CHAPTER 04

Tension Members

4.1 Tension Member

- In a tension member, pulling forces are applied in the opposite directions at the two ends of the member.
- The bending stresses in the tension member due to its self-weight are not taken into consideration. However in case eccentricity exists in the member due to the fact that member is not perfectly straight or there exists eccentricity in connections, then in that case either the flexural stresses are considered in design or the net area of tension member is modified.
- Strength of a tension member gets adversely affected due to the presence of nuts, bolts and pins at connections due to shear lag, reversal of stresses, flexural moments in end connections due to eccentricity etc.

Remember As per theoretical aspects, tension member is considered as the most efficient structural member because there does not occur the possibility of buckling and also the member can be stressed up to and beyond the yield point and also the whole cross section is under a uniform tensile stress.

- The presence of bolt/rivet/pin holes influence the stress distribution at the section passing through these holes.
- Critical tensile stress in a tension member occurs at the section where the cross sectional area is the minimum. As far as possible bolts/rivets/pins are placed in one line to have minimum reduction in the cross sectional area of the member but often the required length of the connection makes it necessary to provid connection in more than one line. Thus in this case, the rivets/bolts/pins are placed in a staggered fashion as shown in Fig. 4.1 (b).

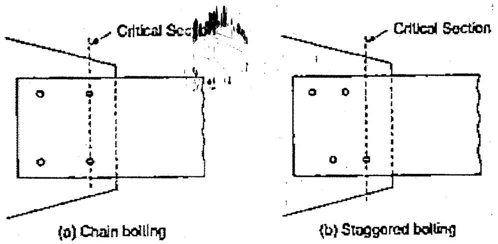


Fig. 4.1 Chain and staggered pattern

4.2 Various Types of Tension Members

(a) **Wire and cable:** Wire and cable are the simplest form of tension member. Steel wires are used as guy wires in steel stacks. Many a times, a number of wires are wound helically to form strands or the ropes as shown in Fig. 4.2.

- **Strand:** The term strand implies number of wires wound helically around a central core (Fig. 4.2(a))
- **Wire:** Wire implies a number of strands wound helically around a core (Fig. 4.2(b)). The biggest advantage of wire and cable is that they are too flexible and also has very high tensile strength.

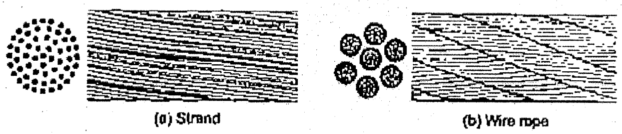


Fig. 4.2 Strand and rope of wires

- Wire ropes are used as suspenders in cable bridges.

(b) **Rods and bars:** Bars and rods are used in lightly loaded structures like the roof trusses.

- They are used as sag rods to support the purlins and also to support girts in industrial buildings.
- Bars can either be threaded and bolted (Fig. 4.3(a)) or they may be given the shape of a loop (Fig. 4.3(b)).

Disadvantages of rods and bars

(i) Large slenderness ratio of tension members make them quite unsuitable to be used as compression members.

(ii) They do not have enough flexural strength even to bear their own weight and thus there occurs a significant amount of sag in these members due to their own weight.

This problem is overcome by limiting the length to thickness ratio (or length to diameter ratio in case of circular sections) or by slightly making the length of tension member less than what is required and then placing the member in the structure to provide some initial tension and the same can also be achieved by the use of a turn buckle (Fig. 4.3(d)).

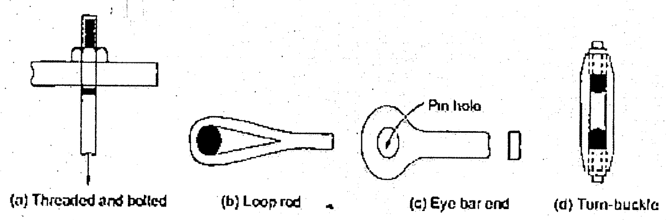


Fig. 4.3 Different forms of connections of bars and rods

(c) **Flat plates and flat bars:** These are predominantly used in foot bridges, transmission towers and also as lacings and battens in built up compression members.

(d) **Rolled sections and built up sections:** The advantage of angle section is that it gives more rigidity to the structure as compared to flat bar and also angle sections do not vibrate under the loads unlike flat bars. Angle sections are used as lacings, in bridges etc.

Built up sections : Built up sections are used primarily to meet the requirement of area required to sustain a load which otherwise cannot be provided by a single section.

- Built up sections are more rigid and offer greater moment of inertia. Built up members are required to be tied up so that all the component members act in unison i.e. as a single unit.
- When the loads are heavy and along with that, rigidity is also required, in that case channel sections are preferred. Closed sections like tubes and pipes are used as main members of trusses and bracings in space frames. With the advent of welding, connection of tubes and pipes has now become easy.

4.3 Net Sectional Area of Tension Members (A_n)

- The net sectional area of a member is the gross area of the section of the member less the area of holes for rivets/bolts.
- Presence of holes in a member increases the stress at the section due to reduction in the sectional area.
- Apart from that, concentration of stresses occurs along the edges of the holes.

4.3.1 Net Sectional Area of Flats and Plates

- Theoretically a tension member without a bolt hole can take tensile load up to the yield point of the material.
- This tensile load carrying capacity of the member gets reduced due to the bolt holes.

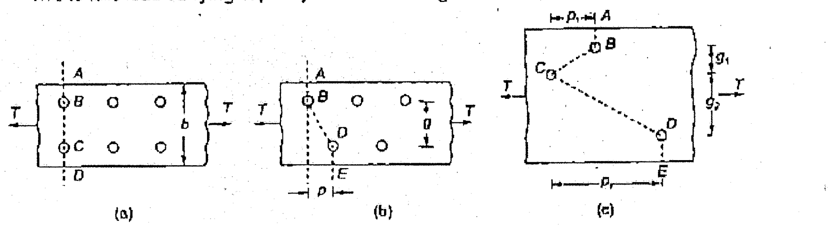


Fig. 4.4 Net sectional area of riveted/bolted plate

- As shown in Fig. 4.4 (a), the plate is provided with bolts along two straight lines called as chain bolting. Section ABCD gives the least net sectional area i.e. section passing through the bolts.
- Thus net sectional area is the gross sectional area of the plate less the area of bolt holes (two bolt holes here), i.e.,

$$A_n = A_g - \text{Area of holes}$$
$$= bt - nd_0t$$
$$= (b - nd_0)t \qquad ...(4.1)$$

where A_n = Net sectional area of plate, A_g = Gross area of the plate, b = Width of the plate, t = Thickness of the plate, d_0 = Diameter of rivet/bolt hole, n = Number of rivet/bolt holes in the section

- Fig. 4.4(b) shows bolting in the same two straight lines but in a staggered manner. The deduction of area in this case will be the sum of the sectional areas of all the holes less $p^2t/4g$ for each inclined line between the two holes except the lines in the direction normal to the direction of applied tensile stress.
- For Fig. 4.4(c), net sectional area will be given as:

$$\text{Deductions} = \text{Total sectional area of the holes} - \left(\frac{p_1^2t}{4g_1} + \frac{p_2^2t}{4g_2}\right)$$

and

$$\text{Net sectional area } (A_n) = A_g - \left[\text{Total sectional area of the holes} - \left(\frac{p_1^2t}{4g_1} + \frac{p_2^2t}{4g_2}\right)\right]$$
$$= bt - nd_0t + \frac{p_1^2t}{4g_1} + \frac{p_2^2t}{4g_2}$$
$$= \left(b - nd_0 + \frac{p_1^2}{4g_1} + \frac{p_2^2}{4g_1}\right)t \qquad ...(4.2)$$

where p = Staggered pitch, g = Gauge distance, n = Total number of holes in the zig-zag line

NOTE: Due to complex nature of stress variation in staggered arrangement of bolts, the term $p^2/4g$ is only an approximate one and the same has been arrived at using the maximum stress theory of failure.

4.3.2 Net Sectional Area of Angles and T-Sections

- Normally, only one leg of the angle section is connected called as **connected leg** and the other leg is kept free called as **outstanding leg**.
- When load is applied through one leg only then non-uniform stress distribution occurs in the angle section because of shear lag effect. Moreover eccentric forces produce bending stresses in the section also.
- When all the legs of the section (angles and T sections) are connected then, eqs. (4.1) and (4.2) can be used for computing the net sectional area. The overall width and gauge length are arrived at as shown in Fig. 4.5.

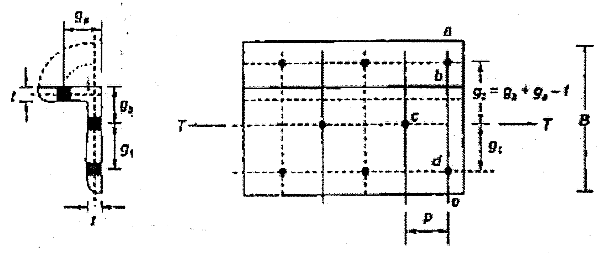


Fig. 4.5 Explanation of gauge for angle section

Remember In case of built up tension members where the two sections are placed back to back in direct contact or separated back to back by a certain distance but not greater than the aggregate thickness of the connected individual members, there tacking bolts are provided in line at a pitch not greater than 1000 mm.

