3 Connections

3.1 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 account for more than half the cost of structural steelwork and so their design and detailing are of primary importance for the economy of the structure.

The type of connection designed has an influence on member design and so must be decided even prior to the design of the structural system and design of members. For example, in the design of bolted tension members, the net area is calculated assuming a suitable number and diameter of bolts based on experience. Therefore, it is necessary to verify the net area after designing the connection. Similarly in the analysis of frames, the member forces are determined by assuming the connections to be pinned, rigid, or semi-rigid, as the actual behaviour cannot be precisely defined.

Just as members are classified as bending members or axially loaded members depending on the dominant force/moment resisted, connections are also classified into idealised types while designing. But the actual behaviour of the connection may be different and this point should always be kept in mind so that the connection designed does not differ significantly from the intended type. Take for example, the connection of an axially loaded truss member at a joint. If the truss is assumed to be pin jointed, then the member should ideally be connected by means of a single pin or bolt. However, in practice, if the pin or bolt diameter works out to be larger than that possible, more than one bolt will be used. The truss can then be considered pin-jointed only if the bending due to self-weight or other superimposed loads beta joints is negligible. Note that the
Connection behaviour will also influence the calculation of the effective length for the buckling analysis of struts.

Connection elements consist of components such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and weld. The connections provided in steel structures can be classified as 1) riveted 2) bolted and 3) welded connections. Riveted connections were once very popular and are still used in some cases but will gradually be replaced by bolted connections. This is due to the low strength of rivets, higher installation costs and the inherent inefficiency of the connection. Welded connections have the advantage that no holes need to be drilled in the member and consequently have higher efficiencies. However, welding in the field may be difficult, costly, and time consuming. Welded connections are also susceptible to failure by cracking under repeated cyclic loads due to fatigue which may be due to working loads such as trains passing over a bridge (high-cycle fatigue) or earthquakes (low-cycle fatigue). A special type of bolted connection using High Strength Friction Grip (HSFG) bolts has been found to perform better under such conditions than the conventional black bolts used to resist predominantly static loading. Bolted connections are also easy to inspect and replace. The choice of using a particular type of connection is entirely that of the designer and he should take his decision based on a good understanding of the connection behaviour, economy and speed of construction. Ease of fabrication and erection should be considered in the design of connections. Attention should be paid to clearances necessary for field erection, tolerances, tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.
3.2 Bolted connections

3.2.1 Connection classification

(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.3.1 (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 [See Fig.3.1 (b)].

The Moment connections are more complex to analyse compared to the above two types and are shown in Fig.3.2 (a) and Fig.3.2 (b). The connection in Fig.3.2 (a) is also known as bracket connection and the resistance is only through shear in the bolts.

The connection shown in Fig.3.2 (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.
(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.

![Fig 3.2 Moment connections](image)

Typical shear connections occur as a lap or a butt joint used in the tension members [See Fig.3.3]. 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. In the of single cover plate (lap) joint, the thickness of the cover plate is chosen to be equal to or greater than the connected plates. While in double cover plate (butt) joint, the combined thickness of the cover plates should be equal to or greater than the connected plates.


**Fig 3.3 Shear connections**

A hanger connection is shown in Fig.3.4 (a). In this connection, load transmission is by pure tension in the bolts. In the connection shown in Fig.3.4 (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). The force transfer in either case is discussed in more detail later.

**Fig 3.4(a) Tension connection (b) Tension plus shear connection**

**3.2.2 Bolts and bolting**

Bolts used in steel structures are of three types: 1) Black Bolts 2) Turned and Fitted Bolts and 3) High Strength Friction Grip (HSFG) Bolts.
The International Standards Organisation designation for bolts, also followed in India, is given by Grade $x.y$. In this nomenclature, $x$ indicates one-tenth of the minimum ultimate tensile strength of the bolt in kgf/mm$^2$ and the second number, $y$, indicates one-tenth of the ratio of the yield stress to ultimate stress, expressed as a percentage. Thus, for example, grade 4.6 bolt will have a minimum ultimate strength 40 kgf/mm$^2$ (392 Mpa) and minimum yield strength of 0.6 times 40, which is 24 kgf/mm$^2$ (235 Mpa).

