 220$ mm

(iii) Lacing design

Let the lacing bars are provided at 45° with the horizontal

Horizontal distance between bolt lines

$$\text{Assuming gauge length} = 50 \text{ mm}$$

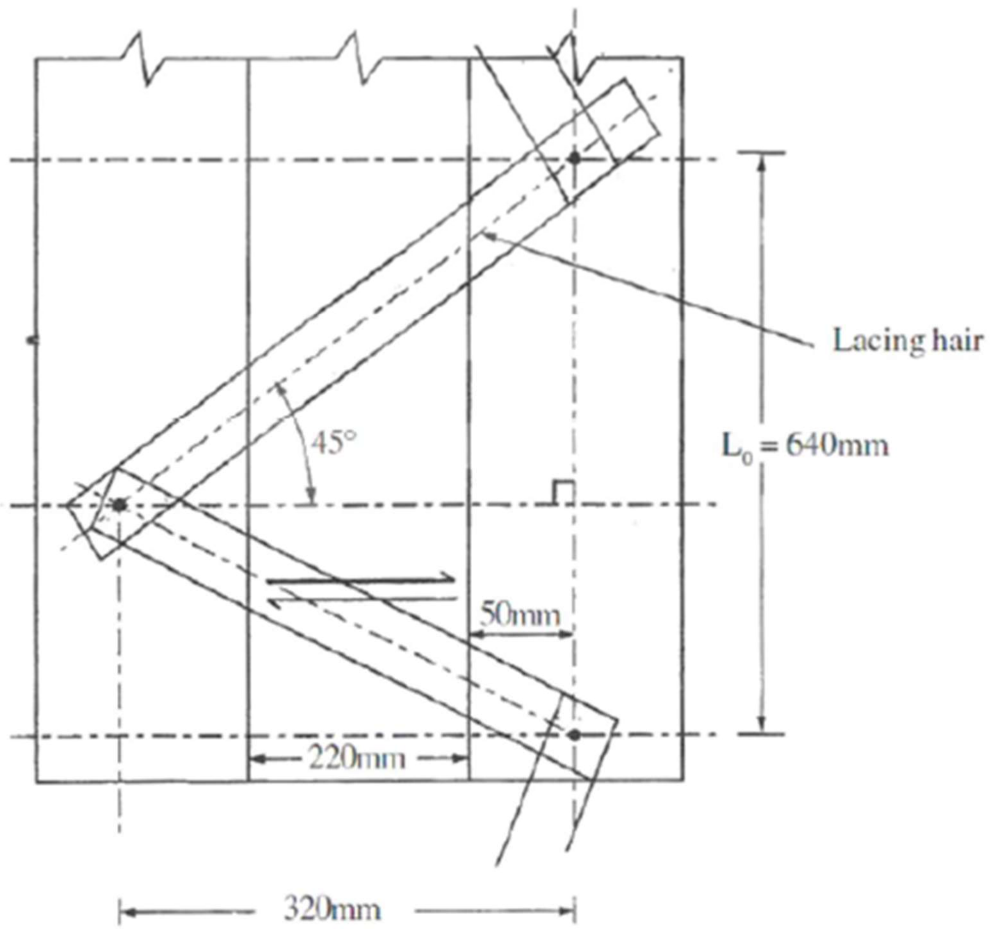
$$= 220 + 50 + 50 = 320 \text{ mm}$$

Spacing between lacing bars

$$L_0 = 320 \tan 45° \times 2 = 640 \text{ mm} \quad \text{(Refer Fig. 3.24)}$$

$$\frac{L_0}{r_{yy}} \text{ should be } < 0.7 \times \frac{L}{r} \text{ of whole column}$$

$$\frac{640}{28.3} = 22.61 < 0.7 \times 42.82 = 29.97 \qquad \therefore \text{ OK}$$



Now, transverse shear to be resisted by lacing bars

$$V_t = 2.5\% \text{ of the axial load}$$

$$= \frac{2.5}{100} \times 1200 = 30 \text{ kN or } 30 \times 10^3 \text{ N}$$

$\therefore$ Transverse shear in each panel

$$= \frac{V}{N} = \frac{30 \times 10^3}{2} = 15 \times 10^3 \text{ N}$$

Compressive force in the lacing bar

$$= 15 \times 10^3 \times \frac{1}{\cos 45^\circ} = 21.21 \times 10^3 \text{ N}$$

