MODULE -3

Design of tension members

As per IS 800- 2007

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1.What Are Tension Members?

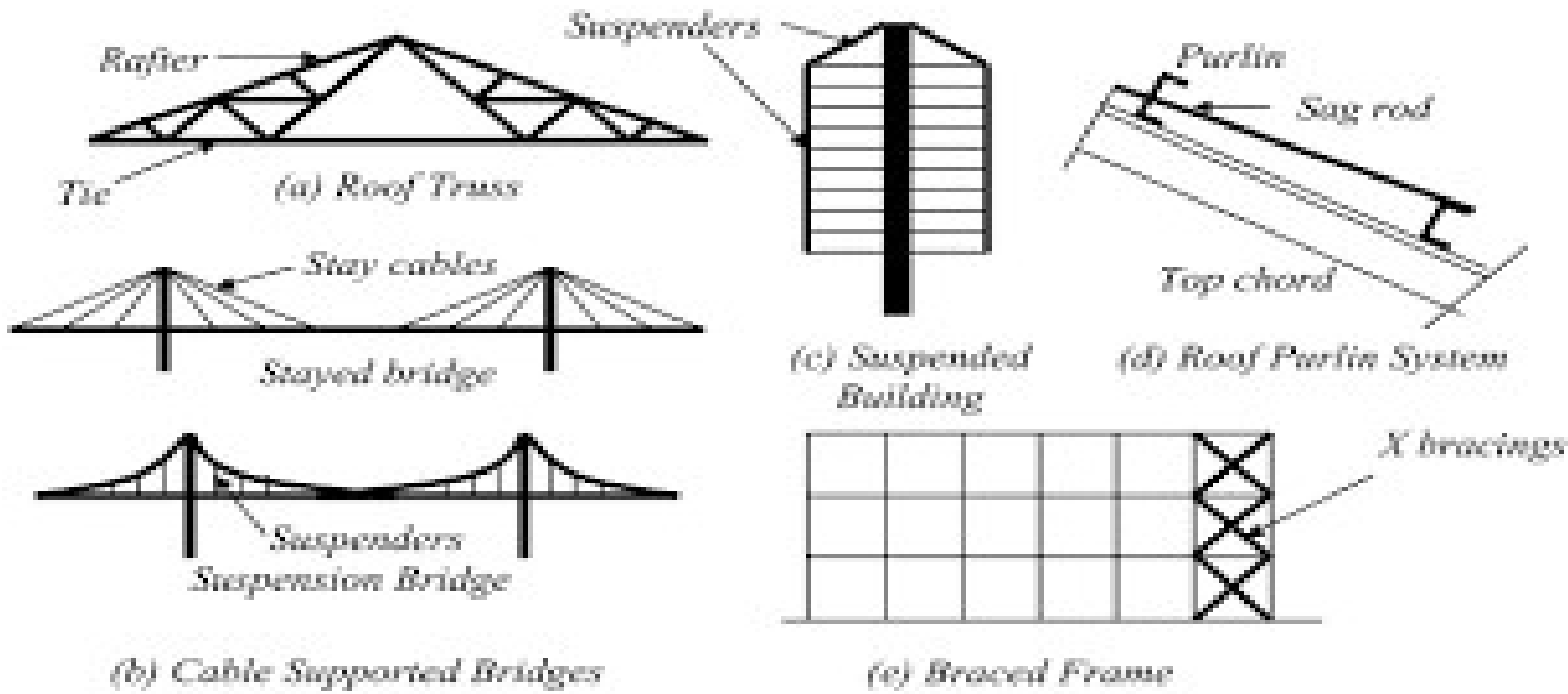
- structural elements that are subjected to direct axial tensile loads, which tend to elongate the members.

- The strength of these members is influenced by several factors such as the length of connection, size and spacing of fasteners, net area of cross section, type of fabrication, connection eccentricity, and shear lag at the end connection.

- The stress concentration near the holes leads to the yielding of the nearby fibres but the ductility of the steel permits redistribution of over stress in adjoining section till the fibres away from the holes progressively reach yield stress. Therefore at ultimate load it is reasonable to assume uniform stress distribution.

2. Types of tension members

1. Wires and cables: wires ropes are exclusively used for hoisting purposes and as guy wires in steel stacks and towers. Strands and ropes are formed by helical winding of wires . A strand consists of individual wires wound helically around the central core. These are not recommended in bracing system as they cannot resist compression. The advantages of wire and cable are flexibility and strength.



2. Bars and rods: These are simplest forms of tension members. Bars and rods are often used as tension members in bracing system, as sag rods to support purlins between trusses. Presently these are not favourite of with the designers because large drift they cause during strong winds and disturbing noise induces by the vibrations.

3.Plates and flat bars: These are used often as tension members in transmission towers, foot bridges, etc. These are also used in columns to keep the component members in their correct position. Eg- lacing flats, batten plates, end tie plates etc. Single and built-up structural shapes: 1. Open sections such as angles, channels and I sections.

- Compound and built-up sections such as double angles and double channels with are without additional plates and jointed with some connection system.
- Closed sections such as circular, square, rectangular or hollow sections.

3. Behaviour of Tension Members

The load-deformation behavior of members subjected to uniform tensile stress is similar to the load-deflection behavior of the corresponding basic material. The typical stress-strain behavior of mild steel under axial tensile load is shown in Fig. 1. The upper yield point is merged with the lower yield point for convenience. The material shows a linear elastic behavior in the initial region (O to A). The material undergoes sufficient yielding in portion A to B. Further deformation leads to an increase in resistance, where the material strain hardens (from B to C). The material reaches its ultimate stress at point C. The stress decreases with increase in further deformation and breaks at D. The high strength steel members do not exhibit the well defined yield point and the yield region (Fig. 1). For such materials, the 0.2 percent proof stress is usually taken as the yield stress (E).

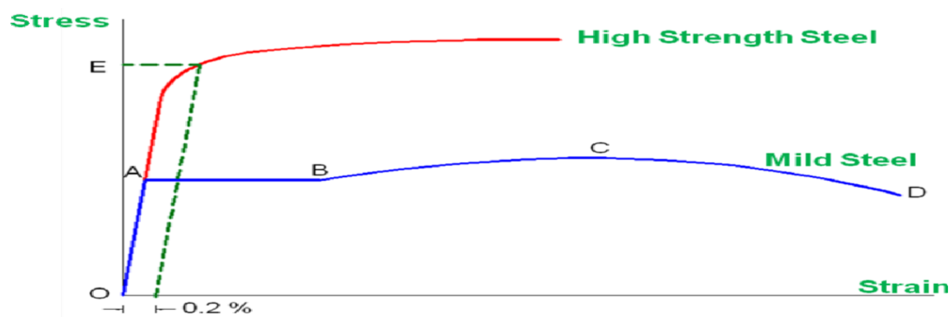


Fig. 1 Typical stress-strain diagram for mild steel and high strength steel

4.0 Slenderness Ratio

Apart from strength requirement, the tension members have to be checked for minimum stiffness by stipulating the limiting maximum slenderness ratio of the member. This is required to prevent undesirable lateral movement or excessive vibration. The slenderness limits specified in IS: 800-2007 for tension members are given in Table 1.

