CHAPTER 11

Eccentric and Moment Connections

11.1 Different types of Connections

Flexible connection: Connection of beam to column that is supposed to resist and transfer the beam reaction only to the column is referred to as **flexible connection** or **shear connection**. This type of connection allows having free rotation at the beam end and do not have any moment restraint.

Rigid connection: Contrary to above type of connection, connection of beam to column which do not permit any rotation at the beam end and is supposed to resist and transfer the beam reaction to the column is referred to as **moment connection** or **rigid connection**. e.g. **Welded moment restraint** connection which is in fact nearly cent percent rigid.

Semi-rigid connection: In between the above two types of connection, there is another type of beam-column connection that partially resists end moments and also allow relative rotation between the beam and column. This type of connection is referred to as **semi-rigid connection**.

NOTE: Most of the connections are neither perfectly rigid nor totally flexible and are partially restrained to one degree or another.

Fig. 11.1 shows moment-rotation relationship for different types of connections which shows moment (M) as a function of relative rotation (θ) of the elastic lines of the connected members at their point of intersection.

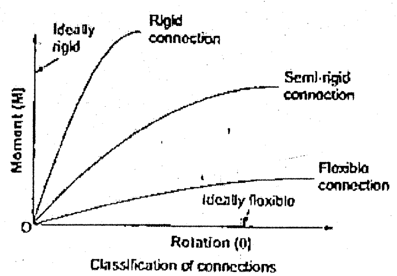


Fig. 11.1 Theoretical moment-rotation characteristics for connections

Fig. 11.2 shows some simple flexible connections, Fig. 11.3 shows some typical rigid connections and Fig. 11.4 shows some semi-rigid connections.

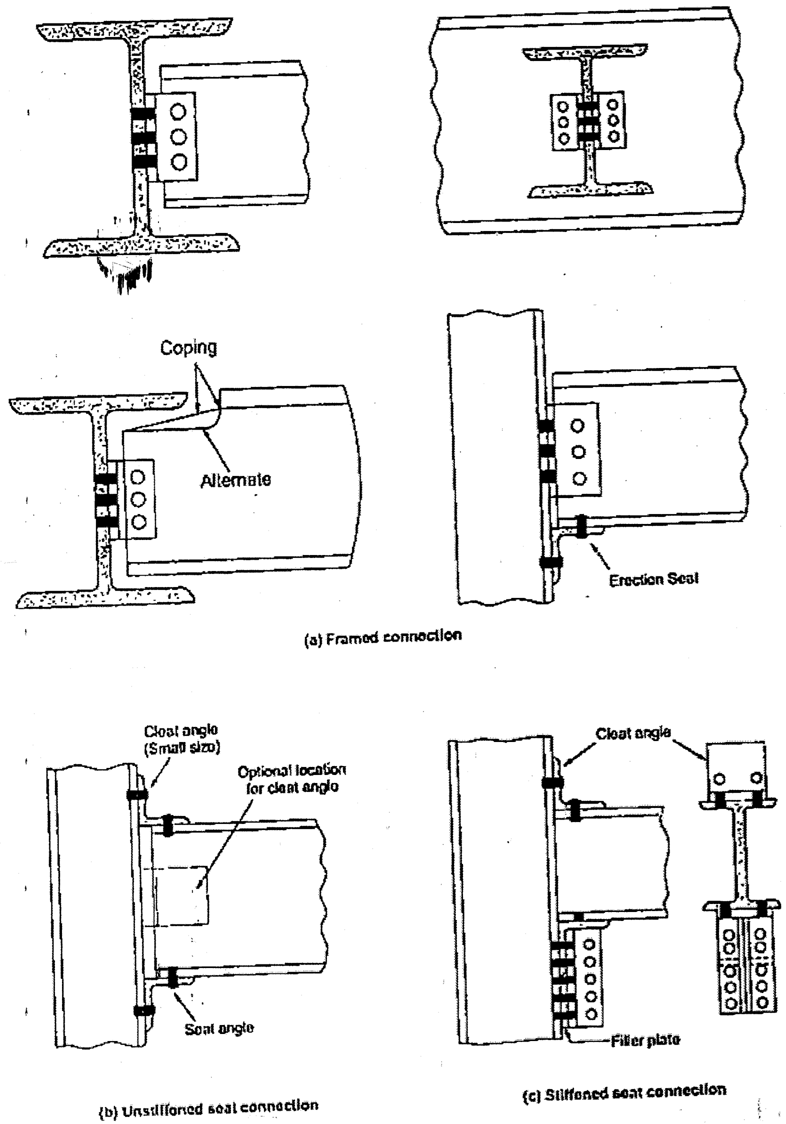


Fig. 11.2 Some simple flexible connections

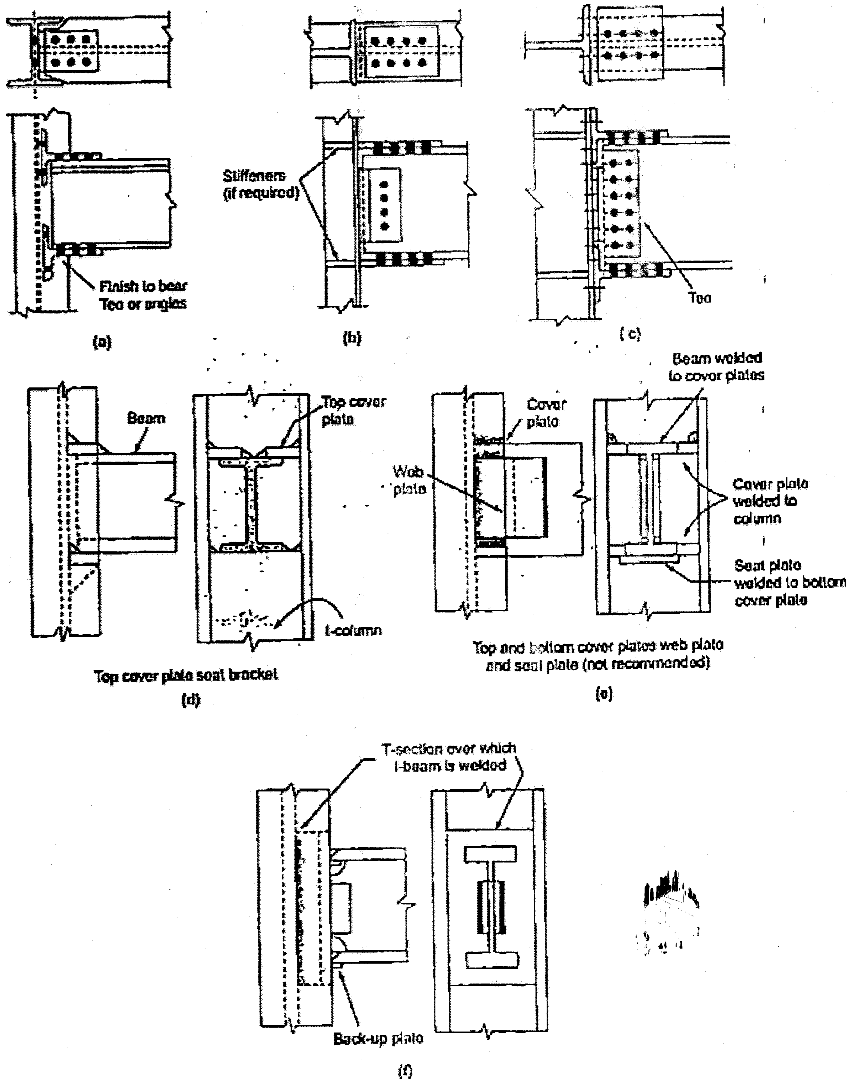


Fig. 11.3 Some rigid connections

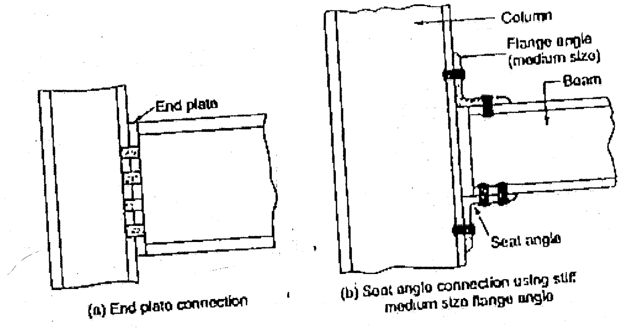


Fig. 11.4 Some semi-rigid connections

- The ratio of the moment developed at the connection to the moment that can be developed by a totally rigid connection is taken as a basis to classify the type of connection.

Do You Know ?

Rigid connections are those which when jointed with absolutely fixed supports, are capable to developing 90% or more of fixed end moment. In a similar way, **simple connections** are defined as those connections which when jointed in a similar way will not develop more than 25% of fixed end moment. In between these two extremes, lies the semi-rigid connection.

- IS 800 differentiates various connections according to their ultimate strength or as per their initial elastic stiffness (**Bjorhovde's classification**) which is based on non-dimensional moment parameter ($m = M_u/M_{pb}$) and the non-dimensional rotation parameter ($\theta = \theta_r/\theta_p$) where M_{pb} is the moment in beam at the intersection of beam-column centerline, θ_p is the plastic rotation, θ_r is defined in **Appendix F 4.3.2** of IS 800:2007.

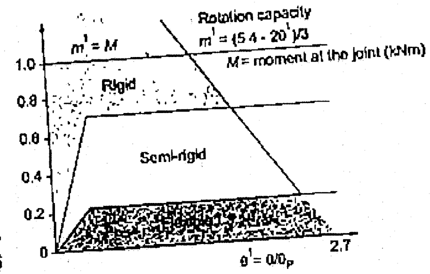


Fig. 11.5 Bjorhovde's classification of connections

- Bjorhovde's classification is based on reference length of beam which is equal to five times the beam depth. Table 11.1 shows the various limits for connection classification and the same is represented in graphical form as shown in Fig. 11.5.

