## 4.2 Behaviour of tension members

Since axially loaded tension members are subjected to uniform tensile stress, their load deformation behaviour (Fig.4.3) 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.4.3). The 0.2% offset load, T, as shown in Fig.4.3 is usually taken as the yield point in such cases.

**Load-elongation of tension member to view click here**

**Fig. 4.3 Load – elongation of tension members**

### 4.2.1 Design strength due to yielding of gross section

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 (C1.62)

$$T_d = f_y A / \gamma_{mO} \qquad \textbf{\textit{(4.1)}}$$

Where, $f_y$ is the yield strength of the material (in MPa), A is the gross area of cross section in $mm^2$ and $\gamma_{mO}$ is the partial safety factor for failure in tension by yielding. The value of $\gamma_{mO}$ according to IS: 800 is 1.10.

## 4.2.2 Design strength due to rupture of critical section

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 Tension Member: Behaviour of Tension Members elastic range, but exhibits stress concentration adjacent to the hole [Fig 4.4(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.

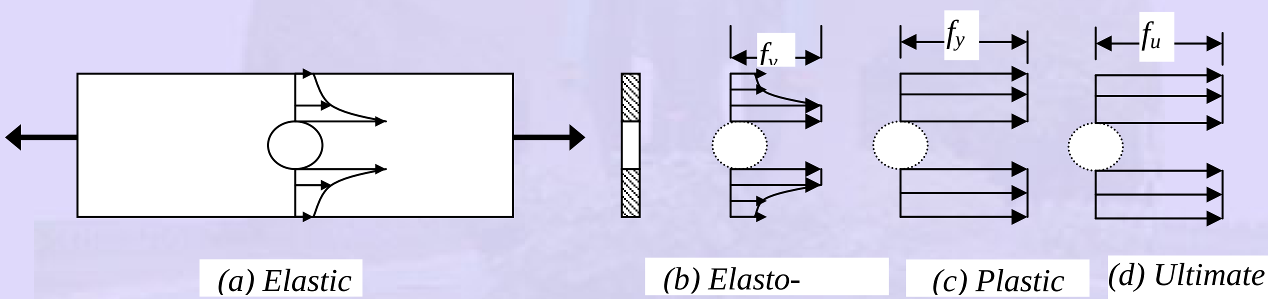


**Fig. 4.4 Stress distribution at a hole in a plate under tension**

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.4.4(b)], until the entire net section at the hole reaches the yield stress, $f_y$, [Fig.4.4(c)]. Finally, the rupture (tension failure) of the member occurs when the entire net cross section reaches the ultimate stress, $f_u$, [Fig.4.4 (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 (C1.6.3)

$$P_{tn} = 0.9 f_u A_n / \gamma_{m1} \qquad \textbf{(4.2)}$$

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.4.4 (b)] and $\gamma_{m1}$ 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.4.5).

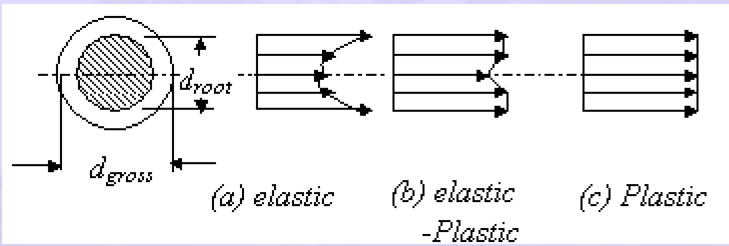


**Fig 4.5 Stress in a threaded rod**

The lower value of the design tension capacities, as given by Eqn.4.1 and 4.2, governs the design strength of a plate with holes.