Black bolts are unfinished and are made of mild steel and are usually of Grade 4.6. Black bolts have adequate strength and ductility when used properly; but while tightening the nut snug tight (“Snug tight” is defined as the tightness that exists when all plies in a joint are in firm contact) will twist off easily if tightened too much. Turned-and-fitted bolts have uniform shanks and are inserted in close tolerance drilled holes and made snug tight by box spanners. The diameter of the hole is about 1.5 to 2.0 mm larger than the bolt diameter for ease in fitting. High strength black bolts (grade 8.8) may also be used in connections in which the bolts are tightened snug fit.

In these bearing type of connections, the plates are in firm contact but may slip under loading until the hole surface bears against the bolt. The load transmitted from plate to bolt is therefore by bearing and the bolt is in shear as explained in the next section. Under dynamic loads, the nuts are liable to become loose and so these bolts are not allowed for use under such loading. In situations where small slips can cause significant effects as in beam splices, black bolts are not preferred. However, due to the lower cost of the bolt and its installation, black bolts are quite popular in simple structures subjected to static loading.

Turned and fitted bolts are available from grade 4.6 to grade 8.8. For the higher grades there is no definite yield point and so 0.2% proof stress is used.

High Strength Friction Grip bolts (HSFG) provide extremely efficient connections and perform well under fluctuating/fatigue load conditions. These bolts should be
tightened to their proof loads and require hardened washers to distribute the load under the bolt heads. The washers are usually tapered when used on rolled steel sections. The tension in the bolt ensures that no slip takes place under working conditions and so the load transmission from plate to the bolt is through friction and not by bearing as explained in the next section. However, under ultimate load, the friction may be overcome leading to a slip and so bearing will govern the design.

HSFG bolts are made from quenched and tempered alloy steels with grades from 8.8 to 10.9. The most common are the so-called, general grade of 8.8 and have medium carbon content, which makes them less ductile. The 10.9 grade have a much higher tensile strength, but lower ductility and the margin between the 0.2% yield strength and the ultimate strength is also lower.

The tightening of HSFG bolts can be done by either of the following methods (IS 4000-..):

1. **Turn-of-nut tightening method**: In this method the bolts are first made snug tight and then turned by specific amounts (usually either half or three-fourth turns) to induce tension equal to the proof load (Fig 3.5(a)).

2. **Calibrated wrench tightening method**: In this method the bolts are tightened by a wrench (Fig 3.5(b)) calibrated to produce the required tension.

3. **Alternate design bolt installation**: In this method special bolts are used which indicate the bolt tension. Presently such bolts are not available in India.

4. **Direct tension indicator method**: In this method special washers with protrusions are used [Fig.3.5(c)]. As the bolt is tightened, these protrusions are compressed and the gap produced by them gets reduced in proportion to the load. This gap is
measured by means of a feeler gauge, consisting of small bits of steel plates of varying thickness, which can be inserted into the gap.

Fig 3.5 Tightening of HSFG bolts

Since HSFG bolts under working loads, do not rely on resistance from bearing, holes larger than usual can be provided to ease erection and take care of lack-of-fit. Typical hole types that can be used are standard, extra large and short or long slotted. These are shown in Fig.3.6. However the type of hole will govern the strength of the connection.

Fig. 3.6 Hole types for HSFG bolts
Holes must also satisfy pitch and edge/end distance criteria (Cl.10.2). A minimum pitch is usually specified for accommodating the spanner and to limit adverse interaction between the bearing stresses on neighbouring bolts. A maximum pitch criterion takes care of buckling of the plies under compressive loads.

### 3.2.3 Shear connections with bearing type bolts

In this section the force transfer mechanisms of bearing and friction type of bolted connections are described. This would help in identifying the modes of failure discussed in the next section.

#### 3.2.3.1 Force transfer of bearing type bolts

Fig. 3.7 shows the free body diagram of the shear force transfer in bearing type of bolted connection. It is seen that tension in one plate is equilibrated by the bearing stress between the bolt and the hole in the plate. Since there is a clearance between the bolt and the hole in which it is fitted, the bearing stress is mobilised only after the plates slip relative to one another and start bearing on the bolt. The section $x-x$ in the bolt is critical section for shear. Since it is a lap joint there is only one critical section in shear (single shear) in the bolt. In the case of butt splices there would be two critical sections in the bolt in shear (double shear), corresponding to the two cover plates.

![Fig. 3.7 Shear transfer by bearing type bolt](image-url)
3.2.3.2 Design shear strength of bearing type bolts

The failure of connections with bearing bolts in shear involves either bolt failure or the failure of the connected plates. In this section, the failure modes are described along with the codal provisions for design and detailing shear connections.