Assuming 16 mm diameter bolts

Minimum width of lacing flat (Refer clause 7.6.2 of IS : 800 - 2007)

$$= 3 \times 16 = 48 \text{ mm say } 50 \text{ mm}$$

$$\text{Minimum thickness} = \frac{1}{40} \times \text{of the effective length of single lacing}$$

$$= \frac{1}{40} \times (220 + 50 + 50) \times \text{cosec } 45^\circ = 11.31 \text{ say } 12 \text{ mm}$$

$\therefore$ Provide lacing bar of 50 × 12 mm (flat bar)

$$\text{Minimum } r = \frac{t}{\sqrt{12}} = \frac{12}{\sqrt{12}} = 3.464 \text{ mm}$$

Slenderness ratio of lacing bar

$$= \frac{l_e}{r} = \frac{320 \times \text{cosec } 45^\circ}{3.464} = \frac{452.54}{3.464}$$

$$= 130.64 < 145 \text{ (As per IS : 800 - 2007)} \quad \therefore \text{Safe}$$

For flat bar, buckling class '*c*' (Refer Table 10, IS : 800-2007)

$\therefore$ For $\lambda = 130.64$ and buckling class '*c*' and $f_y = 250$ N/mm^2

Design compressive stress :

(Refer Table 9(c), IS : 800 - 2007). $f_{cd} = 75$ N/mm^2

$\therefore$ Design compressive strength

$$= A \times f_{cd}$$

$$= (50 \times 12) \times 75$$

$$= 45 \times 10^3 \text{ N} > 21.21 \times 10^3 \text{ N} \quad \therefore \text{Safe}$$

(iv) Check for tension in lacing bar

- Design strength against rupture

$$T_{dn} = \frac{0.9A_n \cdot f_u}{\gamma_{mf}} = \frac{0.9 \times (50-18) \times 12 \times 410}{1.25} = 11.357 \times 10^4 \text{ N}$$

- Design tensile strength against yielding

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{(50 \times 12) \times 250}{1.1} = 13.63 \times 10^4 \text{ N}$$

$\therefore$ Design tensile strength of lacing bar

$11.357 \times 10^4 \text{ N} > 21.21 \times 10^3 \text{ N}$

- Design shear strength of 16 mm bolt (Double shear)

(Double shear because one bolt tie two lacing plates at one joint or point)

$$= 2\left[\frac{f_{ub}}{\sqrt{3} \cdot \gamma_{mb}} \times A_{nb}\right]$$

$$A_{nb} = 0.78 \times \frac{\pi}{4}(16)^2 = 156.82 \text{ mm}^2$$

$$= 2\left[\frac{400}{\sqrt{3} \times 1.25} \times 156.82\right] = 57.54 \times 10^3 \text{ N}$$

- Design bearing strength of bolt

$$= \frac{2.5k_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$

k_b is minimum of the following $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}$ and 1 $\quad |d_0 = d + 2 = 16 + 2 = 18$ mm

Assuming edge distance $= e = 1.5 \times 18 = 27$ mm say 30 mm

pitch $= p = 2.5 \times 16 = 40$ mm

$$\frac{30}{3 \times 18} = 0.55, \frac{40}{3 \times 18} - 0.25 = 0.49, \frac{400}{410} = 0.97 \text{ and } 1$$

$$\therefore \quad k_b = 0.49$$

$$\therefore \quad = \frac{2.5 \times 0.49 \times 16 \times 12 \times 410}{1.25} = 77.145 \times 10^3 \text{ N}$$

$\therefore$ Design strength of bolt $= 57.54 \times 10^3$ N (Smaller of above two values)

$\therefore$ No. of bolts required for lacing connection

$$= \frac{\text{Compressive force in lacing bar}}{\text{Design strength of bolt}} = \frac{21.21 \times 10^3}{57.94 \times 10^3} = 0.36 \text{ say } 1$$

Hence, provide 1 bolt of 16 mm diameter at each end of lacing bar.

(v) Tie plates

Tie plates must be provided at the ends of laced column.

Effective depth of tie plate should not be less than distance between the centroids of the main members.

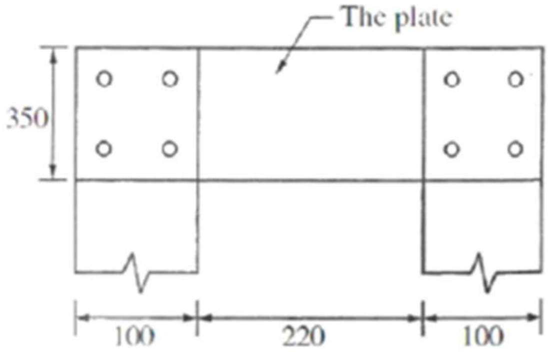


Fig. 3.25

$$= S + 2(C_{yy})$$
$$= 220 + 2(24.4) = 268.8$$

Assuming 30 mm edge distance or clearance at each end.

