5 DESIGN OF TENSION MEMBERS

1.0 INTRODUCTION

Tension members are linear members in which axial forces act so as to elongate (stretch) the member. A rope, for example, is a tension member. Tension members carry loads most efficiently, since the entire cross section is subjected to uniform stress. Unlike compression members, they do not fail by buckling (see chapter on compression members). Ties of trusses [Fig 1(*a*)], suspenders of cable stayed and suspension bridges [Fig.1 (*b*)], suspenders of buildings systems hung from a central core [Fig.1(*c*)] (such buildings are used in earthquake prone zones as a way of minimising inertia forces on the structure), and sag rods of roof purlins [Fig 1(*d*)] are other examples of tension members.

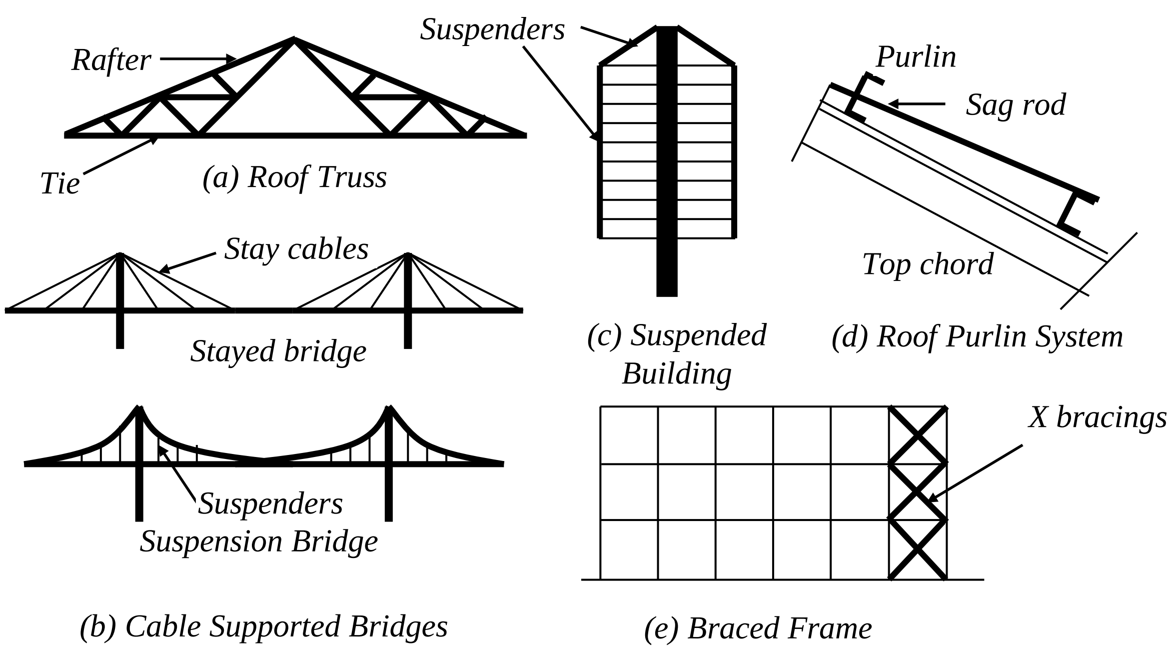

Fig. 1 Tension Members in Structures

Tension members are also encountered as bracings used for the lateral load resistance. In X type bracings [Fig.1 (*e*)] the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice versa. Hence, such members may have to be designed to resist tensile and compressive forces.

The tension members can have a variety of cross sections. The single angle and double angle sections [Fig 2(*a*)] are used in light roof trusses as in industrial buildings. The tension members in bridge trusses are made of channels or I sections, acting individually or built-up [Figs. 2(*c*) and 2(*d*)]. The circular rods [Fig.2 (*d*)] are used in bracings designed to resist loads in tension only. They buckle at very low compression and are not considered effective. Steel wire ropes [Fig.2 (*e*)] are used as suspenders in the cable suspended bridges and as main stays in the cable-stayed bridges.

