UNIT II CONNECTIONS IN STEEL STRUCTURES

Axially loaded bolted connections for Plates and Angle Members using bearing type bolts

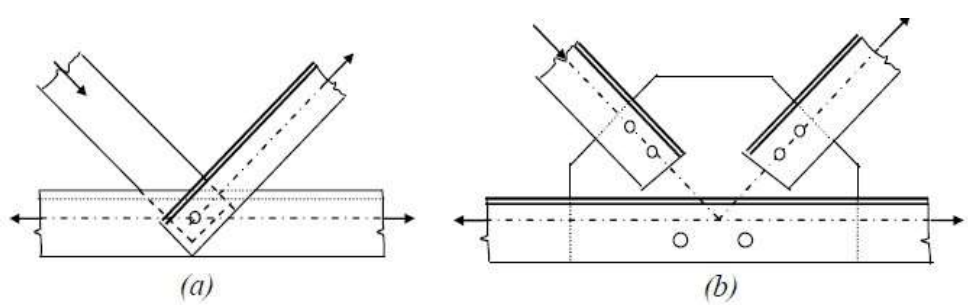
1.0 INTRODUCTION

Connections form an important part of any structure and are designed more conservatively than members. This is because, connections are more complex than members to analyse, and the discrepancy between analysis and actual behaviour is large. Further, in case of overloading, we prefer the failure confined to an individual member rather than in connections, which could affect many members.

Connections can also be classified in the following ways:

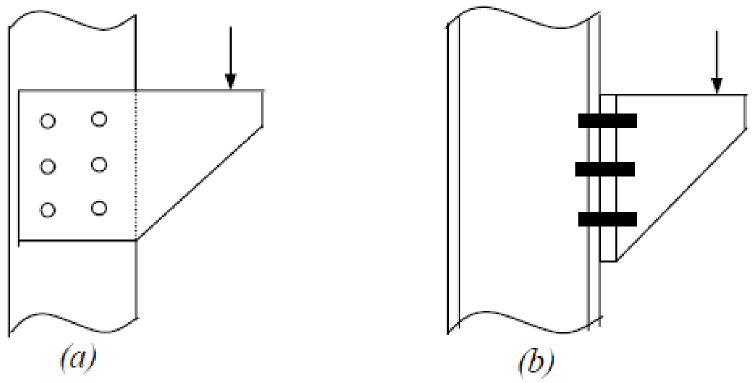
(a) Classification based on the type of resultant force transferred:

The bolted connections are referred to as concentric connections (force transfer in tension and compression member), eccentric connections (in reaction transferring brackets) or moment resisting connections (in beam to column connections in frames). Ideal concentric connections should have only one bolt passing through all the members meeting at a joint [Fig.*a*]. However, in practice, this is not usually possible and so it is only ensured that the centroidal axes of the members meet at one point [Fig. *b*].



Concentric Connections

The Moment connections are more complex to analyse compared to the above two types and are shown in Fig. *a* and Fig. *b*. The connection in Fig. *a* is also known as bracket connection and the resistance is only through shear in the bolts. The connection shown in Fig. *b* is often found in moment resisting frames where the beam moment is transferred to the column. The connection is also used at the base of the column where a base plate is connected to the foundation by means of anchor bolts. In this connection, the bolts are subjected to a combination of shear and axial tension.

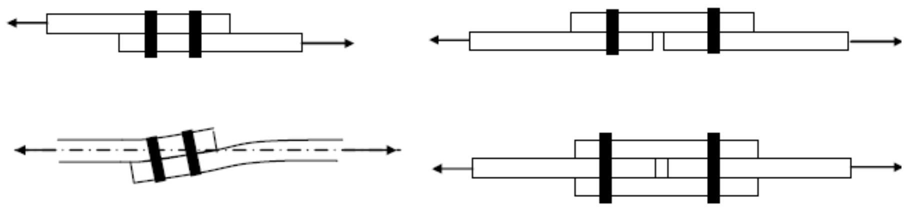


Moment Connections

(b) Classification based on the type of force experienced by the bolts:

The bolted connections can also be classified based on geometry and loading conditions into three types namely, shear connections, tension connections and combined shear and tension connections.

Typical shear connections occur as a *lap* or a *butt* joint used in the tension members. While the lap joint has a tendency to bend so that the forces tend to become collinear, the butt joint requires *cover plates*. Since the load acts in the plane of the plates, the load transmission at the joint will ultimately be through shearing forces in the bolts. In the case of lap joint or a single cover plate butt joint, there is only one shearing plane, and so the bolts are said to be in *single shear*. In the case of double cover butt joint, there are two shearing planes and so the bolts will be in *double shear*. It should be noted that the single cover type butt joint is nothing but lap joints in series and also bends so that the centre of the cover plate becomes collinear with the forces.



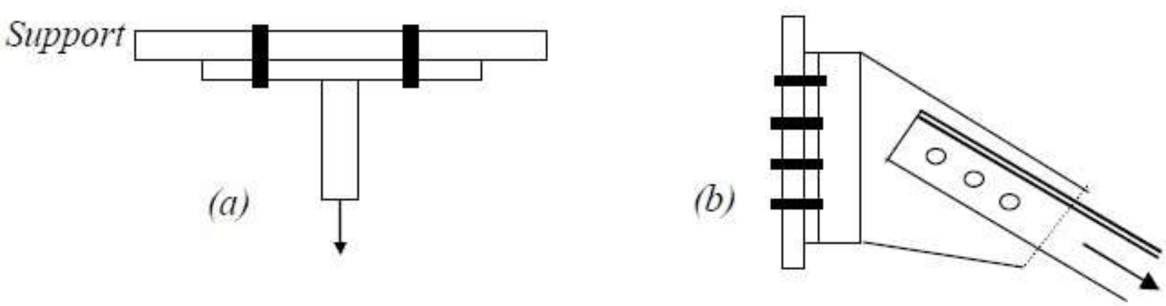
(a) Lap Connection (b) Butt Connection

Shear Connections

A hanger connection is shown in Fig. *a*. In this connection, load transmission is by pure tension in the bolts. In the connection shown in Fig. *b* the bolts are subjected to both tension and shear.

(c) Classification based on force transfer mechanism by bolts: The bolted connections are

classified as bearing type (bolts bear against the holes to transfer the force) or friction type (force transfer between the plates due to the clamping force generated by the pre- tensioning of the bolts).



(a) Tension Connection (b) Tension plus Shear Connection

The design strength in tension of a plate, *Tdn,*as governed by rupture of net cross-sectional area, *An,* at the holes is given by where γ_{ml} = partial safety factor at ultimate stress = 1.25

f_u = ultimate stress of the material of plate

A_n = net effective area at critical section

$$= \left[b - nd_o + \Sigma \frac{p_{si}^2}{4g_i} \right] t$$

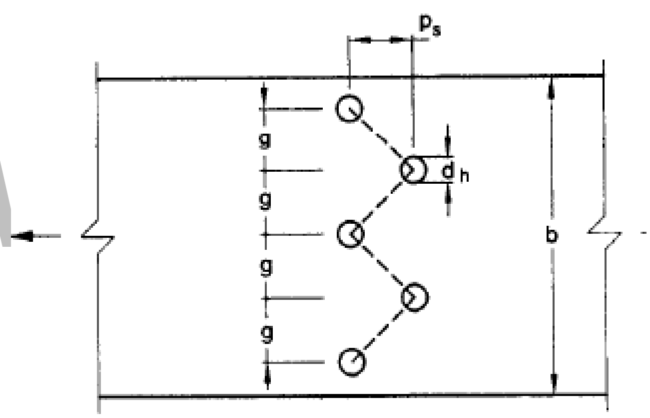
where b = width of plate

t = thickness of thinner plate

d_o = diameter of the bolt hole

n = number of bolt holes

Note: If there is no staggering of the bolts, $p_{si} = 0$ and hence, $A_n = [b - nd_o]\, t$



where

b, t = width and thickness of the plate, respectively,

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),

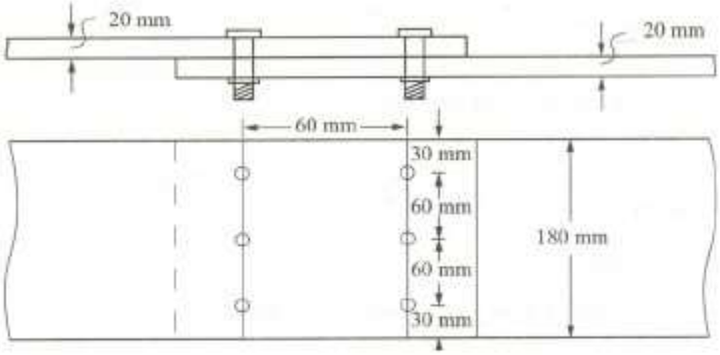
g = gauge length between the bolt holes, as shown in Fig. 5,

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

Find the efficiency of the lap joint shown in fig. given M20 bolt of grade 4.6 and plate of grade Fe410 [E250] are used.



Given Data:-

t = 20mm

Bolt:- M20

Grade 4.6 => fu = 400N/mm^2

fy = 250N/mm^2

Plate:-

Fe 410 [E250]

Fu = 410 N/mm^2

Fy = 250 N/mm^2 [Table 1 – I.S 800 – 2007]

Efficiency of the joint = $\dfrac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$

Strength of connection is least of strength of plate at critical section and strength of bolt in shear & bearing.

Strength of plate @ the joint:-

Tensile force $T_{an} = \dfrac{0.9 A n f u}{\gamma_{ml}}$

$A_n = (b - n d_o) t$

ps = 0 [∵ Bolts are on a straight line]

= (180 – 3x22) 20 [∵ d_o = 20+2=22]

An = 2280mm^2

γ_{ml} = 1.25 [from table 5- I.S 800-2007]

[d_o = Dia of bolt hole = 20+2=22mm]

$T_{dn} = \dfrac{0.9 \times 2280 \times 410}{1.25}$

T_{dn} = 673.056 KN

(i) Strength of bolt in shear $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$$V_{nsb} = \frac{fu}{\sqrt{3}}[N_n A_{nb} + N_s A_{sb}]$$

N_n = No.of shear planes @ the thread = 1

N_s = No.of shear planes @ shank $N_s = 0$ for lap jt $N_s = 1$

$$A_{nb} = 0.78 \times \frac{\pi d^2}{4}$$

$$= 0.78 \times \frac{\pi \times 2^2}{4}$$

$A_{nb} = 245mm^2$

$$V_{nsb} = \frac{400}{\sqrt{3}}[1 \times 245 \times 6]$$

$$V_{nsb} = 339.481 KN \qquad \because V_{dsb} = \frac{339.48}{1.25}$$

$$V_{dsb} = 271.58 \text{ KN}$$

(ii) Strength of bolt in bearing: [cls 10.3.4 IS]

Take

$$\beta_{ij} = \beta_{ig} = \beta_{pk} = 1$$

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

V_{dbp} = 2.5 kb dt fu

Kb = least of $e/3d_o$, $p/3d_o$-0.25, $\frac{fub}{Fu}$, 1.0

$$k_b \Rightarrow \frac{30}{3 \times 22}, \frac{60}{3 \times 22} - 0.25$$

$$k_b \Rightarrow 0.45, 0.659, 0.976, 1$$

Take K_b value of whichever less [$\therefore K_b = 0.45$]

$$V_{nbp} = 2.5 \times 0.45 \times 20 \times 20 \times 410$$

$$V_{nbp} = 186.3 \text{ KN}$$

$$V_{dbp} = \frac{186.3}{1.25}$$

$$V_{dbp} = 149.04 \text{ KN}$$

$\therefore$ Design strength of bolt = 6 x 149.04

V_{dbp} bolt = 894.24 KN

Design strength of the joint = 271.58 KN

Design strength of jt is the least of strength of joint 673.06 KN, 271.58 KN & 894.24 KN

Strength of Solid Plate:-

Strength of Solid Plate = $\frac{fy \times Ag}{\gamma_{ml}}$

[yielding sides the strength of solid plate]

$$= \frac{250}{1.1} \times 180 \times 20$$

Strength of solid plate = 818.18 KN

$\therefore$ Efficiency of joint $\eta = \frac{271.58}{818.18} \times 100$

$$\eta = 33.19$$

2. Find the efficiency of the joint for the above problem if instead of lap joint, a double cover butt joint is provided. Two cover plates each of size 12mm and 6 nos. of bolts are provided on each side.

Given Data:-

[Table 1, I.S 800-2007] [Pg.No.13]

Plate:-

Fe410 [250]

Fu = 410 N/mm^2

Fy = 250 N/mm^2

Bolt:-

M20, Grade 4.6

φ of bolt = 20 mm

fu_b = 400 N/mm^2

fy_b = 240 N/mm^2

The strength of plate at the joints and the strength of bolts in bearing are same as that of the previous problem.