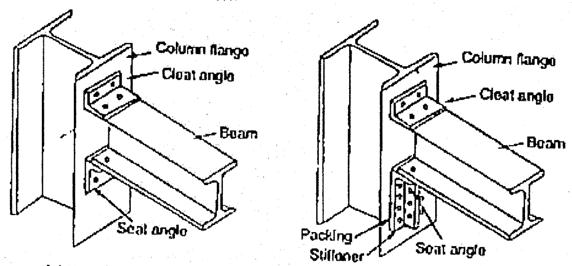
Table 11.1 Limits for classification of connections

S.No.	Type of connection	Classification in terms of strength	Classification in terms of stiffness
1.	Rigid connection	m = 0.7	m = 2.50
2.	Semi rigid connection	0.7 > m > 0.2	2.50 > m > 0.50
3.	Flexible connection	m = 0.2	m = 0.50

11.2 Beam Column Connections

(a) Seat connection:

- In the seat connection, the beam is connected to column with an angle called as seat angle provided at the bottom of beam, where one leg of the angle is used to make seat for the beam and other leg is connected to the column flange.
- In addition to seat angle, another angle is also provided called as **cleat angle/clip angle** provided at the top as shown in **Fig. 11.6(a)**.
- Many a times it may be quite possible that there is not enough space available at the top of the beam for cleat angle. In that case, the cleat angle can be placed in the web portion of the beam as shown in Fig. 11.2(b) by dotted lines. This cleat angle wherever provided i.e. either at the top or in the web portion of the beam helps in keeping the top flange of the beam from being getting twisted accidently during construction etc.
- In addition to that, cleat angles provide lateral stability to compression flange of the beam at supports by providing restraint at the beam ends against torsion. This connection is referred to as **unstiffened seat connection.**
- In unstiffened seat connection, the connection bears the load due to bending strength of horizontal leg of seat angle that is connected to beam.
- When reactions are considerably large, the bending strength of this seat angle proves to be insufficient and thus it is required to be stiffened by another angle(s) making a connection that is known as **stiffened seat connection** as shown in **Fig. 11.6(b)**. Additional packing plates are also required for this type of connection.



(a) Unstiffened seat connection (b) Stiffened seat connection

Fig. 11.6 Seat connection (bolted)

(b) Framed connection:

- Sometimes, angle sections are connected on either side of the web of the beam to connect the beam to the column. This type of connection is referred to as **framed connection** as shown in Fig. 11.7.
- Seat angle may be provided to aid in erection but this does not carry any load.
- When a beam is connected to another beam at right angles to each other through angles on either side of one of the beam then this connection is also called as **framed connection** as shown in Fig. 11.8. This connection is being primarily designed for shear.

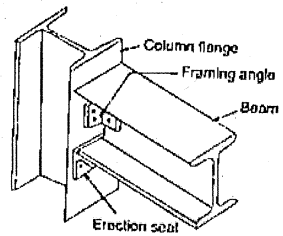
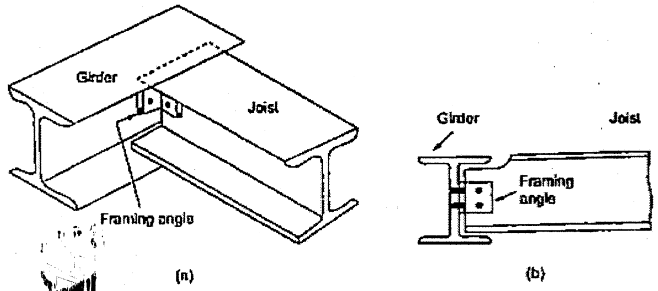


Fig. 11.7 Beam-column framed connection (bolted)



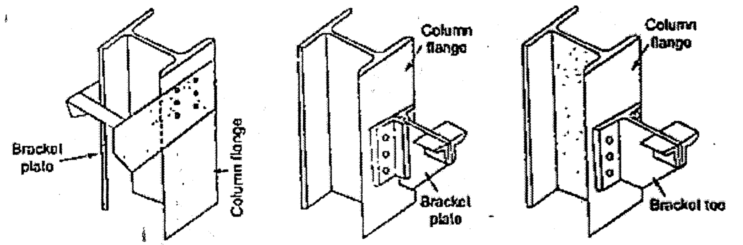
(a) (b)

Fig. 11.8 Beam-beam framed connection (bolted)

- Another recourse of joining the members is welding. In case of welded connections, the members can be welded directly to each other even without any additional plates or angles. But this direct connection requires butting of the member sections together and for this, it is essential to cut the members of exact size which is quite difficult in field practice. This imperfection leads to undesirable stresses which may even alter the behaviour of the connection making the connection prone to failure.

(c) **Bracket connection type-I:** Another possibility of connecting a beam to column is through brackets as shown in **Fig. 11.9(a)**. Brackets are made of two plates connected to column flanges. The type of connection shown in **Fig. 11.9 (a)** is called as **bracket connection type-I**. Here the connectors are subjected to shear force and additional shear due to torsion.

(c) **Bracket connection type-II:** Sometimes, angles are provided on either side of the bracket plate (**Fig. 11.9(b)**) or even a T-section is provided as **bracket plate (Fig. 11.9(c))** and connected to column flange. This type of connection is called as **bracket connection type-II**. Here the connectors are subjected to direct shear and tension due to bending.



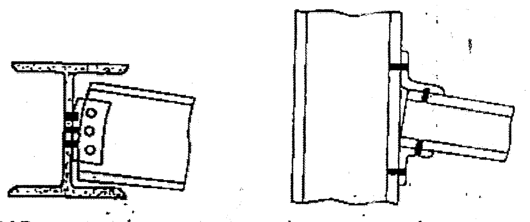
(a) Bracket connection-type I (b) Bracket connection-type II (using plate section) (c) Bracket connection-type II (using tee section)

Fig. 11.9 Bracket connections (bolted)

11.3 Connection Subjected to Eccentric Shear

- Seat connection, framed connection and bracket connection are the examples of connections subjected to shear.

- Seat and framed connections are so designed that they transmit the beam reaction very close to the column face without generating any considerable amount of moment.
- Further to make these connections to have minimum moment of resistance, the angles used for making the connection should be light and flexible as far as possible.
- In order to have simple end connections in these, the connecting beams must be as free to rotate (downwards) as possible as shown in Fig. 11.10. For this to happen, a gap of around 10 – 15 mm is provided.



(a) Framed beam connection (b) Seated beam connection

Fig. 11.10 Connections with simple end support conditions

- The second type of connection i.e. the bracket connection, supports reactions (from beams) at significant distance (i.e. eccentricity) from the column. These type of connections do develop considerable amount of moments.

11.4 Framed Connection-Bolted

- Mainly the connection of floor beams to other beams, columns and girders is made through framed connection.
- The framing angles are used in pairs one on each side of web of the connecting beam as shown in Fig. 11.7 and 11.8.
- The connected leg of the framing angle is connected to the web of the connecting beam (done usually in shop).
- The outstanding leg of the framing angle is connected to the flange of the column (done in field). Generally 8 mm, 10 mm or 12 mm thick angles are used.

Remember The thickness of the framing angle is so adopted that strength of the bolt is not affected by bearing on the framing angles. However thickness of the framing angles are so limited that flexibility is ensured.

11.4.1 Procedure for the Design of Framed Connection

Step-1. Compute the end reaction which the beam will transmit.

Step-2. Connection of the framing angle with the web is designed. This is usually done in the shop. After assuming the bolt diameter, the strength of the bolt in double shear and bearing are calculated.

The computed number of bolts is given by,

$$n = \frac{\text{End reaction}}{\text{Strength of the bolt}}$$

Step-3. In case the outstanding leg of the angle is connected to the web of the girder, the bolt diameter is assumed and strength of the bolt is arrived at.

Step-4. The number of rows of bolt on the connected and the outstanding legs govern the size of the framing angle.

Step-5. The thickness of the angle is arrived at by equating end reaction to the shear strength of the angle leg.

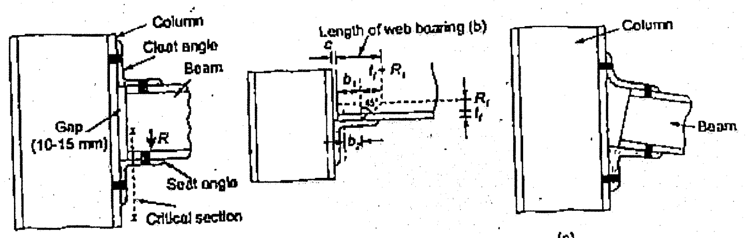
The strength of angle leg (V_d) is given by.

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} h(2t_w) \qquad ...(10.1)$$

where h = Depth of framing angle. t_w = Thickness of framing angle. γ_{m0} = Partial factor safety = 1.1

In Eq. (10.1) the thickness t_w has been multiplied by a factor 2 because of pair of angles on each side of the beam web.

11.5 Seat Connection-Bolted



(a) (b) (c)

Fig. 11.11 Unstiffened seat connection-Bolted

- It is also a flexible connection made by using seat angles below the beam as shown in Fig. 11.6.
- The seat angle aids in erection of beam at correct location. Also the seat legs provide a support on which a beam can rest during the erection process.
- Two types of commonly used seat connections are the unstiffened seat connection where an angle is provided below the beam flange and is designed to transmit the beam reaction to the main beam or column as shown in Fig. 11.11 and stiffened seat connection as shown in Fig. 11.12.

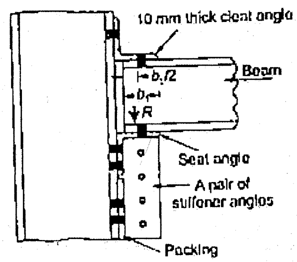


Fig. 11.12 Stiffened seat connection-Bolted

11.5.1 Unstiffened Seat Connection

- In the unstiffened seat connection, the connecting leg of the angle with the beam is usually kept as 100 mm long enough to check the web crippling of the beam. A minimum of two bolts are used for making a connection.

- It is required to keep a clearance of about 10 – 15 mm between the beam and the other main beam or column.
- It is a flexible type of connection wherein the seat angle is assumed to be flexible.
- For stability reasons, another angle is also provided at the top of upper flange of the beam called as **cleat angle/clip angle.** It also provides lateral support to the compression flange of the beam at end support regions.

NOTE It is assumed that unstiffened seat connection does not provide any moment restraint (i.e. rotation restraint) in the vertical plane at the beam ends and is thus designed to transmit the reactions only.