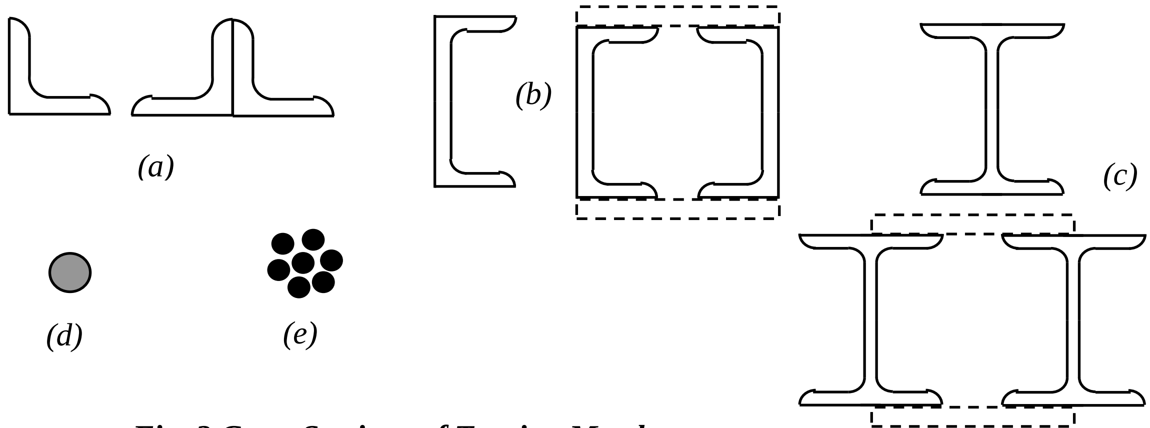

Fig. 2 Cross Sections of Tension Members

2.0 BEHAVIOUR OF TENSION MEMBERS

Since axially loaded tension members are subjected to uniform tensile stress, their load deformation behaviour (Fig.3) is similar to the corresponding basic material stress strain behaviour. Mild steel members (IS: 2062 & IS: 226) exhibit an elastic range (a-b) ending at yielding (b). This is followed by yield plateau (b-c). In the Yield Plateau the load remains constant as the elongation increases to nearly ten times the yield strain. Under further stretching the material shows a smaller increase in tension with elongation (c-d), compared to the elastic range. This range is referred to as the strain hardening range. After reaching the ultimate load (d), the loading decreases as the elongation increases (d-e) until rupture (e). High strength steel tension members do not exhibit a well-defined yield point and a yield plateau (Fig.3). The *0.2%* offset load, *T*, as shown in Fig. 3 is usually taken as the yield point in such cases.

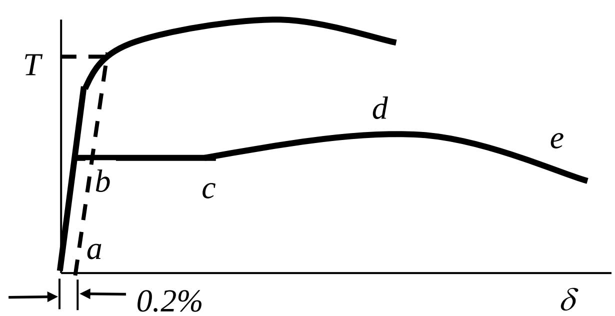

Fig. 3 Load – Elongation of Tension Members

2.1 Design strength of tension members

Although steel tension members can sustain loads up to the ultimate load without failure, the elongation of the members at this load would be nearly *10-15%* of the original length and the structure supported by the member would become unserviceable. Hence, in the design of tension members, the yield load is usually taken as the limiting load. The corresponding design strength in member under axial tension is given by

$$T_d = f_y A / \gamma_{M_0} \tag{1}$$

where, f_y is the yield strength of the material (in *MPa*), A is the gross area of cross section in mm^2 and γ_{M0} is the partial safety factor for failure in tension by yielding. The value of γ_{M0} according to IS: 800 is *1.15*.

2.2 Plates under Tension

Frequently plates under tension have bolt holes. The tensile stress in a plate at the cross section of a hole is not uniformly distributed in the elastic range, but exhibits stress concentration adjacent to the hole [Fig 4 (*a*)]. The ratio of the maximum elastic stress adjacent to the hole to the average stress on the net cross section is referred to as the Stress Concentration Factor. This factor is in the range of 2 to 3, depending upon the ratio of the diameter of the hole to the width of the plate normal to the direction of stress.