The strength of plate at the joints and the strength of bolts in bearing are same as that of the previous problem.

(1) Strength of plate @ the joint:-

$$T_{dn}=\frac{0.9\,Anfu}{\gamma_{ml}}$$

$$A_n=[b-nd_o]t$$

$$= [180 - 3 \times 22]\ 20$$

$$A_n = 2280mm^2$$

$\gamma_{ml}=1.25$ [from tables-5 IS 800-2007 Pg.No:30]

$d_o = 20+2 = 22$

$$= \frac{2280\times0.9\times410}{1.25}$$

$$T_{dn}=673.056\,KN$$

(2) Strength of bolts:-

(i) Strength of bolt in bearing : (cls 10.3.4 IS 800-2007]

$$V_{dbp}=\frac{V_{nbp}}{\gamma_{mb}}$$

$$V_{nbp}=2.5\,kb.dt.fu$$

$$K_b=\frac{o}{3d_o},\frac{p}{3d_o}-0.25,\frac{fub}{fu},1.0$$

$$=\frac{30}{3\times22},\frac{60}{3\times22}-0.25,\frac{400}{410},1.0$$

$$K_b=0.45,0.659,0.976,1$$

Take kb value of whichever is less

$$\because K_b=0.45$$

$$V_{nbp}=2.5\times0.45\times20\times20\times410$$

$$V_{nbp}=186.3\,KN$$

$$V_{dbp}=\frac{186.3}{1.25}$$

$$V_{dbp}=149.04\,KN$$

$\therefore$ Strength of bolt in bearing = 6 x 149.04

$$V_{dbp}=894.24\,KN$$

(ii) Strength of bolt in shear:- [cls:10.3.3 IS 800-2007]

$$V_{dsp}=\frac{V_{nsb}}{\gamma_{mb}}$$

$$V_{nsb}=\frac{fu}{\sqrt{3}}[N_n A_{nb}+N_{sA_{sb}}]$$

$\because$ Double cover butt jt provided each bolts resists shear along two planes, the section at the root & another section at the shank.

$\because n_n = n_s = 1$ for each bolts

$$A_{nb} = 0.78 \times \frac{\pi d^2}{4}$$

$$= \frac{0.78 \times \pi \times 20^2}{4}$$

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

$$A_{sb} = \frac{\pi d^2}{4}$$

$$= \frac{\pi \times 20^2}{4}$$

$$A_{sb} = 314.16 mm^2$$

$$\therefore V_{nsb} = \frac{400}{\sqrt{3}}[6 \times 245 + 6 \times 314.16]$$

$$V_{nsb} = 774.8 KN$$

$$\boxed{V_{dsp} = 619.84 KN}$$

Reduction factors $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$

$\therefore$ Design Strength of the joint = 619.84 KN [least of 673 KN, 894.4 KN, 619.84 KN]

<u>Strength of the solid plate:-</u>

Strength of the solid plate = $\frac{f_y A_g}{\gamma_{ml}}$

$= \frac{250}{1.1} \times 180 \times 20$ [Tks of thinner plate is the least of sum of cover plate 20(or) 24mm]

Strength of the solid plate = 818.18KN

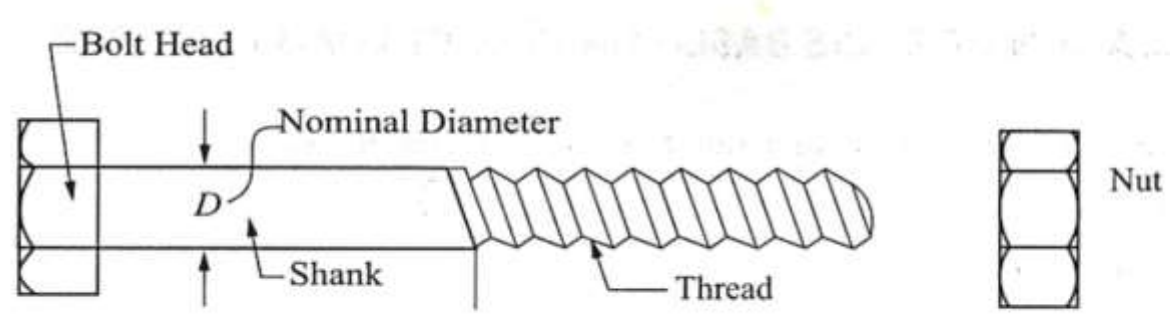
$$\eta = \frac{619.84}{818.18} \times 100$$

$$\eta = 75.76\%$$

UNIT II CONNECTIONS IN STEEL STRUCTURES

Type of Fasteners- Bolts Pins and welds- Types of simple bolted and welded connections Relative advantages and Limitations-Modes of failure-the concept of Shear lag-efficiency of joints- Axially loaded bolted connections for Plates and Angle Members using bearing type bolts –Prying forces and Hanger connection– Design of Slip critical connections with High strength Friction Grip bolts.- Design of joints for combined shear and Tension- Eccentrically Loaded Bolted Bracket Connections- Welds-symbols and specifications- Effective area of welds-Fillet and but Welded connections-Axially Loaded connections for Plate and angle truss members and Eccentrically Loaded bracket connections.

Bolted Connections



Bolt and nut.

- Bolt is a metal pin with a head at one end and a shank threaded at other end to receive a nut,. Steel washers are usually provided under the bolt head and nuts to prevent the treaded portion of the bolt from bearing on the connecting pieces and to distribute the clamping pressure on the bolted member.

- A bolt connection can be used for end connections in tension and compression members. They can also hold down column bases in position and as separator for purlins and beams in foundations.

- Bolts are having the following advantages over rivets and pins:
  - The erection of the structures can be speeded up.
  - Less skilled labour can be employed.
  - Overall cost of bolted connection is lesser than the other alternatives.

  However the following shortcomings are also associated with the bolted connections:

  - Cost of material is high, about double than that of rivets.
  - The tensile strength of bolt is reduced due to the reduced area at the root of the thread and stress concentration.
  - Normally strength reduction will be there for loose fit bolts.
  - Bolts may get loose when subjected to vibrations

Types of bolts:

Block Bolts:- [Unfinished Bolts]

- These bolts are made from mild steel with square or hexagonal heads.
- The nominal dia(d) available are 12,16,20,22,24,27,30 & 36 mm designated as M16 M20 etc.,
- As the shank is unfinished, there is no contact with the members at the entire shown of contact surface.
- Joints remain quite loose result into large deflections & loosening of nuts in course of time.
- Generally the diameter of bolt hole is 1.5mm to 2m larger than the nominal diameter of shank.

Fitted Bolts:- [Turned Bolts]

- These bolts are made from M.S.steel formed from hexagonal rods which are finished by turning to a circular shape within the bolts hole.

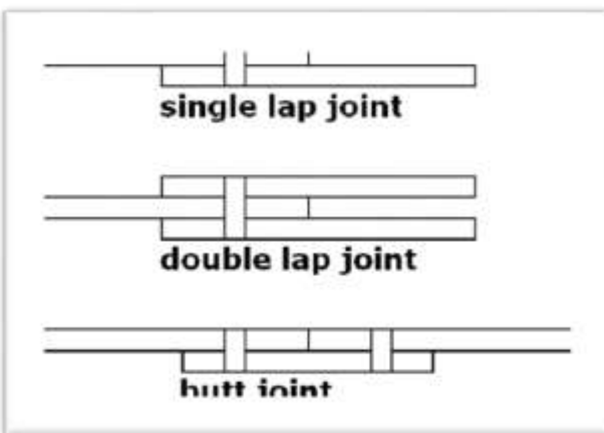
- The actual dimension of the bolt holes are kept 1.2 to 1.3mm larger than the nominal dia. Where the blot hole is kept 1.5mm larger than 'd'
- Aligning the bolt holes needs special care.

Friction Type Bolts: - [HSFG – High Strength Friction Grib bolts]

- These are made from high strength steel rod, where the surface of a shank is kept unfinished and are tightened to a proof load using calculated wrenches.
- Nuts are prevented by using clamping devices.
- The shearing load is first resisted by the frictional force b/w the member and the head.
- It can be used for dynamic moving loads and HSFG Bolts replaces rivets.
- The nominal dia available are 16,20,24,30 & 36mm.

Classification of Bolt Joints:

- ❖ Butt joint
- ❖ Cover joint
- ❖ Tee-Joint
- ❖ Lap- joint



Failure of bolted connections

Failure of bolted connection can be classified broadly in to two:

- Failure of the bolt

- Failure of connecting parts.

Bolted joints may fail in any of the following six ways
- Shear failure of bolts.
- Bearing failure of bolts.
- Bearing failure of plates
- Tension failure of bolts.
- Tension or tearing failure of plates.
- Block shear failure.

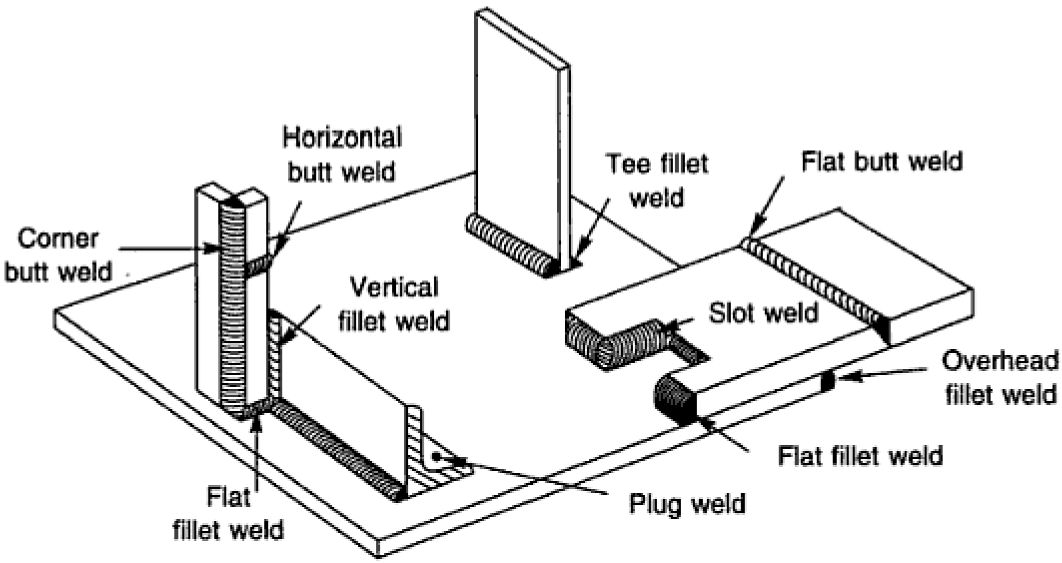
Welded connection

When two members are connected by means of welds, such a connection is known as welded connection. Welding offers an opportunity to the designer to achieve a more efficient use of the materials.

Types of welded connections

The basic types of welded joints can be classified depending on the types of welds, position of welds and type of joint.

Based on type of weld, welds can be classified in to
- fillet weld,
- groove weld (or butt weld),
- plug weld,
- slot weld,
- spot weld



Types and Positions of Welds

Advantages and disadvantages of welded joints

The following are the advantages of welded joints.

- Due to the absence of gusset plates and other connecters, the welds are usually lighter.
- Welding process is quicker as it requires no drilling of holes.
- Welding is more adaptable than other types of connections and can even be used in circular pipes.
- 100% efficiency can be achieved in welding whereas the connection such as bolts can have a maximum efficiency of 70 - 80%.
- Noise produced during the welding process is relatively less.
- Welds usually have good aesthetic appearance.
- Welded joints are air tight and water tight and can be used for water tanks and gas tanks.
- Welded joints are rigid.
- Mismatch of holes will never happen in welded connection.
- Alternation of joints can easily be made in the case of welded connections.

However the welded connection is having the following disadvantages.

• Due to the uneven heating and cooling, members are likely to distort in the process of Welding.
• Possibility of brittle fracture is more in the case of welded connections.
• Welded connections are more prone to failure due to fatigue stresses.
• The inspection of welded joints is difficult and expensive. It can only be done by Employing NDT.
• Highly skilled persons are required for welding.
• Proper welding in field conditions is difficult.
• Welded joints are over rigid.

TERMINOLOGY:

Pitch (p) is the distance between the centers of two consecutive bolts measured along a row of bolts. When the bolts are placed staggered, then the pitch is known as staggered pitch.

Gauge (g) is the distance between the centers of two consecutive bolts measured adjacent row of bolts.

Edge distance (e) is the distance of a bolt hole from the adjacent edge of the plate

End Distance(e') is the distance of the nearest bolt hole from the end of the plate.