4.4 Net Effective Area of Tension Members (A_{ne})

Factors that affect efficiency of the tension member are:

(a) Ductility of the member (b) Shear lag effect
(c) Geometric configuration (d) Method of fabrication

Among all the above factors, shear lag effect is the most prominent one. The net effective area of a section is defined as:

$$A_{ne} = k_1 k_2 k_3 k_4 A_n \qquad ...(4.3)$$

where k_1 = Ductility factor, k_2 = Hole forming factor
k_3 = Geometry factor, k_4 = Shear lag factor
A_{ne} = Net effective area, A_n = Net sectional area

The various factors are as described below:

- **The ductility factor (k_1):** As the ductility of the material of tension member increases, the strength of the tension member also increases since due to high ductility stress distribution around the bolt holes get modified and a more uniform stress distribution takes place when the steel is in plastic state. So for ductile materials, it is assumed that there is a uniform stress distribution when the material is loaded beyond its yield point. For commonly available steel with specified minimum ductility, the ductility factor is unity i.e. k_1 = 1.
- **Hole forming factor (k_2):** This factor is also known as **factor for method of fabrication.** In case of bolted connections, the bolt holes are formed either by punching or drilling. In punched holes, the material around the hole gets deformed in shear and thus the strength of the member get reduced by about 10 to 15% to that compared to drilled holes. This occurs due to strain hardening of the material which causes the loss of ductility. This factor (k_2) is taken as 0.85 for punched holes and 1.0 for drilled holes as some of the standards specify. IS 800 : 2007 recommends to increase the hole diameter by 2 mm while computing the net area of punched hole rather than using k_2 as 0.85.
- **The geometry factor (k_3):** As gauge to diameter ratio (g/d) of a bolted joint increases, the efficiency of bolted joint decreases. For widely spaced bolts i.e. for large gauge distances (or large g/d ratio), the metal in between the holes experiences non uniform strain which leads to local fracture well before the strain gets equalized to the net section. This value of k_3 varies from 0.9 to 1.14 but IS 800:2007 recommends this value as unity.
- **Shear lag factor (k_4):** Leaving members like flats and bars, the actual tensile stress in the tension member till fracture that occurs at its critical section, is less than the claimed tensile strength of the steel except in that case where all the elements of a tension member are so arranged that uniform stress distribution occurs over the entire section.

The phenomenon of shear lag occurs when some elements of the member cross section are not connected. In Fig. 4.6(a), an angle section is acting as a tension member connected with one leg only. Thus, the connected leg will get over stressed and the unconnected leg will not be stressed fully. At connections, large amount of load is carried by the connected leg and it requires some distance called as **transition distance** to spread the stress uniformly across the whole angle section (Fig. 4.9(b)).

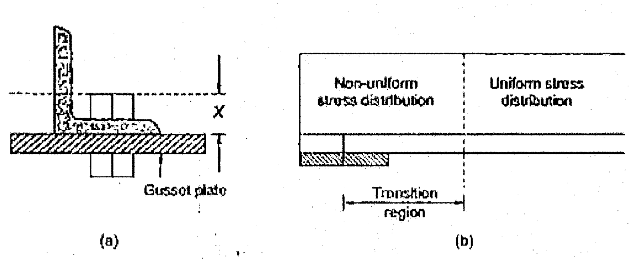


Fig. 4.6 Explanation of shear lag

In the transition length of the angle section, the stress in the connected leg may exceed yield stress (f_y) and may go into strain hardening zone thereby making the member liable to fracture prematurely. As one moves away from the joint connection, the stress distribution becomes more uniform. In the transition region, shear transfer gets lagged. This shear lag reduces the effectiveness of the component element of the tension member that is not connected directly to the gusset plate. Thus outstanding legs of the unequal angle sections are kept small i.e. long leg is made as the connected leg and the short leg as unconnected leg. Shear lag is independent of the type of load and applies to both the welded and bolted connections. But due to reduction in net area bolted connection gets affected more as compared to welded connection. This shear lag is taken care of by using the net effective area. This is done by using the shear lag factor (k_4) as:

$$k_4 = 1 - \frac{\bar{x}}{L} \qquad ...(4.4)$$

where $\bar{x}$ = Distance from the face of the gusset plate to the centroid of the connected leg
L = Length of the connection in the direction of load as shown in Fig. 4.7.

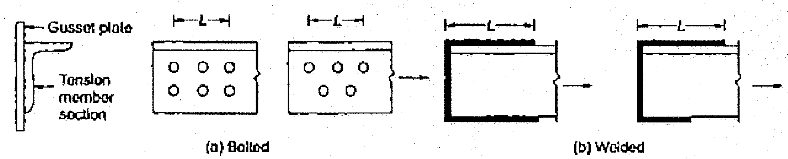


Fig. 4.7 Explanation of 'L' for welded and bolted connection

From Eq. (4.4), as ($\bar{x}/L$) decreases, factor k_4 increases and hence the net effective area of the section and thus the strength of the tension member increases.

Now clubbing all the above four factors viz. k_1, k_2, k_3 and k_4, Eq. (4.3) reduces to,

$$A_{ne} = k_4 A_n \qquad ...(4.5)$$

($\because k_1 = k_3 = 1$, k_2 is taken care of in area of bolt holes)

IS 800:2007 defines this factor k_4 as α and thus Eq.(4.5) can be expressed as,

$$A_{ne} = \alpha A_n$$

where α = 0.6 when number of bolts ≤ 2
= 0.7 when number of bolts = 3
= 0.8 when number of bolts ≥ 4
= 0.8 for welds

4.5 Different Types of Failure in Tension Members

(a) **Yielding of gross section (T_{dg}):** In this type of failure wherein the yielding of the gross section occurs there a significant amount of deformation occurs before the material actually gets fractured. This significant amount of deformation makes the structure unserviceable.

(b) **Rupture of net section (or fracture) (T_{dn}):** When the net section of the member reaches the ultimate stress, then rupture (or fracture) of member takes place.

(c) **Block shear failure (T_{db}):** Here a segment of the block of the material at the corner of the connection shears out due to possible high bearing strength of steel and high strength of bolts.

4.6 Design Strength of a Tension Member

The design strength of a tension member is based on two limit states viz.:

(a) Limit state based on yield strength of the member due to its large elongation.

(b) Limit state based on ultimate strength of the member. This limit state is particularly crucial where the section of the tension member gets reduced due to the presence of bolts holes etc.

A tension member with bolt holes at the ends for connection purpose etc., even if subjected to tensile stresses in excess of yield stress to as high as ultimate stress, the member may not elongate excessively, because of the presence of so called non active metal behind the bolt holes in the direction of tensile force. Just at the point of fracture (i.e. breaking), the metal tends to narrow out at that point where the fracture occurs but the additional metal present behind the bolt hole restricts this contraction of the strained metal thereby increasing the ultimate strength. Thus fracture strength at the net section passing through the bolt holes is considered as another limit state.

The design strength of a tension member is the minimum of the following:

(a) Design strength due to yielding of the gross section (T_{dg})

(b) Rupture strength of critical section (T_{dn})

(c) Block shear strength (T_{db})

4.6.1 Limit state – 1: Design Strength Due to Yielding of the Gross Section (T_{dg})

- This limit state tries to prevent the excessive elongation of the tension member.
- In order to prevent the attainment of this limit state, the stress on the gross section must be less than the yield stress of the material (f_y).
- Though yield stress may reach at the net section prior to the gross section but this length of connection is too small as compared to overall length of the tension member and thus elongation in the unreduced sectional part will be more than the elongation in the holes region.

NOTE: It is the larger elongation and not the attainment of yield first that is considered as a limit state.

- Thus to prevent the excessive elongation in the tension member, the stress on the gross section must be less than the yield stress i.e.

$$\frac{T}{A_g} < f_y$$

$$T < f_y A_g$$

- The design strength due to yielding of the gross section is given by,

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} \qquad \text{...(4.6)}$$

where T_{dg} = Design strength of the tension member

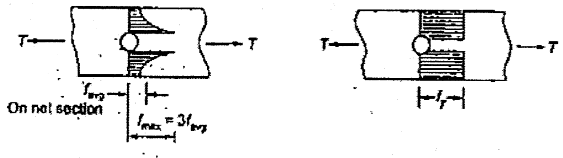
A_g = Gross area of the section

f_y = Yield strength of the material

γ_{m0} = Partial factor of safety for failure in tension member by yielding = 1.1

4.6.2 Limit State -2: Design Strength Due to Rupture of Net Section

- Presence of holes in the section cause stress redistribution around the holes and makes the stress near to the holes to be larger than the average stress at the bolt hole (=P/A) as shown in Fig. 4.8.