Frequently, plates have more than one hole for the purpose of making connections. These holes are usually made in a staggered arrangement [Fig.4.6 (a)]. Let us consider the two extreme arrangements of two bolt holes in a plate, as shown in Fig.4.6 (b) & 4.6(c). In the case of the arrangement shown in Fig.4.6 (b), the gross area is reduced by two bolt holes to obtain the net area. Whereas, in arrangement shown in Fig.4.6c, 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.4.6(c) to Fig.4.6 (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.4.6(c)] to the staggered section 1-2-3-4 [Fig.4.6(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

$$A_n = \left(b - 2d + p^2 / 4g\right)t \qquad \textbf{(4.3)}$$

Where, the variables are as defined in Fig.4.6 (a). In Eqn.4.3 the increase of net effective area due to inclined section is empirical and is based on test results. It can be seen from Eqn.4.3 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

$$p^2 / 4g > d \qquad \textbf{(4.4)}$$

When multiple holes are arranged in a staggered fashion in a plate as shown in Fig.4.6 (a), the net area corresponding to the staggered section in general is given by

$$A_{net} = \left(b - nd + \sum \frac{p^2}{4g}\right)t \qquad \textbf{(4.5)}$$

Where, n is the number of bolt holes in the staggered section [n = 7 for the zigzag section in Fig.4.6 (a)] and the summation over $p^2/4g$ is carried over all inclined legs of the section [equal to n-1 = 6 in Fig.4.6 (a)].

Normally, net areas 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.

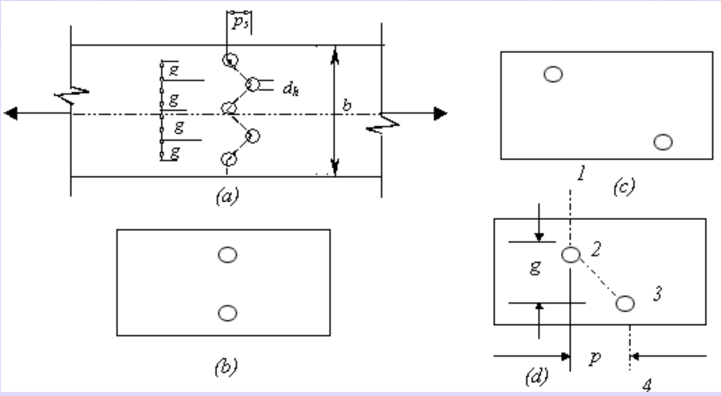


**Fig 4.6 Plates with bolt hole under tension**

### 4.2.3 Design strength due to block shear

A tension member may fail along end connection due to block shear as shown in Fig.4.7. The corresponding design strength can be evaluated using the following equations. The block shear strength $T_{db}$, at an end connection is taken as the smaller of (C1.64)

$$T_{db} = \left( A_{vg} f_y / \left( \sqrt{3}\, \gamma_{m0} \right) + f_u\, A_{tn} / \gamma_{m1} \right) \qquad \textbf{(4.6)}$$

or

$$T_{db} = \left( f_u\, A_{vn} / \left( \sqrt{3}\, \gamma_{m1} \right) + f_y\, A_{tg} / \gamma_{m0} \right) \qquad \textbf{(4.7)}$$

Where, $A_{vg}$, $A_{vn}$ = minimum gross and net area in shear along a line of transmitted force, respectively (1-2 and 4 –3 as shown in Fig 4.6 and 1-2 as shown in Fig 4.7), $A_{tg}$, $A_{tn}$ = minimum gross and net area in tension from the hole to the toe of the angle or next last row of bolt in plates, perpendicular to the line of force, respectively (2-3) as shown in Fig 4.7 and $f_u$, $f_y$ = ultimate and yield stress of the material respectively

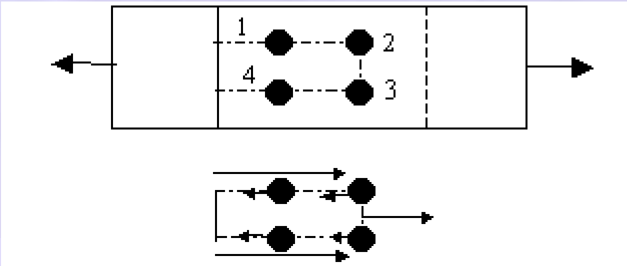


**Fig 4.7 Block shearing failure plates**

### 4.2.4 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. 4.8). 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.