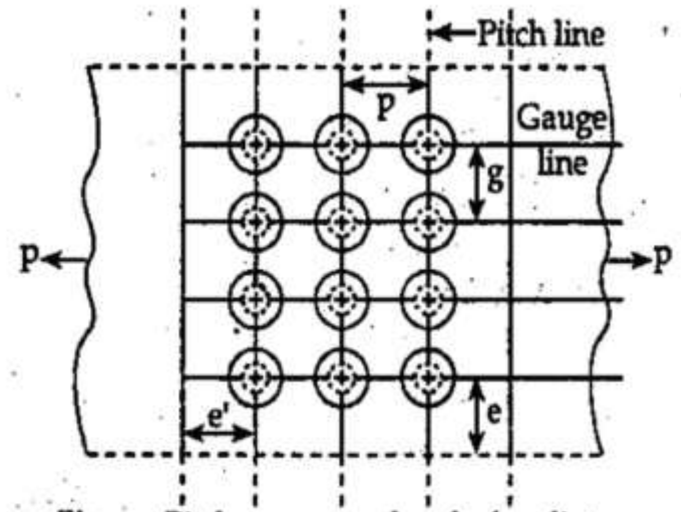
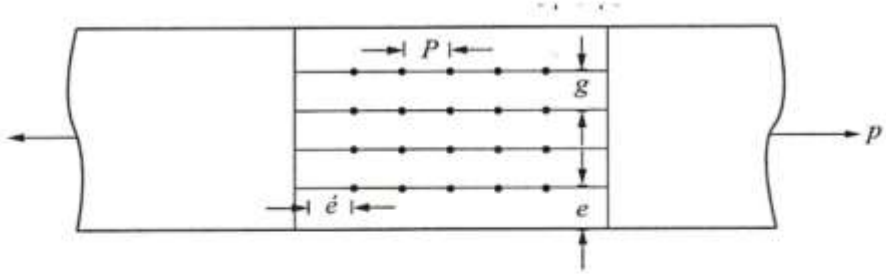
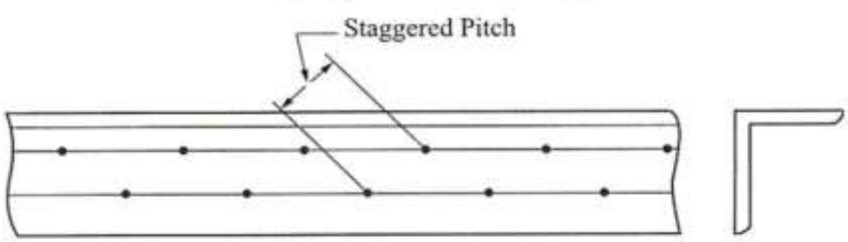


Figure: Pitch, gauge, end and edge distance



Pitch, gauge distance and edge distance.



Bolt distance in staggered bolts.

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Introduction [Section-10 IS 800-2007]

The steel structures are constructed by properly connecting the available standard sections. The connections are an important part of steel structure and are designed more conventionally than any individual members. There is a discrepancy between the actual behavior and the analysis of steel structure is large, therefore the connections are complex to analyze and design. When the structural member fails in case of overloading then there is a general practice to prefer the individual member rather than the connections, therefore this kind of practice affects many structural members. The cost of structural steel consists of major portion of connections and that is the reason primary importance should be given to the design of connections for safety and economy of structure.

The connections are generally provided in the following cases:

- When there is the requirement to cater the heavy load and long span then the built-up sections are to be provided. In this case, this section should be connected together to get a good section.
- In case of longer span, the length of standard section needs to be connected with other section. In this case to connect the multiple sections proper design of connections are important.
- The different members need to be connected at the end (for example secondary beams to be connected to primary beam, column, footings, etc).

The classification in the connections provided in the steel structure is as follows:

- Riveted connections
- Bolted connections
- Welded connections
- Pinned connections

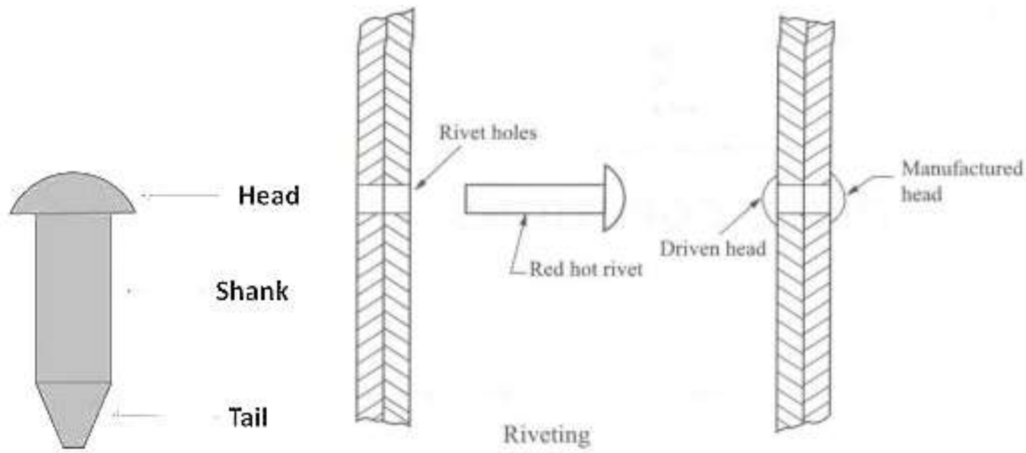
BOLTS PINS AND WELDS

Riveted (Pin) Connections

A piece of round steel forged in place to connect two or more than two steel members together is known as a rivet.

The rivets for structural purposes are manufactured from mild steel and high tensile rivet bars. A rivet consists of a head and a body as shown in Fig. The body of rivet is termed as shank. The rivets are manufactured in different lengths to suit different purposes. The size of rivets is expressed by the diameter of the shank.

- Riveted connections are used because rigid connection are establish since there was lot of disadvantages in riveted connection.
- Requirements of skilled labour Cost increased due to defective rivets, the connections are later preferred



RIVETED JOINT

The riveted joints are of two types:

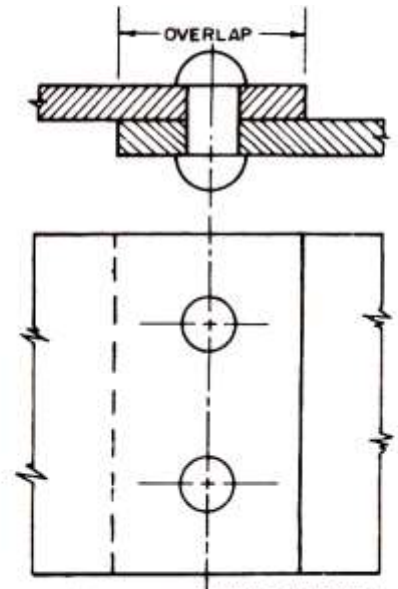
- ❖ **Lap joint**
- ❖ **Butt joint**

Lap joint:

When one member is placed above the other and the two are connected by means of rivets the joint is known as lap joint as shown in figure

These joints are further classified according to the number of rivets used and the arrangement of rivets adopted. Following are the different types of lap joints:

- ❖ Single riveted lap joint
- ❖ Chain riveted lap joint
- ❖ Double riveted lap joint
- ❖ Zig-zag riveted lap joint

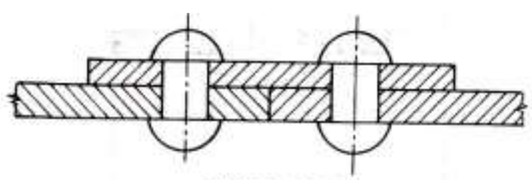


(A) LAP JOINT(SINGLE RIVETED)

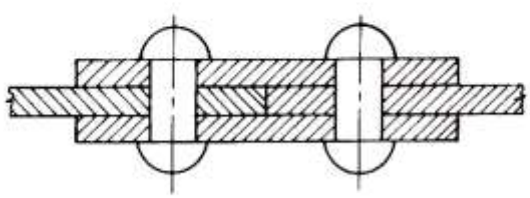
Butt joint:

When plates are placed end to end and flushed with each Other and are joined by means of cover plates, the joint is known as butt joint. The butt joints are of two types

- **Single cover buttjoint**
- **Double cover buttjoint**

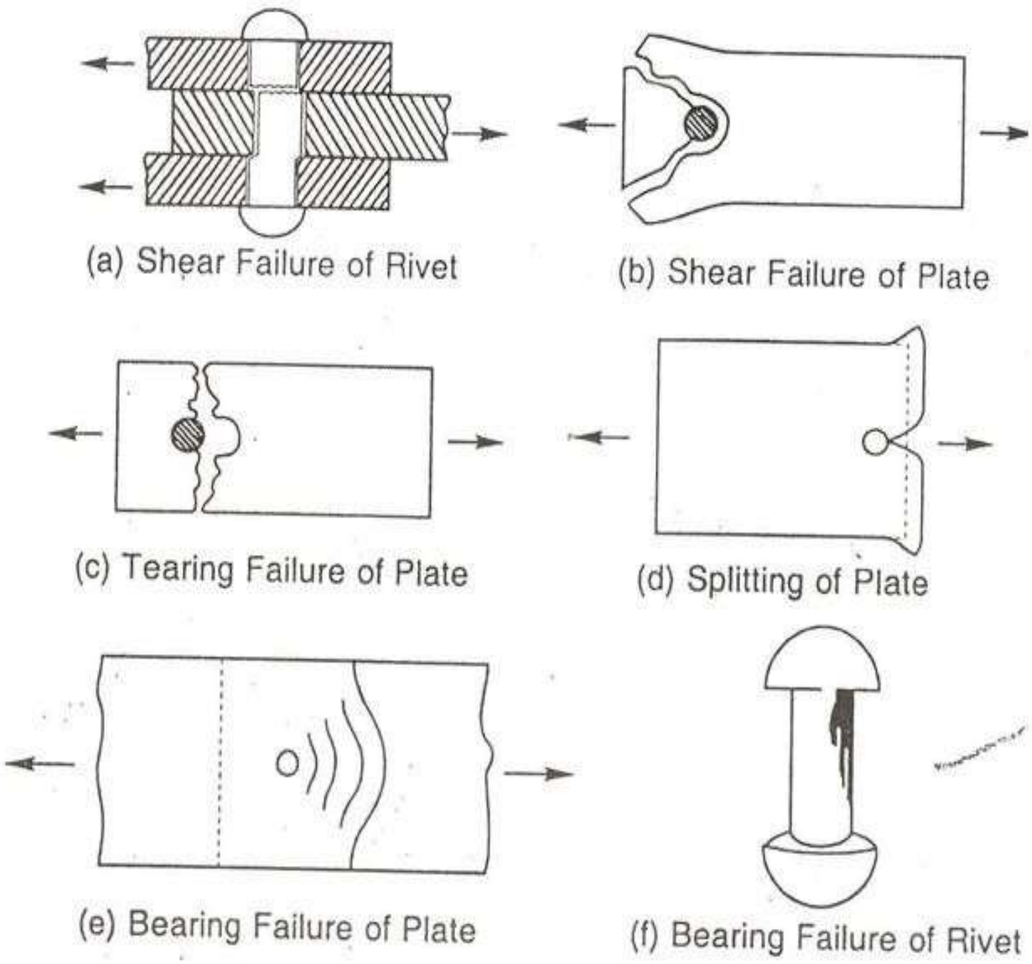


SINGLE COVER PLATE BUTT-JOINT



DOUBLE COVER SINGLE RIVETED BUTT JOINT

- Shear failure of rivets.
- Shear failure of plates.
- Tearing failure of plates.
- Bearing failure of plates.
- Splitting failure of plates at the edges.
- Bearing failure of rivets.



Failure of Riveted Joints

Failure of a Joints

The failure of a riveted joint may take place in any of the following ways:

DESIGN STRENGTH OF BEARING BOLTS

(a) In Shear: It is least of the following:

(i) Shear capacity (strength)

(ii) Bearing capacity (strength)

(i) Shear capacity of bearing bolts (V_{dsb})

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

where γ_{mb} = partial safety factor of bolt

and V_{nsb} = nominal shear capacity of bolt

$$= \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

where f_{ub} = ultimate tensile strength of bolt

n_n = number of shear planes through threads

= 1 for each bolt.

n_s = number of shear planes intercepting non-threaded portion of shank

= 1, for a bolt in double shear

= 0, for a bolt in single shear.

A_{sh} = nominal shank area of the bolt

A_{nb} = net shear area in threaded portion

i.e.,
$$T_{dh} = \frac{0.9 f_{ub} A_n}{\gamma_{mb}} \leq \frac{f_{yb} A_{sh}}{\gamma_{mo}}$$

where f_{yb} = yield stress of the bolt

A_n = net area of the bolt at thread

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

A_{sh} = shank area of the bolt $= \frac{\pi}{4} d^2$.