It can be seen from Fig. 11.11 (c) that as the beam rotates in the vertical plane, the beam ends rotate and induce a moment at the connection resulting in distortion of cleat angle and bolts get elongated. To relieve the bolts from tension, it is preferred to keep the gauge large and angles to be thin i.e. cleat angles to be used must be flexible. The amount of load that can be supported by the seat angle depends on the flexural resistance of the horizontal leg of the seat angle.

11.5.2 Stiffened Seat Connection

- For heavier loads, seat angles only are not sufficient to carry the load and so stiffening angles are required as shown in Fig. 11.12.
- Here the outstanding leg of the seat is stiffened by an angle and thus seat angle does not remain flexible.

NOTE The outstanding leg of the seat angle is required to be stiffened when the reaction to be transmitted is very large (i.e. heavy loads are acting on the beam) and/or 100 mm seating leg is not able to provide the required bearing area. In addition to that, due to large reaction, the number of bolts required to join the connecting leg of the seat angle to the column may be so large that more than two or three rows of bolts are required to be provided. There is a limit on number of bolts that can be provided on the seat angle and thus remaining bolts are provided on another angle called as **stiffener angle.** Packing plates of thickness equal to thickness of the seat angle are provided between the column and stiffener angle.

- The stiffener angle relieves the seat from flexure. But it is difficult to trim the stiffener angle to the profile of fillet of seat angle and thus effective width of the outstanding leg of the seat angle in bearing is taken as actual length of the outstanding leg of the seat angle less the radius of fillet.
- A pair of stiffener angles is provided and outstanding legs are lack riveted/bolted.
- The bolts connecting the column flange to the stiffener angle are subjected to direct shear and moment. If the outstanding leg of the stiffener angles are not tack bolted then the two stiffener angles act independently and the bolts connecting the stiffener angle to the column flange are designed for shear, moment and torsion.

11.5.3 Procedure for the Design of Unstiffened Seat Connection

Step-1. It is assumed that length B of the seat angle is equal to width of beam flange.

Step-2. The length of outstanding leg of the seat angle is arrived at on the basis of web crippling of the beam. It is customary to keep the length of seat angle more than the calculated bearing length which is given by,

$$b = \frac{R\gamma_{m0}}{t_w f_{yw}} \qquad \text{...(11.2)}$$

where R = Reaction coming from the beam, t_w = Web thickness of the beam

f_{yw} = Yield strength of the web, γ_{m0} = Partial factor of safety = 1.1

Step-3. Load dispersion takes place at 45° from the bearing on the seat to the root line as shown in Fig. 11.11 (b). The length of this bearing on seat is given by,

$$b_1 = b - (t_f + R_1) \qquad \text{...(11.3)}$$

where t_f = Flange thickness of the beam

R_1 = Root radius of the beam flange

Also, the distance between the ends of bearing on seat to root of the angle is,

$$b_2 = b_1 + c - (t_a + R_a) \qquad \text{.....(11.4)}$$

where t_a = Thickness of angle

R_a = Radius of root of the angle as given in IS Handbook No. 1

c = Clearance and tolerance

Step-4. The connected leg is so chosen that at least two horizontal rows of bolts can be accommodated.

Step-5. It is assumed that beam reaction is uniformly distributed over the bearing length b_1. The thickness of outstanding leg of the seat angle is so chosen that the seat angle should not get fail by bending on a section at toe of the fillet. The bending moment is,

$$M = \frac{R}{b_1} \cdot b_2 \cdot \frac{b_2}{2} \qquad \text{...(11.5)}$$

The flexural capacity of the angle leg is given by,

$$M_d = 1.2 Z_e \frac{f_y}{\gamma_{m0}} \qquad \text{...(11.6)}$$

where Z_e = Section modulus of the outstanding leg of the seat angle that provides bearing.

If the assumed angle thickness is not sufficient (i.e. $M_d < M$) then section has to be revised.

Step-6. The shear capacity of the outstanding leg of the seat angle is given by,

$$V_{dp} = B t_a \frac{f_y}{\sqrt{3}\gamma_{m0}} \qquad \text{...(11.7)}$$

where B = Length of the angle

= Width of the flange of the supported beam as stated in Step-1 above.

Step-7. Assume a suitable bolt diameter and compute number of bolts required to connect the seat angle with the column flange as,

$$\text{No. of bolts required } (n) = \frac{\text{End reaction}}{\text{Strength of the bolt}}$$

The outstanding leg of the seat angle is connected to the beam flange with two bolts of preferably the same diameter as provided for the angle leg connected to the column flange.

Step-8. A clip/cleat angle of nominal size is provided at the top of the beam flange and is connected by two bolts of preferable the same diameter as provided for the seat angle.

11.5.2 Procedure for the Design of Stiffened Seat Connection

Step-1. On the basis of bearing length required, the size of the seat angle is assumed.

Step-2. The outstanding leg of the stiffener angle must provide the bearing area required and also to avoid its local buckling, the length of the outstanding leg must not exceed $14t_a\varepsilon$ where t_a is the angle thickness.

The required bearing area is worked out as,

$$A = \frac{R\gamma_{m0}}{f_y}$$

where R = Reaction from the beam being supported

f_y = Yield strength

γ_{m0} = Partial factor of safety for the material = 1.1

The thickness of the stiffener angle should be more than the thickness of the web of the beam being supported.

Step-3. Here the seat is not required to be flexible and thus the reaction is assumed to have greater eccentricity. The bolts that connect the angle leg to the column flange are subjected to both moment and direct shear. Behaviourwise, this connection is more or less is like bracket connection type-II. To make computations easier, it is assumed that the reaction from the supported beam acts at the middle of the bearing length.

Step-4. Bending moment at the face of the column is computed.

Step-5. The computation of number of bolts required and the associated checks are similar to that of bracket connection type-II.

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_{sb}}{T_{db}}\right)^2 \le 1 \qquad ...(11.6)$$

Step-6. A clip/cleat angle of nominal size is provided at the top of the beam flange and is connected by two bolts of preferable the same diameter as provided for the seat angle.

11.6 Bolted Bracket Connection

- As shown in Fig. 11.9 bolted bracket connections are required at places where the two members to be connected do not intersect each other. This leads to enhanced eccentricity both in the connection and the member to be jointed and thus structurally it is not preferable.
- The bolts in this type of connection are subjected to shear and torsion/bending due to eccentric shear.
- When the bolts are subjected to direct shear and torque due to shear then the connection is classed as **bracket connection type-I** (Fig. 11.9(a)).
- When the bolts are subjected to shear and tension then the connection is called as **bracket connection type-II** (Fig. 11.9(b)).

- There are two different approaches for analyzing the bracket connection type-I viz. the elastic method and the ultimate strength method.
- In the elastic method, the friction between the connected parts i.e. frictional resistance is altogether ignored and the connected parts are assumed to be perfectly rigid with connectors being perfectly elastic. The elastic method of analysis gives very conservative results.
- The second approach viz. the ultimate strength method gives results that are more close to the realistic values but this method is quite difficult to apply in routine designs. IS 800 does not specify any particular method for the analysis of bracket connection type – I and thus it depends on the discretion of the designer to use any of the analysis method.

11.6.1 Bolted Bracket Connection Type-I

- In this connection, the twisting moment in the bolts is in the plane of the connection as shown in Fig. 11.13.
- This occurs when line of action of load is in the plane of connection and the center of gravity of the connection in **elastic method** or the instantaneous center in the **ultimate strength method** is the center of rotation. The whole bolt group is subjected to shear and torsion.

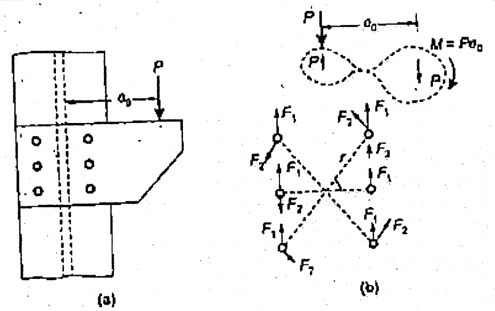


Fig. 11.13 Forces in the bolt of bracket connection type-I

Elastic Analysis

The eccentric axial load P is made concentric along with torque M as shown in Fig. 11.13(b).

Let F_1 = Force in the bolts due to direct shear, F_2 = Force in the bolts due to torsion

The resultant force in each bolt (due to F_1 and F_2) must be less than the strength of the bolt. It is assumed that load on the joint is shared equally by all the bolts and thus force in any bolt due to direct load is,

$$F_1 = \frac{P}{n} \qquad ...(11.9)$$

Also, force in any bolt due to torsion is directly proportional to its distance from the center of rotation of the connection i.e.

$$F_2 \propto r$$

$$\Rightarrow \quad F_2 = kr$$

$$\Rightarrow \quad k = \frac{F_2}{r}$$

Thus torque about center of rotation of the bolt group = $F_2 r = kr^2$

Total resisting torque = $\Sigma kr^2 = k\Sigma r^2 = \frac{F_2}{r}\left(\Sigma r^2\right)$

The resisting torque must be equal to or greater than the torque on the connection i.e.

$$M = \frac{F_2}{r}\left(\Sigma r^2\right)$$

$$\Rightarrow \quad Pe_0 = \frac{F_2}{r}\left(\Sigma r^2\right)$$

$$\Rightarrow \quad F_2 = \frac{Pe_0 r}{\Sigma r^2} \quad ...(11.10)$$

This force F_2 is maximum when distance r is maximum, let it be r_n.

$$\text{Thus,} \quad F_2 = \frac{Pe_0 r_n}{\Sigma r^2} \quad ...(11.11)$$

The two forces F_1 and F_2, act at different angles on various bolts of the connection. Let for any one such typical bolt, the angle between the two said forces is θ. Thus the resultant F of the forces F_1 and F_2 on the critical bolt is,

$$F = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta} \quad ...(11.12)$$

For connection to be safe, this force F must be less than the strength of the bolt.

Here e_o = Perpendicular distance measured from center of rotation of the bolt group to the line of action of load which is called as eccentricity.

$r_1, r_2 ... r_n$ = Distances of the bolts from the center of rotation of the bolt group

n = Number of bolts in the bolt group

F_1 = Force in the bolt due to direct shear P

F_2 = Force in the bolt due to torque M

k = A constant

From Eq. (11.12), it can be seen that resultant force F will be maximum when angle θ is the minimum.

Remember: Bolt subjected to maximum resultant force i.e. the critical bolt is the one that is farthest away from the center of gravity of the bolt group and nearest to the applied line of load.