Table 1 Maximum values of effective slenderness ratio as per IS: 800-2007

Member	Maximum effective slenderness ratio (l/r)
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compressive forces resulting only from a combination of wind/earthquake actions, provided the deformation of such a member does not adversely affect the stresses in any part of the structure	250
A member normally acting as a tie in a roof truss or a bracing member, which is not considered effective when subject to reversal of stress resulting from the action of wind or earthquake forces	350

Members always in tension (other than pre-tensioned members)	400

5. Shear Lag

The tensile force to a tension member is transferred by a gusset plate or by the adjacent member connected to one of the legs either by bolting or welding. This force which is transferred to one leg by the end connection locally gets transferred as tensile stress over the entire cross section by shear. Hence, the distribution of tensile stress on the section from the first bolt hole to the last bolt hole will not be uniform. Hence, the connected leg will have higher stresses at failure while the stresses in the outstanding leg will be relatively lower. However, at sections far away from the end connection, the stress distribution becomes more uniform. Here the stress transfer mechanism, i.e., the internal transfer of forces from one leg to the other (or flange to web, or from one part to the other), will be by shear and because one part 'lags' behind the other, the phenomenon is referred to as '*shear lag*'.

The shear lag reduces the effectiveness of the component plates of a tension member that are not connected directly to a gusset plate. The efficiency of a tension member can be increased by reducing the area of such components which are not directly connected at the ends. The shear lag effect reduces with increase in the connection length.

1. Modes of Failure

The different modes of failure in tension members are

1. Gross section yielding
2. Net section rupture
3. Block shear failure

The strength of tension members under the different modes are failure, i.e., design strength due to yielding of gross section, T_{dg}, rupture of critical section, T_{dn} and block shear T_{db} are first determined. The design strength of a member under axial tension, T_d, is the lowest of the above three values.

6. Gross section yielding

Steel members (plates, angles, etc.) without bolt holes can sustain loads up to the ultimate load without failure. However, the members will elongate considerably (10 to 15 % of its original length) at this load, and hence make the structure unserviceable. Hence the design strength T_{dg} is limited to the yielding of gross cross section which is given by

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

where

f_y = yield strength of the material in MPa

A_g = gross area of cross section in mm^2

γ_{m0} = 1.10 = partial safety factor for failure at yielding

6.2 Net section rupture

This occurs when the tension member is connected to the main or other members by bolts. The holes made in members for bolts will reduce the cross section, and hence net area will govern the failure in this case. Holes in members cause stress concentration at service loads. From the theory of elasticity, the tensile stress adjacent to a hole will be about two to three times the average stress on the net area (Fig. 2a). This depends on the ratio of the diameter of the hole to the width of the plate normal to the direction of the stress.

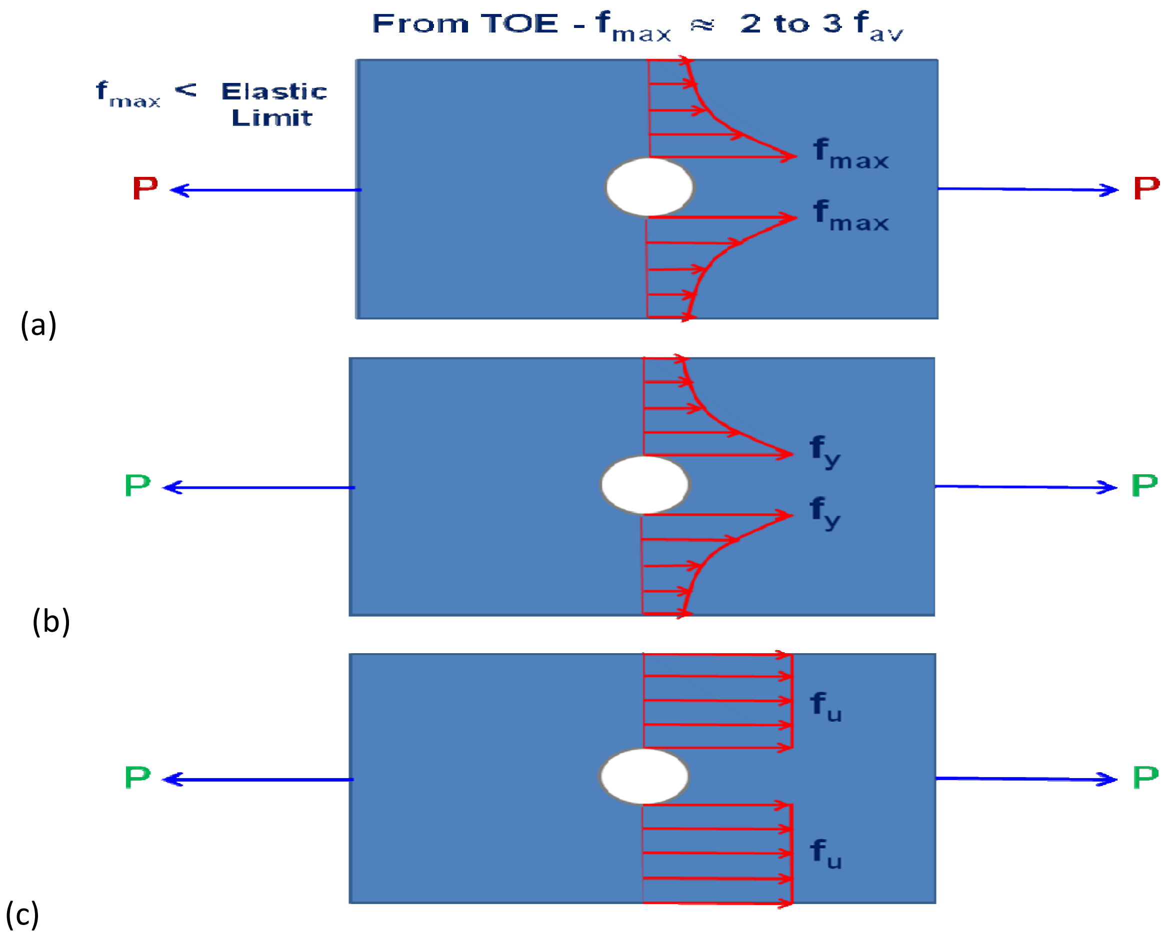


Fig. 2 Stress-distribution in a plate adjacent to hole due to tensile force.