(a) Elastic stress distribution (b) Nominal strength condition

Fig. 4.8 Stress redistribution around the holes

- This stress variation across the net section will become uniform as the stress approaches the yield stress thereby approaching the plastic state.

NOTE: The ratio of maximum elastic stress (f_{max}) to the mean stress (f_{avg}) is often known as stress concentration factor.

- The stress in the fibers near to the hole reaches the yield stress (f_y) and remains constant as the load is increased but the stress in the sectional regions away from the hole gradually increases till they reach the yield stress (f_y).
- Because of the ductility of steel, initially yielded zone undergoes more deformation till the member fractures through the holes. At this stage, the stress in the entire net section reaches the ultimate stress (f_u).
- In order to prevent the net section fracture of a tension member,

$$T < A_n f_u$$

- The design strength due to rupture of net section is given by,

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \qquad \text{...(4.7)}$$

where A_n = Net effective area at the critical section

γ_{m1} = Partial factor of safety for the material strength governed by ultimate stress = 1.25

NOTE: In Eq. (4.7) the factor of 0.9 has been introduced since there is not available any kind of reserve strength beyond ultimate strength and this factor of 0.9 gives some factor of safety.

- **For angle section:** Rupture strength of angle section connected through one leg only gets affected by shear lag. In this case the design strength due to rupture of net section is given by,

$$T_{dn} = \frac{0.9A_{nc}f_u}{\gamma_{m1}} + \frac{\beta A_{go}f_y}{\gamma_{m0}} \quad ...(4.8)$$

where, A_{nc} = Net area of connected leg

A_{go} = Gross area of outstanding leg

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{f_u\gamma_{m0}}{f_y\gamma_{m1}} \ge 0.7 \quad ...(4.9)$$

w = Length of outstanding leg of angle section

b_s = Shear lag width as shown in Fig. 4.9

t = Thickness of the leg

L_c = Length of the end connection i.e. distance of the outermost bolt of the end joint measured in the loading direction or the length of the weld in the direction of load

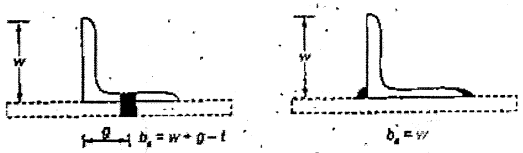


Fig. 4.9 Single angle as tension member

For preliminary design purpose, IS code recommends the following expression:

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} \quad ...(4.10)$$

where α = 0.6 when number of bolts ≤ 2

= 0.7 when number of bolts = 3

= 0.8 when number of bolts ≥ 4

= 0.8 for welds

However it is difficult to determine the equivalent weld length and thus designers are required to judge it rationally.

- Bolts and threaded rods:

$$T_{dn} = \frac{0.9A_n f_u}{\gamma_{m1}} \quad ...(4.11)$$

where A_n = Net effective area at the threaded section which is given by,

$= \frac{\pi}{4}(d - 0.9382p)^2$ where p is the pitch of the thread

$= 0.78\frac{\pi}{4}d^2$ for ISO threads

- For other sections: The rupture strength for other sections like the double angle, T-section, I-section, channel etc. may be determined by the same equations as for single angle section with b_s taken from farthest edge of the outstanding leg to the nearest bolt line or weld line in the connected leg.

4.6.3 Limit State-3: Block Shear Strength

- At the connected end of a tension member, failure may occur along a path which involves shear along one plane and tension on the orthogonal plane along the fastener. This type of failure is referred to as block failure.

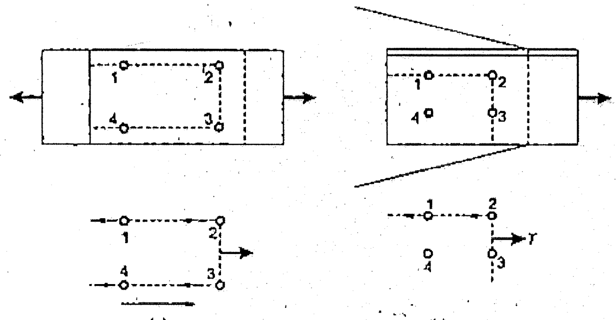


Fig. 4.10 Block shear failure

- In Fig. 4.10(a) failure along planes 1-2 and 3-4 is shear failure and that along plane 2-3 is tension failure.
- In Fig. 4.10 (b), shear failure occurs along 1-2 and tension failure along 2-3.
- For bolted connections, IS 800 : 2007 recommends the following expression for block shear strength:

$$T_{db} = \begin{cases} \dfrac{A_{vg}f_y}{\sqrt{3}\cdot\gamma_{m0}} + \dfrac{0.9A_{tn}f_u}{\gamma_{m1}} \\ \dfrac{0.9A_{vn}f_u}{\sqrt{3}\cdot\gamma_{m1}} + \dfrac{A_{tg}f_y}{\gamma_{m0}} \end{cases} \text{whichever is less} \quad ...(4.12)$$

A_{vg} and A_{vn} = Minimum gross and net area in shear (1-2, 3-4 in Fig. 4.10(a) and 1-2 in Fig. 4.10(b))

A_{tg} and A_{tn} = Minimum gross and net area in tension (2-3 in Fig. 4.10(a) and (b))

NOTE: The block shear strength T_{db} shall be checked for the welded connection by taking into account a suitable section around the weld.

4.7 Slenderness Ratio of Tension Members

- Since buckling is not involved in a tension member and hence theoretically there is no limit on the slenderness ratio of a tension member.
- In case there is a possibility of stress reversal (due to wind, transportation etc.) then stability of the member is a concern and in that case it is needed to put a limit on the maximum slenderness ratio.

Table 4.1 Maximum slenderness ratio for various types of tension members

S.No.	Type of Tension Member	Maximum Slenderness Ratio
1.	Tension member in which there can be reversal of direct stress due to loads other than wind or earthquake force.	180
2.	A member normally acting as a tie in a roof truss or a bracing system but subjected to possible reversal of stress due to wind or earthquake forces.	350
3.	Tension member i.e., members always under tension (other than pretensioned members)	400

4.8 Procedure for the Design of Tension Member

Step-1. Determine the required gross area to carry the factored tensile load (T_u) considering the strength in yielding as:

$$A_g = \frac{T_u}{\left(f_y/\gamma_{m0}\right)} = \frac{1.1T_u}{f_y} \qquad \text{...(4.13)}$$

Step-2. Select a suitable shape of the section depending on structural and architectural requirements with gross area of the section about 25 to 40% more than the gross area computed in Step-1 above.

Step-3. Determine the number of bolts required in case of bolted connection and length and size of the weld required in case of welded connection.

Step-4. Determine the strength of the member and associated connection(s) considering the following types of strength:

(a) Strength in yielding of the gross area

(b) Strength in rupture of critical section

(c) Strength in block shear

In general, if the requirements of edge distance and minimum pitch are maintained then strength in yielding gives the least value and hence the design is safe if gross area provided is more than the gross area required.

Step-5. The strength obtained must be greater than the factored tension but it should not exceed the factored tension too much otherwise revise the section by selecting another suitable section.

Step-6. Check the slenderness ratio of the tension member as per IS 800:2007 which is given in Table 4.1.

4.9 Splicing in Tension Member

- If a single piece of a tension member is not available for the required length then tension members are spliced to transfer the required tension from one piece to another.
- The strength of the splice plates and bolts / welded connections must have strength at least equal to the design load.
- In case, connecting tension members are of different thickness, then filler plates are used.
- As per Cl. 10.3.3.3 of IS 800:2007, the design shear strength of bolts carrying shear through a packing plate of thickness in excess of 6 mm shall be reduced by a factor given by,

$$\beta_{pk} = 1 - 0.0125\, t_{pk} \qquad \text{...(4.14)}$$

where t_{pk} = Thickness of thicker packing plate (in mm)

4.10 Lug Angles

- For a heavily loaded tension member, the length of the connection comes out to be too large to be accommodated on a gusset plate.
- This length of connection can be reduced by using lug angles as shown in **Fig. 4.11**.
- Use of lug angles saves the cost of gusset plate but this saving is offset by additional fasteners and angles required.

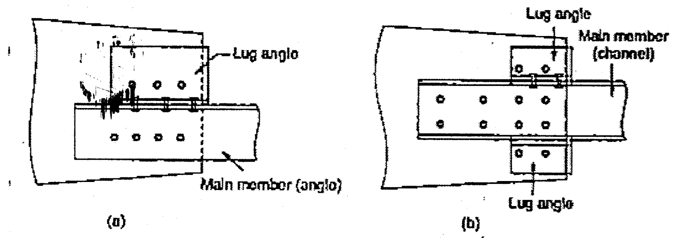


Fig. 4.11 Lug Angles

Cl. 10.12 of IS 800 : 2007 specifies certain requirements for lug angles which are as given below:

(a) The effective connection of the lug angle shall be as far as possible, terminate at the end of the member.

