Unit – II

Design of compression members, Tension members, Roof Trusses - Angular & Tubular, Lattice Girders.

Tension members

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

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),

$$T = fy \times A / Ymo$$

Where, fy is the yield strength of the material (in MPa), A is the gross area of cross section in mm2 and Ymo is the partial safety factor for failure in tension by yielding. The value of Ymo according to IS: 800 is 1.10.

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: Behavior of Tension Members elastic range, but exhibits stress concentration adjacent to the hole. 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.

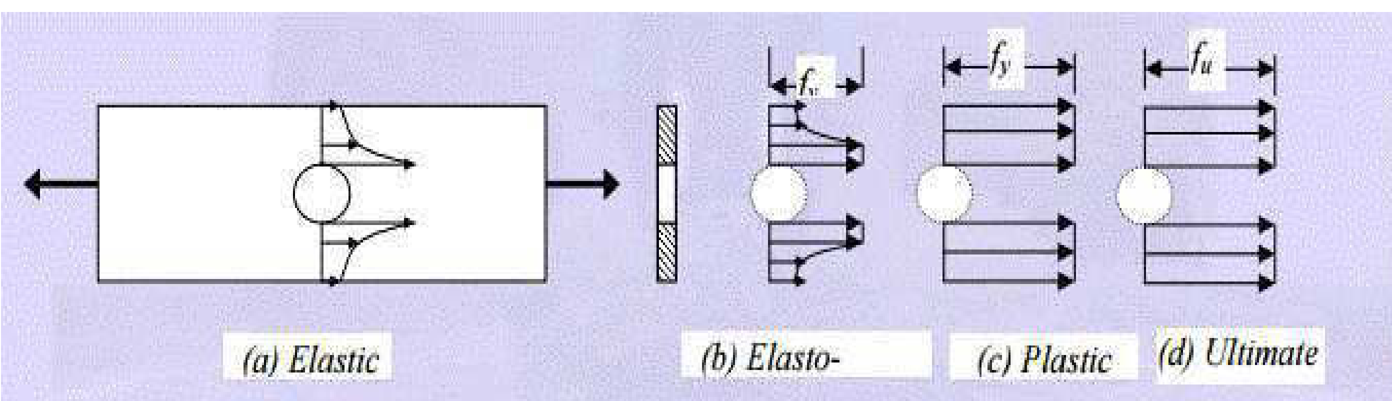


Fig. 4.4 Stress distribution at a hole in a plate under tension

In statically loaded tension members with a hole, the point adjacent to the hole reaches yield stress, fy, 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, fy, Finally, the rupture (tension failure) of the member occurs when the entire net cross section reaches the ultimate stress, fu,. 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)

$$P_{tn} = 0.9\, f_u\, A_n / Y_{ml}$$

Where, fu is the ultimate stress of the material, An is the net area of the cross section after deductions for the hole and Y_{ml} is the partial safety factor against ultimate tension failure by rupture (Y_{ml} = 1.25). Similarly threaded rods subjected to tension could fail by rupture at the root of the threaded region and hence net area, An, is the root area of the threaded section.

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 center to center 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

$$A_n = (b - 2d + P^2 / 4g)\, t$$

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

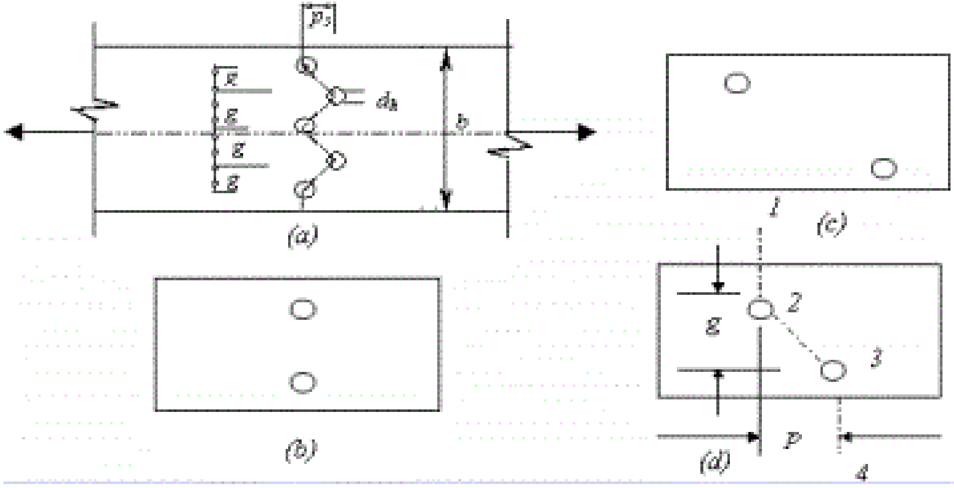
$$P^2 / 4g > d$$

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

$$A_{net} = (b - nd + \sum P^2 / 4g)\, t$$

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 p2 /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.