In connections made with bearing type of bolts, the behaviour is linear until i) yielding takes place at the net section of the plate under combined tension and flexure or ii) shearing takes place at the bolt shear plane or iii) failure of bolt takes place in bearing, iv) failure of plate takes place in bearing and v) block shear failure occurs. Of these, i) and v) are discussed in the chapter on tension members. The remaining three are described below.

1. Shearing of bolts: The shearing of bolts can take place in the threaded portion of the bolt and so the area at the root of the threads, also called the tensile stress area, is taken as the shear area. Since threads can occur in the shear plane, the area for resisting shear should normally be taken as the net tensile stress area, of the bolts. The shear area is specified in the code and is usually about 0.8 times the shank area. However, if it is ensured that the threads will not lie in the shear plane then the full area can be taken as the shear area.

A bolt subjected to a factored shear force \( V_{sb} \) shall satisfy

\[
V_{sb} \leq V_{nsb} / \gamma_{mb}
\]

Where

\[
V_{nsb} = \text{nominal shear capacity of a bolt, calculated as follows:}
\]

\[
\gamma_{mb} = 1.25
\]

\[
V_{nsb} = \frac{f_u}{\sqrt{3}} \left( n_n A_{nb} + n_s A_{sb} \right) \quad (3.1)
\]

Where

\( f_u = \text{ultimate tensile strength of a bolt} \)
\[ n_n = \text{number of shear planes with threads intercepting the shear plane} \]

\[ n_s = \text{number of shear planes without threads intercepting the shear plane} \]

\[ A_{sb} = \text{nominal plain shank area of the bolt} \]

\[ A_{nb} = \text{net tensile area at threads, may be taken as the area corresponding to root diameter at the thread} \]

For bolts in single shear, either \( n_n \) or \( n_s \) is one and the other is zero. For bolts in double shear the sum of \( n_n \) and \( n_s \) is two.

2. **Bearing failure:** If the connected plates are made of high strength steel then failure of bolt can take place by bearing of the plates on the bolts. If the plate material is weaker than the bolt material, then failure will occur by bearing of the bolt on the plate and the hole will elongate. The bearing area is given by the nominal diameter of the bolt times the combined thickness of the plates bearing in any direction. A bolt bearing on any plate subjected to a factored shear force \( V_{sb} \) shall satisfy

\[ V_{sb} \leq V_{npb} / \gamma_{mb} \quad (3.2) \]

Where, \( \gamma_{mb} = 1.25 \)

\[ V_{npb} = \text{bearing strength of a bolt, calculated as} \]

\[ V_{npb} = 2.5df_u \quad (3.3) \]

Where

\( f_u = \text{smaller of the ultimate tensile stress of the bolt and the ultimate tensile stress of the plate} \)

\( d = \text{nominal diameter of the bolt} \)

\( t = \text{summation of the thicknesses of the connected plates experiencing bearing stress in the same direction} \).
Fig. 3.8 Types of failures in a shear connection (a) Shearing of bolts (b) Bearing failure of plate (c) Bearing failure of bolt

The underlying assumption behind the design of bolted connections, namely that all bolts carry equal load is not true in some cases as mentioned below.

In long joints, the bolts farther away from the centre of the joint will carry more load than the bolts located close to the centre. Therefore, for joints having more than two bolts on either side of the building connection with the distance between the first and the last bolt exceeding 15\(d\) in the direction of load, the nominal shear capacity \(V_{ns}\) shall be reduced by the factor, \(\beta_{lj}\), given by (Cl.10.3.2.1)

\[
\beta_{lj} = 1.075 - \frac{lj}{(200 \, d)} \quad \text{but} \quad 0.75 < \beta_{lj} < 1.0
\]

Where, \(d\) = nominal diameter of the bolt

Similarly, if the grip length exceeds five times the nominal diameter, the strength is reduced as specified in IS 800. In multibolt connections, due to hole mismatch, all the bolts may not carry the same load. However, under ultimate load, due to high bearing ductility of the plates considerable redistribution of the load is possible and so the assumption that all bolts carry equal load may be considered valid.