(b) The connection of the lug angle to the main member shall preferably start in advance of the member to the gusset plate.

(c) For connecting lug angle to the gusset, a minimum of two bolts or rivets or equivalent weld length should be used.

(d) In case, the main member is an angle then:

   (i) The whole area of the main member shall be taken as effective instead of net effective section. The whole area of the member is the gross area of the member less deduction for the rivet/bolt holes.

   (ii) The strength of lug angles and fasteners connecting lug angle to gusset plate must be at least 20% more than the force in the outstanding leg.

   (iii) The strength of fastener connecting lug angle and main member shall be at least 40% more than the force carried by the outstanding leg.

(e) In case the main member is a channel section then,

   (i) As far as possible, the channel should be symmetric.

   (ii) The strength of fasteners connecting lug angle to the gusset should be at least 10% more than the force in the outstanding leg.

   (iii) The strength of fasteners connecting lug angle to the main member should be at least 20% more than the force in the outstanding leg.

4.11 Gusset Plate

- Gusset plate aids in making the connection where more than one member are required to be joined.
- The lines of action of force in the members are assumed to be coinciding at a point as shown in Fig. 4.12.

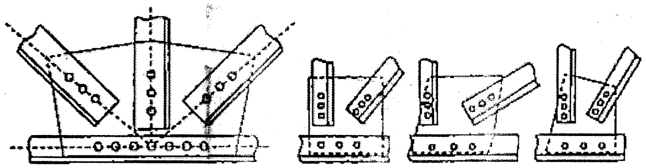


Fig. 4.12 Gusset plate connecting the members

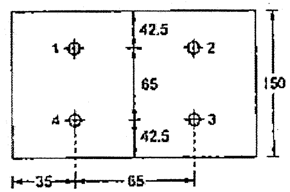
- When these lines of action do not coincide then this gives rise to secondary stresses.
- The size and shape of the gusset plate depends on the direction of the members meeting at a joint.
- The profile of the plate is so fixed that specification of edge distances are complied with.

NOTE: The top edge of the gusset plate is stiffened if it exceeds $42\,t\varepsilon$ where $\varepsilon = \sqrt{250/f_y}$

- There are no elaborate design specifications for the thickness design of gusset plate but in any case it should not be less than 12 mm.
- Structurally a gusset plate should be able to resist shear stresses, flexural stresses and the direct stresses.

Illustrative Examples

Example 4.1 Determine the design tensile strength of plate 150 mm × 12 mm size with holes for 16 mm diameter bolts as shown in figure below. Use Fe410



(All dimensions in mm)

Solution:

Strength of plate is the minimum of

(a) Yielding of gross-section (T_{dg})
(b) Rupture of critical-section (T_{dn})
(c) Block shear strength (T_{db})

(a) Design strength due to yielding of gross-section

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

where $A_g = 150 \times 12 = 1800\ \text{mm}^2$

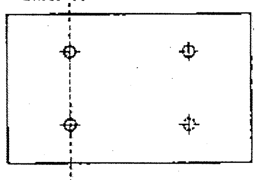
For Fe410, $f_y = 250\ \text{N/mm}^2$

$\gamma_{m0} = 1.1$

$$\therefore \quad T_{dg} = \frac{1800 \times 250}{1.1} N = 409.1\ \text{kN}$$

(b) Design strength due to rupture of critical section

Critical section



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

Critical section is having two holes

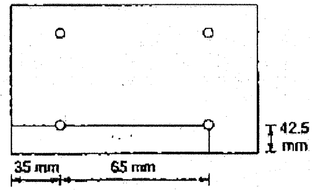
Gross diameter of holes (d_0) = 18 mm (From Table 2.5)

$$\therefore \quad \text{Net effective area } (A_n) = \left[b - nd_0 + \Sigma \frac{p^2}{4g}\right] t$$

$$= [150 - 2 \times 18 + 0]12 = 1368\ \text{mm}^2$$

$$\therefore \quad T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9(1368)410}{1.25} N = 403.8\ \text{kN}$$

(c) Block shear strength



Gross area in shear (A_{vg}) = (65 + 35)12 = 1200 mm²

Net area in shear (A_{vn}) = (65 + 35 − 1.5 × 18)12 = 876 mm²

($\because$ 1.5 bolt holes are getting intercepted in shear areas)

Gross area in tension (A_{tg}) = 42.5 × 12 = 510 mm²

Net area in tension (A_{tn}) = (42.5 − 0.5 × 18)12 = 402 mm²

Block shear strength is minimum of the following:

(i) Shear yield and tension fracture

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = \frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 402 \times 410}{1.25}$$

$$= (157.5 + 118.7)\ \text{kN} = 276.2\ \text{kN}$$

(ii) Tension yield and shear fracture

$$T_{db2} = \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} = \frac{510 \times 250}{1.1} + \frac{0.9 \times 876 \times 410}{\sqrt{3} \times 1.25}$$

$$= (115.9 + 149.3)\ \text{kN} = 265.2\ \text{kN}$$

$\therefore \quad T_{db}$ = Minimum of T_{db1} and T_{db2} = 265.2 kN

Thus strength of plate = Minimum of (a), (b) and (c) = 265.2 kN

Example 4.2 An unequal angle ISA 90 × 60 × 6 is connected to a 10 mm gusset plate at the end with four nos. 16 mm diameter bolts. The angle is to be used as a tension member. Find the design tensile strength of the angle if

(a) 90 mm leg is connected to the gusset plate (g = 50 mm)
(b) 60 mm leg is connected to the gusset plate (g = 30 mm).

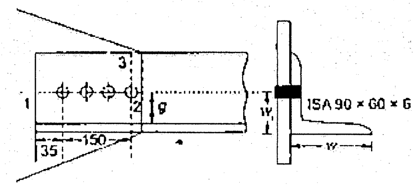
Use Fe410.

Solution:

(a) 90 m leg is connected to the gusset

Strength of the angle is the minimum of

(i) Yielding of gross-section (T_{dg})
(ii) Rupture of critical section (T_{dn})
(iii) Block shear strength (T_{db})



(i) Design strength due to yielding of gross-section

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

From steel table, for ISA 90 × 60 × 6, A_g = 865 mm²

$$\therefore \quad T_{dg} = \frac{865 \times 250}{1.1} N = 196.59 \text{ kN}$$

(ii) Design strength due to rupture of critical section

$$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

Where net area of connected leg

$$A_{nc} = \left(90 - \frac{6}{2}\right)6 = 522 \text{ mm}^2$$

Gross area of outstanding leg

$$A_{go} = \left(60 - \frac{6}{2}\right)6 = 342 \text{ mm}^2$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{0.9 f_u \gamma_{mo}}{f_y \gamma_{m1}} \ge 0.7$$

w = width of outstanding leg = 60 mm

w_1 = 50 mm

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

= 60 + 50 − 6 = 104 mm

L_c = 3 × 50 = 150 mm

$$\therefore \quad \beta = 1.4 - 0.076\left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\left(\frac{104}{150}\right) = 1.0787$$

$$\frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{0.9 \times 410 \times 1.1}{250 \times 1.25} = 1.2988 \ge 0.7$$

Thus $\quad \beta \le \dfrac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} \ge 0.7$

$\therefore \quad \beta = 1.0787$

$$\therefore \quad T_{dn} = \frac{0.9\, A_{nc} f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{m0}} = \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.0787 \times 342 \times 250}{1.1}$$

$$= (154.09 + 83.84) \text{ kN} = 237.93 \text{ kN}$$

(iii) Block shear strength

Failure may occur along the section 1 - 2 - 3

For 16 mm dia. bolt, $\quad d_0$ = 18 mm

Tearing length in tension = 90 − 50 = 40 mm

Gross area in shear (A_{vg}) = (150 + 35)6 = 1110 mm²

Net area in shear (A_{vn}) = (150 + 35 − 3.5 × 18)6 = 732 mm²

Gross area in tension (A_{tg}) = 40 × 6 = 240 mm²

Net area in tension (A_{tn}) = (40 − 0.5 × 18)6 = 186 mm²

Shear yield and tension fracture (T_{db1})

$$= \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = \frac{1110 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$$

$$= (145.65 + 54.91)\text{kN} = 200.56 \text{ kN}$$

Tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} = \frac{240 \times 250}{1.1} + \frac{0.9 \times 732 \times 410}{\sqrt{3} \times 1.25}$$

$$= (54.55 + 124.76)\text{kN} = 179.31 \text{ kN}$$

$\therefore$ Block shear strength (T_{db}) = Minimum of T_{db1} and T_{db2}

= 179.31 kN

$\therefore$ Strength of angle ISA 90 × 60 × 6 with 90 mm leg as connected leg

= Minimum of (i), (ii) and (iii)