When the tension member with a hole is loaded statically, the point adjacent to the hole reaches the yield stress f_y first (Fig. 2b). On further loading, the stress in other fibers away from the hole progressively reaches the yield stress f_y. Deformations of the member continue with increasing load until final rupture of the member occurs when the entire net cross section of the member reaches the ultimate stress f_u (Fig. 2c).

6.2.1 Net section rupture in plates

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross sectional area, A_n, at the holes is given by

$$T_{dn} = 0.9 f_u A_n / \gamma_{m1}$$

where

γ_{m1} = 1.25 = partial safety factor for failure at ultimate stress

f_u = ultimate stress of the material in MPa

A_n = net effective area of the member in mm^2 is given by

$$A_n = \left[b - n\, d_h + \sum_i \frac{p_{si}^2}{4 g_i} \right] t$$

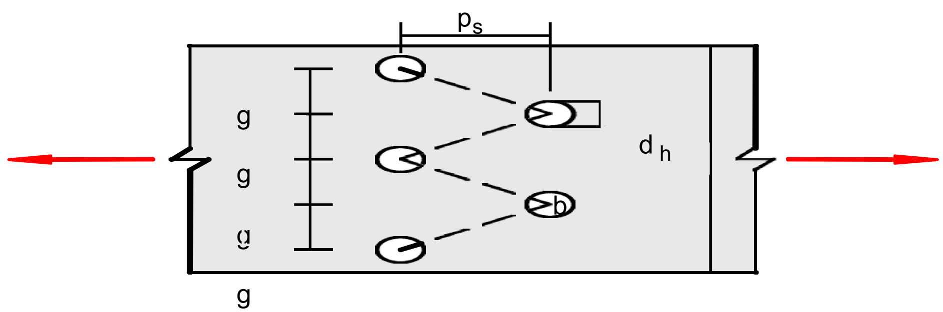


Fig. 3 Plate with bolt holes in tension

where

b, t = width and thickness of the plate, respectively

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case of directly punched holes)

g = gauge length between the bolt holes, as shown in Fig. 3

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 3

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs

The '0.9' factor included in the design strength equation is based on a statistical evaluation of a large number of test results for net section failure of members.

2. Net section rupture in threaded rods

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

$$T_{dn} = 0.9 f_u A_n / \gamma_{m1}$$

where A_n = net root area at the threaded section

3. Net section rupture in single angles

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, $T_{dn,}$ as governed by rupture at net section is given by

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

where

$$\beta = 1.4 - 0.076 (w/t) (f_y / f_u) (b_s / L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$$

$$\geq 0.7$$

where

w = outstand leg width

b_s = shear lag width, as shown in Fig. 4

L_c = Length of the end connection, i.e., distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction

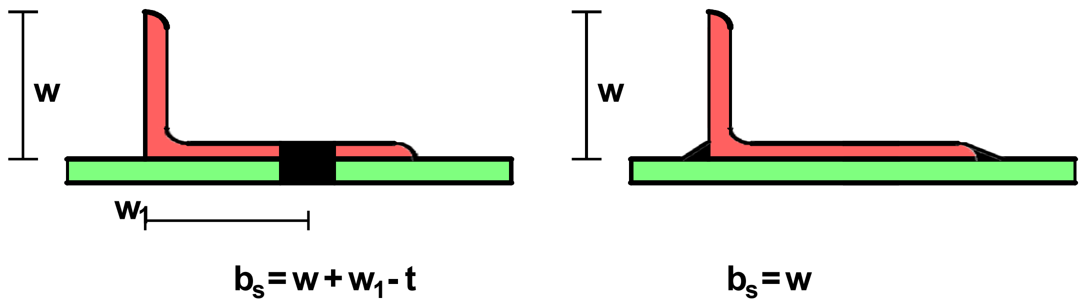


Fig. 4 Angles with single leg connections

For preliminary sizing, the rupture strength of net section may be approximately taken as

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

where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length

A_n = net area of the total cross section

A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg, and

t = thickness of the leg

6.2.4 Net section rupture in other sections

The tearing strength, T_{dn}, of the double angles, channels, I sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in 6.2.3, where β is calculated based on the shear lag distance, b_s taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross section.

6.3 Block shear failure

Block shear failure is considered as a potential failure mode at the ends of an axially loaded tension member. In this failure mode, the failure of the member occurs along a path involving tension on one plane and shear on a perpendicular plane along the fasteners. A typical block shear failure of a gusset plate is shown in Fig. 5. Here plane B-C is under tension whereas planes A-B and C-D are in shear.

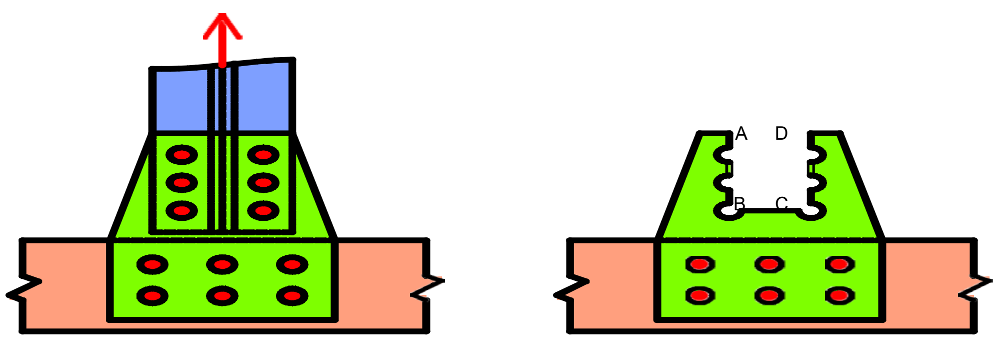


Fig. 5 Block shear failure in gusset plate

Typical block shear failure of angles in a bolted connection is shown in Fig. 6. Here plane 1-2 is in shear and plane 2-3 is in tension.