Total depth of tie plate $= 268.8 + 2(30) = 328.8$ say 350 mm

Total width of tie plate $= 220 + 2(b_w) = 220 + 2(100) = 420$ mm

Required thickness of tie plate $= \frac{1}{50}$ of effective length or width

$$= \frac{1}{50}(220 + 2g), \quad g = 50 \text{ mm gauge distance}$$

$$= \frac{1}{50}(220 + 100)$$

$$= 6.4 \text{ mm say } 8 \text{ mm}$$

∴ Provide a tie plate of 420 × 350 × 8 mm at both ends with six no. of bolts of 16 mm diameter.

(vi) Details of Connection

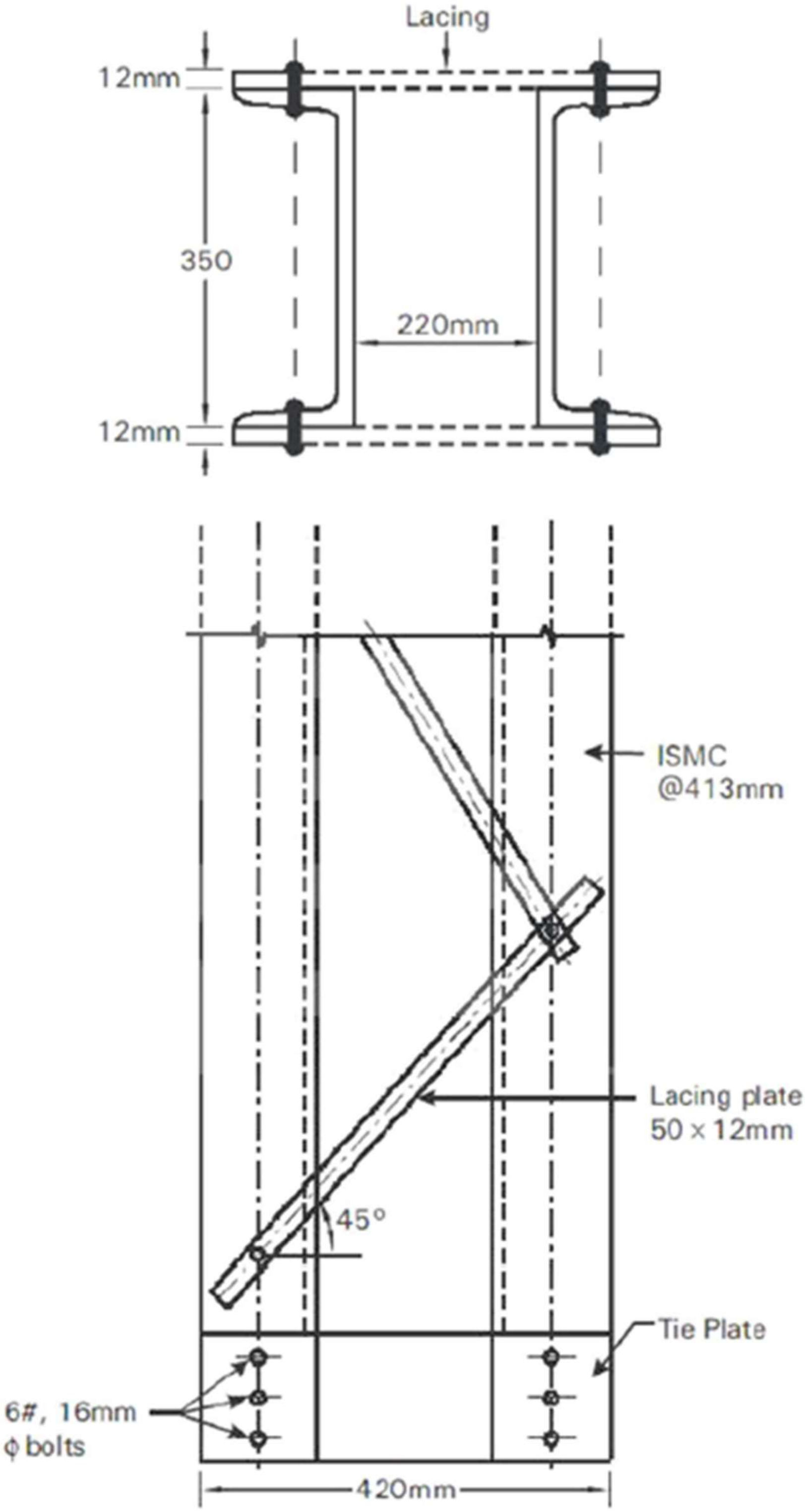


Fig. 3.26 : Column and lacing connection

Q 3) Similar solution

$f_u = 410$ N/mm², $f_y = 250$N/mm², $T = 100$ kN

Partial safety factors : $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$, $\gamma_{mb} = 1.25$