DESIGN STRENGTH OF BOLTS IN SHEAR (V_{dsf})

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}$$

where γ_{mf} = 1.10, if slip resistance is designed at service load

= 1.25, if the slip resistance is designed at ultimate load.

and $V_{nsf} = \mu_f \, n_e \, K_h \, F_o$

where μ_f = coefficient of friction as specified in Table 20 (Clause 10.4.3) in IS 800-2007.

n_e = number of effective interfaces offering frictional resistance to the slip.

[Note: n_e = 1 for each bolt in lap joint and 2 for each bolt in double cover bolt joint]

K_h = 1.0 for fasteners in clearance hole

= 0.85 for fasteners in oversized and short slotted holes and for long slotted holes loaded perpendicular to the slot.

F_o = minimum bolt tension at installation and may be taken as $A_{nb} f_o$

A_{nb} = net area of the bolt at threads

$$\left(0.78 \frac{\pi}{4} d^2\right)$$

f_o = proof stress = $0.7 f_{ub}$.

Note:

1. All the reduction factors specified for bearing bolted connection hold good for HSFG bolted connection also.
2. Since the bearing strength of HSFG bolts is greater than the plates, no check on bearing strength of the bolt is necessary.

Bolt Strength in Tension (T_{df})

$$T_{df} = \frac{0.9 f_{ub} A_n}{\gamma_{mb}} \le \frac{f_{yb} A_{sh}}{\gamma_m}$$

$$\gamma_{mb} = 1.25, \gamma_m = 1.1$$

Note: In the design of HSFG bolts subjected to tensile forces additional force Q called prying forces is to be considered.

$$Q = \frac{l_v}{2l_c}\left(T_e - \frac{\beta \eta f_0 b_e t^4}{27 l_c l_v^2}\right)$$

where

Q = prying force

$2T_e$ = total applied tensile force

l_v = distance from the bolt centre line to the toe of the fillet weld or to half the root radius for a rolled section

l_c = distance between prying forces and bolt centre line and is the minimum of either the end distance or the value given by:

$$l_c = 1.1t\sqrt{\frac{\beta f_0}{f_y}}$$

β = 2 for non-pretensioned bolts and 1 for pretensioned bolts

η = 1.5

b_e = effective width of flange per pair of bolts

f_0 = proof stress in consistent units

t = thickness of end plate.

PRINCIPLES TO BE OBSERVED IN THE DESIGN

1. Design strength should be more than design load.
2. The centre of gravity of bolts should coincide with the centre of gravity of the connected members.
3. The length of connection should be kept as small as possible.
4. It should satisfy requirements specified in clause 10.2, regarding spacing, such as
   a. Pitch shall not be less than 2.5 d.
   b. Minimum edge distance = 1.7 d_o, in case of hand cut edges and 1.5 d_o in case of rolled or machine cut edges.
5. Diameter of bolt hole for various bolts shall be taken as shown below:

diameter of bolt (d) :	12	14	16	20	22	24	30	36
diameter of bolt hole (d_o) :	13	15	18	22	24	26	33	39

6. Area of bolt at shank = $\frac{\pi}{4} d^2$

   Area of bolt at threads = 0.78 $\frac{\pi}{4} d^2$

7. Material properties of bolts.

Grade 4.6	f_{yb} = 240 MPa	f_{ub} = 400 MPa
Grade 4.8	f_{yb} = 320 MPa	f_{ub} = 420 MPa
Grade 5.6	f_{yb} = 300 MPa	f_{ub} = 500 MPa
Grade 5.8	f_{yb} = 400 MPa	f_{ub} = 520 MPa.

Design a lap joint to connect two plates each of width 100 mm, if the thickness of one plate is 12 mm and the other is 10 mm. The joint has to transfer a working load of 100 kN. The plates are of f_e 410 grade. Use bearing type of bolts and draw connection details.

Solution: Using M16 bolts of grade 4.6,

d = 16 mm $\quad d_o$ = 18 mm $\quad f_{ub}$ = 400 N/mm^2

Since it is a lap joint, the bolt is in single shear, the critical section being at the roots of the thread of the bolts.

∴ Nominal strength of a bolt in shear

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(1 \times 0 + 0.78\,\frac{\pi}{4}\,d^2\right)$$

$$= \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2$$

$$= 36218 \text{ N}$$

∴ Design strength of a bolt in shear

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{36218}{1.25} = 28974 \text{ N}$$

Minimum pitch to be provided = 2.5 d = 2.5 × 16 = 40 mm

Minimum edge distance = 1.5 d_o = 1.5 × 18 = 27 mm

Provide p = 40 mm and e = 30 mm.

Strength in bearing:

k_b is least of $\frac{30}{3 \times 18}, \frac{40}{3 \times 18} - 0.25, \frac{400}{410}$ and 1.0

i.e., $k_b = 0.4907$

Now, thickness of thinner plate = 10 mm, f_u = 400 N/mm^2

∴ Normal bearing strength of a bolt

$$V_{npb} = 2.5\,k_b\,d\,t\,f_u$$

$$= 2.5 \times 0.4907 \times 16 \times 10 \times 400$$

$$= 78512 \text{ N}$$

∴ Design strength of M16 bolts

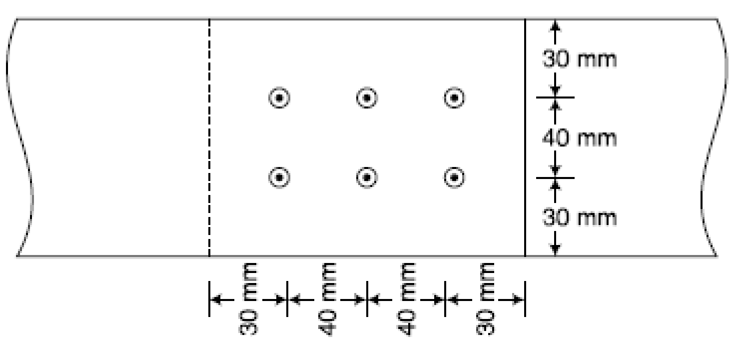
= 28974 N

Working (nominal load) = 100 kN.

∴ Design load = 100 × 1.5 = 150 kN.

Hence, no. of bolts required $= \frac{150 \times 1000}{28974} = 5.18$

Provide 6 bolts. They may be provided as shown in Fig.



Check for the Strength of Plate

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

There are two holes along the critical section,

$$\therefore \quad T_{dn} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

$$= 188928 \text{ N} = 188.928 \text{ kN} > 150 \text{ kN}.$$

Hence safe.

Design a single bolted double cover butt joint to connect boiler plates of thickness 16 mm for maximum efficiency. Use M 20 bolts of grade 4.6. Boiler plates are of Fe 410 grade. Draw the connection details.

Solution: $d = 20$ mm $\therefore d_o = 22$ mm. $f_{ub} = 400$ N/mm^2, $f_u = 410$ N/mm^2, $t = 16$ mm. Since it is double cover but joint, the bolts are in double shear, one section at shank and another at root of thread are resisting shear.

$\therefore$ Nominal strength of bolt in shear,

$$V_{nsb} = \frac{400}{\sqrt{3}}\left(1 \times \frac{\pi}{4} \times 20^2 + 1 \times 0.78 \times \frac{\pi}{4} \times 20^2\right) = 129143 \text{ N}$$

$\therefore$ Design strength

$$V_{dsb} = \frac{V_{nsb}}{1.25} = \frac{129143}{1.25} = 103314 \text{ N}$$

Assuming bearing strength is more than this, to get maximum efficiency, strength of plate per pitch width is equated to V_{dsb}.

To avoid failure of the cover plates, the total thickness of the cover plates should be more than the thickness of the main plate. Provide cover plates of 12 mm thickness.

$\therefore$ Design strength of plate per pitch width

$$= \frac{0.9 \times 410\,(p - 22)}{1.25} \times 16$$

$$= 4723.2\,(p - 22)$$

Equating it to strength of bolt, we get

$$4723.2\,(p - 22) = 103314$$

$$\therefore \quad p = 43.87 \text{ mm.}$$

Minimum pitch to be provided $= 2.5\, d$

$$= 2.5 \times 20 = 50 \text{ mm}$$

Provide $\quad p = 50$ mm.

Check for Bearing Strength of Bolt

Minimum edge distance $e = 1.7\ d_o = 1.7 \times 22 = 37.4$ mm

Provide $e = 40$ mm.

Then, k_b is minimum of $\frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$$\therefore \quad k_b = 0.5076$$

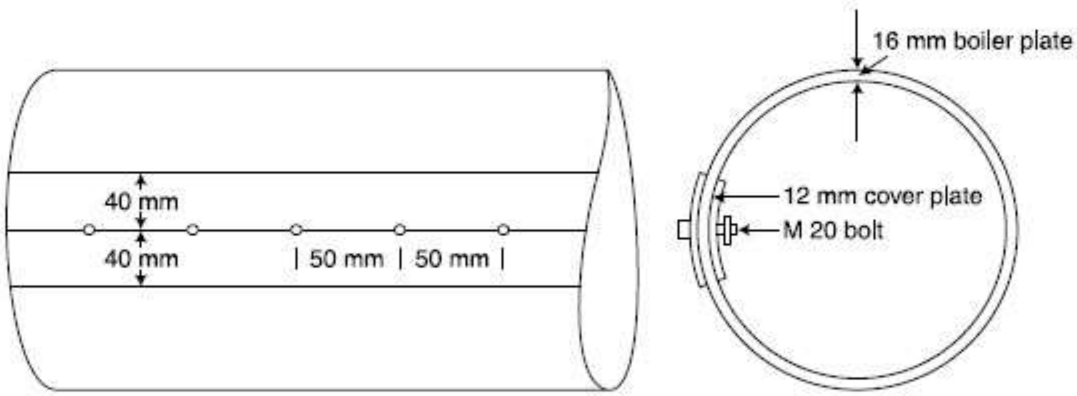
$\therefore$ Design strength of bolt in bearing

$$= \frac{2.5 \times 0.5076 \times 20 \times 16 \times 400}{1.25}$$

$$= 129939 \text{ N} > 103314 \text{ N}$$

Hence, the assumption that design strength of bolt = 103314 N is correct.

Figure 2.4 shows the connection details.



ECCENTRIC CONNECTION WITH BEARING BOLTS

There are two cases of eccentric connections:

1. Moment in the plane of bolts
2. Moment at right angles to the plane of bolt.

When Moment is in the Plane of Bolts

If P is the eccentric load and 'e' is the eccentricity, moment to be resisted by bolts

$$M = Pe$$

The number of bolts per row required is given by $n = \sqrt{\frac{[illegible]}{2Vp}}$ where p is the pitch.

If r_i is the radial distance of the bolts, and r the radial distance of extreme bolt, then force in the extreme bolt in radial direction is

$$F_2 = \frac{Per}{\Sigma r_i^2}$$

Direct shear in vertical direction

$$F_1 = \frac{P}{n}, \text{ where } n \text{ is the total number of bolts.}$$

$\therefore$ Resultant force on extreme bolt

$$F = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

where θ is the angle between F_1 and F_2.

For safe design $F \leq V$

Example A bracket is to be bolted to the flange of the column, which is of ISHB 300 @ 577 N/m. If the bracket has to carry a design load of 800 kN at an eccentricity of 250 mm, design the connection using 8 mm cover plates and M 20 bolts of grade 4.6.

Solution: Factored load on each plate of bracket $\frac{800}{2} = 400$ kN

Eccentricity = 250 mm.

$\therefore$ On each plate $\quad M = 400 \times 250 = 100000$ kN-mm

$$= 100{,}000 \times 1000 \text{ N-mm}$$

Flange thickness of ISHB 300 @ 577 N/m is 10.6 mm and thickness of cover plate is 12 mm. Hence, the thickness of thinner member is 10.6 mm.

$d = 20$ mm $\quad d_o = 22$ mm $\quad f_{ub} = 400$ N/mm^2

For rolled section $\quad f_u = 410$ N/mm^2.

Bolts are in single shear.