3.2.4 Shear connection with HSFG bolts

3.2.4.1 Force transfer of HSFG bolts

The free body diagram of an HSFG connection is shown in Fig. 3.9. It can be seen that the pretension in the bolt causes clamping forces between the plates even
before the external load is applied. When the external load is applied, the tendency of
two plates to slip against one another is resisted by the friction between the plates. The
frictional resistance is equal to the coefficient of friction multiplied by the normal
clamping force between the plates. Until the externally applied force exceeds this
frictional resistance the relative slip between the plates is prevented. The HSFG
connections are designed such that under service load the force does not exceed the
frictional resistance so that the relative slip is avoided during service. When the external
force exceeds the frictional resistance the plates slip until the bolts come into contact
with the plate and start bearing against the hole. Beyond this point the external force is
resisted by the combined action of the frictional resistance and the bearing resistance.

![Diagram](image)

**Fig. 3.9 Shear transfer by HSFG Bolt**

### 3.2.4.2 Design shear strength of HSFG bolts

HSFG bolts will come into bearing only after slip takes place. Therefore if slip is
critical (i.e. if slip cannot be allowed) then one has to calculate the slip resistance, which
will govern the design. However, if slip is not critical, and limit state method is used then
bearing failure can occur at the Limit State of collapse and needs to be checked. Even
in the Limit State method, since HSFG bolts are designed to withstand working loads
without slipping, the slip resistance needs to be checked anyway as a Serviceability
Limit State.
1. Slip Resistance: Design for friction type bolting in which slip is required to be limited, a bolt subjected only to a factored design shear force, \( V_{sf} \) in the interface of connections shall satisfy the following (Cl.10.4.3):

\[
V_{sf} \leq V_{nsf} / \gamma_{mf}
\]

Where \( \gamma_{mf} = 1.25 \)

\( V_{nsf} \) = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

\[
V_{nsf} = \mu_f n_e K_h F_o
\]  

(3.4)

Where, \( \mu_f \) = coefficient of friction (slip factor) as specified in Table 3.1 (\( \mu_f \leq 0.55 \))(Table 3.1 of code).

\( n_e \) = number of effective interfaces offering frictional resistance to slip

\( K_h \) = 1.0 for fasteners in clearance holes

\[= 0.85 \text{ for fasteners in oversized and short slotted holes, and for fasteners in long slotted holes loaded perpendicular to the slot} \]

\[= 0.7 \text{ for fasteners in long slotted holes loaded parallel to the slot.} \]

\( \gamma_{mf} = 1.10 \) (if slip resistance is designed at service load)

\( \gamma_{mf} = 1.25 \) (if slip resistance is designed at ultimate load)

\( F_o \) = minimum bolt tension (proof load) at installation and may be taken as \( 0.8 A_{sb} F_o \)

\( A_{sb} \) = shank area of the bolt in tension

\( f_o \) = proof stress (= 0.70 \( f_{ub} \))

\( V_{ns} \) may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.
2. Bearing strength: The design for friction type bolting, in which bearing stress in the ultimate limit state is required to be limited, \( V_{ub} = \text{factored load bearing force} \) shall satisfy (Cl.10.4.4)

\[
V_{bf} \leq V_{nbf} / \gamma_{mf}
\]

Where \( \gamma_{mf} = 1.25 \)

\( V_{nbf} \) = bearing capacity of a bolt, for friction type connection, calculated as follows:

\[
V_{nbf} = 2.2 \cdot d \cdot t \cdot f_{up} \leq 3 \cdot d \cdot t \cdot f_{yp} \quad (3.5)
\]

Where

- \( f_{up} \) = ultimate tensile stress of the plate
- \( f_{yp} \) = tensile yield stress of the plate
- \( d \) = nominal diameter of the bolt
- \( t \) = summation of thicknesses of all the connected plates experiencing bearing stress in the same direction

The block shear resistance of the edge distance due to bearing force shall also be checked.

### 3.2.5 Tension connections with bearing and HSFG bolts

#### 3.2.5.1 Force transfer by bearing and HSFG bolts

The free body diagram of the tension transfer in a bearing type of bolted connection is shown in Fig. 3.10(a). The variation of bolt tension due to externally applied tension is shown in Fig.3.10(c). It is seen that before any external tension is applied, the force in the bolt is almost zero, since the bolts are only snug tight. As the external tension is increased it is equilibrated by the increase in bolt tension. Failure is

---

**Table 3.1 Typical average values for coefficient of friction (\( \mu_f \))**

<table>
<thead>
<tr>
<th>Treatment of surface</th>
<th>Coeff. of friction (( \mu_f ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean mill scale</td>
<td>0.33</td>
</tr>
<tr>
<td>Sand blasted surface</td>
<td>0.48</td>
</tr>
<tr>
<td>Surfaces blasted with shot or grit and hot-dip galvanized</td>
<td>0.10</td>
</tr>
</tbody>
</table>
reached due to large elongation when the root of the bolt starts yielding. Depending on the relative flexibility of the plate and the bolt, sometimes the opening of the joint may be accompanied by prying action [Fig. 3.10(d)].