Ultimate Analysis

- The elastic method of analysis is quite easy to apply and gives conservative results but the assumption made in elastic analysis that applied load and bolt deformation has a linear relationship and yield stress is not exceeded are not true.
- This drawback is overcome by the ultimate method of analysis wherein the method uses experimentally determined non-linear load deformation relationship for individual bolts and thus gives quite realistic values. The following are the assumptions made in the ultimate load method:

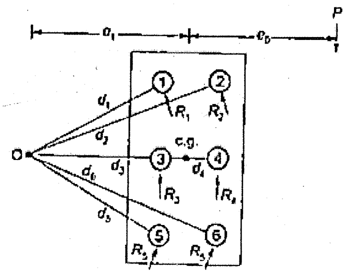


Fig. 11.14 Instantaneous center of rotation in the ultimate method of analysis

(a) **At failure, the bolt group rotates about a point called as instantaneous center of rotation:** In the ultimate method of analysis, if one of the outermost bolts starts to slip or yield, the connection does not fail at all. But the inner bolts will now take more loads under increasing load and failure occurs when all the bolts of the bolt group slip or yield. The eccentric load causes relative rotation and translation of the connected parts which is equivalent to pure rotation of the connection about a point called as instantaneous center of rotation, represented by point O situated at a distance e_1 from the CG of the bolt group as shown in Fig. 11.14.

(b) **The deformation in each bolt is proportional to its distance from the instantaneous center of rotation and acts normal to the radius of rotation:** The ultimate shear force that a bolt in a joint can resist is not the pure shear force that it can resist but it is dependent on the load deformation characteristics. The load applied to a bolt depends on its position in the joint w.r.t. instantaneous center of rotation.

(c) **The capacity of the connection is said to have reached when ultimate strength of the bolt farthest from the instantaneous center is reached:** The resisting force in the bolts of the connection are shown as R_1, R_2, . . . R_n in Fig. 11.14. Each of these force is assumed to act in the direction normal to the line drawn from point O to the center of the bolt.

(d) **The connected parts remain rigid:** In a symmetrical connection, the instantaneous center of rotation will lie on a horizontal line through the CG of the bolt group. This is so because the sum of the moments of the horizontal components of the force R must be zero and also the sum of the moments of the horizontal components about point O. The position of point O is found by trial and error which is a quite difficult process.

11.6.2 Procedure for the Design of Bolted Bracket Connection Type-I with Elastic Method

Step-1 Assume a suitable diameter of the bolt and number of bolts required. Place the bolts in two or more vertical rows at suitable pitch and edge distance.

The number of bolts in one vertical row can be had from Eq. (11.13)

$$n = \sqrt{\frac{6M}{n' p V_{sd}}} \quad ...(11.13)$$

where p = Pitch of the bolts

n' = Number of rows of bolts

V_{sd} = Design strength of the bolt

Step-2 The resultant force on the critical bolt is computed from Eq. (11.12).

Step-3 Strength of the bolt is computed and it should be more than the resultant force on the critical bolt.

11.6.3 Bolted Bracket Connection Type-II

- Here moment is in the plane normal to the plane of connection.
- The line of action of load does not lie in the plane of bolt group. Also the line of rotation does not pass through C.G. of the bolt group.

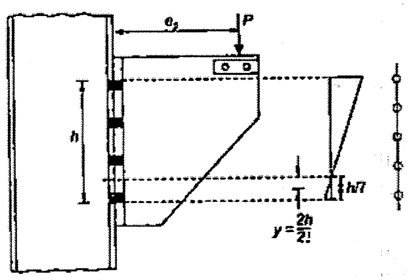


Fig. 11.15 Bolted bracket connection type-II

The forces acting on bolts are

(i) Direct shear

(ii) Tension due to moment

Now force is shared equally by all the bolts thus,

$$F = \frac{P}{n} \quad ...(11.14)$$

Bolts above the line of rotation are in tension and bracket section below the line of rotation will be in compression. It is assumed that line of rotation lies at ($h/7$) from the bottom of bracket where,

h = Distance from bottom of bracket to top most bolt in the connection

Thus bolts above line of rotation will experience direct shear and tension due to moment. Bracket below the line of rotation provides necessary compression.

Now tensile force in i^{th} bolt, $T_i \propto y_i$

$\Rightarrow$ $T_i = ky_i$

$\Rightarrow$ $k = T_i/y_i$

$\therefore$ Moment of resistance due to tensile force in i^{th} bolt,

$$M_i = T_i y_i = ky_i^2$$

Total moment of resistance due to tensile force in bolts,

$$M' = \Sigma M_i = k\Sigma y_i^2 = \frac{T_i}{y_i} \Sigma y_i^2$$

$$\Rightarrow \quad T_i = \frac{M' y_i}{\Sigma y_i^2} \quad ...(11.15)$$

$\therefore$ Tensile force in farthest bolt, $T_n = \dfrac{M' y_n}{\Sigma y_i^2}$ ...(11.16)

For equilibrium, Total compression = Total tension

$$\Rightarrow \quad C = T$$

$$\Rightarrow \quad C = \frac{M' \Sigma y_i}{\Sigma y_i^2}$$

Also, External moment = Moment of resistance due to tensile force in bolts + Moment of resistance due to compression force on column flange

$$\Rightarrow \quad M = M' + C\bar{y}$$

$$\Rightarrow \quad M = M' + \left(\frac{M'\Sigma y_i}{\Sigma y_i^2}\right)\bar{y}$$

$$\Rightarrow \quad M = M'\left(1 + \frac{\Sigma y_i}{\Sigma y_i^2} \times \frac{2h}{21}\right) \qquad \left(\because \bar{y} = \frac{2h}{21}\right)$$

$$\Rightarrow \quad M' = \frac{M}{1 + \frac{2h}{21} \times \frac{\Sigma y_i}{\Sigma y_i^2}}$$

where, $M = Pe_0$ (due to eccentric load P)

e_0 = eccentricity of load P from the plane of bolt group.

11.6.4 Procedure for the Design of Bolted Bracket Connection Type-II

Step-1. Assume bolt diameter, pitch and edge distance

Step-2. Compute strength of the bolt

Step-3. Compute number of bolts required from Eq. (11.13)

Step-4. Compute shear force F in the extreme bolt (= P/n)

Step-5. Compute tensile force T in the extreme bolt from Eq. (11.16)

Step-6. Check the connection for collective effect of shear and tension as:

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

Where, V_{sb} = Shear force in the bolt

V_{dsb} = Design shear strength of bolt

T_b = Tensile force in the bolt

T_{db} = Design tensile strength of bolt

11.7 Welded Bracket Connection

11.7.1 Welded Bracket Connection Type-I

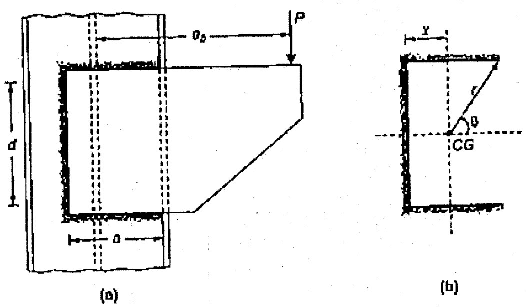


Fig. 11.16 Welded bracket connection type-I

Direct shear stress, $q_1 = \dfrac{P}{(2a+d)t}$ ...(11.18)

Shear stress due to twisting moment is computed as

$$\frac{T}{I_p} = \frac{q_2}{r}$$

$$\Rightarrow \quad q_2 = \frac{T_r}{I_p} = \frac{Pe_0 r}{I_p} \quad ...(11.19)$$

where I_p = Polar MOI of the weld group about C.G.

Resultant stress, $q = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta} \le f_{wd} = \left(\dfrac{f_u}{\sqrt{3}\gamma_{mw}}\right)$ ...(11.20)

11.7.2 Procedure for the Design of Welded Bracket Connection Type-I

Step-1. Assume overlap of bracket plate on the column flange and also the weld size.

Step-2. Compute distance of centroid of weld group ($\bar{x}$ in Fig. 11.16(b))

Step-3. Compute polar moment of inertia of the weld group (I_p).

Step-4. Compute distance 'r' of the extreme weld from C.G. of weld group.

Step-5. Shear stress 'q_1' and 'q_2' are computed from Eqs. (11.18) and (11.19)

Step-6. Resultant stress (q) is computed from Eq. (11.20) and $q \le \dfrac{f_u}{\sqrt{3}\gamma_{mw}}$

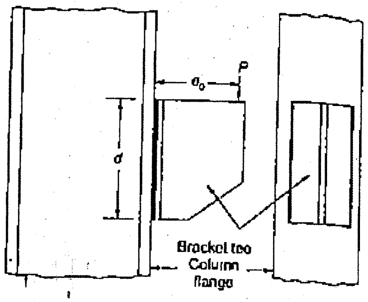


Fig. 11.17 Bracket connection Type-II (welded)

Step-7. Compute weld size as $t_t = KS$ where t_t is the throat thickness and S is the size of weld.

11.7.3 Welded Bracket Connection Type-II

- In this connection, the moment is in a plane normal to the plane of weld i.e. the CG of the weld group lies in a plane normal to the plane of line of action of load (external) and the weld group is acted upon by direct shear and bending as shown in Fig. 11.17.
- Here the shear stresses and maximum bending moment occur at different locations in the joint. So combining these two stresses at a point is not possible instead these stresses are checked individually.
- Thus, when an I-section or channel sections are welded to the plate element, it is customary to assume that flange weld carries the bending moment and web weld carries the shear force. But IS code outlines a procedure to combine these two stresses. Welding can be either butt weld or the fillet weld.