$\therefore$ Design strength of bolt in single shear,

$$V_{db} = \frac{1}{1.25}\frac{400}{\sqrt{3}}\left(0 + 0.78 \times \frac{\pi}{4} \times 20^2\right)$$

$$= 45272 \text{ N}$$

Strength of the bolt in bearing:

k_b is the least of $\dfrac{e}{3d_o}, \dfrac{p}{3d_o} - 0.25, \dfrac{f_{ub}}{f_u}, 1.0$

Adopting two rows of bolts each at 70 mm from the centre line of the column and pitch 50 mm ($\geq 2.5d$),

$$k_b = 0.5076$$

$$\therefore \quad V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.5076 \times 20 \times 8 \times 400$$

$$= 64973 \text{ N} > V_{db}$$

$$V = V_{db} = 45272 \text{ N}$$

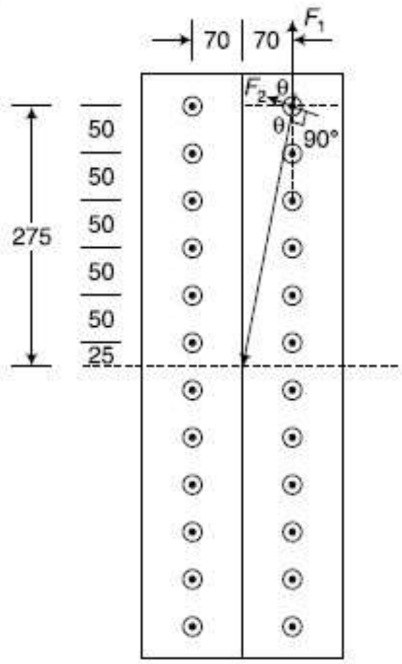


Fig. 2.5

$\therefore$ Number of bolts required per row

$$n = \sqrt{\frac{6M}{2Vp}}$$

$$= \sqrt{\frac{6 \times 100{,}000 \times 1000}{2 \times 45272 \times 50}} = 11.51$$

Provide 12 bolts in each row as shown in Fig. 2.5.

Distance of extreme bolt from the centre of gravity of bolt,

$$r = \sqrt{70^2 + 275^2} = 283.77 \text{ mm.}$$

$$\Sigma r_i^2 = 4\left[\sum_{i=1}^{6} (x_i^2 + y_i^2)\right]$$

$$= 4\,[6 \times 70^2 + 25^2 + 75^2 + 125^2 + 175^2 + 225^2 + 275^2]$$

$$= 832600 \text{ mm}^2$$

$$\therefore \quad F_2 = \frac{Per}{\Sigma r_i^2} = \frac{400 \times 10^3 \times 250 \times 283.77}{832600} = 34082 \text{ N}$$

Direct shear $\quad F_1 = \dfrac{P}{2n} = \dfrac{400 \times 1000}{2 \times 12} = 16667 \text{ N}$

$$\tan\theta = \frac{275}{70} \therefore \theta = 75.719° \quad \text{Hence, } \cos\theta = 0.24668$$

$\therefore$ Resultant force on extreme bolt

$$= \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$$= \sqrt{16667^2 + 34082^2 + 2 \times 16667 \times 34082 \times 0.24668}$$

$$= 41467 \text{ N} < V$$

$\therefore$ Design is safe.

Hence, provide 24 M 20 bolts as shown in Fig.

When the Moment is at Right Angles to the Plane of Bolts

In this case bolts are subjected to shear and tension. If P is the eccentric load, n number of bolts in the connection, direct shear

$$V_{sb} = \frac{P}{n}$$

If e is the eccentricity of load, moment is

$$M = P \times e$$

Since on tension side only bolts resist the force while on compression side entire connecting angle in contact with column resists the force, the centre of gravity is assumed at $\frac{1}{7}$ th the depth of the connection. Hence, moment resisted by bolts in tension,

$$M' = \frac{1}{\left[1 + \frac{2h}{21}\frac{\Sigma y_i}{\Sigma y_i^2}\right]}$$

where y_i is distance of ith bolt from CG and h is depth of connecting angle from topmost bolt. Then tensile force in the extreme bolt due to bending moment

$$T_b = \frac{M'y}{\Sigma y_i^2}$$

Design criteria to be satisfied is

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

where V_{db} = shear strength of bolt

T_{db} = tensile strength of bolt.

Steps to be Followed in the Design

1. Select nominal diameter d of the bolt.
2. Adopt pitch = 2.5 d to 3.0 d.

3. Number of bolts in each row

$$= \sqrt{\frac{6M}{(2V)p}}$$

4. Find V_{sb}, T_b, T_{dh} and check for satisfying interaction formula.

Design a suitable bolted bracket connection for connecting a ISST-200 section to the flange of a ISHB 300 @ 577 N/m to carry a vertical factored load of 400 kN at an eccentricity of 150 mm. Use M20 bolts of grade 4.6 [Ref. Fig. 2.6]

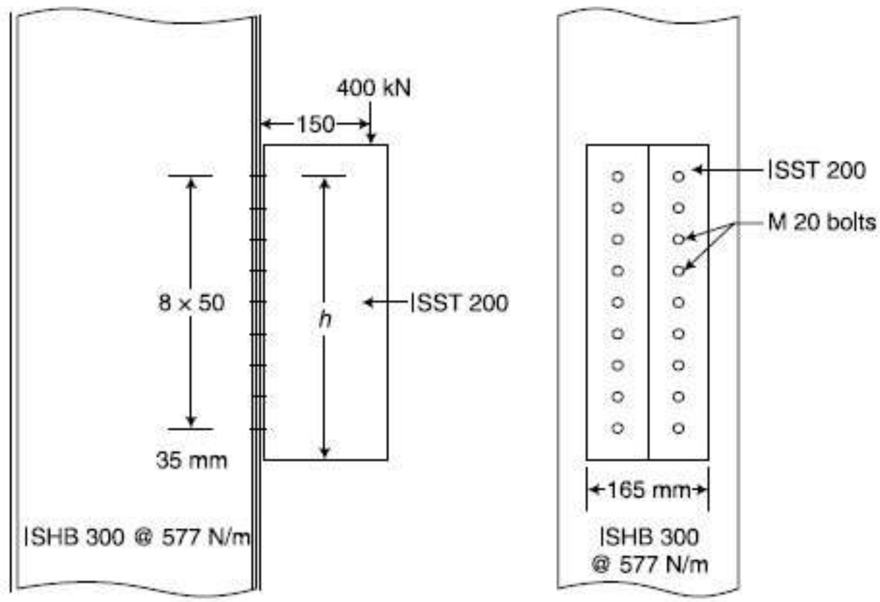


Fig. 2.6

Solution: For M 20 bolts of grade 4.6, $d = 20$ mm, $d_o = 22$ mm, $f_{ub} = 400$ N/mm^2, $f_{yb} = 240$ N/m

For rolled steel section, $f_u = 410$ N/mm^2.

Thickness of flange of ISST 200 is 12.5 mm

Thickness of flange of ISHB 300 @ 577 N/m = 10.6 mm.

∴ Thickness of thinner member = 10.6 mm.

Design strength of M20 bolts in single shear

$$= \frac{1}{1.25} \frac{400}{\sqrt{3}} \left[0 + 0.78 \times \frac{\pi}{4} \times 20^2\right]$$

$$= 45272 \text{ N}$$

Minimum edge distance $= 1.5\, d_o = 1.5 \times 22 = 33$ mm

Minimum pitch $= 2.5\, d = 2.5 \times 20 = 50$ mm.

Let $e = 35$ mm and $p = 50$ mm.

k_b is the smaller of $\frac{35}{3 \times 22}, \frac{50}{3 \times 22} - 0.5, \frac{400}{410}, 1.0$

i.e., $k_b = 0.5076$

Design strength of bolts in bearing against 10.6 mm thick flange of I-section

$$= \frac{1}{1.25} \times 2.5 \times 0.5076 \times 20 \times 10.6 \times 400$$

$$= 86088 \text{ N} > V_{db}.$$

$\therefore$ Design strength of bolt in shear $V = V_{db} = 45272$ N

Design tension capacity of bolt

$$T_{db} = \frac{0.90 f_{ub} A_n}{\gamma_m} < \frac{f_{yb} A_{sb}}{\gamma_{mo}}$$

$$= 0.90 \times 400 \times 0.78 \times \frac{\pi}{4} \times 20^2 < \frac{240 \times \frac{\pi}{4} \times 20^2}{1.1}$$

$$= 88216 < 68544.$$

$\therefore$ $T_{db} = 68544$ N

Using two rows of bolts, number of bolts required in each row,

$$n = \sqrt{\frac{6M}{(2V)p}}$$

Now, $M = 400 \times 150$ kN-mm

$$= 400 \times 150 \times 10^3 \text{ N-mm.}$$

$\therefore$ $$n = \sqrt{\frac{6 \times 400 \times 150 \times 10^3}{2 \times 45272 \times 50}} = 8.91$$

Provide 9 bolts in each row as shown in Fig. 2.6.

$$h = 35 + 50 \times (9 - 1) = 435 \text{ mm.}$$

$$\frac{h}{7} = \frac{435}{7} = 62.14 \text{ mm}$$

i.e., neutral axis lies between bolt nos. 1 and 2 counted from bottom.

$\therefore$ $y_2 = 35 + 50 - 62.14 = 22.86$ mm

Bolt No.	2	3	4	5	6	7	8	9
y	22.86	72.86	122.86	172.86	222.86	272.86	322.86	372.86

$\therefore$ $\sum y_i = 1582.86 \times 2$ mm

$$\sum y_i^2 = 418188 \times 2 \text{ mm}^2$$

Design Shear Strength of Bolts

$$V_{dsf} = \frac{1}{1.25} \cdot \mu_f n_e k_n F_o$$

$$\mu_f = 0.48,\ n_e = 1,\ k_n = 1 \text{ for fasteners in clearance}$$

$$F_o = A_{ub} f_o = 0.78 \times \frac{\pi}{4} \times 24^2 \times 0.7 \times 800$$

$$= 197604 \text{ N}$$

$$\therefore \quad V_{dsF} = 1/1.25 \times 0.48 \times 1 \times 1 \times 197604$$

$$= 75880 \text{ N}$$

Since there are two rows of bolts in the connection, number of bolts required per row when p = 70 mm and taking $V = V_{dsf}$, we get

$$n = \sqrt{\frac{6M}{(2V)p}} = \sqrt{\frac{6 \times 600 \times 1000 \times 250}{2 \times 75880 \times 70}} = 9.2$$

Provide 10 bolts in each row with edge distance 40 mm as shown in Fig. 2.7.

Tensile capacity of bolts

$$= \frac{1}{1.25} \times 0.9 f_{ub} A_{ub}$$

$$= \frac{1}{1.25} \times 0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 24^2$$

$$= 203250 \text{ N}$$

When there is no load, the bracket is held on to the column by compression developed due to the bolt tension. This phenomenon continues even after the load is applied. Hence, the interface of the area 150 × 710 mm may be considered a plane in a monolithic beam. The stress diagram is shown in Fig. 2.7.

$$\text{Max. bending stress} = \frac{6M}{bd^2} = \frac{6 \times 600 \times 10^3 \times 250}{150 \times 710^2}$$

$$= 11.9 \text{ N/mm}^2$$

$$\text{Bending stress at } 40 + \frac{70}{2} = 75 \text{ mm from top flange}$$

$$= 11.9 \times \frac{355 - 75}{355} = 9.39 \text{ N/mm}^2.$$

$$\therefore \text{ Average stress} = \frac{11.9 + 9.39}{2} = 10.64 \text{ N/mm}^2$$

This average bending stress could be considered tension in the bolt.

$\therefore$ Tension in extreme two bolts, $2T_e = 10.64 \times 150 \times 75$

$$2T_e = 119744 \text{ N.}$$

$$\therefore \quad T_e = 59872 \text{ N}$$

Prying Forces

Plate width = 150 mm. Plate thickness = 12 mm

$$l_v = \frac{150}{2} - 6 - 8 - 40 = 21 \text{ mm.}$$

where 8 mm is assumed thickness of weld

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

$$\therefore \quad l_c = 1.1 \times 12 \times \frac{\sqrt{1 \times 0.7 \times 410}}{250} = 14.14 \text{ mm} < \text{edge distance}$$

$$\therefore \quad l_c = 14.14 \text{ mm.}$$

$\beta = 1.0$ for pretensioned bolts.

$b_e = 150$ mm, $f_o = 0.7, f_{ub} = 0.7 \times 800 = 560$ N/mm^2

$t = 12$ mm.

$\therefore$ Prying force Q is given by

$$Q = \frac{l_v}{2l_c}\left[T_e - \frac{\beta n f_o b_e t^4}{27\, l_c l_v^2}\right]$$

$$= \frac{21}{2 \times 14.14}\left[59872 - \frac{1.0 \times 1.5 \times 560 \times 150 \times 12^4}{27 \times 14.14 \times 21^2}\right]$$

$$= 32936 \text{ N}$$

$\therefore$ Total tensile force in the bolt

$$T_f = 59872 + 32936 = 92808 \text{ N}$$

Tension capacity $T_{df} = \dfrac{1}{1.25} \times 0.9 f_{ub} A_n$

$$= \frac{1}{1.25} \times 0.9 \times 800 \times 0.78 \times \frac{\pi}{4} \times 24^2$$

$$= 203249 \text{ N}$$

Direct shear in the bolt $= \dfrac{600 \times 1000}{2 \times 10} = 30000$ N

$$\therefore \quad \left(\frac{V_{sb}}{V_{dsf}}\right)^2 + \left(\frac{T_f}{T_{dsf}}\right)^2 = \left(\frac{30000}{75880}\right)^2 + \left(\frac{92808}{203249}\right)^2$$

$$= 0.365 < 1.0$$

Hence, OK.