Design strength due to block shear

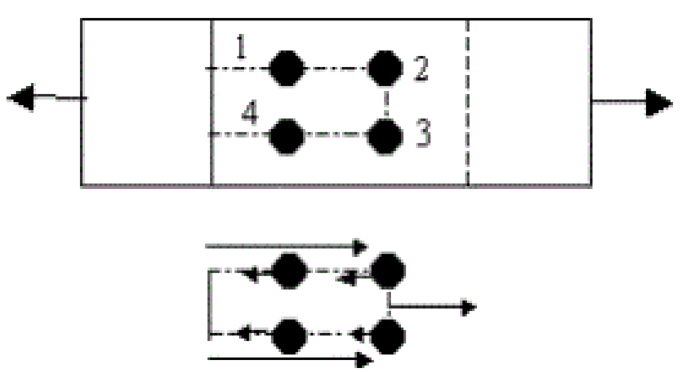
A tension member may fail along end connection due to block shear as shown in Fig.4.7. The corresponding design strength can be evaluated using the following equations. The block shear strength T_{db}, at an end connection is taken as the smaller of (C1.64)

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

or

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

Where, A_{vg}, A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively (1-2 and 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 fu, fy = ultimate and yield stress of the material respectively.



Compression Member Design

Structural elements that are subjected to axial compressive forces only are called columns. Columns are subjected to axial loads thru the centroid.

- Stress: The stress in the column cross-section can be calculated as

$$F = P / A$$

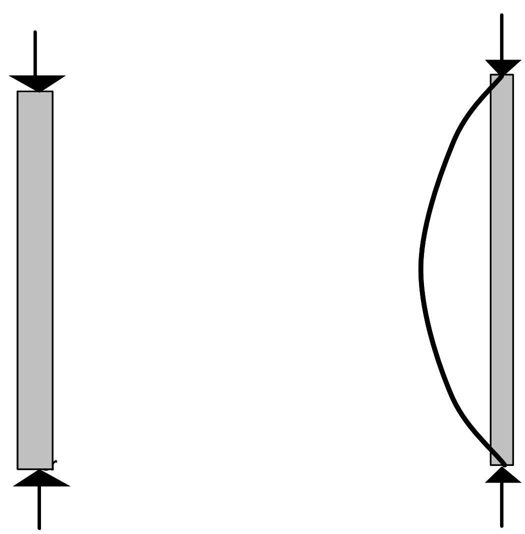
Where, f is assumed to be uniform over the entire cross-section.

- This ideal state is never reached. The stress-state will be non-uniform due to:
  - Accidental eccentricity of loading with respect to the centroid
  - Member out-of –straightness (crookedness), or
  - Residual stresses in the member cross-section due to fabrication processes.

- Accidental eccentricity and member out-of-straightness can cause bending moments in the member. However, these are secondary and are usually ignored.
- Bending moments cannot be neglected if they are acting on the member. Members with axial compression and bending moment are called beam-columns.

Column Buckling

Consider a long slender compression member. If an axial load P is applied and increased slowly, it will ultimately reach a value P_{cr} that will cause buckling of the column. P_{cr} is called the critical buckling load of the column. P



Buckling of axially loaded compression members

Buckling occurs when a straight column subjected to axial compression suddenly undergoes bending. Buckling is identified as a failure limit-state for columns.