The free body diagram of an HSFG bolted connection is shown in Fig. 3.10(b). It is seen that even before any external load is applied, the force in the bolt is equal to proof load. Correspondingly there is a clamping force between the plates in contact. When the external load is applied, part of the load (nearly 10%) of the load is equilibrated by the increase in the bolt force. The balance of the force is equilibrated by the reduction in contact between the plates. This process continues and the contact between the plates is maintained until the contact force due to pre-tensioning is reduced to zero by the externally applied load. Normally, the design is done such that the externally applied tension does not exceed this level. After the external force exceeds this level, the behaviour of the bolt under tension is essentially the same as that in a bearing type of joint.

\[
\begin{align*}
(a) & \text{ Bearing type connection} & (b) & \text{ HSFG Connection} \\
\end{align*}
\]

\[
\begin{align*}
(c) & \text{ External Tension} \\
& \text{versus bolt force} & (d) & \text{Prying Effect} \\
\end{align*}
\]
Fig. 3.10 Bolts under tension and prying effect

Where prying force, $Q$, is significant, prying force shall be calculated as given below and added to the tension in the bolt (Cl.10.4.7).

\[
Q = \frac{l_v}{2 l_c} \left[ T_c - \frac{\beta \gamma f_o b_e t^4}{27 l_c l_v^2} \right] \quad (3.7)
\]

Where, $l_v = \text{distance from the bolt centreline to the toe of the fillet weld or to half the root radius for a rolled section}$; $l_o = \text{distance between prying force and bolt centreline and is the minimum of, either the end distance, or the value given by}$

\[
l_e = 1.1 t \sqrt[4]{\frac{f_o}{f_y}} \quad (3.8)
\]

Where,

- $\beta = 2$ for non pre-tensioned bolt and 1 for pre-tensioned bolt
- $\gamma = 1.5$
- $b_e = \text{effective width of flange per pair of bolts}$
- $f_o = \text{proof stress in consistent units}$
- $t = \text{thickness of the end plate}$

Even if the bolts are strong enough to carry the additional prying forces, the plate can fail by developing a mechanism with yield lines at the centreline of the bolt and at the distance $b$ from it. Therefore, the minimum thickness of the end plate ($t$), to avoid yielding of the plate, can be obtained by equating the moment in the plate at the bolt centreline (point A) and at the distance $b$ from it (point B), to the plastic moment capacity of the plate $M_p$. Thus,

\[
M_A = Qn; \quad M_B = T_b - Qn \quad (3.9)
\]

\[
M_A = M_B = \frac{T_b}{2} = M_p \quad (3.10)
\]

taking $M_p$ as
the minimum thickness for the end plate can be obtained as

\[ t_{\min} = \sqrt{\frac{1.15 \times 4 \times M_p}{f_y \times w}} \]  \hspace{1cm} (3.12)

The corresponding prying force can then be obtained as \( Q = \frac{M_p}{n} \). If the total force in the bolt \((T+Q)\) exceeds the tensile capacity of the bolt, then the thickness of the end plate will have to be increased.

### 3.2.5.2 Design tensile strength of bearing and HSFG bolts

In a tension or hanger connection, the applied load produces tension in the bolts and the bolts are designed as tension members. If the attached plate is allowed to deform, additional tensile forces called prying forces are developed in the bolts.

**Tension Capacity** – A bolt subjected to a factored tension force \( (T_b) \) shall satisfy

\[ T_b \leq \frac{T_{nb}}{\gamma_{mb}} \hspace{1cm} \gamma_{mb} = 1.25 \]  \hspace{1cm} (Cl.10.3.4)

Where, \( T_{nb} \) = nominal tensile capacity of the bolt, calculated as follows:

\[ T_{nb} = 0.90 f_{ub} A_n < f_{yb} A_{sb} \left( \frac{\gamma_{m1}}{\gamma_{m0}} \right) \hspace{1cm} \gamma_{mo} = 1.10 \quad \text{and} \quad \gamma_{mf} = 1.25 \]