UNIT II CONNECTIONS IN STEEL STRUCTURES

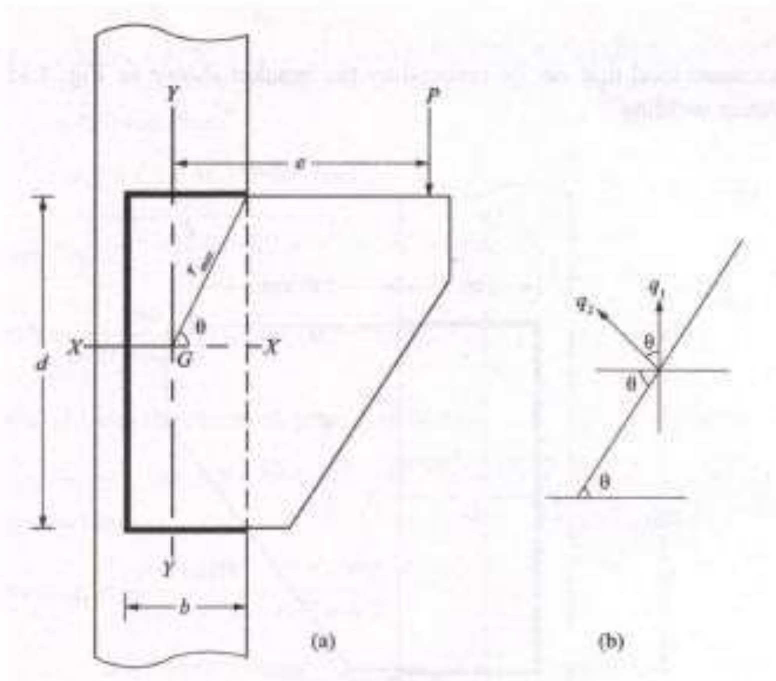
ECCENTRIC CONNECTION:-

Plane of Moment is the same:-

The eccentric load 'P' is equivalent to

(i) A direct axial load acting along the C.G of the group of weld.

(ii) A twisting Moment, M=P x e



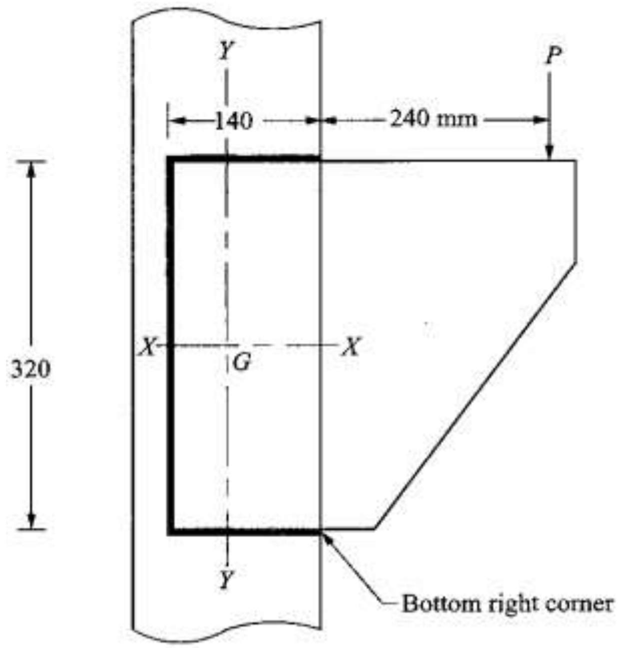
Assuming uniform size of weld with throat tks, 't' and length of weld provided as shown in the fig with sides b&d, the direct shear stress,

$$q_1 = \frac{P}{(2b+d)t}$$

The stress due to twisting moment acting ¿ lr to the C.G of the weld group and the radius vector,

$$q_2 = \frac{p \times e \times y_{max}}{I_{zz}}, \quad \text{Resultant stress } q = \sqrt{q_1^2 \times q_2^2 + 2q_1 q_2 \cos\theta}$$

1. Determine the max load that can be resisted by a bracket shown in fig. Fillet weld of size 6mm is provided as shop welding.



Given:-

Size of weld = 6mm

Depth = 300mm

Sln:-

Here the weld group is provided such that the plane of welding is parallel to the plane of moment.

$\therefore$ Direct shear stress, $q_1 = \frac{P}{A}$

Shear due to moment $q_2 = \frac{P \times e \times y_{max}}{I_{zz}}$

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

<u>Weld Group:-</u>

t = threat tks

t = 0.7S

= 0.7(6) = 4.2m

$\bar{y} = \frac{320}{2} = 160\,mm$

$\bar{x} = \frac{a_1 x_1 + a_2 x_2 + a_3 x_3}{a_1 + a_2 + a_3}$

$= \frac{140 \times 4.2 \times 70 + 3116 \times 4.2 \times 2.1 + 4.2 \times 140 \times 70}{588 + 1308.72 + 588}$

$\bar{x}$ = 34.24mm

$I_{xx} + I_{yy} = I_{zz}$

$$I_{xx}=\frac{140\times4.2^3}{12}+140\times4.2\times(317.9-160)^2$$
$$+\frac{311.6\times4.2}{12}+4.2\times311.6(160-160)^2$$
$$+\frac{140\times4.2^3}{12}+140\times4.2(160-2.1)^2$$
$$= 14661121.44+10589132.71+14661121.44$$
$$I_{xx} = 39911375.59mm^4$$
$$I_{yy}=\left[\frac{4.2\times(140)^3}{12}+4.2\times140(70-34.2)^2\right]\times2$$
$$+\frac{311.6\times(4.2)^3}{12}+311.6\times4.2(34.2-2.1)^2$$
$$I_{yy}=4.78\times10^6 mm^4$$
$$\therefore I_{zz}=44.69\times10^6 mm^4$$

Direct shear stress $q_1=\frac{P}{A}$

A = Total area of the weld group

$$=\frac{P}{[140+320+140]\times4.2}$$
$$q_1=3.968\times10^{-4} PKN$$
$$q_1 = 0.3968\ PN$$
$$q_2=\frac{p\times e\times y_{max}}{I_{zz}}$$

Where,

e = (140-34.24) + 240

= 345.76mm

$$y_{max}=\sqrt{(160)^2+(140-34.24)^2}$$
$$y_{max} = 191.79mm$$
$$=\frac{P\times345.76\times191.79}{44.69\times10^6}$$
$$= 1.483\times10^{-3} PKN/mm^2$$
$$q_2=1.483\ PN/mm^2$$

Resultant stress $q=\sqrt{(0.3968P)^2+(1.483P)^2+2\times0.3968\times1.483P\times\cos\theta}$

Where,

θ = The angle made by radial distance with the C.G

$$\tan\theta=\frac{160}{140-34.24}$$
$$\tan\theta = 1.51285$$
$$\theta = 56^\circ32'$$
$$= P\sqrt{2.3567+0.649}$$

q = 1.7336P -→ (1)

The max load that can be applied to resist the stress the weld can take is

$$= \frac{410/\sqrt{3}}{1.29}$$

$$= 189.37\, N/mm^2 \rightarrow (2)$$

Equating (1) & (2)

1.7336P = 189.37

$\therefore$ P = 109.24 NS

Eccentric Connection - Plane of weld group ¿ lr to the plane of moment:-

For eccentric connection with plane of weld group ¿ lr to the plane of moment 2 types of stresses are developed,

(i) Direct shear stress, $q = \frac{P}{A}$

Where, A =2bt

(ii) The bending stress @ the extreme end of weld

$$F = \frac{M}{Z} = \frac{p \times e}{\frac{2t \times b^2}{6}}$$

$$F = \frac{6Pe}{2tb^2}$$

The equivalent stress, $f_e = \sqrt{f^2 + 3q^2}$

For the weld to be safe the above equivalent stress is equated to the design stress of weld.

Design stress of weld = $\frac{fu/\sqrt{3}}{\gamma_m}$

1. Design a suitable fillet weld for an eccentrically loaded bracket plate. The working load P=100KN and eccentricity, e = 150mm.Tks of bracket plate is 12mm & tha column used is ISHB300@618 N/m [Plane of weld group is ¿ lr to the plane of moment]

NOTE:-

To find the eff. depth of weld (b) considering only the moment case, the eff. depth is assumed as b = 1.1 $\sqrt{\frac{6M}{2tf_{wd}}}$

1. A 18mm tk plate is joint to a 16mm plate by 200mm long [effective] butt weld. Determine the strength of the joint, if

(i) A double 'V' butt joint is provided

(ii) A single "V' butt joint is provided

Assume the grade Fe410 for the plates and for the welds which are shop welded

Given Data:-

L_e= 200mm

Grade of plate Fe=410

$Fu = 410\ N/mm^2$

Weld:- Shop welded

Sln:-

(i) Double 'V' butt joint:-

Strength of weld = Design stress of weld X Eff. Area

= fwd x lw x t

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = \frac{fu}{\sqrt{3}}$$

For double 'V' butt joint complete penetration of the takes place.

$$f_{wn} = 236.71 N/mm^2$$

$$f_{wd} = \frac{236.71}{1.25}$$

$$f_{wd} = 189.368 N/mm^2$$

Strength of weld = 189.368 x 200 x 16

= 605.977 KN

(ii) Single 'V' butt joint:-

t = 5/8 s [Incomplete Penetration]

= 5/8 x 16

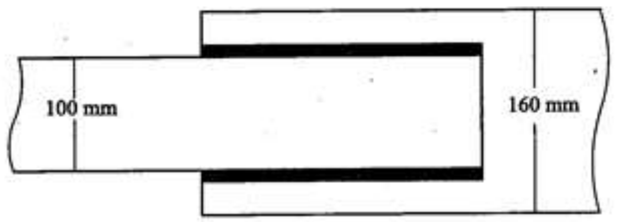
t = 10mm

$$\text{Strength of weld} = \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

= 189.368 x 200 x 10

Strength of weld = 378.74 KN

2. Design a suitable longitudinal fillet weld to connect the plates as shown in fig. The pull to be transmitted is equal to the full strength of the small plate. Given the plates are 12mm tk, grade of plates is Fe410 and welding is made in the factories.



Given:-

TKS of Plate = 12mm

Grade of Plate Fe410

$fu = 410\ N/mm^2$

$fy = 250\ N/mm^2$

The strength of the weld is equated to the design strength of the smaller plate.

Design strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$

Mini size of weld = 5mm [from Table-21 pg.No:78]

Maxi size of weld = t_p- 1.5 = 12-1.5 = 10.5mm

Assume the size of weld = 10mm

t = 0.7s

= 0.7 x 10

t = 7mm

Strength of smaller plate [yielding criteria] = $\frac{fuAg}{\gamma_o}$

Where, $\gamma_o = 1.1$

Strength of smaller plate

(yielding criteria) = $\frac{250 \times 1200}{1.1}$

= 272.72 KN

Strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$

$272.72 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 7$

$$\boxed{lw = 205.7\ mm \simeq 205\ mm}$$

Provide an over lap of 105mm.

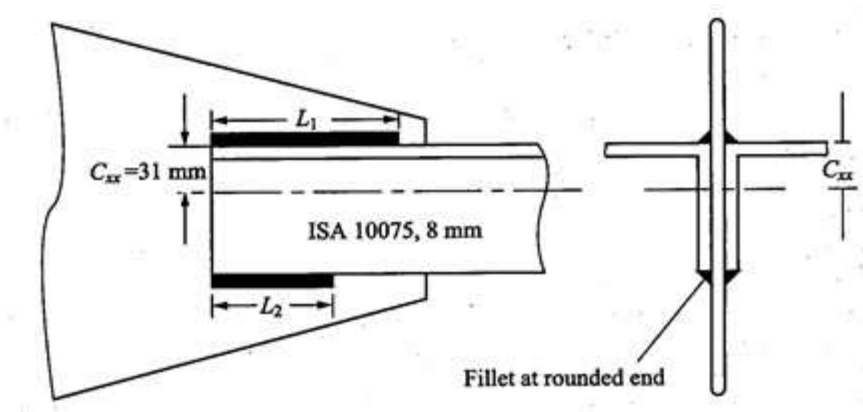
3. A tie member of a roof truss consist of 2Nos of ISA 100x75x8mm. The angles are connected to either side of a 10mm tk guset plate and the member is subjected to a working pull of 300KN. Design the welded connection. Assume the connections are made in the shop.