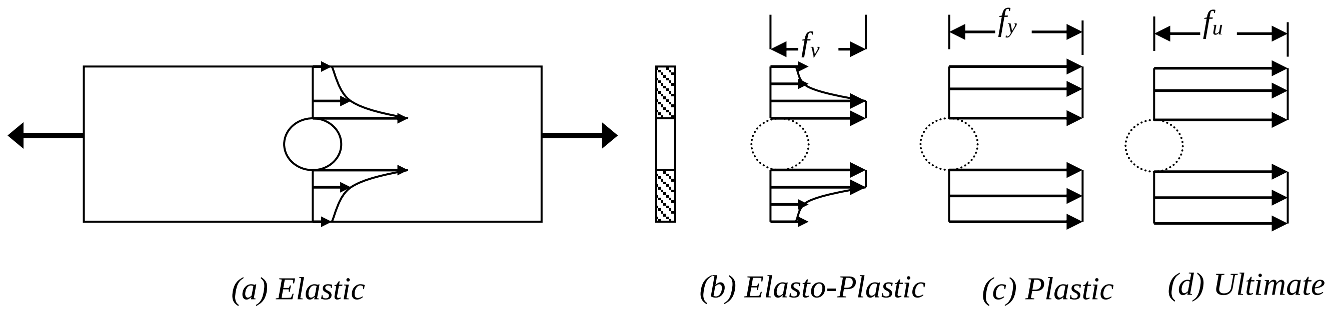

Fig. 4 Stress Distribution at a Hole in a Plate under Tension

In statically loaded tension members with a hole, the point adjacent to the hole reaches yield stress, f_y, first. On further loading, the stress at that point remains constant at the yield stress and the section plastifies progressively away from the hole [Fig.4 (*b*)], until the entire net section at the hole reaches the yield stress, f_y, [Fig. 4(*c*)]. Finally, the rupture (tension failure) of the member occurs when the entire net cross section reaches the ultimate stress, f_u, [Fig. 4(*d*)]. Since only a small length of the member adjacent to the smallest cross section at the holes would stretch a lot at the ultimate stress, and the overall member elongation need not be large, as long as the stresses in the gross section is below the yield stress. Hence, the design strength as governed by net cross-section at the hole, T_{dn}, is given by

$$P_{tn} = 0.9 \; f_u \; A_n / \gamma_{M1} \tag{2}$$

where, f_u is the ultimate stress of the material, A_n is the net area of the cross section after deductions for the hole [Fig.4(b)] and γ_{M1} is the partial safety factor against ultimate tension failure by rupture ($\gamma_{M1} = 1.25$). Similarly threaded rods subjected to tension could fail by rupture at the root of the threaded region and hence net area, A_n, is the root area of the threaded section (Fig.5).

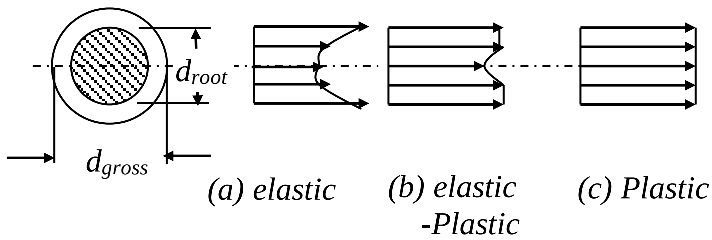

Fig. 5 Stress in a threaded Rod

The design tension of the plates with hole or the threaded rod could also be governed by yielding of the gross cross section beyond the thread (with area equal to A_g) above which the member deformation becomes large and objectionable and the corresponding design load is given by

$$P_{tg} = f_y \; A_g / \gamma_{M_0} \tag{3}$$

where, $\gamma_{M0}=1.15$. The lower value of the design tension capacities, as given by Eqn. 2 and 3, governs the design strength of a plate with holes.