= 179.31 kN

(b) 60 mm leg is connected to the gusset

Strength of the angle is the minimum of T_{dg}, T_{dn} and T_{db}

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{865 \times 250}{1.1} N = 196.59 \text{ kN}$$

Design strength due to rupture of critical section (T_{dn})

Net area of connected leg (A_{nc}) $= \left(60 - \dfrac{6}{2}\right) \times 6 = 342 \text{ mm}^2$

Gross area of outstanding leg (A_{go}) $= \left(90 - \dfrac{6}{2}\right) \times 6 = 522 \text{ mm}^2$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{0.9 f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

where width of outstanding leg = w = 90 mm

w_1 = 30 mm

$b_s = w + w_s - t$

= 90 + 30 − 6 = 114 mm

L_c = 3 × 50 = 150 mm

$$\therefore \quad \beta = 1.4 - 0.076\left(\frac{90}{6}\right)\left(\frac{250}{410}\right)\frac{114}{150} = 0.8717$$

$$\frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{0.9 \times 410 \times 1.1}{250 \times 1.25} = 1.2988 < 0.7$$

Thus $\quad \beta < \dfrac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}}$

$\therefore \quad \beta = 0.8717$

$$\therefore \quad T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}} = \frac{0.9 \times 342 \times 410}{1.25} + \frac{0.8717 \times 522 \times 250}{1.1}$$

$$= (100.96 + 103.42)\text{kN} = 204.38 \text{ kN}$$

Block shear strength (T_{db})

Tearing length in tension = 60 − 30 = 30 mm

Diameter of bolt hole for 16 mm dia bolt

$d_0 = 16 + 2 = 18$ mm

Gross area in shear (A_{vg}) = (150 + 35)6 = 1110 mm²

Net area in shear (A_{vn}) = (150 + 35 − 3.5 × 18)6 = 732 mm²

Gross area in tension (A_{tg}) = 30 × 6 = 180 mm²

Net area in tension (A_{tn}) = (30 − 0.5 × 18)6 = 126 mm²

Shear yield and tension fracture (T_{db1})

$$= \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = \frac{1110 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}$$

= (145.65 + 37.2) kN = 182.85 kN

Tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} = \frac{180 \times 250}{1.1} + \frac{0.9 \times 732 \times 410}{\sqrt{3} \times 1.25}$$

= (40.91 + 124.76)kN = 165.67 kN

∴ Block shear strength (T_{db}) = Minimum of T_{db1} and T_{db2} = 165.67 kN

∴ Strength of angle ISA 90 × 60 × 6 with 60 mm leg as connected leg

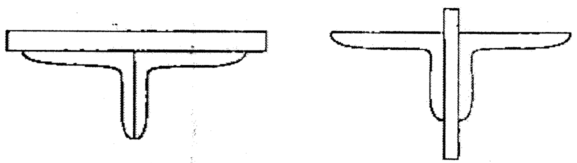
= Minimum of T_{dg} and T_{dn} and T_{db} = 165.67 kN

Example 4.3 **In the example 4.2, if instead of single angle, two angles are connected then determine the tensile strength if**

(a) the two angles are connected on the same side of the gusset plate through 90 mm leg

(b) the two angles are connected on either side of the gusset plate through 60 mm leg

Solution:



As per IS 800:2007, strength of double angle is twice of single angle as obtained in example 2 above. The earlier version of the code i.e. IS 800:1984 used to give strength more than double.

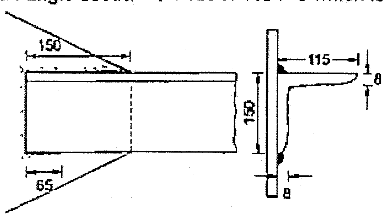
In case tacking rivets/bolts are provided, even then there is no change in strength of tensile member.

Example 4.4 **Determine the tensile strength of an angle section ISA 150 × 115 × 8 which is connected to gusset plate for the following cases:**

(a) yielding of gross-section

(b) rupture strength of critical section

Use Fe410.



Solution:

For Fe410 steel, $f_u = 410$ N/mm²

$f_y = 250$ N/mm²

From steel tables, for ISA 150 × 115 × 8, A_g = 2058 mm²

(a) Tensile strength due to gross-section yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2058 \times 250}{1.1} \text{ N} = 467.73 \text{ kN}$$

(b) Tensile strength due to rupture of critical section

Length of outstanding leg (w) = 115 mm

Weld length along the direction of load (L_c) = 150 mm

Width of shear lag (b_s) = w = 115 mm

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} \ge 0.7$$

$$= 1.4 - 0.076\left(\frac{115}{8}\right)\left(\frac{250}{410}\right)\left(\frac{115}{150}\right) = 0.8893$$

$$\frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{0.9 \times 410 \times 1.1}{250 \times 1.25} = 1.2988 > 0.7$$

∴ $\beta < \dfrac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} > 0.7$ (OK)

∴ $\beta = 0.8893$

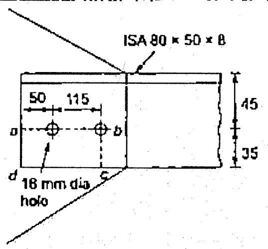
Net area of connected leg (A_{nc}) = $\left(150 - \frac{8}{2}\right)8$ = 1168 mm²

Gross area of outstanding leg (A_{go}) = $\left(115 - \frac{8}{2}\right)8$ = 888 mm²

∴ $$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{m0}} = \frac{0.9(1168)410}{1.25} + \frac{0.8893(888)250}{1.1}$$

= (344.79 + 179.48)kN = 524.27 kN

Example 4.5 **Find the block shear strength of a tension member as shown in figure below. The steel is of grade Fe410.**



Solution:

Gross area as shear (A_{vg})

= (115 + 50)8 = 1320 mm²

Net area as shear (A_{vn}) = (115 + 50 − 1.5 × 18)8 = 1104 mm²

Gross area in tension (A_{tg}) = 35 × 8 = 280 mm²

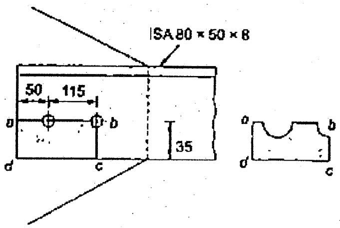
Net area in tension (A_{tn}) = (35 − 0.5 × 18)8 = 208 mm²

Shear yield and tension fracture (T_{db1})

$$= \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{1320 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 208 \times 410}{1.25}$$

$$= (173.21 + 61.4)\text{kN} = 234.61 \text{ kN}$$



Tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}}$$

$$= \frac{280 \times 250}{1.1} + \frac{0.9 \times 1104 \times 410}{\sqrt{3} \times 1.25}$$

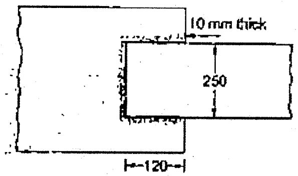
$$= (63.64 + 188.16)\text{kN} = 251.8 \text{ kN}$$

∴ Block shear strength (T_{db})

= Minimum of T_{db1} and T_{db2}

= 234.61 kN

Example 4.6 Find the block shear strength of welded tension member as shown in figure. Use Fe410 steel.



Solution:

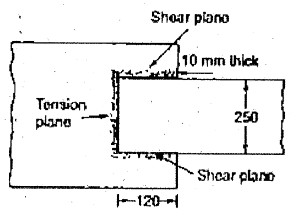
For Fe410, $f_u = 410 \text{ N/mm}^2$, $f_y = 250 \text{ N/mm}^2$

Gross area in shear (A_{vg}) = 2 × 120 × 10 = 2400 mm²

Net area in shear (A_{vn}) = 2 × 120 × 10 = 2400 mm²

Gross area in tension (A_{tg}) = 250 × 10 = 2500 mm²

Net area in tension (A_{tn}) = 250 × 10 = 2500 mm²



Shear yield and tension fracture (T_{db1})

$$= \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{2400 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2500 \times 410}{1.25}$$

$$= (314.92 + 738)\text{kN} = 1052.92 \text{ kN}$$

Tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} = \frac{2500 \times 250}{1.1} + \frac{0.9 \times 2400 \times 410}{\sqrt{3} \times 1.25}$$

$$= (568.18 + 409.04)\text{kN} = 977.22 \text{ kN}$$

∴ Block shear strength = Minimum of T_{db1} and T_{db2} = 977.22 kN

Example 4.7 A tension member 1 m long is required to resist a service dead load of 20 kN and a service live load of 50 kN. Design a rectangular bar of steel grade Fe410 assuming that the member is connected by one line of 16 mm diameter bolts of grade 4.6.