Given Data:-

Working load = 300KN

2 ISA 100x75x8mm

Tks of guset plate = 10mm



Sln:-

Factored load = 1.5x300

= 450 KN

Each ISA 100x75x8mm takes 450/2 = 225 KN

Min. size of weld = 3mm [From table-21 IS 800-2007]

Max. size of weld = 8-1.5

= 6.5mm

Also, max. size of weld (rounded edger) = 3/4 x 8 = 6mm

Throat tks, t = 0.7 x S [$\because$ Angle of fusion = 90^0]

= 0.7 x 6

t = 4.2mm

Strength of weld = Design stress of weld x Eff. Area

$$= \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

$$225 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 4.2$$

$$\because lw = 282.89\,mm \simeq 283\,mm$$

Since the C.G of angle section does not lie at the centre of the connected leg, the weld length at top & bottom need to be such that the C.G of weld.

C.G of angle ISA 100x75x8 = 31mm from the outstanding leg.

<u>To find C.G of weld:-</u>

Let L1 = length of weld @ top

L2 = length of weld @ bottom

$\therefore$ For the C.G of the weld to lie at 31mm from the outstanding leg

$$\therefore L_1 \times 31 = L_2(100-31)$$

$$L_1 = 2.23 l_2$$

$$L_1 \times L_2 = 283$$

$$2.23 l_2 + l_2 = 283$$

$$3.23 L_2 = 283$$

$L_2 = 87.62mm$ ¿$90\,mm$
$L_1 = 195.39\,mm \simeq 200\,mm$

Provide 200mm length of weld @ the top and 90mm length of weld @ the bottom

∴ The min. over lap length is required 200mm

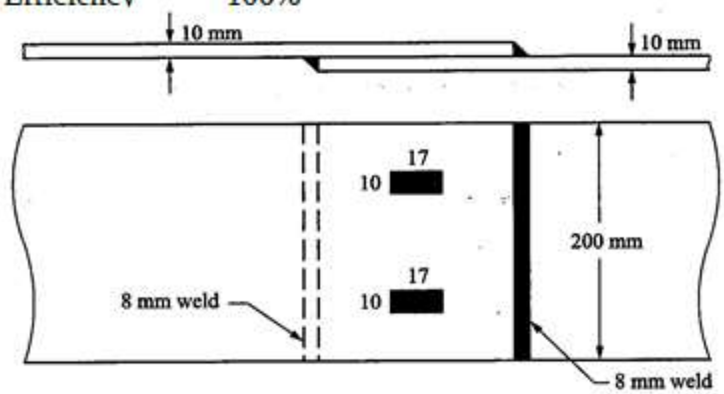
NOTE:-

In case the length of weld is limited, (length of overlap) end fillet weld can be provided which should also satisfy the condition C.G of weld = C.G of member.

4. Design the welded connection to connect 2 plates of width 200mm & tks 10mm for 100% efficiency.

Given:-

Width of plate = 200mm
Tks of plate = 10mm
Efficiency = 100%



Sln:-

1. Strength of the solid plate:-

$$= \frac{fyAg}{\gamma_o}$$
$$= \frac{250 \times 200 \times 10}{1.1}$$
$$= 454.5\ KN$$

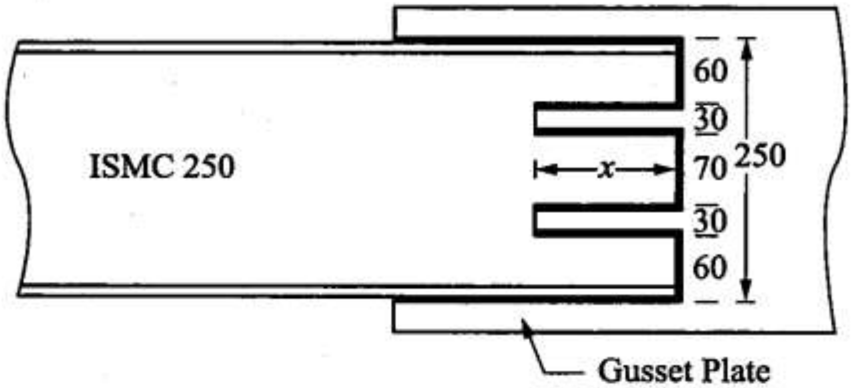
Mini. Size of weld = 3mm
Maxi. Size of weld = 10 -1.5=8.5mm

Assume size of weld as 8mm 78.5mm

Strength of the weld = Design stress of weld x Eff. Area

$$= \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

$454.5\times10^3=\frac{410/\sqrt{3}}{}\times lw\times0.7\times8$



γ_{mw}

$= \frac{410/\sqrt{3}}{1.25}\times368\times5.6$

Sln:-

ISMC 250 – Properties:-

$A = 3867mm^2$

$t_f = 14.1mm$

$t_w = 7.1mm$

Strength of solid Plate:-

Strength of solid plate [channel] = $\frac{fyAg}{\gamma_o}$

$= \frac{250\times3867}{1.1}$

= 878.86KN

∴ Strength of weld read = 878.86KN

Mini size of weld = 3mm [from table-21]

Maxi size of weld = 7.1-1.5=5.6mm

∴ Provide size of weld S = 4mm

∴ Throat tks, t = 0.7 x S

= 0.7 x 4

t = 2.8mm

Strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}}\times lw\times t$

$878.86 \times 10^3 = \frac{410/\sqrt{3}}{1.25}\times lw\times2.8$

lw = 1657.48mm

The available length along sides & end = 400+250+400 = 1050mm

[Since overlap is limited to 400mm]

[Either plug weld or slot weld can be provided]

Assuming 2 nos of 30mm wide to be provided along the end of the channel at equal spacing.

∴ Reqd length of slot = $\frac{1657-1050+2\times4}{4}$

$= 153.75mm$

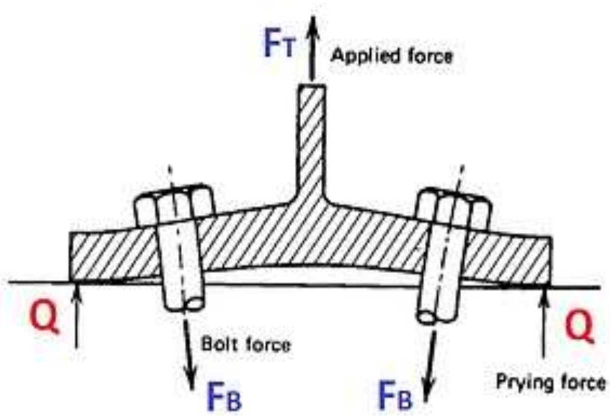
$\therefore$ Length reqd. for the slot = 153.75mm

$\therefore$ Provide 2 slots of length 154mm

UNIT II CONNECTIONS IN STEEL STRUCTURES

PRYING FORCES:

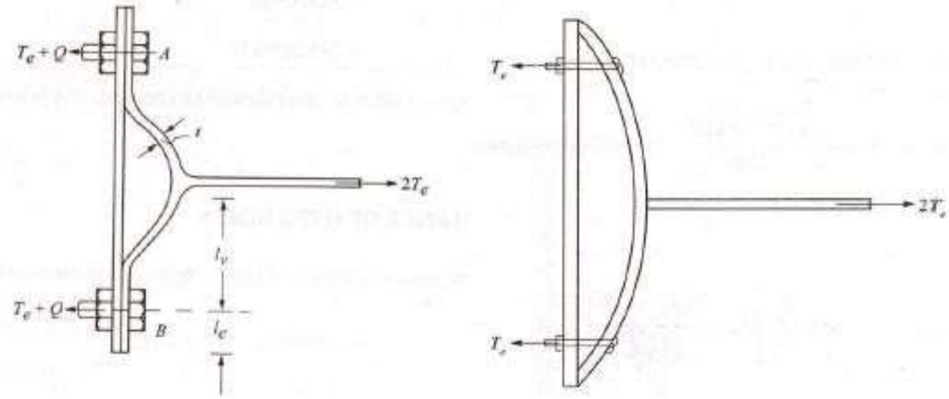
The concept of prying action can be most easily presented in terms of a T-stub loaded by a tensile force FT. The applied force is not concentric to the centre of the bolt and additional forces (Q) may be developed near the flange tip, increasing the bolt force itself. The tension induced in the bolts, from equilibrium, is thus FB= FT/2 + Q. This phenomenon is illustrated in Fig.



There are some important comments and notes regarding prying action:

• Prying action in high strength friction grip bolts is reduced only at a relatively low level of load and the behaviour of the bolts at ultimate load is not significantly affected.
• Even if prying action is low, the distortion of the connected parts results in significant bending of the bolt and local bending of the bolt nut or head.
• Prying action in Eurocode 1993-1-8:2005 is implicitly taken into account when determining the design tension resistance.
• Base plate connections usually have long anchor bolts and a thick base plate when compared to an
end plate. This results in the uplift of the T-stub from the concrete foundation and in these situations, prying of anchor bolts is not observed.

PRYING FORCE:- [cls 10.4.7 IS 800-2007]

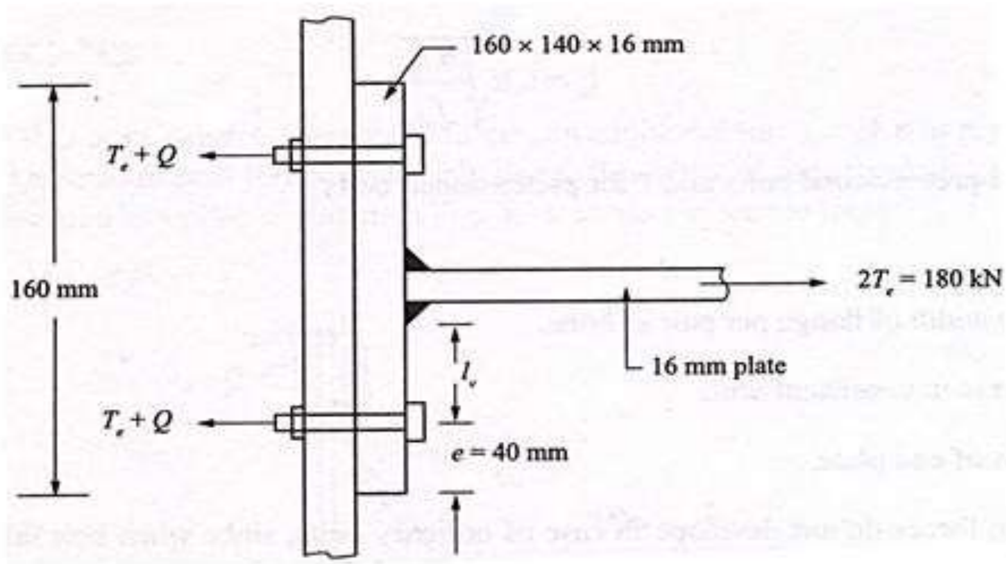


$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta \eta f_o b_e t^4}{27 l_e l_v^2}\right] [Q\ f_o = 0.7 f_{ub}]$$

When,

$$l_e = 1.1t\sqrt{\frac{\beta f_o}{fy}} < e[Q\ f_o = 0.7 f_u]$$

1. The jt shown in fig. as to carry a factored load of 180KN. End plate used size 160x140x16mm. The bolts used are M20 HSFG bolts of grade 8.8 check whether the design is safe.