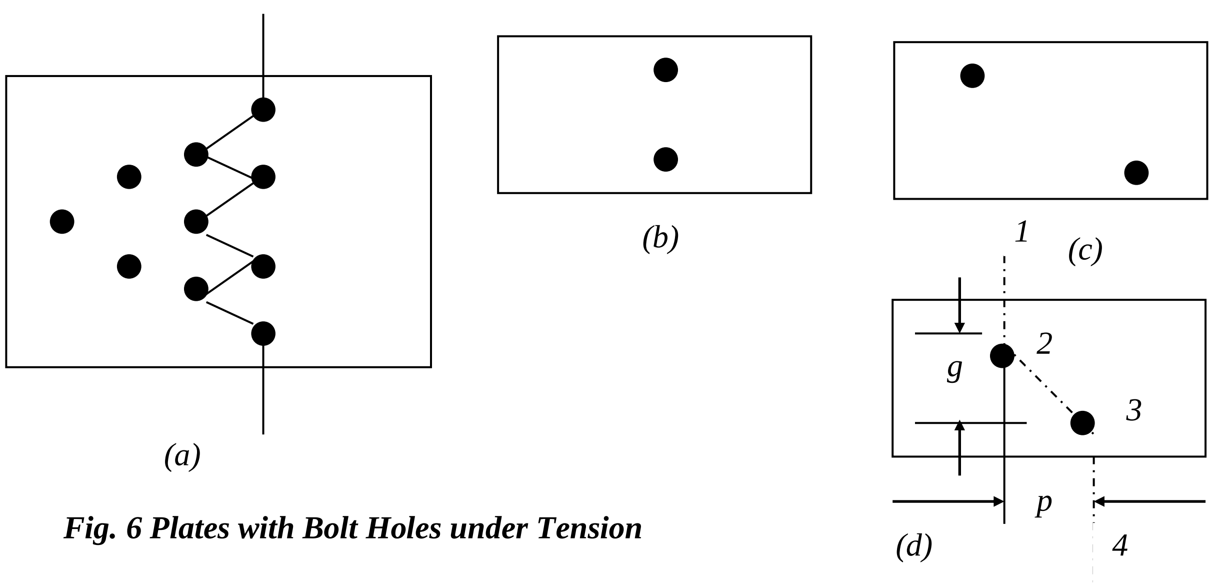

Fig. 6 Plates with Bolt Holes under Tension

Frequently, plates have more than one hole for the purpose of making connections. These holes are usually made in a staggered arrangement [Fig.6 (*a*)]. Let us consider the two extreme arrangements of two bolt holes in a plate, as shown in Fig.6 (*b*) & 6(*c*). In the case of the arrangement shown in Fig.6 (*b*), the gross area is reduced by two bolt holes to obtain the net area. Whereas, in arrangement shown in Fig.6c, deduction of only one hole is necessary, while evaluating the net area of the cross section.

Obviously the change in the net area from the case shown in Fig.6(*c*) to Fig.6 (*b*) has to be gradual. As the pitch length (the centre to centre distance between holes along the direction of the stress) p, is decreased, the critical cross section at some stage changes from straight section [Fig.6(*c*)] to the staggered section 1-2-3-4 [Fig.6 (*d*)]. At this stage, the net area is decreased by two bolt holes along the staggered section, but is increased due to the inclined leg (2-3) of the staggered section. The net effective area of the staggered section 1-2-3-4 is given by

$$A_n = (b - 2d + p^2/4g)\,t \tag{4}$$

where, the variables are as defined in Fig.6(*d*). In Eqn. 4 the increase of net effective area due to inclined section is empirical and is based on test results. It can be seen from Eqn.4, that as the pitch distance, p, increases and the gauge distance, g, decreases, the net effective area corresponding to the staggered section increases and becomes greater than the net area corresponding to single bolt hole. This occurs when

$$p^2/4g > d \tag{5}$$

When multiple holes are arranged in a staggered fashion in a plate as shown in Fig.6 (a), the net area corresponding to the staggered section in general is given by

$$A_{net} = \left(b - nd + \sum \frac{p^2}{4g}\right) t \tag{6}$$

where, n is the number of bolt holes in the staggered section [n = 7 for the zigzag section in Fig. 6(*a*)] and the summation over $p^2/4g$ is carried over all inclined legs of the section [equal to $n-1$ = 6 in Fig.6(*a*)]. Normally, net area of different staggered and straight sections have to be evaluated to obtain the minimum net area to be used in calculating the design strength in tension. An example analysis of a plate with holes under tension is illustrated in Appendix I.

2.3 ANGLES UNDER TENSION

Angles are extensively used as tension members in trusses and bracings. Angles, if axially loaded through centroid, could be designed as in the case of plates. However, usually angles are connected to gusset plates by bolting or welding only one of the two legs (Fig. 7).

This leads to eccentric tension in the member, causing non-uniform distribution of stress over the cross section. Further, since the load is applied by connecting only one leg of the member there is a shear lag locally at the end connections.