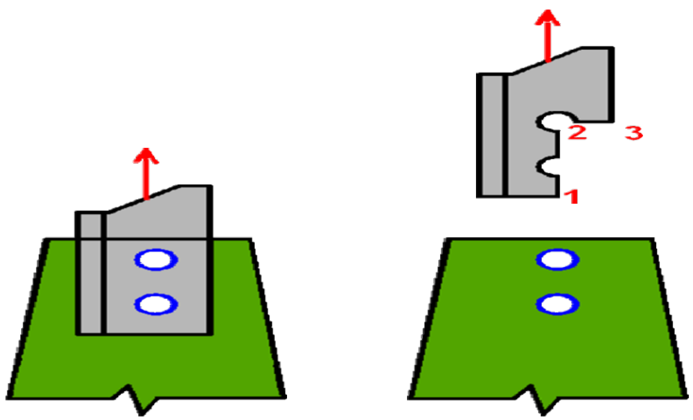


Fig. 6 Block shear failure in angle with bolted connection

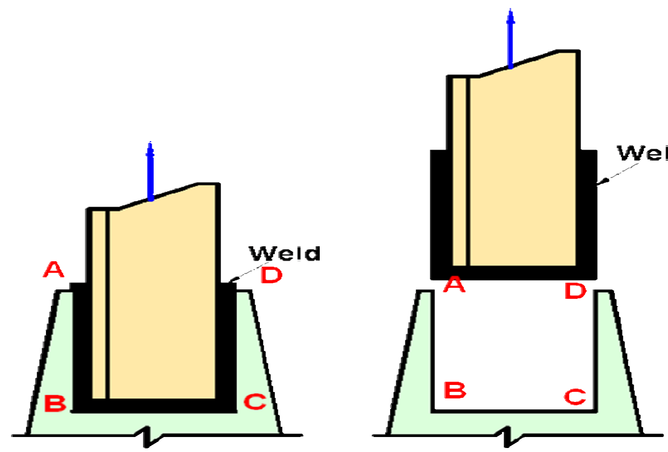


Fig. 7 Block shear failure of gusset plate in welded connections

The block shear failure is also seen in welded connections. A typical failure of a gusset in the welded connection is shown in Fig. 7. The planes of failure are chosen around the weld. Here plane B-C is under tension and planes A-B and C- D are in shear.

6.3.1 Design strength due to block shear in bolted connections

The block shear strength, T_{db}, of connection shall be taken as the smaller of

$$T_{db} = \left(A_{vg} f_y / (\sqrt{3}\, \gamma_{m0}) + f_u A_{tn} / \gamma_{m1} \right) \text{ or}$$

$$T_{db} = \left(f_u A_{vn} / (\sqrt{3}\, \gamma_{m1}) + f_y A_{tg} / \gamma_{m0} \right)$$

Where

A_{vg}, A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively (1-2 and 3–4 as shown in Fig. 8 and 1-2 as

shown in Fig. 9)

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force

(2-3 as shown in Figs. 8 and 9)

f_u, f_y = ultimate and yield stress of the material respectively

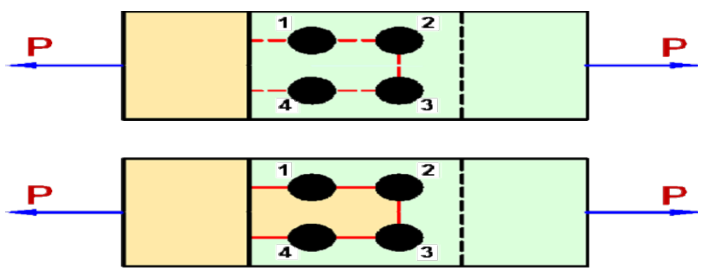


Fig. 8 Block shear failure in plate

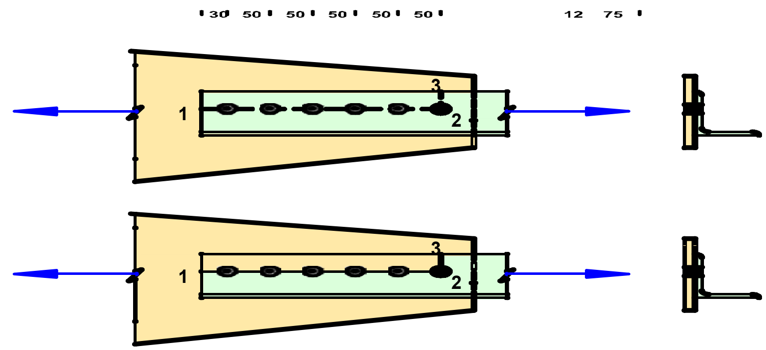


Fig. 9 Block shear failure in angle

6.3.1 Design strength due to block shear in welded connections

The block shear strength, T_{db}, shall be checked for welded connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

7.0 Lug Angles

Lug angles are short angles used to connect the gusset and the outstanding leg of the main member as shown in Fig. 10. The lug angles help to increase the efficiency of the outstanding leg of angles or channels. They are normally provided when the tension member carries a very large load. Higher load results in a larger end connection which can be reduced by providing lug angles. It is ideal to place the lug angle at the beginning of the connection than at any other position.

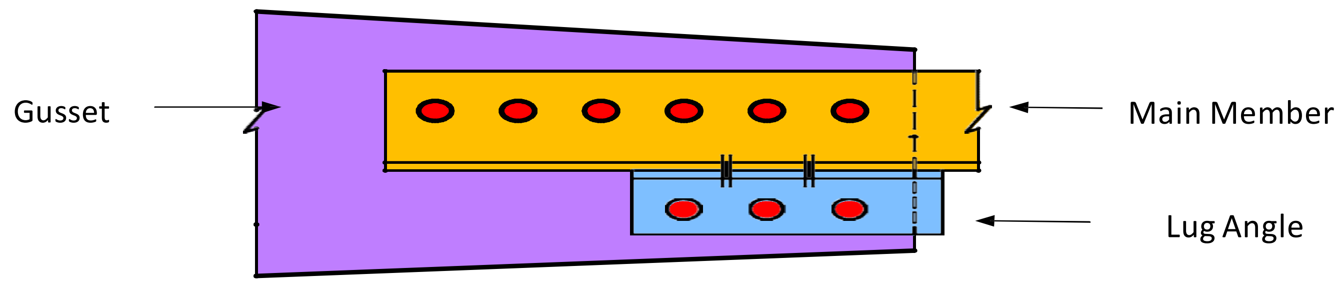


Fig. 10 Lug angle connecting Main member with Gusset

8.0 Numercial Problems

Problem 1

Determine the design tensile strength of the plate 120 mm x 8 mm connected to a 12 mm thick gusset plate with bolt holes as shown in Fig. 11. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa. The diameter of the bolts used is 16 mm.