Step : 1 For preliminary sizing, the rupture strength of net section may be approximately taken as (As per IS : 800 - 2007 Clause 6.3.3)

$$T_{dn} = \alpha \frac{A_n f_u}{\gamma_{m1}}$$

Assuming $\alpha = 0.8$

$$T_{dn} = T = 100 \text{ kN} = 100 \times 10^3 \text{ N (Given)}$$

$\therefore$ Required Net area, $$A_n = \frac{T_{dn}\gamma_{m1}}{0.8 f_u} = \frac{100 \times 10^3 \times 1.25}{0.8 \times 410}$$

$$= 381 \text{ mm}^2$$

The net area may be increased by 25% to determine the expected gross area

$$A_g = 1.25 \times 381 = 476.25 \text{ mm}^2$$

Step : 2 Required gross area on the basis of gross section yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$

$$T_{dg} = T$$

$$A_g = \frac{T\gamma_{m0}}{f_y} = \frac{100 \times 10^3 \times 1.1}{250} = 440 \text{ mm}^2$$

Required gross area is maximum value of step (1) and (2).

$$\therefore \; A_g, \text{ required} = 476.25 \text{ mm}^2$$

Step : 3 Selection of angle section

Refer steel table and select equal or unequal angle section having gross sectional area more than 476.25 mm²

Let us try ISA 50 × 50 × 6 mm (steel table) or Hand book

$$A_g = 568 \text{ mm}^2 > 476.25 \text{ mm}^2$$

Step : 4 Design of bolted connection

$$d = 16 \text{ mm}$$

Assuming 4.6 grade bolts, $f_{ub} = 400$ N/mm²

$$d_0 = 16 + 2 = 18 \text{ mm}$$

$$A_{nb} = 0.78 \times \frac{\pi \times 16^2}{4} = 157 \text{ mm}^2$$

i. Shear strength of bolt

$$= \frac{A_{nb} f_{ub}}{\sqrt{3} \gamma_{mb}}$$

$$= \frac{157 \times 400}{\sqrt{3} \times 1.25} = 29006 \text{ N} = 29 \text{ kN}$$

ii. Bearing strength of bolt

Edge distance, $e = 1.5\ d_0 = 1.5 \times 18 = 27$ mm say 30 mm

pitch, $p = 2.5\ d = 2.5 \times 16 = 40$ **mm**

$$k_b \text{ is least of, } \frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.56,\ \frac{p}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49,$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \text{ and } 1.0$$

$$\therefore \quad k_b = 0.49$$

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

$$= \frac{2.5 \times 0.49 \times 16 \times 6 \times 410}{1.25} = 38572.8 \text{ N}$$

$$= 38.57 \text{ kN}$$

$$\text{Bolt value} = 29 \text{ kN (least of (i) and (ii))}$$

$$\therefore \text{Number of bolts required} = \frac{\text{Load transmitted (Factored load)}}{\text{bolt value}}$$

$$= \frac{100}{29} = 3.44, \quad \therefore \text{ Provide 4 bolts}$$

Step : 5 Check for strength against yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{568 \times 250}{1.1} = 129090 \text{ N} = 129.09 \text{ kN} > 100 \text{ kN}.$$

Step : 6 Check for the strength of plate in rupture

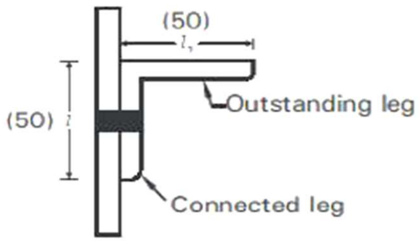


Fig. 4.18 (a)