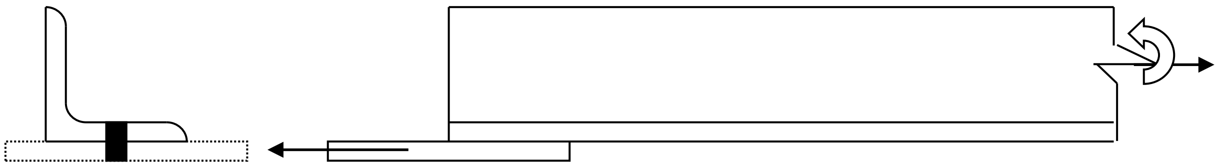

Fig. 7 Angles Eccentrically Loaded through Gussets

Kulak and Wu (1997) have reported, based on an experimental study, the results on the tensile strength of single and double angle members. Summary of their findings is:

- The effect of the gusset thickness, and hence the out of plane stiffness of the end connection, on the ultimate tensile strength is not significant.
- The thickness of the angle has no significant influence on the member strength.
- The effect of shear lag, and hence the strength reduction, is higher when the ratio of the area of the outstanding leg to the total area of cross-section increases.
- When the length of the connection (the number of bolts in end connections) increases, the tensile strength increases up to 4 bolts and the effect of further increase in the number of bolts, on the tensile strength of the member is not significant. This is due to the connection restraint to member bending caused by the end eccentric connection.
- Even double angles connected on opposite sides of a gusset plate experience the effect of shear lag.

Based on the test results, Kulak and Wu (1997) found that the shear lag due to connection through one leg only causes at the ultimate stage the stress in the outstanding leg to be closer only to yield stress even though the stress at the net section of the connected leg may have reached ultimate stress. They have suggested an equation for evaluating the tensile strength of angles connected by one leg, which accounts for various factors that significantly influence the strength. In order to simplify calculations, this formula has suggested that the stress in the outstanding leg be limited to f_y (the yield stress) and the connected sections having holes to be limited to f_u (the ultimate stress). The design tensile strength, T_d, should be the minimum of the following:

<u>Strength as governed by tearing at net section:</u>

$$P_{tn} = A_{nc} f_u / \gamma_{M1} + \beta A_o f_y / \gamma_{M0} \qquad (7a)$$

where, f_y and f_u are the yield and ultimate stress of the material, respectively. A_{nc} and A_o, are the net area of the connected leg and the gross area of the outstanding leg, respectively. The partial safety factors $\gamma_{M0} = 1.15$ and $\gamma_{M1} = 1.25$. β accounts for the end fastener restraint effect and = 1.0, if the number of fasteners is ≥ 4, $\beta = 0.75$ if the number of fasteners = 3 and $\beta = 0.5$, if number of fasteners = 1 or 2. In case of welded connection, $\beta = 1.0$.

Strength as governed by yielding of gross section:

$P_{tg} = A_g \ f_y/\gamma_{M0}$ (7 b)

where, A_g is the gross area of the angle section.

Strength as governed by block shear failure:

A tension member may fail along end connection due to block shear as shown in Fig. 8. The corresponding design strength can be evaluated using the following equations. If the centroid of bolt pattern is not located between the heel of the angle and the centerline of the connected leg, the connection shall be checked for block shear strength given by

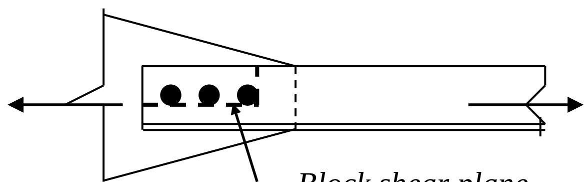

Fig. 8 Block Shear Failure

$$P_{tv} = (0.62\ A_{vg}\ f_y/\gamma_{M0} + A_{tn}\ f_u/\gamma_{M1})$$

or

$$P_{tv} = (0.62\ A_{vn}\ f_u/\gamma_{M1} + A_{tg}\ f_y/\gamma_{M0}) \quad (7c)$$

where, A_{vg} and A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively, and A_{tg} and A_{tn} = minimum gross and net area in tension from the hole to the toe of the angle, perpendicular to the line of force, respectively.

The design strength of an angle loaded in tension through a connection in one leg is given by the smallest of the values obtained from Eqns. 7(*a*) to 7(*c*). These equations are valid for both single angle and double angles in tension, irrespective of whether they are on the same side or opposite sides of the gusset. A sample design of angle tension member is given in worked example 2.