Solution:

For Fe410 grade of steel, $f_u = 410 \text{ N/mm}^2$, $f_y = 250 \text{ N/mm}^2$

Partial safety factors for material, $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$

Partial safety factor for dead and live load, $\gamma_f = 1.5$

For 16 mm diameter bolts, diameter of hole, $d_0 = 18$ mm

Design factored tensile load, $T = 1.5 \times (20 + 50) = 105$ kN

Area required on the basis of net section fracture

$$T = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$\Rightarrow \quad A_n = \frac{T\gamma_{m1}}{0.9 f_u} = \frac{105 \times 1000 \times 1.25}{0.9 \times 410} = 355.69 \text{ mm}^2$$

This net area required should be increased by 25% to have gross area required

$$= 1.25 \times 355.69 = 444.61 \text{ mm}^2$$

Area required on the basis of gross-section yielding

$$T = \frac{A_g f_y}{\gamma_{m0}}$$

$$\Rightarrow \quad A_g = \frac{T\gamma_{m1}}{f_y} = \frac{105 \times 1000 \times 1.1}{250} = 462 \text{ mm}^2$$

Thus required trial area = Larger of 444.61 mm² and 462 mm² = 462 mm²

Let thickness of rectangular bar = 10 mm

$$\therefore \quad \text{Width of bar required} = \frac{462}{10} = 46.2 \text{ mm} \approx 50 \text{ mm}$$

∵ Member is connected by one line of bolts

∴ Rectangular section is weakened by one bolt hole only

∴ Net area provided (A_n) = $(B - nd_0)t$

= (50 − 1 × 18)10

= 320 mm² < 365.69 mm² (Not OK)

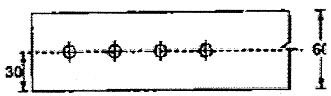
Thus revise width of bar to 60 mm

∴ Net area provided (A_n) = (60 − 1 × 18) 10 = 420 mm² > 355.69 mm² (OK)

Connection

Given 16 mm diameter bolts of grade 4.6

Area of 16 mm diameter bolts (A_{nb}) = 157 mm²



$$\text{Shear capacity of 16 mm diameter bolt } (V_{dsb}) = \frac{A_{nb} f_{ub}}{\sqrt{3}\gamma_{mb}}$$

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

$$\therefore \quad \text{No of bolts required} = \frac{105}{29} = 3.62 \simeq 4 \text{ nos (say)}$$

Provide 4-16 mm diameter bolts at a pitch of 45 mm with an end distance of 35 mm and edge distanc of 30 mm.

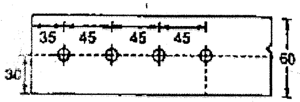
Block shear strength

Gross shear area $(A_{vg}) = (35 + 3 \times 45)10 = 1700$ mm²

Net shear area $(A_{vn}) = (3 \times 45 + 35 - 3.5 \times 18)10 = 1070$ mm²

Gross tension area $(A_{tg}) = 30 \times 10 = 300$ mm²

Net tension area $(A_{tn}) = (30 - 0.5 \times 18)10 = 210$ mm²



Strength due to shear yield and tension fracture

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}} = \frac{1700 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 210 \times 410}{\sqrt{3} \times 1.25}$$

$$= (223.07 + 61.99) \text{ kN} = 285.06 \text{ kN} \qquad \text{(OK)}$$

Strength due to tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} = \frac{300 \times 250}{1.1} + \frac{0.9 \times 1070 \times 410}{\sqrt{3} \times 1.25}$$

$$= (68.18 + 182.36) \text{ kN} = 250.54 \text{ kN}$$

∴ Block shear strength (T_{db}) = Minimum of T_{db1} and T_{db2} = 250.54 kN > 105 kN

Thus section is safe against block shear failure

$$\text{Radius of gyration } (\lambda) = \sqrt{\frac{I}{A}} = \sqrt{\frac{bt^3/12}{bt}} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.887 \text{ mm}$$

Length of the rectangular bar, $(L) = 1$ m $= 1000$ mm

$$\therefore \text{Slenderness ratio } (\lambda) = \frac{L}{r} = \frac{1000}{2.887} = 346.38 < 400 \qquad \text{(OK)}$$

Example 4.8 It is required to design a bridge truss diagonal to carry a factored tensile load of 250 kN. The length of the diagonal is 2.8 m. The tension member is connected to a gusset plate 16 mm thick with one line of 20 mm diameter bolts of grade 8.8. Design the member using angle section and use steel of grade 410.

Solution:

For Fe410, $f_u = 410$ N/mm² and $f_y = 250$ N/mm²

Partial safety factor for material

$$\gamma_{m0} = 1.1$$
$$\gamma_{ml} = 1.25$$
$$\gamma_{mb} = 1.25$$

For bolt of grade 8.8, ultimate tensile stress $(f_{ub}) = 830$ N/mm² for $d > 16$ mm

For 20 mm diameter bolt, stress area of bolt $(A_{nb}) = 245$ mm²

Assuming the threads to be included in the shear plane,

$$\text{Shear strength of bolt} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} A_{nb} = \frac{830}{\sqrt{3}} \frac{(245)}{1.25} N = 93.92 \text{ kN}$$

Let Pitch of the bolts = 65 mm

End distance = 45 mm

Edge distance = 50 mm

$$k_b = \text{minimum of} \begin{cases} \dfrac{e}{3d_0} = \dfrac{45}{3 \times 22} = 0.682 \\ \dfrac{p}{3d_0} - 0.25 = \dfrac{65}{3 \times 22} - 0.25 = 0.735 \\ \dfrac{f_{ub}}{f_u} = \dfrac{830}{410} = 2.024 \\ 1.0 \end{cases} = 0.682$$

Design bearing strength of the bolt

$$\text{Design bearing strength of the bolt} = \frac{2.5 k_b d t f_u}{\gamma_{mb}} = \frac{2.5 \times 0.682 \times 20 \times 8 \times 410}{1.25} \text{N}$$

(Assuming thickness of member section as 8 mm)

$$= 89.48 \text{ kN}$$

$$\therefore \quad \text{Number of bolts required} = \frac{250}{89.48} = 2.8 \simeq 3 \text{ bolts (say)}$$

Provide 3 - 20 mm diameter bolts of grade 8.8 in one line

Required net area on the basis of net section fracture is given by,

$$T = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

$$\Rightarrow \quad A_n = \frac{T\gamma_{ml}}{0.9 f_u} = \frac{250 \times 1000 \times 1.25}{0.9 \times 410} = 846.88 \text{ mm}^2$$

This net area should be increased by 20% to get the gross area

∴ Gross area required = $1.2 \times 846.88 = 1016.26$ mm²

Required gross area on the basis of gross-section yielding is given by,

$$T = \frac{A_g f_y}{\gamma_{m0}}$$

$$\Rightarrow \quad A_g = \frac{T\gamma_{m0}}{f_y} = \frac{250 \times 1000 \times 1.1}{250} = 1100 \text{ mm}^2$$

Try ISA 100 × 75 × 8

Gross area, (A_g) = 1336 mm²

Diameter of hole (d_0) = 22 mm

$$\text{Net area of connected leg } (A_{nc}) = \left(100 - 22 - \frac{8}{2}\right)8 = 592 \text{ mm}^2$$

$$\text{Gross area of outstanding leg } (A_{go}) = \left(75 - \frac{8}{2}\right)8 = 568 \text{ mm}^2$$

Now the tearing strength of the angle section, connected through one leg only is affected by shear lag also. The design strength due to net section fracture (T_{dn}) which is governed by tearing at net section is given by,

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

where,

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{ml}} \ge 0.7$$

Length of outstanding leg (w) = 75 mm

w_1 = 50 mm

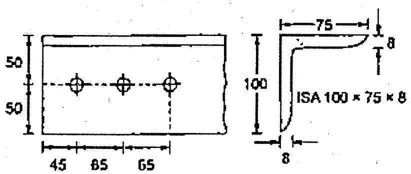
∴ Shear lag width (b_s) = $w + w_s - t$

= 75 + 50 − 8

= 117 mm

Distance between end bolts (L_c) = 2 × 65

= 130 mm



∴ $$\beta = 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{117}{130}\right) = 1.009$$

$$\frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{0.9\times 410\times 1.1}{250\times 1.25} = 1.2988 > 0.7 \qquad \text{(OK)}$$

∴ $$\beta < \frac{0.9 f_u \gamma_{m0}}{f_y \gamma_{m1}}$$

∴ $$\beta = 1.009$$

∴ $$T_{dn} = \frac{0.9\times 592\times 410}{1.25} + \frac{1.009\times 568\times 250}{1.1}$$

$$= (174.76 + 130.25)\text{kN} = 305.01 \text{ kN} > 250 \text{ kN} \qquad \text{(OK)}$$

Check for block shear strength

Gross area in shear (A_{vg}) = (2 × 65 + 45)8 = 1400 mm²

Net area in shear (A_{vn}) = (2 × 65 + 45 − 2.5 × 22)8 = 960 mm²

Gross area in tension (A_{tg}) = 50 × 8 = 400 mm²

Net area in tension (A_{tn}) = (50 − 0.5 × 22)8 = 312 mm²

Shear yield and tension fracture (T_{db1})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{1400\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 312\times 410}{1.25}\text{N}$$

$$= (183.7 + 92.1)\text{ kN} = 275.8 \text{ kN}$$

Tension yield and shear fracture (T_{db2})

$$= \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} = \frac{400\times 250}{1.1} + \frac{0.9\times 960\times 410}{\sqrt{3}\times 1.25}N$$

$$= (90.9 + 163.6)\text{kN} = 254.5 \text{ kN}$$

∴ Block shear strength (T_{db}) = Minimum of T_{db1} and T_{db2}

= 254.5 kN

> 250 kN

∴ Member is safe in block shear

Example 4.9 Two tension members of sections 170 × 10 and 250 × 16 are required to be joined through splicing. The member is carrying a factored tensile load of 280 kN. Design the splice using Fe410 grade of steel and provide 20 mm diameter bolts of 4.6 grade for correction purpose.