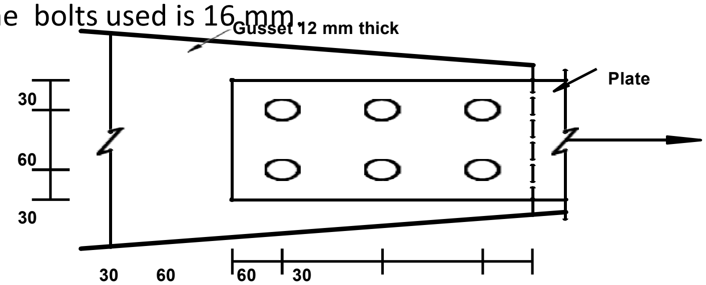


Fig. 11 Details of end connection

Solution

The design tensile strength T_d of the plate is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of plate limited to the yielding of gross cross section A_g is given by

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

Here f_y = 250 MPa, A_g = 120 x 8 = 960 mm^2 and γ_{m0} = 1.10 Hence T_{dg} = 218.18 kN

(ii) Net section rupture

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

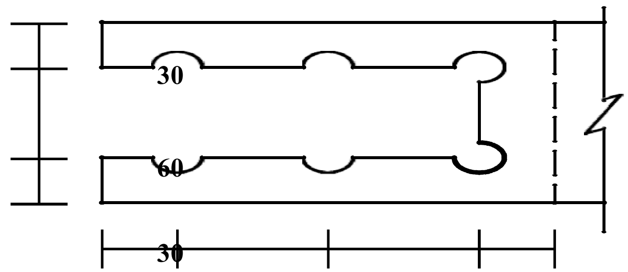
$$T_{dn} = 0.9 f_u A_n / \gamma_{m1}$$

Here f_u = 400 MPa, γ_{m1} = 1.25

Further, diameter of bolt hole = 16 + 2 = 18 mm

Therefore, A_n = (120 – 2 x18) 8 = 672 mm^2. Hence, T_{dn} = 193.54 kN

(iii) Block shear failure



30 60 60 30

Fig. 12 Failure of plate in block shear

The design strength T_{dg} of connection shall be taken as smaller of

$$T_{db1} = (A_{vg} f_y /(\sqrt{3}\, \gamma_{m0}) + 0.9\, A_{tn} f_u /\gamma_{m1}) \text{, OR}$$

$$T_{db2} = (0.9\, A_{vn} f_u /(\sqrt{3}\, \gamma_{m1}) + A_{tg} f_y /\gamma_{m0})$$

Here, $A_{vg} = (150 \text{ x } 8)\, 2 = 2400\ \text{mm}^2$,

$A_{vn} = [(150 - 2.5 \text{ x } 18) \text{ x } 8]\, 2 = 1680\ \text{mm}^2$,

$A_{tg} = (60 \text{ x } 8) = 480\ \text{mm}^2$,

$A_{tn} = (60 - 1.0 \text{ x } 18) \text{ x } 8 = 336\ \text{mm}^2$

Therefore, $T_{db1} = 411.69$ kN and $T_{db2} = 388.44$ kN Hence $T_{db} = 388.44$ kN

Design tensile strength T_d

The tensile design strength T_d is the least of T_{dg}, T_{dn} and T_{db}

Hence, $T_d = T_{dn} = 193.54$ kN

Problem 2

A single unequal angle 100 x 75 x 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 numbers of 20 mm diameter bolts to transfer tension as shown in Fig. 13. Determine the design tensile strength of the angle if the gusset is connected to the 100 mm leg. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa. The diameter of the bolts used is 20 mm.

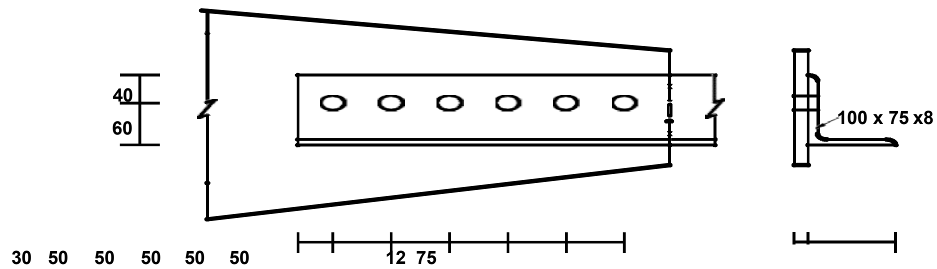


Fig. 13 Details of end connection

Solution

The design tensile strength T_d of the angle is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of angle limited to the yielding of gross cross section A_g is given by

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

Here f_y = 250 MPa, A_g = (100 + 75 – 8) 8 = 1336 mm^2, γ_{m0} = 1.10

Hence T_{dg} = 303.64 kN

(ii) Net section rupture

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

$$T_{dn} = 0.9\, f_u A_{nc} / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

$$\beta = 1.4 - 0.076\, (w/t)\, (f_y/f_u)\, (b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$$

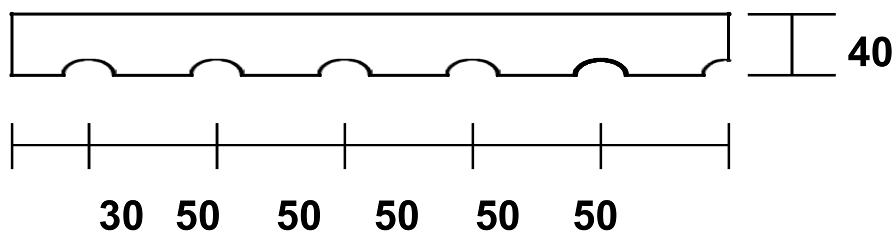
Here f_u = 400 MPa, f_y = 250 MPa, γ_{m1} = 1.25, and γ_{m0} = 1.10

w = 75 mm, t = 8 mm, b_s = (75 + 60 – 8) = 127 mm, L_c = 250 mm Further, diameter of bolt hole = 20 + 2 = 22 mm.

A_{nc} = (100 – 8/2 – 22) 8 = 592 mm^2, A_{go}= (75 – 8/2) 8 = 568 mm^2
Hence, β = 1.17. Since $0.7 \le \beta \le 1.41$, β = 1.17

Hence, T_{dn} = 321.53 kN

(iii) <u>Block shear failure</u>



<u>Fig. 14 Failure of plate in block shear</u>

The design strength T_{dg} of connection shall be taken as smaller of

$T_{db1} = (A_{vg} f_y /(\sqrt{3}\, \gamma_{m0}) + 0.9\, A_{tn} f_u / \gamma_{m1})$, OR

$T_{db2} = (0.9\, A_{vn} f_u /(\sqrt{3}\, \gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$ Here, A_{vg}

= 280 x 8 = 2240 mm^2,

A_{vn} = (280 – 5.5 x 22) x 8 = 1272 mm^2,

A_{tg} = 40 x 8 = 320 mm^2,

A_{tn} = (40 – 0.5 x 22) 8 = 232 mm^2

Therefore, T_{db1} = 360.74 kN and T_{db2} = 284.23 kN Hence T_{db} = 284.23 kN

Design tensile strength T_d

The tensile design strength T_d is the least of T_{dg}, T_{dn} and T_{db}

Hence, $T_d = T_{db}$ = 284.23 kN

Problem 3

A tie member in a bracing system consists of two angles 75 x 75 x 6 mm bolted to a 10 mm thick gusset plate one on each side using a single row of bolts and tack bolted. Determine the tensile capacity of the member and the number of bolts required to develop full capacity of the member. The yield strength and ultimate strength of the material is 250 MPa and 410 MPa, respectively.