The efficiency, η, of an angle tension member is calculated as given below:

$$\eta = P_t / (A_g\ f_y / \gamma_{M_0}) \quad (8)$$

Depending upon the type of end connection and the configuration of the built-up member, the efficiency may vary between 0.85 and 1.0. The higher value of efficiency is obtained in the case of double angles on the opposite sides of the gusset connected at the ends by welding and the lower value is usual in the bolted single angle tension members. In the case of threaded members the efficiency is around 0.85.

In order to increase the efficiency of the outstanding leg in single angles and to decrease the length of the end connections, some times a short length angle at the ends are connected to the gusset and the outstanding leg of the main angle directly, as shown in Fig. 9. Such angles are referred to as lug angles. The design of such end connections is discussed in the chapter on connections.

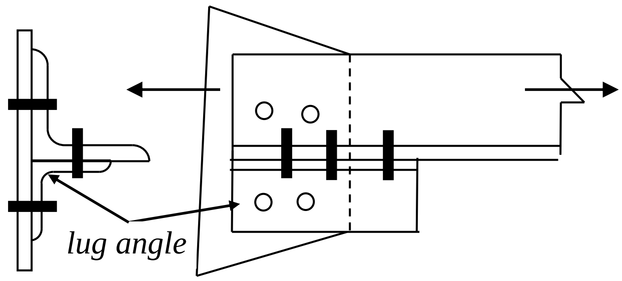

Fig. 9 Tension Member with Lug

3.0 DESIGN OF TENSION MEMBERS

In the design of a tension member, the design tensile force is given and the type of member and the size of the member have to be arrived at. The type of member is usually dictated by the location where the member is used. In the case of roof trusses, for example, angles or pipes are commonly used. Depending upon the span of the truss, the location of the member in the truss and the force in the member either single angle or double angles may be used in roof trusses. Single angle is common in the web members of a roof truss and the double angles are common in rafter and tie members of a roof truss.

Plate tension members are used to suspend pipes and building floors. Rods are also used as suspenders and as sag rods of roof purlins. Steel wires are used as suspender cables in bridges and buildings. Pipes are used in roof trusses on aesthetic considerations, in spite of fabrication difficulty and the higher cost of such tubular trusses. Built-up members made of angles, channels and plates are used as heavy tension members, encountered in bridge trusses.

3.1 Trial and Error Design Process

The design process is iterative, involving choice of a trial section and analysis of its capacity. This process is discussed in this section. Initially, the net effective area required is calculated from the design tension and the ultimate strength of the material as given below.

$$A_n = P_t / (f_u / \gamma_{M_1}) \qquad (9)$$

Using the net area required, the gross area required is calculated, allowing for some assumed number and size of bolt holes in plates, or assumed efficiency index in the case of angles and threader rods. The gross area required is also checked against that required from the yield strength of the gross sections as given below.

$$A_g = P_t / (f_y / \gamma_{M_0}) \qquad (10)$$

A suitable trial section is chosen from the steel section handbook to meet the gross area required. The bolt holes are laid out appropriately in the member and the member is analysed to obtain the actual design strength of the trial section. The design strength of the trial section is evaluated using Eqs. 1 to 6 in the case of plates and threaded bars and using Eqs. 7 in the case of angle ties. If the actual design strength is smaller than or too large compared to the design force, a new trial section is chosen and the analysis is repeated until a satisfactory design is obtained.

3.2 Stiffness Requirement

The tension members, in addition to meeting the design strength requirement, frequently have to be checked for adequate stiffness. This is done to ensure that the member does not sag too much during service due to self-weight or the eccentricity of end plate connections. The IS: 800 imposes the following limitations on the slenderness ratio of members subjected to tension:

(a) In the case of members that are normally under tension but may experience compression due to stress reversal caused by wind / earthquake loading $\ell/r \leq 250$.

(b) In the case of members that are designed for tension but may experience stress reversal for which it is not designed (as in X bracings) $\ell/r \leq 350$.

(c) In the case of members subjected to tension only $\ell/r \leq 400$

In the case of rods used as a tension member in X bracings, the slenderness ratio limitation need not be check for if they are pretensioned by using a turnbuckle or other such arrangement.

4.0 SUMMARY

The behaviour and design of various types of tension members were discussed. The important factors to be considered while evaluating the tensile strength are the reduction in strength due to bolt holes and due to eccentric application of loads through gusset plates attached to one of the elements. It was shown that the yield strength of the gross area or the ultimate strength of the net area may govern the tensile strength. The effect of connecting the end gusset plate to only one of the elements of the cross section was empirically accounted for by the reduction in the effectiveness of the out standing leg, while calculating the net effective area. The methods for accounting for these factors in the design of tension members were discussed. The iterative method of design of tension members was presented.