Given:-

M20 HSFG Grade 8.8

f_{ub}= 800 N/mm^2

Plate:-

f_u= 400 N/mm^2

27 = 180 KN

Assuming 8mm size weld a edge distance are of 40mm

$$l_v = \frac{160}{2} - 8 - 8 - 40$$

$$l_v = 24mm$$

l_e least of $1.1t\sqrt{\frac{\beta f_o}{fy}}$(or)40mm

For β = 1 for pretensioned bolt

$f_o = 0.7fu$ [∵ fu for plate assumed as 410 N/mm^2 fy=250 N/mm^2]

= 0.7 x 410

f_o= 287 N/mm^2

$$\because l_e = 1.1 \times 16\sqrt{\frac{1 \times 287}{250}}$$

= 18.86mm (or) 40mm

$$\boxed{l_e = 18.86mm}$$

Prying force $Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_o b_e t^4}{27 l_e l_v^2}\right]$

When,

$$\beta = 1, \eta = 1.5, T_e = 90KN, f_o = 0.7 \times 800 = 560N/mm^2$$

$$b_e = 140mm, t = 16mm, l_e = 18.86, l_v = 24mm$$

$$\therefore Q = \frac{24}{2 \times 18.86}\left[90 - \frac{1 \times 1.5 \times 560 \times 140 \times (16)^4}{27 \times 18.86 \times (24)^2}\right]$$

$$\boxed{Q = 40.55 \ KN}$$

Total force on bolt = T + a

= 90+40.55

= 130.5 KN

Tension capacity of bolt = $\frac{0.9 f_{ub} A_n}{\gamma_m}$

$= \frac{0.9 \times 800 \times A_n}{\gamma_m}$

$$A_n = \frac{0.78 \times \pi \times 20^2}{4}$$

= 245 mm^2

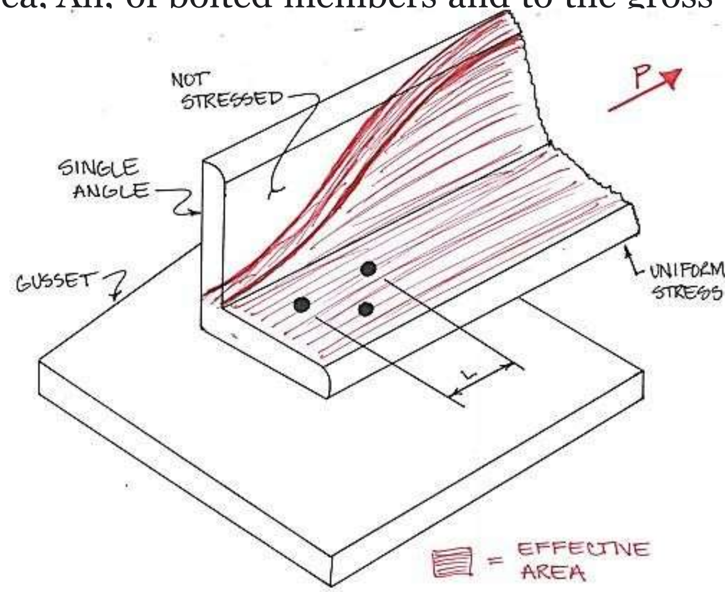
$= \frac{0.9 \times 800 \times 245}{1.25}$

$T_e = 141.12KN$

Hence the design is safe.

Shear lag

Shear lag is a concept used to account for uneven stress distribution in connected members where some but not all of their elements (flange, web, leg, etc.) are connected. The reduction coefficient, U, is applied to the net area, An, of bolted members and to the gross area, Ag, of welded members.



This occurs when some element of the member is not connected. Consider an angle section tension member connected with one leg only as shown in figure. Hence at the joint/connection more of the load is carried by the connection leg and it takes a transition distance as indicated in fig, for the stress to spread uniformly across the whole angle, stress distribution in the two legs of the angle would be different. In the transition region the stress in the connected part may even exceed f_y and go into strain hardening range, the member may fracture prematurely. Away from the joint/connection the stress transfer is uniform. In the transition zone shear transfer lags. Since shear lag reduces the effectiveness of the outstanding leg it is kept smaller length generally. For this reason unequal angles with long leg as connecting leg is preferred.

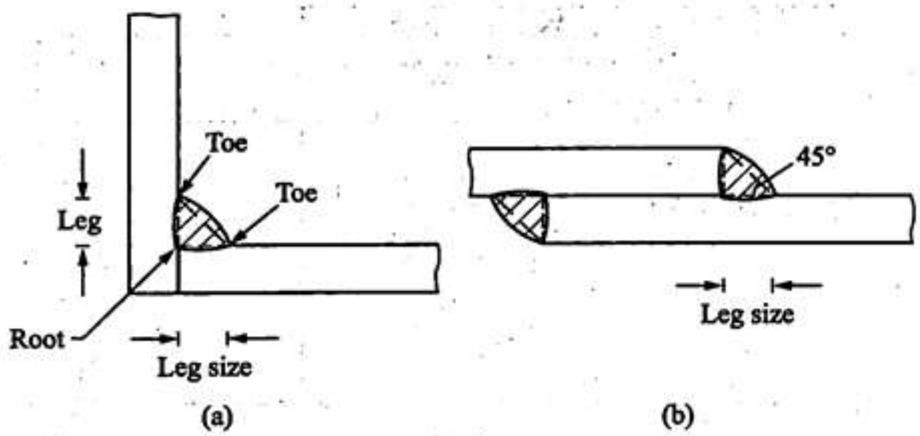
Hence shear lag is the function of distribution of steel and length of the load transfer L. It is the independent of type of load and applied to both bolted and welded connections.

UNIT II CONNECTIONS IN STEEL STRUCTURES

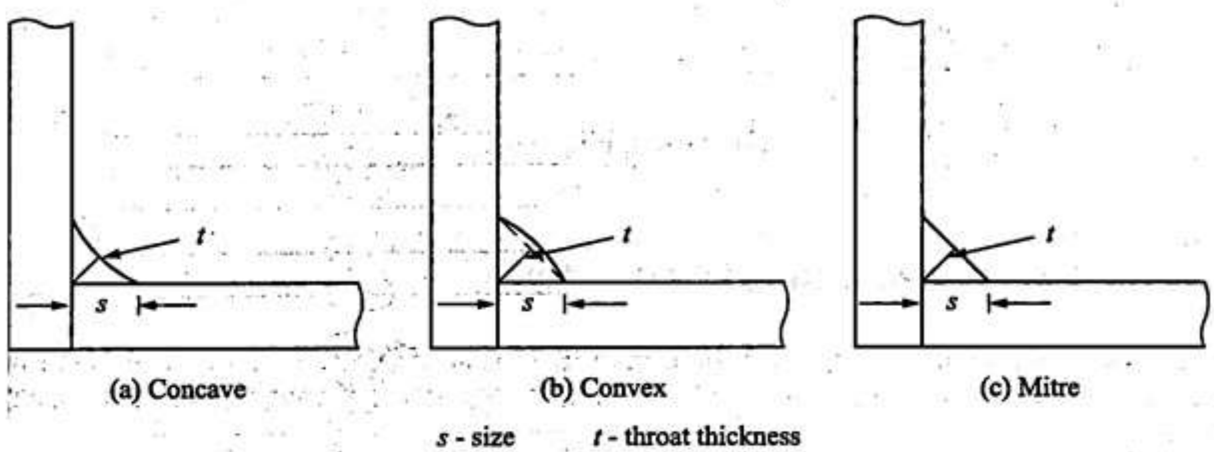
WELDED CONNECTIONS:- [IS 10.5, IS 800-2007]

Welded connections are advantageous in most of the cases, since

(i) Self wt. reduces due to absence of guest plates, connecting angles etc.
(ii) The connection is rigid.
(iii) The process is quicker
(iv) Asthetic appearance is good.
(v) Relatively lesser
(vi) Welded connections are air tight & water tight
(vii) Welded connections are preferable for trusses with circular c/s.



Typical fillet welds.



Types of fillet welds.

DISADVANTAGES OF WELDED CONNECTIONS:-

a) Due to uneven heating & cooling members are likely to distart.
b) There is possibility of brittle fracture at the welded joint.
c) A welded connection fails earlier than a bolted connection, due to fatigue.
d) Inspection of welded its is difficult and expensive.
e) Highly skilled labour is regd. For weld.
f) Proper welding in the field condition is required.

Types of Welds:-

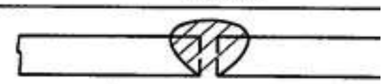
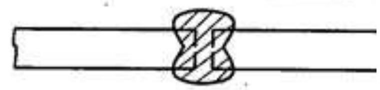
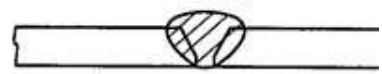
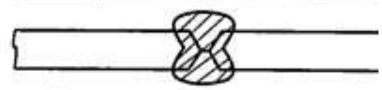
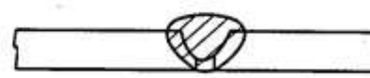
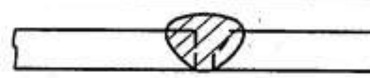
i) Lap weld
ii) Butt weld
iii) Slot weld
iv) Plug weld

(i) Lap weld:-

(ii) Butt weld:-

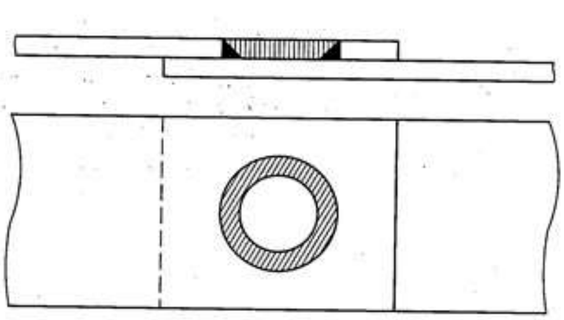
1. Single square butt weld
2. Double square butt weld
3. Single 'V' butt weld
4. Double 'V' butt weld
5. Single 'U' butt weld
6. Double 'U' butt weld
7. 'J' Butt weld

Types of butt weld

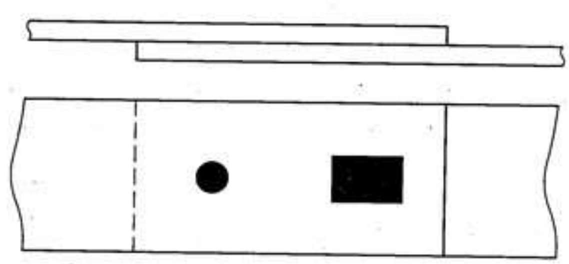
Sl. No.	Type of Butt Weld	Sketch
(a)	Square butt weld, on one side	
(b)	Square butt weld, both sides	
(c)	Single V butt joint	
(d)	Double V-butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint	
(g)	Single bevel butt joint	

Note: Similarly there can be double U, double J and double bevel butt joints.

(iii) Slot & Plug weld:-

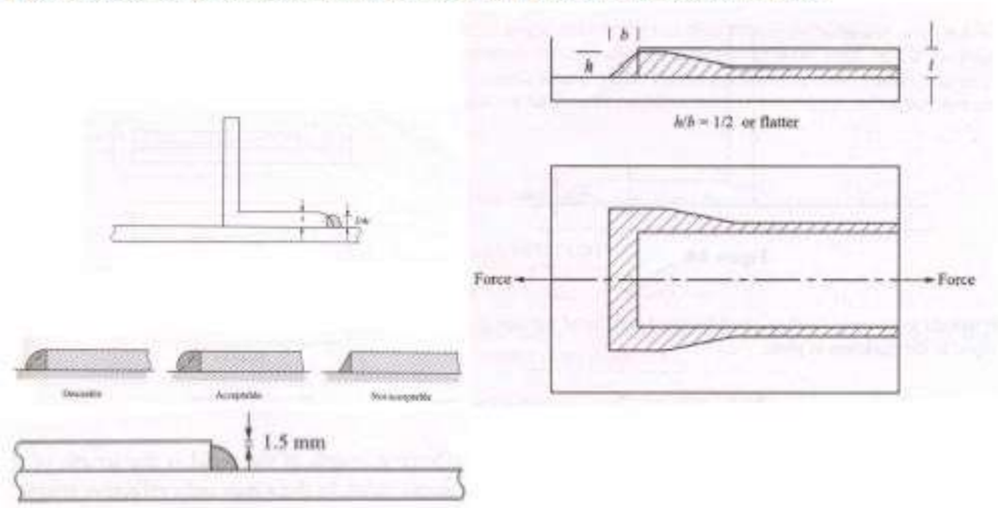


Slot weld.



Plug welds.

I.S. 800-2007 PROVISIONS FOR WELDED CONNECTION:-



1. Butt weld:-

- The size of weld is specified by effective throat tks.
- In case of complete penetration butt weld, it is taken as tks of the thinner part jt.
- Double U & Double V Double J type butt welds are regarded as complete penetration butt welds.
- For incomplete penetration butt welds, is taken as 5/8t
- The eff. Length of butt weld is taken as length of full size weld.
- The mini. Length of butt weld shall be 4 times the size of weld.

2. Filled weld:-

(a) Size of fillet weld:-

- The size of normal fillet weld is taken as mini weld leg size
- For deep penetration weld with penetration not less than 2.4mm, the size of weld is mini. leg size + 2.4mm

(b) Mini size of weld is 3mm. for plates of tks 10 to 20mm, min size is 5mm, for 20 to 32mm plates min size is 6mm & greater than 32mm plates min size is 8mm

(c) Eff. Threat tks:-

- It shall not be less than 3mm and shall not exceed 0.7t [upto 90^o] where, t=tks of thinner plate.

(d) Eff. Length:-

- The eff. Length of the weld is the length of weld for which the specified size and throat tks exist.
- The welding length provided is equal to the eff. Length t twice the size of weld

$$L = leff + 2s$$

- Eff. Length should not be less than 4 times the size of weld.

(e) The min. lap should be 4 times the tks of thinner part jt (or) 40mm whichever is more.