Stress in the Weld Due to Shear

The direct shear stress in the weld is given by,

$$q = \frac{\text{Load } (P)}{\text{Effective weld area}}$$

For fillet weld,

$$q_{cal} = \frac{P}{2l_w t_t} \quad ...(11.21)$$

For butt (or groove) weld,

$$q_{cal} = \frac{P}{dt_e} \quad ...(11.22)$$

where l_w = Effective length of fillet weld
t_t = Effective throat thickness of the fillet weld
d = Depth of butt weld
t_e = Effective throat thickness of butt weld

Stress in the weld due to bending moment

For fillet weld,

$$f_t = \frac{P}{l_w t_t} \quad ...(11.23)$$

For butt weld,

$$f_b = \frac{\text{Moment}}{\text{Section modulus}}$$

$$f_{b,cal} = \frac{Pe_0}{\frac{1}{6}t_e l_w^2} = \frac{6Pe_0}{t_e l_w^2} \quad ...(11.24)$$

Combined stress in the weld

As per Cl. 10.5.10.1.1 of IS 800:2007 for fillet weld, the equivalent stress (f_e) is given by

$$f_e = \sqrt{f_{a,cal}^2 + 3q_{cal}^2} \le \frac{f_u}{\sqrt{3}\gamma_{mw}} \quad ...(11.25)$$

where l_w = Length of the weld connecting the bracket plate with column flange generally equal to depth of the bracket plate

For butt weld, the equivalent stress is given by,

$$f_e = \sqrt{f_{b,cal}^2 + 3q_{cal}^2} \le \frac{f_y}{\gamma_{m0}} \quad ...(11.26)$$

As per Cl. 10.5.10.1.2 of IS 800:2007 check for the combination of stresses need not be done for:

(a) side fillet welds joining cover plates and flange plates, and
(b) fillet welds where sum of normal and shear stresses does not exceed f_{wd}
where f_{wd} is strength of a fillet weld.

11.7.4 Procedure for the Design of Welded Bracket Connection Type-II

- In the design of bracket connection type-II, the size of the weld is assumed beforehand and length of the weld required is determined.
- If the weld length required is more than twice the depth of the bracket then the weld size assumed earlier needs to be revised.

Step-1 Assume a size of weld and compute throat thickness. Determine the design strength of fillet weld per unit length (f_{wd}). The depth of the bracket is determined as given below.

For butt weld,

$$d = \sqrt{\frac{6M}{f_b t}} \quad ...(11.27)$$

where $f_b = \dfrac{f_y}{\gamma_{m0}}$, $\gamma_{m0} = 1.1$

For fillet weld,

$$d = \sqrt{\frac{6M}{2l_{wd} t_t}} \quad ...(11.28)$$

where f_{wd} = Design strength of the fillet weld per unit weld length. A reduced value of about 80% is used in order to take into account the effect of direct shear force

$$= 0.8\left(0.7S\frac{f_u}{\sqrt{3}\gamma_{m0}}\right) \quad ...(11.29)$$

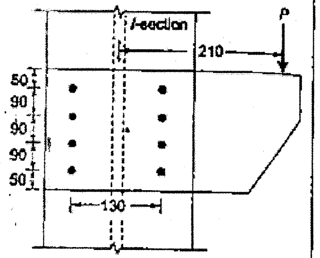
Step-2. The direct shear stress is determined in terms of throat thickness from **Eqs. 11.21 and 11.22.**

Step-3. The bending stress is determined also in terms of throat thickness from **Eqs. 11.23 and 11.24.**

Step-4. Determine the equivalent stress as per **Eqs. 11.25 and 11.26.**

Illustrative Examples

Example 11.1 Find the safe load P that can be carried by the bracket connection as shown. The bolts are of 20 mm diameter of grade 4.6. The thickness of flange of I-section is 13.7 mm and that of bracket plate is 16 mm.



Solution:

$\because$ Steel grade is not given and thus assume Fe410 steel is used with

$f_u = 410$ N/mm², $f_y = 250$ N/mm²

For bolts of grade 4.6, $f_{ub} = 400$ N/mm²

Partial factor to safety for bolt material $\gamma_{mb} = 1.25$

Stress area of 20 mm dia bolt, $A_{nb} = 245$ mm²

From figure, pitch of bolts, $p = 90$ mm

Edge distance, $e = 50$ mm

Shear strength of bolt in single shear

$$V_{dsb} = \frac{A_{nb} \cdot f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \frac{245 \times 400}{\sqrt{3} \times 1.25} N = 45.26 \text{ kN}$$

Strength of bolt in bearing, $V_{dpb} = 2.5 k_b \dfrac{d t f_u}{\gamma_{mb}}$

For 20 mm diameter bolt, diameter of bolt hole, $d_0 = 22$ mm

$$k_b = \text{Minimum of} \begin{cases} \dfrac{e}{3d_0} = \dfrac{50}{3 \times 22} = 0.758 \\[2mm] \dfrac{p}{3d_0} - 0.25 = \dfrac{90}{3 \times 22} - 0.25 = 1.114 \\[2mm] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976 \\[2mm] 1 \end{cases} = 0.758$$

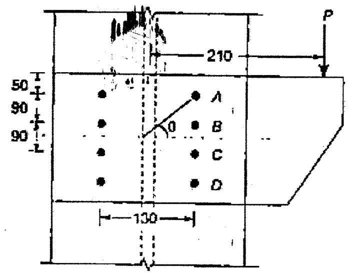
$\therefore$ $k_b = 0.758$

$\therefore$ $V_{dpb} = 2.5 k_b \dfrac{d t f_u}{\gamma_{mb}} = 2.5 \times 0.758 \times 20 \times 13.7 \times \dfrac{410}{1.25} N = 170.31 \text{ kN}$

$\therefore$ Strength of bolt minimum of V_{dsb} and $V_{dpb} = 45.26$ kN

The highly stressed bolt will be the bolt A as shown.

Total number of bolts in the joint, $n = 8$.



Direct force in each bolt, $F_1 = \dfrac{P}{n} = \dfrac{P}{8}$

Force in the bolt due to moment,

$$F_2 = \frac{P e_0 \cdot r_n}{\Sigma r^2}$$

$$r_n = \sqrt{\left(\frac{130}{2}\right)^2 + (45+90)^2} = 149.83 \text{ mm}$$

$$\Sigma r^2 = 4\left[\left\{\left(\frac{130}{2}\right)^2 + 45^2\right\} + \left\{\left(\frac{130}{2}\right)^2 + (90+45)^2\right\}\right]$$

$$= 4[6250 + 22450] = 1,14,800 \text{ mm}^2$$

$\therefore$ $F_2 = \dfrac{P e_0 \cdot r_n}{\Sigma r^2} = \dfrac{P(210)149.83}{114,800} = 0.2741$ P Newtons (N)

$$\cos\theta = \frac{65}{\sqrt{65^2 + (45+90)^2}} = 0.4338$$

$\therefore$ Resultant force on bolt, $F = \sqrt{F_1^2 + F_2^2 + 2F_1 \cdot F_2 \cdot \cos\theta}$

$$= \sqrt{\left(\frac{P}{8}\right)^2 + (0.2741P)^2 + 2\left(\frac{P}{8}\right)(0.2741P)(0.4338)} = 0.3471\,P$$

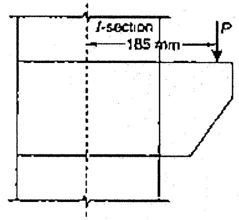
But $F \le$ strength of bolt

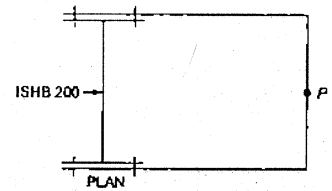
$\Rightarrow$ $0.3471 P \le 45.26 \times 10^3$ N

$\Rightarrow$ $P \le 130.39 \times 10^3$ N $= 130.39$ kN

$\therefore$ Maximum service load $= \dfrac{130.39}{1.5} = 86.93$ kN $\simeq 86$ kN (say)

Example 11.2 It is required to design a bolted bracket connection for an end-support reaction of 350 kN due to factored dead and live load on the beam. The eccentricity of end reaction is 185 mm as shown in figure. Use bolts of grade 4.6. The column section is ISBH200 @ 365.91 N/m.





Solution:

Let steel grade of Fe410 is used

$\therefore$ $f_u = 410$ N/mm², $f_y = 250$ N/mm²

For bolts of grade 4.6, $f_{ub} = 400$ N/mm²

Partial factor of safety for bolt material,

$\gamma_{mb} = 1.25$

For ISBH @ 200 @ 365.91 N/m,

Thickness of flange, $t_f = 9$ mm

The factored end reaction is transferred to the two bracket plates as shown.

$\therefore$ Design load coming on one bracket $= \dfrac{350}{2} = 175$ kN

Let 20 mm diameter bolts are used

$\therefore$ Diameter of bolt hole, $d_0 = 22$ mm

Edge distance, $e = 33$ mm

Stress area, $A_{nb} = 245$ mm²

∴ Minimum pitch, $p = 2.5\ d = 2.5 \times 20 = 50 \text{ mm} \simeq 55 \text{ mm (say)}$

Shear strength of bolt in single shear

$$V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\gamma_{mw}} = \frac{245 \times 400}{\sqrt{3} \times 1.25} \text{ N} = 45.26 \text{kN}$$

Strength of bolt in bearing, $V_{pdb} = 2.5 k_b \dfrac{d t f_u}{\gamma_{mb}}$

$$k_b = \text{Minimum of} \begin{cases} \dfrac{e}{3d_0} = \dfrac{33}{3 \times 22} = 0.5 \\ \dfrac{p}{3d_0} - 0.25 = \dfrac{55}{3 \times 22} - 0.25 = 0.583 \\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976 \end{cases}$$

$= 0.5$

∴ $$V_{dpb} = 2.5 \times 0.5 \times 20 \times \frac{9 \times 410}{1.25} \text{N} = 73.8 \text{ kN}$$

∴ Strength of bolts V_{sd} = Minimum of V_{dsb} and $V_{dpb} = 45.26$ kN

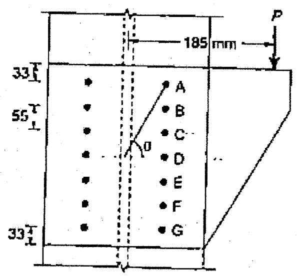
Provide bolts in two vertical rows

∴ Number of bolts required in one row.

$$n = \sqrt{\frac{6M}{pn' V_{sd}}} = \sqrt{\frac{6 \times 175 \times 1000 \times 185}{55 \times 2 \times 45.26 \times 1000}} = 6.25 \simeq 7 \text{ bolts per row}$$

Thus provide 7 bolts in each row.