5.0 REFERENCES

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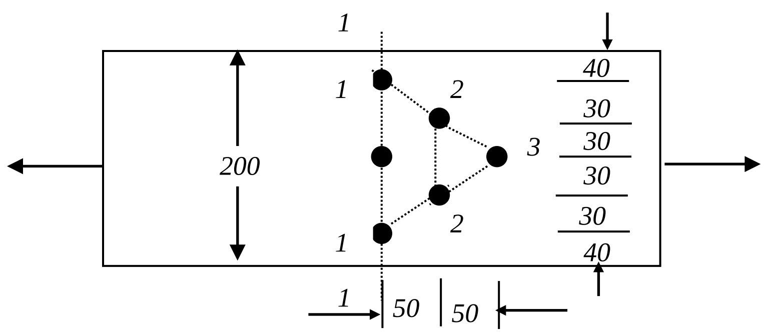
PROBLEM 1:

Determine the desig tensile strength of the plate (200 X 10 mm) with the holes as shown below, if the yield strength and the ultimate strength of the steel used are 250 MPa and 420 MPa and 20 mm diameter bolts are used.

$f_y = 250$ MPa

$f_u = 420$ MPa



Calculation of net area, A_{net}:

$$A_n \text{ (section 11)} = (200 - 3*21.5)*10 = 1355\,mm^2 \text{ (governs)}$$

$$A_n \text{ (section 1221)} = \left(200 - 4*21.5 + \frac{2*50^2}{4*30}\right)*10 = 1557\,mm^2$$

$$A_n \text{ (section 12321)} = \left(200 - 5*21.5 + \frac{4*50^2}{4*30}\right)*10 = 1758\,mm^2$$

P_t is lesser of

$$\text{(i) } A_g\, f_y / \gamma M_0 = \frac{200*10*250/1.15}{1000} = 434.8\,kN$$

$$\text{(ii) } 0.9\,A_n\, f_u / \gamma\, M_1 = \frac{0.9 * 1342*420/1.25}{1000} = 405.8\,kN$$

$$P_t = 405.8\ kN$$

Efficiency of the plate with holes $= \dfrac{P_t}{A_g\, f_y/\gamma_{M_0}} = \dfrac{409.8}{434.8} = 0.93$

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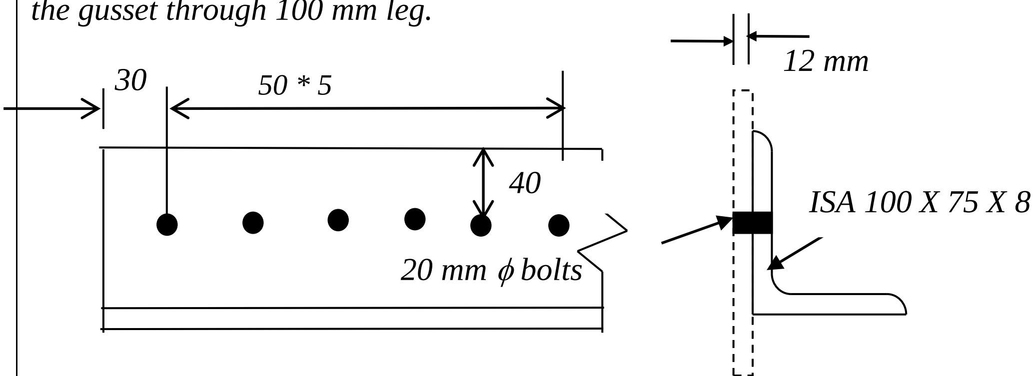
PROBLEM 2:

Analysis of single angle tension members

A single unequal angle 100 X 75 X 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 nos. 20 mm diameter bolts to transfer tension. Determine the design tensile strength of the angle. (a) if the gusset is connected to the 100 mm leg, (b) if the gusset is connected to the 70 mm leg, (c) if two such angles are connected to the same side of the gusset through the 100 mm leg. (d) if two such angles are connected to the opposite sides of the gusset through 100 mm leg.



a) *The 100mm leg bolted to the gusset :*

A_{nc} = (100 - 8/2 - 21.5) *8 = 596 mm^2.