Solution:

For Fe410 grade of steel, f_u = 410 N/mm² and f_y = 250 N/mm²

Partial safety factor for material

γ_{m0} = 1.1

γ_{m1} = 1.25

Partial safety factor for bolt γ_{mb} = 1.25

For bolt of grade 4.6, ultimate tensile stress (f_{ub}) = 400 N/mm²

Diameter of 20 mm bolt hole (d_0) = 22 mm

Splice plates are provided on both the sides of tension members

Design tensile strength of the member

Design strength due to gross-section yielding will be governed by strength of smaller section i.e. 170 × 10.

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = (170\times 10)\frac{250}{1.1}N = 386.4 \text{ kN}$$

Design strength due to net section rupture,

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

Now the arrangement of bolts at this stage is not known and thus it is assumed that section is weakened by one bolt hole only

∴ $$A_n = (b - d_0)t = (170 - 22)10 = 1480 \text{ mm}^2$$

∴ $$T_{dn} = \frac{0.9\times 1480\times 410}{1.25}N = 436.9 \text{ kN}$$

∴ Design tensile strength of the member = Minimum of T_{dg} and T_{dn} = 386.4 kN

Now as per IS specifications, the splice connection should be designed for a force of atleast 0.3 times (i.e. 30%) the design capacity of the member in tension or the design action, which ever is more.

∴ Splice connection will be designed for 0.3 × 386.4 kN (= 115.92 kN) or factored tensile load (= 280 kN), which ever is more i.e. 280 kN. Now the members connected are of different thickness and thus packing will be required.

Thickness of packing (t_{pkg}) = 16 − 10 = 6 mm but ≤ 6 mm

The bolts will be in double shear and bearing

For 20 mm diameter bolts, A_{nb} = 245 mm²

Strength of bolt in double shear (V_{dsb})

$$= \frac{2\times A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \frac{2\times 245\times 400}{\sqrt{3}\times 1.25}N = 90.53 \text{ kN}$$

$$\text{Strength of bolt in bearing } (V_{dpb}) = \frac{2.5 k_b\, dt f_u}{\gamma_{mb}} = \frac{2.5\times 1\times 20\times 10\times 410}{1.25}N = 164 \text{ kN}$$

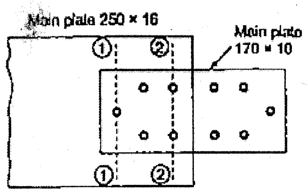
(Assuming k_b = 1 and aggregate thickness of splice plates should be more than minimum thickness of member plates i.e. 10 mm and 16 mm)

Thus strength of bolt = Minimum of V_{dsb} and V_{dpb}

= 90.53 kN

$\therefore$ Number of bolts required $= \dfrac{280}{90.53} = 3.09 \approx 5$ bolts (say)

Provide 5 - 20 mm diameter bolts with arrangement as shown



Size of the splice plate

Width of splice plate = Width of main member

= 170 mm

Critical section for main plate will be 1 - 1 and critical section for splice plate will be 2 - 2.

Let t_{sp} = Thickness of splice plate

$\therefore$ Design action at (2)-(2) = Design tensile strength

$$\therefore \quad 280 \times 1000 = \frac{0.9\, A_n f_u}{\gamma_{mb}} = \frac{0.9(170 - 2 \times 22)2t_{sp} \times 410}{1.25 \times 10^{-3}}$$

$$\Rightarrow \quad 280 \times 1000 = 74390.4\, t_{sp}$$

$$\Rightarrow \quad t_{sp} = 3.76 \text{ mm}$$

The aggregate thickness of the splice plate should not be less than the maximum thickness of main connected member (= 16 mm)

Thus provide 10 mm thick splice

Provide bolts at a pitch of 2.5 d = 2.5 × 20 = 50 mm

Edge distance for 20 mm bolts = 33 mm ≈ 35 mm

Length of splice plate = (35 + 50)4 = 340 mm ≈ 350 mm (say)

Provide 350 × 170 × 10 mm splice plate on each side of the main member with 6 mm packing plate in between the splice plates.

Example 4.10 **Design the section for diagonal member of a roof turns which carries a tensile load of 270 kN. Design the connection of member to 16 mm thick gusset plate also. The length of the connection is not to exceed 350 mm. Provide lug angles if required. Use Fe410 grade of steel and bolts of grade 4.6.**

Solution:

For Fe410 grade steel, f_u = 410 N/mm² and f_y = 250 N/mm²

For 4.6 grade bolt, f_{ub} = 400 N/mm²

Factored tensile load = 1.5 × 270 = 405 kN

Net area required on the basis of net section fracture

$$A_n = \frac{T\gamma_{m1}}{0.9 f_u} = \frac{405 \times 1000 \times 1.25}{0.9 \times 410} = 1371.95 \text{ mm}^2$$

Increase this area by about 25% to have gross area

$$\therefore \quad A_g = 1.25 \times 1371.95 = 1714.94 \text{ mm}^2$$

Try ISA 150 × 75 × 10

$$A_g = 2156 \text{ mm}^2$$
$$r = 15.9 \text{ mm}$$

Let 20 mm diameter bolts are provided

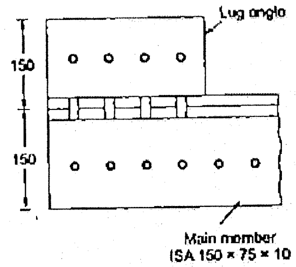
$$\therefore \quad A_{nb} = 245 \text{ mm}^2$$

Diameter of bolt hole (d_0) = 22 mm

Connection of diagonal member with gusset plate

Strength of bolt in single shear $(V_{dsb}) = \dfrac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \dfrac{245 \times 400}{\sqrt{3} \times 1.25}$ N = 45.26 kN

$\therefore$ Number of bolts required $= \dfrac{405}{45.26} = 8.95 \simeq 10$ (say)



Provide 10 - 20 mm diameter bolts at a pitch of 50 mm and edge distance of 45 mm

$\therefore$ Length of gusset plate required for making the connection

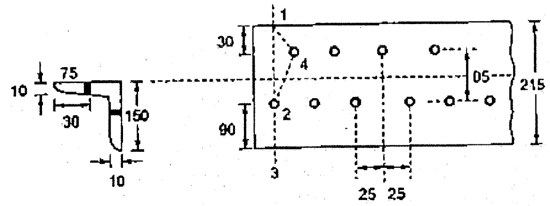
= 9 × 50 + 45 + 45 = 540 mm

> 350 mm

$\therefore$ Provide lug angles

Let us provide 6 bolts for connecting the main member to the gusset plate and 4 bolts for connecting lug angle with the gusset plate

Let the bolts are provided in staggered pattern as shown



Length of gusset plate required = 5 × 50 + 2 × 45 = 340 mm < 350 mm (OK)

Net section along 1 - 2 - 3 = A_{n1} = (215 − 22)10 = 1930 mm²

$$\text{Net section along 1 - 4 - 2 - 3} = A_{n2} = \left(215 - 2 \times 22 + \frac{1 \times 25^2}{4 \times 95}\right)10$$

$$= 1726.45 \text{ mm}^2$$

$$> 1371.95 \text{ mm}^2 \qquad \text{(OK)}$$

Fracture strength of the section (T_{dn})

$$= \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1726.45 \times 410}{1.25} \text{N}$$

$$= 509.65 \text{ kN} > 405 \text{ kN} \qquad \text{(OK)}$$

Design of lug angle

For the design of lug angle, the load in the outstanding leg of the main angle is determined by proportioning the loads in the legs as per the areas.

$$\therefore \text{Gross area of connected leg of main angle} = \left(150 - \frac{10}{2}\right)10 = 1450 \text{ mm}^2$$