The highly stressed bolt will be bolt A as shown



Direct force on bolt A, $F_1 = \dfrac{P}{14} = \dfrac{175}{14} \text{kN} = 12.5 \text{ kN}$

Force in bolt A due to torque,

$$F_2 = \frac{Pe_0 \cdot r_n}{\Sigma r^2}$$

Here eccentricity, $e_0 = 185$ mm

Let gauge of ISBH200@365.91 N/m, g = 90 mm

(check from steel table)

∴ $$r_n = \sqrt{(3 \times 55)^2 + 45^2} = 171.026 \text{ mm}$$

$$\Sigma r^2 = 4\left[\left\{(3 \times 55)^2 + 45^2\right\} + \left\{(2 \times 55)^2 + 45^2\right\} + \left\{(1 \times 55)^2 + 45^2\right\}\right] + 2(0^2 + 45^2)$$

$$= 4[29250 + 14125 + 5050] + 4050 = 197750 \text{ mm}^2$$

∴ $$F_2 = \frac{Pe_0 r_n}{\Sigma r^2} = \frac{170 \times 185 \times 171.026}{197750} \text{kN} = 27.2 \text{ kN}$$

$$\cos\theta = \frac{45}{r_n} = \frac{45}{171.026} = 0.2631$$

∴ Resultant force on bolt, $F = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

$$= \sqrt{(12.5)^2 + (27.2)^2 + 2(12.5)(27.2)(0.2631)} = 32.787 \text{ kN}$$

$< V_{sd} (= 45.26 \text{ kN})$ (OK)

Hence provide 20 mm diameter bolts with 07 bolts each in two vertical rows.

Example 11.3 **Design a bolted seat connection for a beam reaction of 105 kN under factored loads. The beam section is ISMB 300 @ 433.6 N/m which is connected to flange of column section ISHB350 @ 661.19 N/m. Use steel of grade Fe410 and bolts of grade 4.6.**

Solution:

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

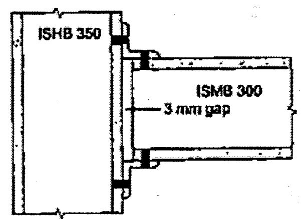
For bolts of grade 4.6, $f_{ub} = 400$ N/mm²

Partial factor of safety for material, $\gamma_{mo} = 1.1$

Partial factor of safety for bolt material, $\gamma_{mb} = 1.25$

The section properties of beam (ISMB 300 @ 433.6 N/mm) and column (ISBH 350 @ 661.19 N/m) are as follows.

	ISMB 300 @ 433.6 N/mm	ISBH 350 @ 661.19 N/m
Depth of section, D	300 mm	350 mm
Width of flange, b_f	140 mm	250 mm
Flange thickness, t_f	12.4 mm	11.6 mm
Web thickness, t_w	7.5 mm	8.3 mm
Radius of root, R_1	14 mm	12 mm
MOI about z-zaxis, I_z	8603.6 × 10⁴ mm⁴	18159.7 × 10⁴ mm⁴
MOI about y-yaxis, I_y	453.9 × 10⁴ mm⁴	2451.4 × 10⁴ mm⁴
Elastic section modulus, z_{ez}	573.6 × 10³ mm³	1094.8 × 10³ mm³
Plastic section modulus, z_{pz}	651.74 × 10³ mm³	1213.53 × 10³ mm³
Web depth, d	247.2 mm	302.8 mm



Length of seat angle = Width of connected beam

= 140 mm (Normal to plane of paper)

Bearing leg length of seat angle,

$$b = \frac{R\gamma_{mo}}{t_w f_{yw}} = \frac{(105 \times 1000)1.1}{7.5 \times 250} \text{mm}$$

$= 61.6 \text{ mm} = 75 \text{ mm (say)}$

A clearance of 3 mm between column and beam is provided

∴ Required leg length of outstanding leg = 61.6 + 3 = 64.6 mm < 75 mm (OK)

Length of bearing on seat, $b_1 = b - (t_f + R_1) = 61.6 - (12.4 + 14) = 35.2$ mm

The factored reaction of 105 kN is assumed to be uniformly distributed over a bearing length of 35.2 mm.

Try a seat angle of 150 × 75 × 12

$\therefore$ Radius of root of seat angle $R_1 = 50$ mm

$\therefore$ End of bearing is at a distance of b_2 on seat from root of angle

where, $b_2 = 35.2 + 3 - (12 + 10) = 16.2$ mm

$$\therefore \text{Moment at critical section}, M = R \times \frac{b_2}{b_1} \times \frac{b_2}{2} = 105 \times \frac{16.2}{35.2} \times \frac{16.2}{2} = 391.42 \text{ kN.mm}$$

Moment capacity of leg of angle,

$$M_d = 1.2 Z_e \frac{f_y}{\gamma_{m0}} = 1.5 \times 140 \times \frac{12^2}{6} \times \frac{250}{1.1} \text{N.mm}$$

$$= 916.36 \text{ kN.mm} > 391.42 \text{ kN.mm} (= M)$$

Design shear capacity of outstanding leg,

$$V_{dp} = \frac{(140 \times 12) \times 250}{\sqrt{3} \times 1.1} \text{N} = 220.44 \text{ kN} > 105 \text{ kN} \qquad \text{(OK)}$$

Thus seat angle of 150 x 75 x 12 is adequate.

Connection of seat angle with column flange

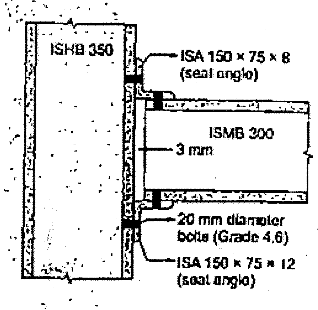
Provide 20 mm diameter bolts of grade 4.6 with

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

Shear strength of bolt in single shear,

$$V_{dsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\gamma_{mb}} = \frac{245 \times 400}{\sqrt{3} \times 1.25} \text{N}$$

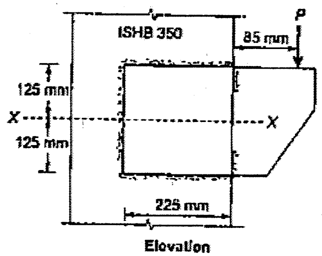
$$= 45.26 \text{ kN}$$

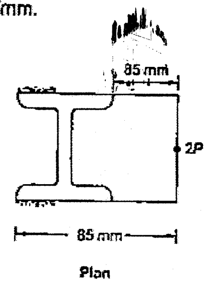


$$\therefore \text{Number of bolts required} = \frac{105}{45.26} = 2.32 \simeq 4 \text{ bolts (say)}$$

Also provide a nominal cleat angle of ISA × 150 × 75 × 8 on top of beam and connect it with column flange for cleat angle, provide 2 nos. 20 mm diameter bolts on each leg.

Example 11.4 A welded bracket connection is as shown in figure. The bracket plate is connected to the column flange through 6 mm fillet weld. A concentrated load P acts at a distance of 85 mm from the column flange. Compute the maximum value of load P that can be placed on this bracket connection. Column section is ISHB 350 @ 661.19 N/mm.



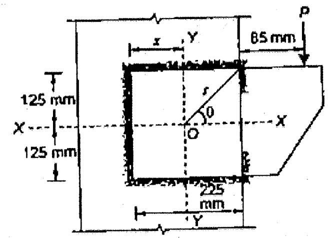


Solution:

Let, size of weld = 6 mm

Effective throat thickness of weld

$$t_t = 0.7 \times 6 = 4.2 \text{ mm}$$



Let distance of centroid of weld group (point O) from left weld edge = $\bar{x}$

$$\therefore \quad \bar{x} = \frac{(2 \times 225 \times 4.2)\left(\frac{225}{2}\right) + (250 \times 4.2)0}{(2 \times 225 \times 4.2) + (250 \times 4.2)}$$

$$= \frac{212625}{2940} = 72.32 \text{ mm}$$

Computing polar moment of inertia of weld group

Polar moment of inertia, $I_p = I_x + I_y$

$$I_x = \frac{4.2 \times 250^3}{12} + 2\left[\frac{225 \times 4.2^3}{12} + (225 \times 4.2)125^2\right] \text{mm}^4$$

$$= (5.46875 \times 10^6 + 29.534 \times 10^6) \text{ mm}^4 = 35 \times 10^6 \text{ mm}^4$$

$$I_y = \left[\frac{250 \times 4.2^3}{12} + 250 \times 4.2 \times 72.32^2\right] + 2\left[4.2 \times \frac{225^3}{12} + 4.2 \times 225 \times (112.5 - 72.32)^2\right]$$

$$= \left(5.4932 \times 10^6 + 11.0247 \times 10^6\right) \text{mm}^4 = 16.52 \times 10^6 \text{ mm}^4$$

$$\therefore \quad I_p = I_x + I_y = (35 \times 10^6 + 16.52 \times 10^6) \text{mm}^4 = 51.52 \times 10^6 \text{ mm}^4$$

Distance of extreme point from CG of weld group,

$$r = \sqrt{(225 - 72.32)^2 + 125^2} = 197.32 \text{ mm}$$

Computation of stresses

Direct shear stress,

$$q_1 = \frac{P}{A} = \frac{P}{(2 \times 225 + 250)4.2} = 0.0003401361P \text{ N/mm}^2$$

Shear stress due to twisting moment,

$$q_2 = \frac{P(225 - \bar{x} + 85)}{51.52 \times 10^6} \times 197.32 = \frac{P(225 - 72.32 + 85) \times 197.32}{51.52 \times 10^6} = 0.00091031P \text{ N/mm}^2$$

$$\cos\theta = \frac{(225 - \bar{x})}{r} = \frac{225 - 72.32}{197.32} = 0.7738$$

$\therefore$ Resultant shear stress,

$$q = \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos\theta}$$

$$= \sqrt{(0.0003401361P)^2 + (0.00091031P)^2 + 2(0.0003401361P)(0.00091031P)(0.7738)}$$

$$= 1.193 \times 10^{-3} P \text{ N/mm}^2$$

But resultant shear stress,

$$q \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.371 \text{ N/mm}^2$$

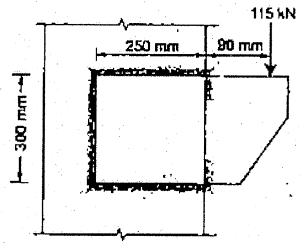
$\therefore \quad 1.193 \times 10^{-3} P \le 189.371$

$\Rightarrow \quad P \le 158.735 \times 10^3 \text{ N} = 158.735 \text{ kN}$

$$\therefore \quad \text{Service load} = \frac{P}{1.5} = \frac{158.735}{1.5} = 105.82 \text{ kN}$$

$\therefore$ Maximum service load that can be supported $= 2 \times 105.82 \text{ kN} = 211.64 \text{ kN}$

Example 11.5 A bracket plate is welded to the column flange as shown. The column section is ISHB 400 @ 759.29 N/m. If a factored load of 115 kN is required to be supported by the bracket connection then determine the size of the weld.