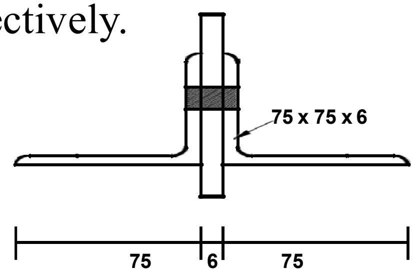


Fig. 15 Details of connection at end

Solution

The design tensile strength T_d of the angles is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of angles limited to the yielding of gross cross section A_g is given by

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

Here f_y = 250 MPa, A_g = 2 x 866 = 1732 mm^2, γ_{m0} = 1.10.

Hence T_{dg} = 393.64 kN

(ii) Net section rupture

The design strength T_{dn} of angle governed by rupture of net cross sectional area. Since the number of rivets is not known, the rupture strength of net section is approximately calculated as

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

Assuming a single line of 4 numbers 20 mm dia bolts, α = 0.8. Dia of the bolt hole = 20 + 2 = 22mm

A_n = [(75 – 22 – 6/2) x 6 + (75 – 6/2) x 6] x 2 = 1464 mm^2

Also, f_u = 410 MPa and γ_{m1} = 1.25

Hence, T_{dn} = 384.15 kN

Design of bolts

Bolts are in double shear.

Hence, strength of single 20 mm dia bolt = 2 x 45.3 = 90.6 kN

For the strength of connection to be larger than the strength of member, Number of bolts required = 384.15 / 90.6 = 4.24

Hence provide 5 numbers of 20 mm bolts. Hence the connection is safe.

Assume edge and end distances = 35 mm and pitch = 50 mm

(iii) Block shear failure

The design strength T_{dg} of connection shall be taken as smaller of

$$T_{db1} = \left(A_{vg} f_y / (\sqrt{3}\, \gamma_{m0}) + 0.9\, A_{tn} f_u / \gamma_{m1} \right), \text{ OR}$$

$$T_{db2} = \left(0.9\, A_{vn} f_u / (\sqrt{3}\, \gamma_{m1}) + A_{tg} f_y / \gamma_{m0} \right)$$

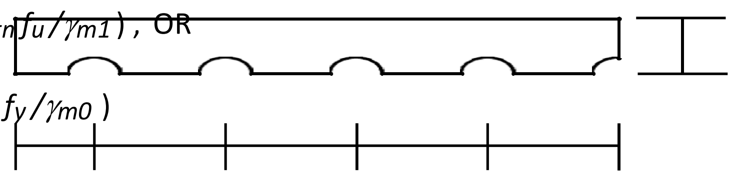


Fig. 16 Failure of angle in block shear

Consider one angle. Here,

A_{vg} = 235 x 6 = 1410 mm^2, A_{vn} = (235 – 4.5 x 22) x 6 = 816 mm^2,

A_{tg} = 35 x 6 = 210 mm^2, A_{tn} = (35 –22/2) 6 = 144 mm^2

Therefore, T_{db1} = 227.50 kN and T_{db2} = 186.80 kN Hence T_{db} = 186.80 kN

Considering the two angles, the block shear strength is

T_{db} = 186.80 x 2 = 373.60 kN

Tensile capacity of member T_d

The tensile capacity T_d is the least of T_{dg}, T_{dn} and T_{db}.

Here, T_{dg} = 393.64 kN, T_{dn} = 384.15 kN and T_{db} = 373.60 kN Hence, $T_d = T_{db}$ = 373.60 kN

Problem 4

Design a suitable angle section to carry a factored tensile force of 210 kN assuming a single row of M20 bolts. The yield strength and ultimate strength of the material is 250 MPa and 410 MPa, respectively. The length of the member is 3 m.

Solution Step 1:

Obtain the net area, A_n, required to carry the design load T_u from the equation using the ultimate stress.

$$T_u = f_u A_n / \gamma_{m1}$$

Here, T_u = 210 kN, f_u = 410 MPa, and γ_{m1} = 1.25 Therefore, A_n = 619.8 mm^2

Increase the net area, A_n, by 25 percent to obtain the gross area. Hence, A_g= 774.8 mm^2

Step 2:

Obtain the gross area, A_g, required to carry the design load T_u from the equation using the yield stress.

$$T_u = f_y A_g / \gamma_{m0}$$

Here, T_u = 210 kN, f_y = 250 MPa, and γ_{m0} = 1.10 Therefore, A_g = 924.0 mm^2

Step 3:

From steps 1 and 2,

Required gross area $A_{g,req.}$ = 924.0 mm^2 (max. value)

Select an angle 65 x 65 x 8 with A_g = 976 mm^2 (> 924.0 mm^2)

Step 4:

The strength of 20 mm diameter bolts in single shear = 45.3 kN

Hence required number of bolts = 210/45.3 = 4.64 Provide 5 bolts at a pitch of 60 mm

Step 5:

The design strength T_{dg} of plate limited to the yielding of gross cross section A_g is given by

$T_{dg} = f_y A_g / \gamma_{m0}$ Here f_y = 250 MPa, A_g = 976 mm^2 and γ_{m0} = 1.10 Hence T_{dg} = 221.80 kN

Step 6:

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

$$T_{dn} = 0.9\, f_u A_{nc} / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

$$\beta = 1.4 - 0.076\ (w/t)\ (f_y / f_u)\ (b_s / L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$$

Here f_u = 410 MPa, f_y = 250 MPa, γ_{m1} = 1.25, and γ_{m0} = 1.10

w = 65 mm, t = 8 mm, b_s = (65 + 35 – 8) = 92 mm,

L_c = 4 x 60 = 240 mm

Further, diameter of bolt hole = 20 + 2 = 22 mm

A_{nc} = (65 – 8/2 – 22) 8 = 312 mm^2, A_{go}= (65 – 8/2) 8 = 488 mm^2

Hence, β = 1.26. Since $0.7 \leq \beta \leq 1.44$, β = 1.26

Hence, T_{dn} = 231.85 kN

Step 7:

The design strength T_{dg} of connection shall be taken as smaller of

$$T_{db1} = (A_{vg} f_y / (3\ \gamma_{m0}) + 0.9\, A_{tn} f_u / \gamma_{m1})\text{ , OR}$$

$$T_{db2} = (0.9\, A_{vn} f_u / (3\ \gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$ Assuming an edge distance of 40 mm,

A_{tg} = 30 x 8 = 240 mm^2, A_{tn} = (30 – 22/2) x 8 = 152 mm^2,

Therefore, T_{db1} = 338.79 kN and T_{db2} = 301.33 kN Hence T_{db} = 301.33 kN

Step 8:

The tensile capacity of member ISA 65 x 65 x 8 with 5 bolts of 20 mm diameter is the least of T_{dg}, T_{dn} and T_{db}.

Therefore, $T = T_{dg}$ = 221.80 kN > 210 kN.

Hence the angle and the connection is safe.

Step 9:

Check for stiffness.