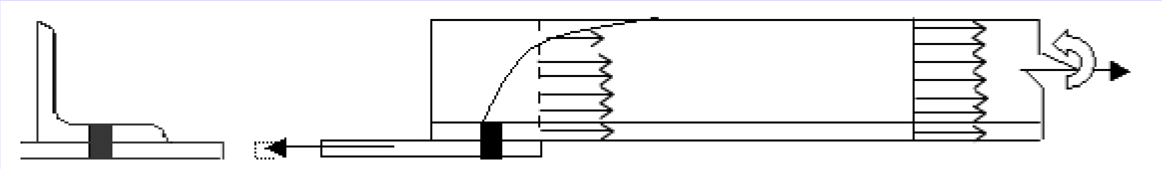

**Fig 4.8 Angles eccentrically loaded through gussets**

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 effects 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 strength of an angle connected by one leg as governed by tearing at the net section is given by (C1.6.3.3)

$$T_{tn} = \left( A_{nc} f_u / \gamma_{m1} + \beta A_0 f_y / \gamma_{m0} \right) \qquad \textbf{\textit{(4.8)}}$$

Where, $f_y$ and $f_u$ are the yield and ultimate stress of the material, respectively. $An_{nc}$ and $A_o$, are the net area of the connected leg and the gross area of the outstanding leg, respectively. The partial safety factors $\gamma_{m0} = 1.10$ and $\gamma_{m1} = 1.25$ $\beta$ accounts for the end fastener restraint effect and is given by

$$\beta = 1.4 - 0.035 (w/t)(f_u / f_y)(b_s / L) \qquad \textbf{\textit{(4.9)}}$$

Where w and $b_s$ are as shown in Fig 4.9. L = Length of the end connection, i.e., distance between the outermost bolts in the joint along the length direction or length of the weld along the length direction

Alternatively, the tearing strength of net section may be taken as

$$T_{dn} = \alpha A_n f_u / \gamma_{m1} \qquad \textbf{\textit{(4.10)}}$$

Where, $\alpha = 0.6$ for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts in the end connection or equivalent weld length, $A_n$ = net area of the total cross section, $A_{nc}$= net area of connected leg, $A_{go}$= gross area of outstanding leg, t = thickness of the leg.

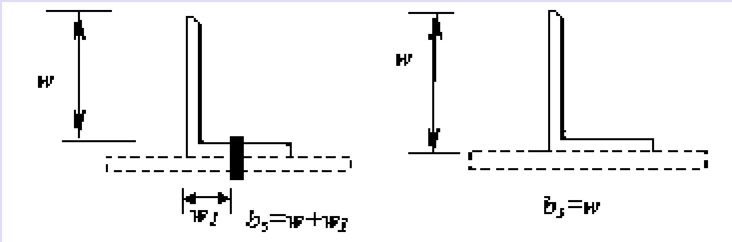

**Fig 4.9 Angles with ended connections**

$\beta = 1.0$, if the number of fasteners is $\leq 4$,

$\beta = 0.75$ if the number of fasteners = 3 and

"Above is not recommended in code anywhere" $\beta = 0.5$, if number of fasteners = 1 or 2.

In case of welded connection, $\beta = 1.0$

The strength $\eta$ as governed by yielding of gross section and block shear may be calculated as explained for the plate. The minimum of the above strengths will govern the design.

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

$$\eta = F_d / \left( A_g f_y / \gamma_{m0} \right) \qquad \textbf{\textit{(4.11)}}$$

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.4.10. Such angles are referred to as lug angles. The design of such connections should confirm to the codal provisions given in C1.10.12.

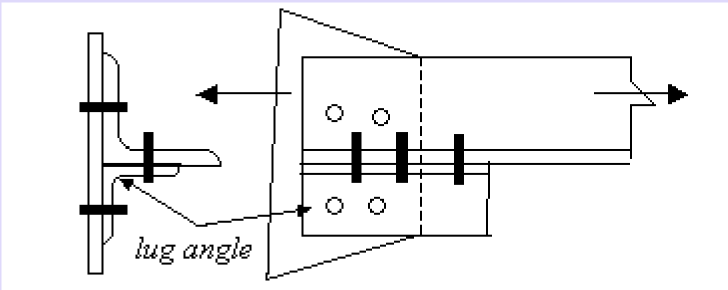


**Fig 4.10 Tension member with lug**