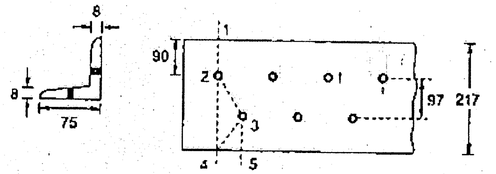
$$\text{Gross area of outstanding leg of main angle} = \left(75 - \frac{10}{2}\right)10 = 700 \text{ mm}^2$$

$\therefore$ Load taken up by outstanding leg of main angle = $\left(\frac{700}{1450+700}\right)405 = 131.86$ kN

$\therefore$ Design load for lug angle $(T_1) = 1.2 \times 131.86$ kN = 158.23 kN

$\therefore$ Net area required for lug angle = $A_{n2} = \frac{T_1 \gamma_{m1}}{0.9 f_u} = \frac{158.23 \times 1000 \times 1.25}{0.9 \times 410} = 536.01$ mm^2

Try ISA 150 × 75 × 8 with 150 mm leg connected to the gusset plate



Net section along 1 - 2 - 4, (A_{n1})

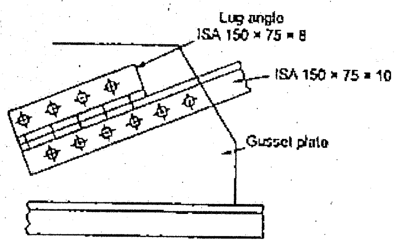
$$= (217 - 22)8 = 1560 \text{ mm}^2$$

Net section along 1 - 2 - 3 - 5 (A_{n2})

$$= \left(217 - 2 \times 22 + \frac{1 \times 25^2}{4 \times 97}\right)8$$

$$= 1396.89 \text{ mm}^2 > 536.01 \text{ mm}^2$$

$\therefore$ Number of bolts required for connecting the lug angle to gusset plate



$$= \frac{158.23}{45.26} = 3.5 \simeq 4 \text{ (say)}$$

Number of bolts required for connecting outstanding leg of the main angle to the leg angle

$$= \frac{1.4 \times 131.86}{45.26} = 4.08 \simeq 6 \text{ (say)}$$

Objective Brain Teasers

Q.1 The design tensile strength of a steel section due to gross-section yielding T_{dg} is given by

(a) $\frac{A_g f_y}{\gamma_{m0}}$ (b) $\frac{A_g f_y}{\gamma_{m1}}$

(c) $\frac{\gamma_{m0}}{A_g f_y}$ (d) $\frac{\gamma_{m1}}{A_g f_y}$

Q.2 The design strength of a tension member is given by minimum of

(i) Block shear strength of end region
(ii) Rupture of critical section
(iii) Yielding of gross-section

Of the above statements, the correct one(s) is/are

(a) (i) only (b) (i) and (ii)
(c) (ii) and (iii) (d) (i), (ii) and (iii)

Q.3 The design tensile strength of plate due to rupture of critical section is given by

(a) $\frac{0.75 A_g f_u}{\gamma_{m1}}$ (b) $\frac{0.8 A_n f_u}{\gamma_{m0}}$

(c) $\frac{0.8 A_n f_u}{\gamma_{m1}}$ (d) $\frac{0.9 A_n f_u}{\gamma_{m1}}$

Q.4 For the design tensile strength of plate due to rupture of critical section which of the following is/are considered?

(i) Shear stress
(ii) Bearing stress
(iii) Direct stress

The correct one(s) is/are

(a) (i) and (ii) (b) (ii) and (iii)
(c) (i) and (iii) (d) (i), (ii) and (iii)

Q.5 The minimum thickness of gusset plate to be used should not be less than

(a) 6 mm (b) 8 mm
(c) 10 mm (d) 12 mm

Q.6 A tension splice section is designed for

(a) Maximum factored tensile load
(b) Design strength of the tension member
(c) Maximum service load
(d) Maximum of factored tensile load and 0.3 times the design strength of tension member

Q.7 For arriving at net area of flat provide with staggered bolts, ______ area is reduced from the gross-sectional area.

(a) $\left(nd_n - \frac{n' p^2}{4g}\right)t$ (b) $\frac{n' p^2}{4g}t$

(c) $\left(nd_n + \frac{n' p^2}{4g}\right)t$ (d) None of these

Q.8 The preferred leg of unequal angle section for making connection to be used as a tension member is

(a) Short leg
(b) Long leg
(c) Any of short a long leg
(d) Depends on magnitude of tensile force

Q.9 Block shear strength of a tension member is

(i) Fracture strength of shear plane and yield strength of tension plane
(ii) Fracture strength of tension plane and yield strength of shear plane

Choose the correct statement

(a) Maximum of (i) and (ii)
(b) Minimum of (i) and (ii)
(c) Summation of (i) and (ii)
(d) None of these

Q.10 The block shear failure of tension member involves failure planes which are

(a) two mutually perpendicular tension planes
(b) two mutually perpendicular shear plane
(c) tension on one plane and shear on other perpendicular plane
(d) tension on one plane and compression on other perpendicular plane

Q.11 Which limit states allow the excessive elongation of a tension member?

(a) T_{dn} (b) T_{db}
(c) T_{dg} (d) All of the above

Q.12 The top edge of gusset plate is stiffened if length of top edge exceeds

(a) $42 t\varepsilon$ (b) $30 t\varepsilon$
(c) $10 t\varepsilon$ (d) $9.4 t\varepsilon$

Q.13 Strictly, a gusset plate must be able to resist

(a) shear stress (b) bending stress
(c) direct stress (d) All of the above

Q.14 Ductility enhances the strength of bolted member by

(a) having a more uniform stress distribution in the plastic state
(b) concentrating stress distribution around the bolt holes
(c) carrying more load in elastic state
(d) resulting in reduced net sectional area

Q.15 Geometry factor is defined as

(a) pitch/bolt diameter
(b) gauge/bolt hole diameter
(c) gauge/bolt diameter
(d) pitch/bolt hole diameter

Q.16 The effect of shear lag is

(a) less in small length of connection
(b) less in large length of connection

(c) nonexistent in angle sections
(d) nonexistent in channel section

Q.17 Which is more preferable as a tension member?
(a) Channel section connected with web
(b) Equal legged angle section
(c) Unequal angle section with smaller leg as connected leg
(d) Unequal angle section with larger leg as connected leg

Q.18 When less number of high strength bolts are used for making the connection with smaller connection length, which of the following types of failure is prove to occur?
(a) Yielding of gross-section
(b) Rupture failure
(c) Tearing failure of plate
(d) Block shear failure

Q.19 Which of the following is considered as a limit state?
(i) Attainment of yield point of tension member
(ii) Excessive elongation of the tension member
(a) (i) only (b) (ii) only
(c) (i) and (ii) (d) Neither (i) nor (ii)

Q.20 "Stress concentration factor" is defined as
(a) $\frac{f_{avg}}{f_{max}}$ (b) $|f_{avg} - f_{max}|$
(c) $\frac{f_{max}}{f_{avg}}$ (d) $f_{max} \cdot f_{avg}$

Answers

1. (a)	2. (d)	3. (d)	4. (d)	5. (d)
6. (d)	7. (a)	8. (d)	9. (b)	10. (c)
11. (c)	12. (a)	13. (d)	14. (a)	15. (c)
16. (b)	17. (d)	18. (d)	19. (b)	20. (c)

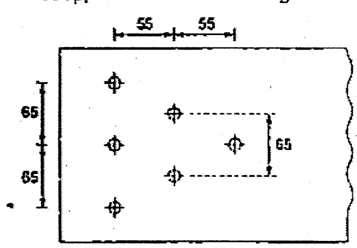
Conventional Practice Questions

Q.1 A T-section 3.2 m long is required to carry a tensile load of 280 kN. Design the section.

Q.2 Design a tension member with channel section which is 3.6 m long and is required to carry a tensile load of 565 kN.

Q.3 Two angles are placed back to back with long legs as connected leg. The length of the member is 3.5 m and is required to carry a tensile force of 220 kN. Design the tension member.

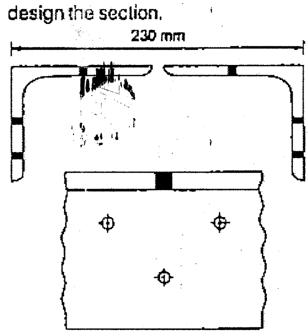
Q.4 20 mm diameter bolts of grade 4.6 are used to connect two plates of thickness 12 mm. Determine the maximum service load that can be applied under tension through the member.



Q.5 Design a lug angle to be provided with a tension member subjected to a tensile load of 320 kN. The length of the member being 2.95 m.

Q.6 250 kN service tensile load is required to be transmitted through steel flats 250 mm wide x 10 mm thick. Design a suitable splice to connect the flats.

Q.7 Two equal angle sections are used as tension member to carry a tensile force of 710 kN. Using 20 mm diameter bolts of grade 4.6, design the section.



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