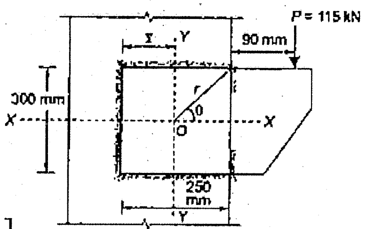
Solution:

Let $\quad t$ = Throat thickness of weld

Let $\quad \bar{x}$ = Distance of centroid of weld group from left edge of weld

$$= \frac{(2 \times 250 \times t)125 + (300 + t) \times 0}{2 \times 250 \times t + 300 + t}$$

$= 78.125$ mm



$\therefore$ Eccentricity of load,

$e = (1250 - \bar{x}) + 90$

$= (250 - 78.125) + 90$

$= 261.875$ mm

$$I_x = \frac{t \times 300^3}{12} + 2\left[\frac{250 \times t^3}{12} + (250 \times t)150^2\right]$$

$$= 13.5 \times 10^6 t \text{ mm}^4 \qquad \left(\text{Neglecting the term } \frac{250 \times t^3}{12}\right)$$

$$I_y = \left[\frac{300 \times t^3}{12} + (300 \times t)78.125^2\right] + 2\left[\frac{t \times 250^3}{12} + (250 \times t)(125 - 78.125)^2\right]$$

$$= [1.831\, t \times 10^6 + 3.703\, t \times 10^6] \qquad \left(\text{Neglecting the term } \frac{300 \times t^3}{12}\right)$$

$= 5.534\, t \times 10^6 \text{ mm}^4$

$\therefore$ Polar moment of inertia

$$I_p = I_x + I_y = 13.5\, t + 10^6 + 5.534 t \times 10^6 = 19.034\, t + 10^6 \text{ mm}^4$$

$$r = \sqrt{(250 - 78.125)^2 + 150^2} = 228.125 \text{ mm}$$

$$\cos\theta = \frac{250 - \bar{x}}{N} = \frac{250 - 78.125}{228.125} = 0.7534$$

Direct shear stress,

$$q_1 = \frac{115 \times 10^3}{(2 \times 250 \times t) + (300 \times t)} = \frac{143.75}{t} \text{ N/mm}^2$$

Shear stress due to twisting moment,

$$q_2 = \left(\frac{115 \times 10^3 \times 261.875}{19.034 t \times 10^6}\right)228.125 \qquad \left(\because \frac{f_s}{r} = \frac{I}{I_p} \Rightarrow f_s = \frac{T_r}{I_p}\right)$$

$$= \frac{360.94}{t} \text{ N/mm}^2$$

$\therefore \quad$ Resultant shear stress, $q = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$

$$= \sqrt{\left(\frac{143.75}{t}\right)^2 + \left(\frac{360.94}{t}\right)^2 + 2\left(\frac{143.75}{t}\right)\left(\frac{360.94}{t}\right)0.7534} = \frac{478.67}{t} \text{ N/mm}^2$$

Assuming shop welding

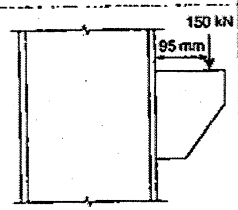
$\therefore \quad \gamma_{mw} = 1.25$

$$\therefore \quad \frac{478.67}{t} \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.371$$

$\therefore \quad t \ge 2.53$ mm

$$\therefore \text{ Size of weld, } s = \frac{t}{0.7} = \frac{2.53}{0.7} = 3.61 \simeq 5 \text{ mm (say)}$$

Example 11.6 The figure shows a bracket plate to transmit a reaction of 150 kN at an eccentricity of 95 mm from the column flange. Design the suitable weld. Assume Fe410 steel grade and thickness of bracket plate as 10 mm. Assume shop welding.



Solution:

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

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

Partial factor of safety for material,

$\gamma_{mo} = 1.1$

Partial factor of safety for shop weld,

$\gamma_{mw} = 1.25$

Now either butt weld or fillet weld can be provided

Let size of fillet weld = 6 mm

$\therefore$ Effective throat thickness of weld,

$t = 0.7 \times 6 = 4.2$ mm

$$\therefore \text{ Depth of bracket plate, } \quad d = \sqrt{\frac{6M}{2t f_{wd_1}}} \qquad \text{where } f_{wd_1} = \frac{0.8 f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{0.8 \times 410}{\sqrt{3} \times 1.25} = 151.5 \text{ N/mm}^2$$

$$= \sqrt{\frac{6\times150\times1000\times95}{2\times4.2\times151.5}} = 259.2 \text{ mm} \simeq 280 \text{ mm} \quad \text{(say)}$$

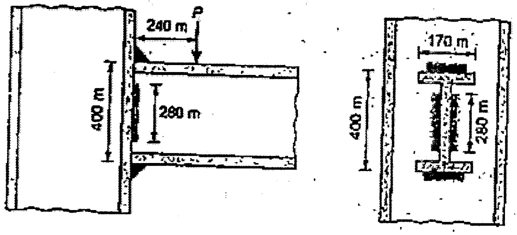
∴ Provide a bracket plate of depth 280 mm

Direct shear stress, $q_1 = \dfrac{150\times1000}{2\times280\times4.2} = 63.78 \text{ N/mm}^2$

Flexural stress, $f_b = \dfrac{150\times1000}{2\times280\times4.2} = 63.78 \text{ N/mm}^2$

∴ Equivalent stress, $q = 63.78 + 63.78 = 127.56 \text{ N/mm}^2$

Example 11.7 An I-section bracket is connected to the column as shown. The size of weld is 6 mm on web and 10 mm on flange. Find the safe load that can be carried by the connection.



Solution:

Size of weld on web (s_w) = 6 mm

∴ Throat thickness of web weld (t_{web}) = 0.7 × 6 = 4.2 mm

Size of weld on flage (s_f) = 10 mm

∴ Throat thickness of flange weld (t_{flange}) = 0.7 × 10 = 7 mm

∴ Total weld area = (280 × 4.2) × 2 + (170 × 7) × 2 = 4732 mm²

Direct shear stress on weld $= \dfrac{P}{4732}$ N/mm²

Flexural stress on weld $= \dfrac{P\times240}{(I/200)}$

where $I = (170\times7)200^2\times2 + \dfrac{4.2\times280^3\times2}{12} = 110.57\times10^6 \text{ mm}^4$

∴ Flexural stress on weld $= \dfrac{P\times240\times200}{110.57\times10^6} = 0.43411P\times10^{-3} = \dfrac{P}{2303.54}$

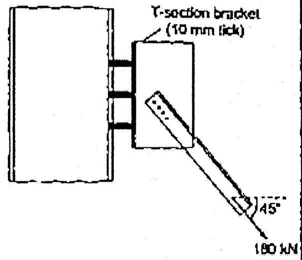
∴ Equivalent shear stress $(f_e) = \sqrt{3\times\left(\dfrac{P}{4732}\right)^2 + \left(\dfrac{P}{2303.54}\right)^2} = \dfrac{P}{1761.08}$

∴ $\dfrac{P}{1761.08} \le 189.37$

⇒ $P \le 333.5$ kN

∴ Permissible load in working conditions $= \dfrac{333.5}{1.5} = 222.33 \text{ kN} \simeq 222 \text{ kN (say)}$

Example 11.8 Check the safety of joint as shown. Eight bolts of 18 mm diameter used to connect the T-section bracket with the column face. Use steel of grade Fe410 and bolts of grade 4.6



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

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

For bolt of grade 4.6, $f_{ub} = 400 \text{ N/mm}^2$

$f_y = 240 \text{ N/mm}^2$

Hole diameter for 18 mm dia. bolt

$d_0 = 20$ mm

Partial factor of safety of material resistance governed by yield,

$\gamma_{m0} = 1.1$

Partial factor of safety for material resistance governed by ultimate stress

$\gamma_{m1} = 1.25$

Partial factor of safety for bolt material

$\gamma_{mb} = 1.25$

Factored tensile load (T) = 180 kN

∴ Horizontal component of T,

$T_H = 180 \cos 45° = 127.28$ kN

Vertical component of T,

$T_v = 180 \sin 45° = 127.28$ kN

∴ Tension coming on each bolt due to $T_H (T_b) = \dfrac{127.28}{8} = 15.91$ kN

Shear coming on each bolt due to $T_v (V_{sb}) = \dfrac{127.28}{8} = 15.91$ kN

Here the bolts are in single shear.

∴ Shear strength of bolt in single shear $V_{dsb} = \dfrac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}}$

Tensile stress area of 18 mm bolt, $A_{nb} = 0.78\times\dfrac{\pi}{4}(18)^2 = 198.5 \text{ mm}^2$

Note: Tensile area of bolt of dia. d, $A_{nb} \simeq 0.78\times\dfrac{\pi}{4}d^2$

∴ $V_{dsb} = \dfrac{A_{nb} f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \dfrac{198.5\times400}{\sqrt{3}\times1.25} \text{ N} = 36.67 \text{ kN}$

Tensile strength of bolt, $T_{db} = 0.9 f_{ub}\dfrac{A_{nb}}{\gamma_{mb}} = 0.9(400)\dfrac{198.5}{1.1} \text{ N} = 64.96 \text{ kN}$

$\le A_{sb} f_{yb}\dfrac{\gamma_{mb}}{\gamma_{m0}} = \left(\dfrac{\pi}{4}\times18^2\right)250\times\dfrac{1.25}{1.1} \text{ N} = 72.29 \text{ kN}$ (OK)

For bolts subjected to tension and shear, the following interaction formula must be satisfied.