A_o = (75 - 8/2) * 8 = 568.mm^2

A_g = ((100-8/2) + (75 – 8/2)) * 8 =1336 mm^2

Strength as governed by tearing of net section:

Since the number of bolts = 4; β = 1.0

$$P_t = A_{nc} f_u/\gamma_{M1} + \beta A_o f_y/\gamma_{M0}$$

$$= 596 * 420/1.25 + 1.0 * 568 * 250 / 1.15$$

$$= 323734 \text{ N (or) } 323.7 \text{ kN}$$

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Strength as governed by yielding of gross section:

$P_t = A_g f_y/\gamma_{M0}$

$= 1336 * 250 / 1.15 = 290435$ N (or) 290.4 kN

Block shear strength

$P_v = (0.62\, A_{vg}\ f_y/\gamma_{M0} + A_{tn}\ f_u/\gamma_{M1})$

$= 0.62 * (5 * 50 + 30) * 8 * 250/1.15 + (40 - 21.5/2) * 8 * 420/1.25$

$= 380537$ N $= 380.5$ kN

or

$P_v = (0.62\, A_{vn}\ f_u/\gamma_{M1} + A_{tg}\ f_y/\gamma_{M0})$

$= (0.62\,(5 * 50 + 30 - 5.5 * 21.5) * 8 * 420 / 1.25$

$+ 40 * 8 * 250/ 1.15)$

$= 339131$ N $= 339.1$

The design tensile strength of the member = 290.4 kN

The efficiency of the tension member, is given by

$$\eta = \frac{P_t}{A_g\ f_y} = \frac{290.4 * 1000}{(100 + 75 - 8) * 8 * 250/1.15} = 1.0$$

b) *The 75 mm leg is bolted to the gusset:*

$A_{nc} = (75 - 8/2 - 21.5) * 8 = 396\ mm^2$

$A_o = (100 - 8/2) * 8 = 768\ mm^2$

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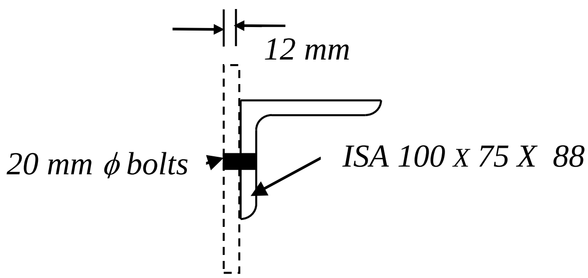

__Strength as governed by tearing of net section:__

Since the number of bolts = 6 $\beta = 1.0$

$$P_t = A_{nc} f_u/\gamma_{M1} + \beta A_o f_y/\gamma_{M0}$$

$$= 396 * 420/1.25 + 1.0 * 768 * 250 / 1.15$$

$$= 300123 \text{ N (or) } 300.1 \text{ kN}$$

__Strength as governed by yielding of gross section:__

$$P_t = A_g f_y/\gamma_{M0}$$

$$= 1336 * 250 / 1.15 = 290435 \text{ N (or) } 290.4 \text{ kN}$$

__Block shear strength:__

$$P_v \le (0.62 A_{vg} f_y/\gamma_{M0} + A_{tn} f_u/\gamma_{M1})$$

$$= 0.62 * (5 * 50 + 30) * 8 * 250/1.15$$

$$+ (35 - 21.5/2) * 8 * 420/1.25$$

$$= 367097 \text{ N} = 367.1 \text{ kN}$$

$$P_v \le (0.62 A_{vn} f_u/\gamma_{M1} + A_{tg} f_y/\gamma_{M0})$$

$$= (0.62 (5 * 50 + 30 - 5.5 * x\ 21.5) * 8 * 420 / 1.25$$

$$+ 35 * 8 * 250/ 1.15$$

$$= 330435 \text{ N} = 330.4 \text{ kN}$$

The design tensile strength of the member = 290.4 kN

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Even though the tearing strength of the net section is reduced, the yielding of the gross section still governs the design strength.

The efficiency of the tension member is as before 1.0

Note: *The design tension strength is more some times if the longer leg of an unequal angle is connected to the gusset (when the tearing strength of the net section governs the design strength).*

An understanding about the range of values for the section efficiency, η, is useful to arrive at the trial size of angle members in design problems.

(c & d)The double angle strength would be twice single angle strength as obtained above in case (a)

$P_t = 2 * 290.4 = 580.8\ kN$