- The critical buckling load Pcr for columns is theoretically given by Equation

$$P_{cr} = (\pi^2 E I) / (KL)^2$$

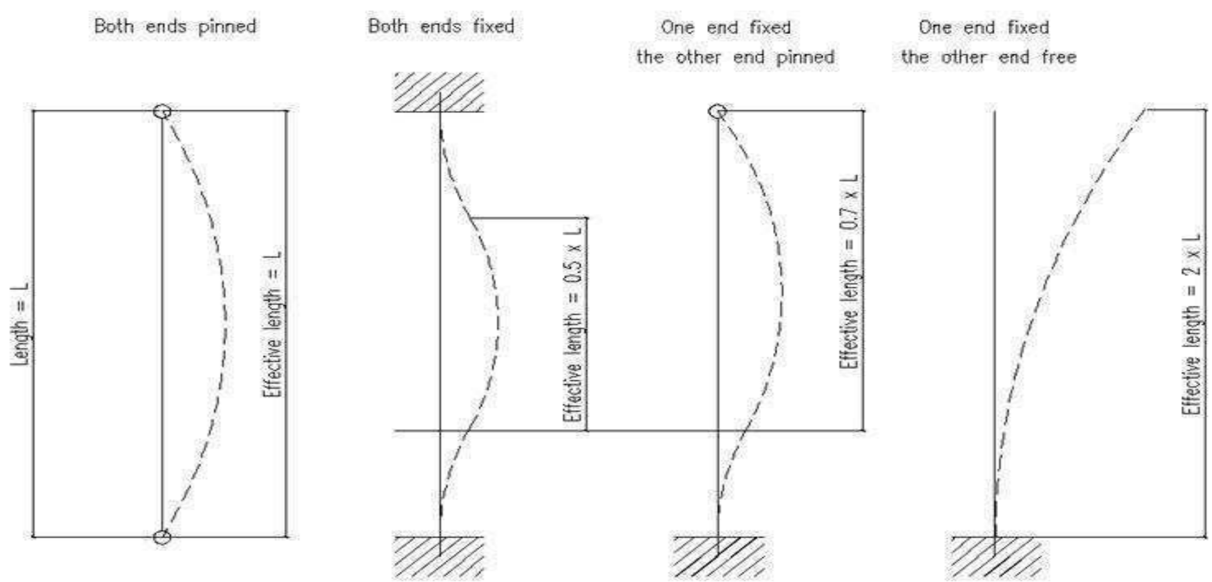
Where, I = moment of inertia about axis of buckling

K = effective length factor based on end boundary conditions

- Effective length factors are

Table C-C2.1

K Values for Columns



Compression Members: - are structural elements that are pushed together or carry a load, more technically they are subjected only to axial compressive forces. That is, the loads are applied on the longitudinal axis through the centroid of the member cross section, and the load over the cross sectional area gives the stress

on the compressed member. In buildings posts and columns are almost always compression members as are the top chord of trusses.

Primarily Occur as:

1. Columns in buildings;
2. Chord Members in trusses and diagonal members in end panels of trusses
3. Stability is an important consideration in design and behavior of compression members
4. Area is generally spread out to maximize Radius of Gyration

Types of compression Members: - On the basis of slenderness ratio (KL / r), where, KL is effective length and 'r' is the least radius of gyration of cross-section; different types of compression member are;

1. Short Compression Member (Column): - These are compression members having low slenderness ratio and fail by crushing.

2. Intermediate Compression Member (Column): - These are the compression members having medium slenderness ratio. They show axial shortening at the initial stage and with the increase in axial load; deformation in the direction normal to an axis of loading develops to cause failure.

3. Long Compression Member: - These are the compression members having high slenderness ratio and fail by elastic buckling in the Euler mode on attaining the crippling load.

Lattice Girder

The lattice girder type of design has been supplanted in modern construction with welded or bolted plate girders, which use more material but have lower fabrication and maintenance costs. The lattice girder was used prior to the development of larger rolled steel plates.

The term is also sometimes used to refer to a structural member commonly made using a combination of structural sections connected with diagonal lacing. This member is more correctly referred to as a laced strut[2] or laced tie, as it normally resists axial compression (strut) or axial tension (tie); the lattice girder, like any girder, primarily resists bending.

The component sections may typically include metal beams, channel and angle sections, with the lacing elements either metal plate strips, or angle sections. The lacing elements are typically attached using either hot rivets or threaded locator bolts. As with lattice girders, laced struts and ties have generally been supplanted by hollow box sections, which are more economic with modern technology. In some case seismic retrofit modifications replace riveted lacing with plates bolted in place.