Area of connected leg, $A_{nc} = \left(l - d_0 - \frac{t}{2}\right)t$

$$= \left(50 - 18 - \frac{6}{2}\right)6$$

$$= 174 \text{ mm}^2$$

Area of out standing leg, $A_{go} = \left(l_1 - \frac{t}{2}\right)t$

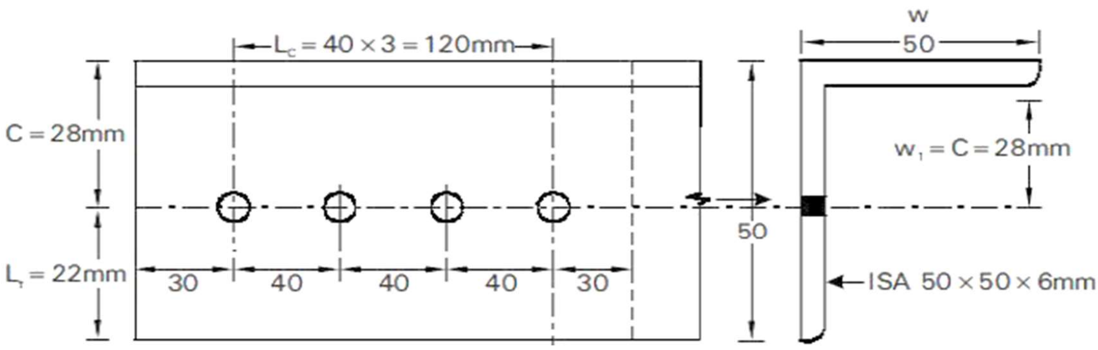
$$= \left(50 - \frac{6}{2}\right)6 = 282 \text{ mm}^2$$

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

where w = outstanding **leg width = 50 mm, w_1 = 28 mm (from table** - 4.2) or Hand book Table-31

b_s = shear lag width = $w + w_1 - t$

$= 50 + 28 - 6 = 72$ mm



$$\beta = 1.4 - 0.076\left(\frac{50}{6}\right)\times\left(\frac{250}{410}\right)\times\left(\frac{72}{120}\right) = 1.16$$

$$\beta = 1.16 > 0.7$$

$$\leq \frac{f_u\gamma_{m0}}{f_y\gamma_{m1}} = \frac{410\times 1.10}{250\times 1.25} = 1.44$$

$$\beta = 1.16$$

The design strength T_{dn} of angle governed by rupture of net cross sectional area, A_n

$$T_{dn} = 0.9\frac{A_{nc}f_u}{\gamma_{m1}} + \frac{\beta A_{g0}f_y}{\gamma_{m0}}$$

$$= 0.9\times\frac{174\times 410}{1.25} + \frac{1.16\times 282\times 250}{1.10}$$

$$= 51364.8 + 74345.4 = 125710.2 \text{ N}$$

$$= 125.71 \text{ kN} > 100 \text{ kN}$$

Hence safe.

Step : 7 Check for block shear

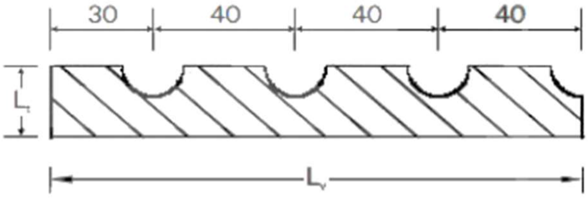


Fig. 4.18 (c)

$L_t = l - w_1 = 50 - 28 = 22$

$L_v = 30 + 40 \times 3 = 150$ mm

As per IS : 800-2007, clause 6.4.1, the Block shear strength, T_{db} of connection shall be taken as minimum of

$$T_{db1} = \left[\frac{A_{vg}f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9A_{tn}f_u}{\gamma_{m1}}\right]$$

$$T_{db2} = \left[\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\gamma_{m0}}\right]$$

$$A_{vg} = L_v \times t = 150 \times 6 = 900 \text{ mm}^2$$

$$A_{vn} = \left(L_v - \left(4 - \tfrac{1}{2}\right)\times d_0\right)t$$

$$= (150 - 3.5 \times 18) \times 6 = 522 \text{ mm}^2$$

$$A_{tg} = L_t \times t = 22 \times 6 = 132 \text{ mm}^2$$

$$A_{tn} = \left(L_t - \frac{1}{2} \times d_0\right) t = \left(22 - \frac{1}{2} \times 18\right) \times 6 = 78 \text{ mm}^2$$

$$T_{db1} = \frac{900 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 78 \times 410}{1.25} = 141119 \text{ N} = 141.12 \text{ kN}$$

$$T_{db2} = \frac{0.9 \times 522 \times 410}{\sqrt{3} \times 1.25} + \frac{132 \times 250}{1.1} = 118966 \text{ N} = 118.97 \text{ kN}$$

Therefore the block shear strength is 118.97 kN

$$T_{db} = 118.97 \text{ kN} > 100 \text{ kN}$$

Hence the member is safe in block shear.