L = 3000 mm, r_{min} = 12.5 mm L/ r_{xx} = 240 < 250

Hence the section is safe.

Problem 5

A single unequal angle 100 x 75 x 6 mm is connected to a 8 mm thick gusset plate at the ends by 4 mm welds as shown in Fig. 17. The average length of the weld is 225 mm. Determine the design tensile strength of the angle if the gusset is connected to the 100 mm leg. The yield strength and ultimate strength of the steel used are 250 MPa and 400 MPa.

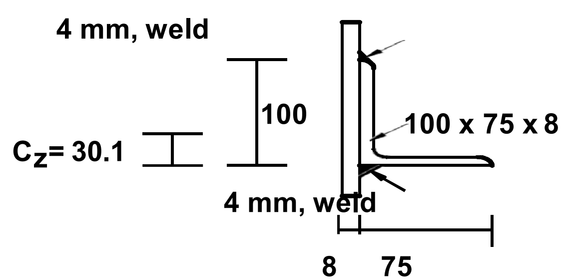


Fig. 17 Details of connection at end

Solution

The design tensile strength T_d of the angle is calculated based on the following criteria.

(i) Gross section yielding

The design strength T_{dg} of angle limited to the yielding of gross cross section A_g is given by

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

Here f_y = 250 MPa, A_g = 1010 mm^2, γ_{m0} = 1.10 Hence T_{dg} = 229.55 kN

(ii) Net section rupture

The design strength T_{dn} of plate governed by rupture of net cross sectional area is given by

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

$\beta = 1.4 - 0.076\ (w/t)\ (f_y / f_u)\ (b_s / L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$

Here f_u = 400 MPa, f_y = 250 MPa, γ_{m1} = 1.25, and γ_{m0} = 1.10

w = 75 mm, t = 6 mm, b_s = 75 mm, L_c = 225 mm

A_{nc} = (100 – 6/2) 6= 582 mm^2, A_{go}= (75 – 6/2) 6 = 432 mm^2

Hence, $\beta = 1.20.$ *Since* $0.7 \leq \beta \leq 1.41$, $\beta = 1.20$ Hence, T_{dn} = 306.39 kN

(iii) Block shear failure

The design strength T_{dg} of connection shall be taken as smaller of

$T_{db1} = (A_{vg} f_y / (\sqrt{3}\, \gamma_{m0}) + 0.9\, A_{tn} f_u / \gamma_{m1})$, OR

$T_{db2} = (0.9\, A_{vn} f_u / (\sqrt{3}\, \gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$ Here, A_{vg} = (225 x

8) 2 = 3600 mm^2,

$A_{vn} = A_{vg}$ = 3600 mm^2,

A_{tg} = 100 x 8 = 800 mm^2,

$A_{tn} = A_{tg}$ = 800 mm^2

Therefore, T_{db1} = 702.78 kN and T_{db2} = 780.41 kN

Hence T_{db} = 702.78 kN

Design tensile strength T_d

The tensile design strength T_d is the least of T_{dg}, T_{dn} and T_{db}.

Hence, T_d = T_{dg} = 229.55 kN Proportioning of

weld

Tensile capacity = 229.55 kN, Capacity of 4 mm weld = 0.53 kN/mm Hence,

Length of weld on upper side of angle = (229.55 x 30.1/100)/0.53

= 130 mm, say 140 mm

Length of weld on bottom side of angle = (229.55 x 69.9/100)/0.53

= 302.7 mm, say 310 mm

Problem 6

A tie member of a roof truss consists of 2 ISA 100x75x8 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.

Solution Step 1:

To obtain the thickness of weld:

Working Load = 300 kN

Factored Load = 300 x 1.5 = 450 kN

At the rounded toe of the angle section,

size of weld should not exceed = ¾ x thickness s = ¾ x 8 = 6 mm

At top the thickness should not exceed

s = t – 1.5 = 8 – 1.5 = 6.5 mm

Hence provide s = 6 mm weld.

Step 2:

To obtain the total length of the weld required:

Each angle carries a factored pull of 450/2 = 225 kN Let L_w be the total length of the weld required.

Assuming normal weld, t = 0.7 x 6 mm

Design strength of the weld = L_w t f_u/√3 x 1/1.25

= L_w x 0.7 x 6 x 410/√3 x 1/1.25 Equating it to the factored load,

L_w x 0.7 x 6 x 410/√3 x 1/1.25 = 225 x 10^3 L_w = 283 mm

Step 3:

To obtain the length of top and lower weld:

Centre of gravity of the 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 lower weld. To make centre of gravity of weld to coincide with that of angle,

L_1 x31 = L_2 (100-31) L_1 = (69/31) x L_2

Required $L_1 + L_2 = 283$

L_2 ((69/31) + 1) = 283

Hence, L_2 = 87 mm L_1 =195 mm

Provide 6 mm weld of L1 =195 mm and L2 = 87 mm as shown in the Fig. 18

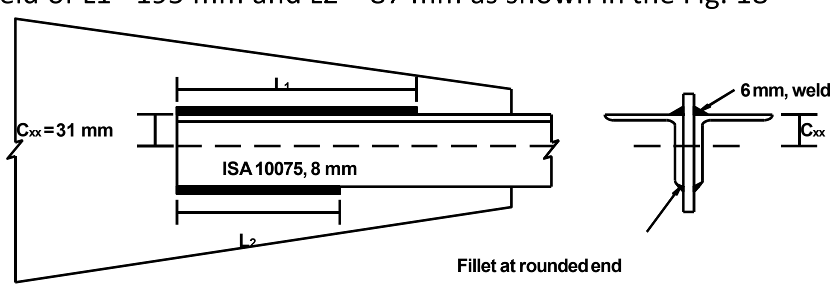


Fig. 18 Details of weld at the end connection

__Problem 7__

A tie member consists of 2 ISMC 250. The channels are connected on either side of a 12 mm thick gusset plate. Design the welded joint to develop the full strength of the tie. However the overlap is to be limited to 400 mm.

__Solution Step 1:__

Obtain the tensile design strength of each channel:

For ISMC 250, [from steel tables] Thickness of weld = 7.1 mm Thickness of flange =

14.1 mm Sectional area = 3867 mm^2

Tensile design strength of each channel = $A_g f_y / 1.1$

= 3867 x 250 /1.1

= 878864 N

__Step 2:__

Obtain the weld thickness:

Minimum thickness = 3 mm

Maximum thickness = 0.7 t =0.7 x 7.1 = 4.97 mm Provide s = 4 mm weld.