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2+\left(\frac{T_b}{T_{db}}\right)^2 \le 1$$

$$\text{LHS} = \left(\frac{V_{sb}}{V_{dsb}}\right)^2+\left(\frac{T_b}{T_{db}}\right)^2 = \left(\frac{15.91}{36.67}\right)^2+\left(\frac{15.91}{64.96}\right)^2$$

$$= 0.1882 + 0.05999$$

$$= 0.24819$$

$$< 1 \qquad \text{(OK)}$$

Thus bolts connecting the T-bracket to the column face are safe.

Example 11.9 In above question, double angle section are used to connect to the T-section bracket with bolts of grade 4.6. The angle sections are 10 mm thick. Find the number of 18 mm dia. bolts required.

Solution:

The bolts connecting the double angle sections with T-section bracket will be in double shear.

∴ Shear strength of bolt in double shear,

$$V_{dsb} = 2\times\frac{A_{nb}f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \frac{2\times198.5\times400}{\sqrt{3}\times1.25}\ \text{N} = 73.35\ \text{kN}$$

Bearing strength of bolt, $V_{dpb} = 2.5k_b dt\dfrac{f_u}{\gamma_{mb}}$

Minimum pitch $(p) = 2.5\,d = 2.5\times18 = 45$ mm = 50 mm (say)

Maximum edge distance for 18 mm bolt $(e) = 30$ mm ≃ 35 mm (say)

$$k_b = \text{Minimum of}\begin{cases}\dfrac{e}{3d_0}=\dfrac{35}{3\times20}=0.583\\[2mm] \dfrac{p}{3d_0}-0.25=\dfrac{50}{3\times20}-0.25=0.583\\[2mm] \dfrac{f_{ub}}{f_u}=\dfrac{400}{410}=0.976\\[2mm] 1\end{cases} = 0.583$$

Thickness (t) = Minimum of thickness of T-bracket and aggregate thickness of two angle section
= minimum of 10 mm and (10 + 10) mm
= 10 mm

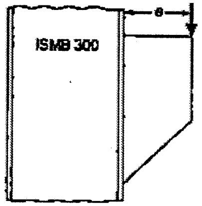
$$\therefore\quad V_{dpb} = 2.5k_b dt\frac{f_u}{\gamma_{mb}} = 2.5(0.583)18(10)\frac{410}{1.25}\ \text{N} = 86.05\ \text{kN}$$

∴ Strength of bolt = Minimum of V_{dsb} and V_{dpb}
= 73.35 kN

$$\therefore\quad \text{No. of bolt required} = \frac{180}{73.35} = 2.45\ \text{bolts} = 4\ \text{bolts (say)}$$

∴ Provide 4 nos. 18 mm dia bolts

Example 11.10 Design a suitable fillet welded bracket connection if a working load of 105 kN is acting at an eccentricity of 140 mm. The thickness of bracket plate is 12 mm. The column section is ISHB 300 @ 618 N/m.



Solution:

Service load = 105 kN

∴ Factored load (P) = 1.5 × 105 kN = 157.5 kN

Let size of weld (S) = 8 mm

∴ Throat thickness of weld $(t) = 0.7\,S = 0.7\times8 = 5.6$ mm

$$\text{Strength of weld } (f_{wd}) = \frac{f_u}{\sqrt{3}\,\gamma_{mb}} = \frac{410}{\sqrt{3}\times1.25}\ \text{N/mm}^2 = 189.37\ \text{N/mm}^2$$

$$\text{Depth of weld required } (h) = \sqrt{\frac{6M}{2tf_{wd}}} = \sqrt{\frac{6\times157.5\times1000\times140}{2\times5.6\times189.37}} = 249.76\ \text{mm} \simeq 270\ \text{mm (say)}$$

$$\text{Direct shear stress } (q) = \frac{P}{2th} = \frac{157.5\times1000}{2\times5.6\times270} = 52.08\ \text{N/mm}^2$$

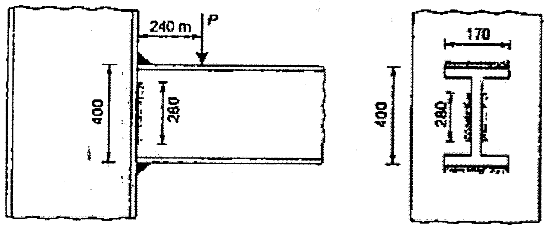
$$\text{Bending stress } (f) = \frac{M}{Z} = \frac{6M}{2th^2} = \frac{6\times157.5\times1000\times140}{2\times5.6\times270^2} = 162.04\ \text{N/mm}^2$$

$$\therefore\quad \text{Equivalent stress } (f_e) = \sqrt{f^2+3q^2} = \sqrt{162.04^2+3\times52.08^2}$$

$$= 185.46\ \text{N/mm}^2$$

$$< 189.37\ \text{N/mm}^2 \qquad \text{(OK)}$$

Example 11.11 An I-section bracket is connected to the column as shown. The size of weld is 6 mm on web and 10 mm on flange. Find the safe load that can be carried by the connection.



Solution:

Size of weld on web (s_w) = 6 mm

∴ Throat thickness of web weld (t_{web}) = 0.7 × 6 = 4.2 mm

Size of weld on flange (s_f) = 10 mm

∴ Throat thickness of flange weld (t_{flange}) = 0.7 × 10 = 7 mm

∴ Total weld area = (280 × 4.2) × 2 + (170 × 7) × 2 = 4732 mm²

$$\text{Direct shear stress on weld} = \frac{P}{4732}\ \text{N/mm}^2$$

$$\text{Flexural stress on weld} = \frac{P \times 250}{(I/200)}$$

where

$$I = (170\times 7)200^2\times 2 + \frac{4.2\times 280^3}{12} = 102.8832\times 10^6\ \text{mm}^4$$

$$\text{Flexural stress on weld} = \frac{P\times 250\times 200}{102.8832\times 10^6} = 0.48599P\times 10^{-2} = \frac{P}{2057.664}$$

$$\text{Equivalent shear stress } (f_e) = \sqrt{\left(\frac{P}{4732}\right)^2 + \left(\frac{P}{2057.664}\right)^2} = \frac{P}{1886.98}$$

$$\text{Design stress of weld} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}\times 1.25} = 189.37\ \text{N/mm}^2$$

$$\therefore \quad \frac{P}{186.98} \le 189.37$$

$$\Rightarrow \quad P \ge 357.34\ \text{kN}$$

∴ Permissible load in working conditions

$$= \frac{357.34}{1.5} = 238.23\ \text{kN}$$

Objective Brain Teasers

Q.1 The moment of resistance of clip angle is:

(a) $0.2lt^2 \frac{f_y}{\gamma_{m0}}$ (b) $0.6lt^2 \frac{f_y}{\gamma_{m0}}$

(c) $0.8lt^2 \frac{f_y}{\gamma_{m0}}$ (d) $1.0lt^2 \frac{f_y}{\gamma_{m0}}$

Q.2 The beam-column flexible connection resist and transfer

(a) Only moment
(b) Both shear and moment
(c) Shear only
(d) 10% shear and 50% moment

Q.3 In framed connections, the bolts connecting the web of beam to connecting angles are subjected to:

(a) Double shear with no bearing at all
(b) Single shear and bearing
(c) Single shear with no bearing at all
(d) Double shear and bearing

Q.4 Given below are some of the statements regarding end plate connections.

(i) End plate is welded to the beam end
(ii) End plates extend above and below the beam section.
(iii) End plate is bolted to the column flange.

This connection is classified as:

(a) Rigid and unstiffened
(b) Rigid and stiffened
(c) Flexible and stiffened
(d) Flexible and unstiffened

Q.5 The major difficulty in analyzing the bracket connection type-I by ultimate analysis method is:

(i) Determination on non-linear bolt deformation relationship for each and every bolt.
(ii) Locating the instantaneous center of rotation.
(iii) a cumbersome trial and error method is required.

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

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

Q.6 The bearing length of seat angle in beam-column connection is given by,

(a) $\frac{R_1\gamma_{m0}}{t_w f_{yw}}$ (b) $1.5\frac{R_1\gamma_{m0}}{t_w f_{yw}}$

(c) $1.5\frac{R\gamma_{m0}}{t_w f_{yw}}$ (d) $1.1\frac{R\gamma_{m0}}{t_w f_{yw}}$

Q.7 In practice, all of the connections fall in the category of

(a) Rigid connection
(b) Semi-rigid connection
(c) Flexible connection
(d) Data insufficient

Q.8 Which of the following statement(s) is(are) true w.r.t. design of moment and eccentric connections?

1. Moment resistant connections are assumed to develop 90% or more of the fixed end moment.
2. Flexible connections are assumed to develop not more 25% of the moment.

(a) 1 only
(b) 2 only
(c) Both 1 and 2
(d) Neither 1 and nor 2

Answers

1. (a) 2. (c) 3. (d) 4. (b) 5. (d)
6. (a) 7. (b) 8. (c)

Conventional Practice Questions

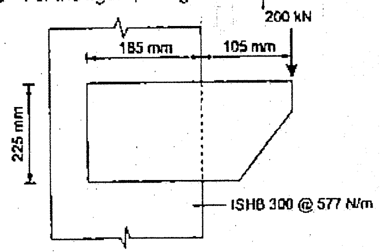
Q.1 ISMB 300 @ 402 N/m transmits an end reaction of 402 kN to the flange of ISHB 300 @ 618 kN/m. Design a suitable:

(i) Unstiffened seat connection.
(ii) Stiffened seat connection.

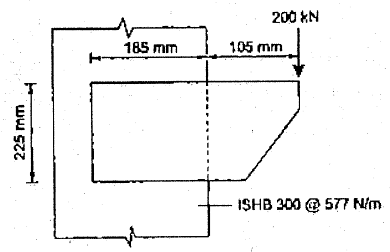
Q.2 ISMB 200 @ 249 N/m transmits a working reaction of 60 kN and a working moment of 90 kNm to the flange of ISHB 225 @ 423 N/m. Design the

(i) shop welded connection
(ii) site welded connection
(iii) bolted connection

Q.3 For the figure, design the suitable weld.



Q.4 In the figure, design the connection with 4.6 grade bolts.



Q.5 A load of 160 kN is to be transmitted through a bracket plate 14 m thick welded to flange of column section ISHB 400 @ 806 N/m. The eccentricity of load from the column face is 115 mm. Design the connection using

(i) weld to be done at site
(ii) bolts of grade 4.6.

■■■■