Therefore the angle and the connection is safe.

Q 4) Similar solution

$$f_y = 250 \text{ MPa}, \gamma_{m0} = 1.1, \gamma_f = 1.5$$

$$\text{Effective span, } L = \text{clear span + bearing}$$

$$L = 6.5 + 0.2 = 6.7 \text{ m}$$

$$\text{Total udl per } m^2 = 12 + 3 = 15 \text{ kN/m}^2$$

$$\text{Total udl per } m = 15 \times \text{C/C spacing of beams}$$

$$= 15 \times 3 = 45 \text{ kN/m}$$

$$\text{Factored udl, } W_U = \text{Total udl per } m \times \gamma_f$$

$$W_U = 45 \times 1.5 = 67.5 \text{ kN/m}$$

$$\text{Maximum BM, } M = \frac{W_U L^2}{8} = \frac{67.5 \times 6.7^2}{8} = 378.75 \text{ kN.m}$$

$$= 378.75 \times 10^6 \text{ N.mm}$$

Required plastic modulus of section

$$Z_p = \frac{M\gamma_{m0}}{f_y} = \frac{378.75 \times 10^6 \times 1.1}{250} = 1666.54 \times 10^3 \text{mm}^3$$

Let us try ISLB 500 @ 75 kg/m

$h = 500$ mm, $b_f = 180$ mm, $t_f = 14.1$ mm, $t_w = 9.2$ mm, $Z_e = 1543.2 \times 10^3$ mm^3.

$Z_p = 1773.67 \times 10^3$ mm^3

$$I_{zz} = 38579 \times 10^4 \text{ mm}^4, r_1 = 17 \text{ mm}$$

- **Section classification**

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

$$\text{Outstand of flange, } b = \frac{b_f}{2} = \frac{180}{2} = 90 \text{mm}$$

$$\text{Depth of web, } d = h - 2(t_f + r_1)$$

$$= 500 - 2(14.1 + 17) = 437.8 \text{ mm}$$

$$\frac{b}{t_f} = \frac{90}{14.1} = 6.38 < 9.4$$

$$\frac{d}{t_w} = \frac{437.8}{9.2} = 47.58 < 84$$

Hence, the section is classified as plastic section, $\beta_b = 1.0$

Since $\frac{d}{t_w} = 47.58 < 67$, check for web buckling is not required

- **Check for shear capacity**

Maximum Shear Force, $V = \frac{W_U L}{2} = \frac{67.5 \times 6.7}{2} = 226.13\,\text{kN}$

Design shear strength of the section

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} h t_w = \frac{250}{\sqrt{3} \times 1.1} \times 500 \times 9.2$$

$$= 603593\ \text{N} = 603.59\ \text{kN} > 226.13\ \text{kN}$$

Hence safe

- **Check for high or low shear**

$$0.6\ V_d = 0.6 \times 603.59 = 362.15\ \text{kN} > 226.13\ \text{kN}$$

Therefore the case is of low shear

- **Design bending strength**

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} = \frac{1.0 \times 1773.67 \times 10^3 \times 250}{1.1}$$

$$= 403.10 \times 10^6\ \text{N.mm} = 403.1\ \text{kN.m}$$

$$\leq \frac{1.2 Z_e f_y}{\gamma_{m0}} = \frac{1.2 \times 1543.2 \times 10^3 \times 250}{1.1}$$

$$= 420.87 \times 10^6\ \text{N.mm} = 420.87\ \text{kN.m}$$

Therefore $M_d = 403.1\ \text{kN.m} > 378.75\ \text{kN.m}$

Hence section is safe

- **Check for deflection**

Maximum deflection, $\delta_{max} = \frac{5}{384} \times \frac{WL^4}{EI}$

$W = 45$ kN/m (working load)

$$\delta_{max} = \frac{5}{384} \times \frac{45 \times 6700^4}{2 \times 10^5 \times 38579 \times 10^4} = 15.3\,\text{mm}$$

Permissible deflection, $\delta = \frac{L}{300} = \frac{6700}{300} = 22.33\,\text{mm}$

$$\delta_{max} = 15.3 < 22.33\ \text{mm}$$

Hence safe against deflection