Throat thickness, t = 0.7 x 4 = 2.8 mm

__Step 3:__

Obtain the strength of weld:

Weld strength = $(L_w t\ fu/\sqrt{3}) \times 1 / \gamma_{mw}$

$= L_w \times 2.8 \times (410/\sqrt{3}) \times 1/1.25$

Equating strength of weld to tensile strength of the channel, we get $L_w \times 2.8 \times (410/\sqrt{3}) \times 1/1.25 = 878804$

i.e, $L_w = 1658$ mm

Since allowable length is limited to 400 + 400 mm it needs slot weld. The arrangement can be as shown in the Fig.19 with slots of length 'x'. Then

400 + 400 + (250-2x30) + 4x = 1658

i.e., x = 167 mm

Provide x = 170 mm as shown in the Fig. 19.

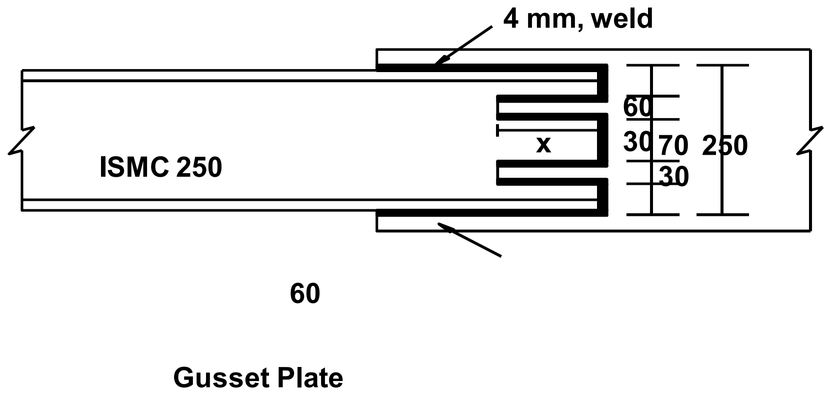


Fig. 19 Details of welding at the connection

Problem 8

A single angle member carries a factored axial force of 400 kN. Design the member and the connection with a gusset plate and a lug angle. The yield strength and ultimate strength of the material is 250 MPa and 410 MPa, respectively.

Solution

Sizing of Single Angle

Factored load = 400 kN

For preliminary sizing of single angle use the relation (Cl. 6.3.3 of IS 800 : 2007)

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

where T_{dn} = 500 kN, α = 0.8 ($\geq$ 4 bolts), f_u = 410 MPa and γ_{m1} = 1.25 Hence, Required net area is A_n = 1524.4 mm2

The gross area is arrived by increasing the net area by 15% (say) Therefore, Required gross area is A_g = 1753.1 mm2

Therefore provide ISA 125 x 75 x 10 Hence actual gross area A_g = 1902 mm^2

Here, the 125 mm side is connected to the gusset and 75 mm side is the outstanding leg.

The design strength T_{dg} of angle limited to the yielding of gross cross section A_g is given by

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

Here f_y = 250 MPa, A_g = 1902 mm^2, γ_{m0} = 1.10 Hence T_{dg} = 432.30 kN > 400 kN O.K.

Sizing of Lug Angle

Total factored load = 400 kN

Gross area of outstanding leg in single angle = [75 – (10/2)] x 10

= 700 mm2

Load carried by the outstanding leg of the single angle is proportional to its area in comparison with the total area.

Hence, load carried by outstanding leg = (700/1902) x 400 = 147.2 kN

Lug angle should be designed to take a load not less than 20% in excess of load carried by outstanding leg (Cl. 10.12.2 of IS 800 : 2007)

Hence, Load considered for lug angle = 1.2 x 147.2 = 176.64 kN

For preliminary sizing of lug angle use the relation (Cl. 6.3.3 of IS 800 : 2007) $T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$

where T_{dn} = 176.64 kN, α = 0.8 (□ 4 bolts), f_u = 410 MPa and □$_{m1}$ = 1.25 Hence, Required net area is A_n = 673.17 mm2

The gross area is arrived by increasing the net area by 15% (say) Therefore, Required gross area is A_g = 774.15 mm2

Therefore provide ISA 75 x 75 x 8 Hence actual gross area A_g = 1140 mm^2

Design of connections

Assume one row of 20 mm diameter bolt. Use a pitch of 2.5 x 20 = 50 mm, and an edge distance of 30 mm (Cl. 10.2.2 and Cl. 10.2.4.2 of IS 800 : 2007)

Strength of 20 mm bolt in single shear = 45.30 Kn

a) Connection of main angle member with gusset

Load carried by the connecting leg of the main member is proportional to its area in comparison with the total area.

Gross area of connected leg = [125 – (10/2)] x 10 = 1200 mm2

Hence, load carried by connecting leg = (1200/1902) x 400 = 252.37 kN Required number of 20 mm bolts = 252.37/45.30 = 5.57, say 6 nos.

b) <u>Connection of lug angle with gusset</u>

Load carried by outstanding leg = 147.2 kN

The connection should be designed to take a load not less than 20% in excess of load carried by outstanding leg (Cl. 10.12.2 of IS 800 : 2007)

Hence load considered in the design for connection = 1.2 x 147.2

= 176.74 kN

Required number of 20 mm bolts = 176.74/45.30 = 3.89, say 5 nos.

c) <u>Connection of main angle member with lug angle</u>

Load carried by outstanding leg = 147.2 kN

The connection should be designed to take a load not less than 40% in excess of load carried by outstanding leg (Cl. 10.12.2 of IS 800 : 2007)

Hence load considered in the design for connection = 1.4 x 147.2

= 206.08 kN

Required number of 20 mm bolts = 206.08/45.30 = 4.55, say 5 nos.

The details of the connection are shown in Fig. 20.

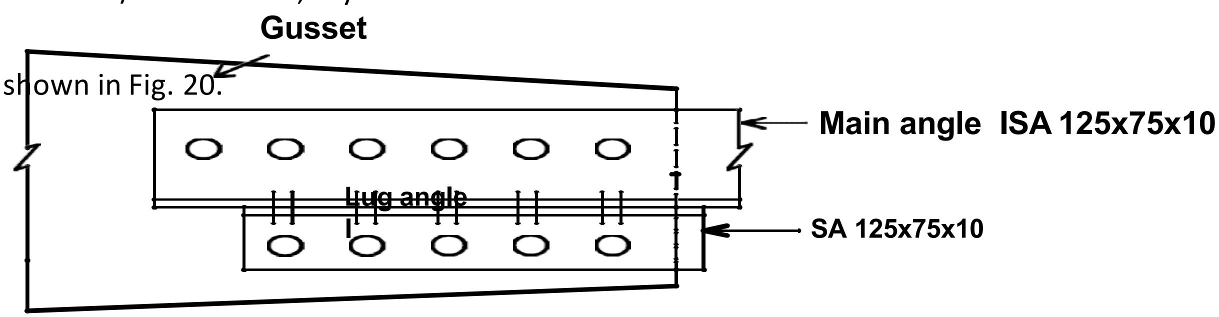


Fig. 20 Details of connection of main angle with lug angle and gusset

9.0 References

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