Where,

- \( f_{ub} \) = ultimate tensile stress of the bolt
- \( f_{yb} \) = yield stress of the bolt
- \( A_n \) = net tensile stress area as specified in the appropriate Indian Standard. For bolts where the tensile stress area is not defined, \( A_n \) shall be taken as the area at the root of the threads (explained in next - chapter)
- \( A_{sb} \) = shank area of the bolt
3.2.5.3 Combined shear and tension failure

**Bolt Subjected to Combined Shear and Tension** – A bolt required to resist both design shear force ($V_{sd}$) and design tensile force ($T_{nd}$) at the same time shall satisfy

$$\left(\frac{V}{V_{sd}}\right)^2 + \left(\frac{T_e}{T_{nd}}\right)^2 \leq 1.0 \quad (3.13)$$

Where, $V =$ applied shear; $V_{sd} =$ design shear capacity; $T_e =$ externally applied tension and $T_{nd} =$ design tension capacity. This gives a circular interaction curve as shown in Fig. 3.11.

Bolts in a connection for which slip in the serviceability limit state shall be limited, which are subjected to a tension force, $T$, and shear force, $V$, shall satisfy (Cl.10.4.6)

$$\left(\frac{V}{V_{sdf}}\right)^2 + \left(\frac{T_e}{T_{ndf}}\right)^2 \leq 1.0 \quad (3.14)$$

Where, $V =$ applied shear at service load; $V_{sdf} =$ design shear strength; $T_e =$ externally applied tension at service load; $T_{ndf} =$ design tension strength.

![Fig. 3.11 Shear and Tension Interaction Curve](image-url)
3.3 Welding and welded connections

Welding is the process of joining two pieces of metal by creating a strong metallurgical bond between them by heating or pressure or both. It is distinguished from other forms of mechanical connections, such as riveting or bolting, which are formed by friction or mechanical interlocking. It is one of the oldest and reliable methods of joining.

Welding offers many advantages over bolting and riveting. Welding enables direct transfer of stress between members eliminating gusset and splice plates necessary for bolted structures. Hence, the weight of the joint is minimum. In the case of tension members, the absence of holes improves the efficiency of the section. It involves less fabrication cost compared to other methods due to handling of fewer parts and elimination of operations like drilling, punching etc. and consequently less labour leading to economy. Welding offers air tight and water tight joining and hence is ideal for oil storage tanks, ships etc. Welded structures also have a neat appearance and enable the connection of complicated shapes. Welded structures are more rigid compared to structures with riveted and bolted connections. A truly continuous structure is formed by the process of fusing the members together. Generally welded joints are as strong or stronger than the base metal, thereby placing no restriction on the joints. Stress concentration effect is also considerably less in a welded connection.

Some of the disadvantages of welding are that it requires skilled manpower for welding as well as inspection. Also, non-destructive evaluation may have to be carried out to detect defects in welds. Welding in the field may be difficult due to the location or environment. Welded joints are highly prone to cracking under fatigue loading. Large residual stresses and distortion are developed in welded connections.

3.3.1 Fundamentals of welding

A welded joint is obtained when two clean surfaces are brought into contact with each other and either pressure or heat, or both are applied to obtain a bond. The tendency of atoms to bond is the fundamental basis of welding. The inter-diffusion
between the materials that are joined is the underlying principle in all welding processes. The diffusion may take place in the liquid, solid or mixed state. In welding the metallic materials are joined by the formation of metallic bonds and a perfect connection is formed. In practice however, it is very difficult to achieve a perfect joint; for, real surfaces are never smooth. When welding, contact is established only at a few points in the surface, joins irregular surfaces where atomic bonding occurs. Therefore the strength attained will be only a fraction of the full strength. Also, the irregular surface may not be very clean, being contaminated with adsorbed moisture, oxide film, grease layer etc. In the welding of such surfaces, the contaminants have to be removed for the bonding of the surface atoms to take place. This can be accomplished by applying either heat or pressure. In practical welding, both heat and pressure are applied to get a good joint.

As pointed out earlier, any welding process needs some form of energy, often heat, to connect the two materials. The relative amount of heat and pressure required to join two materials may vary considerably between two extreme cases in which either heat or pressure alone is applied. When heat alone is applied to make the joint, pressure is used merely to keep the joining members together. Examples of such a process are Gas Tungsten Arc Welding (GTAW), Shielded Metal Arc Welding (SMAW), Submerged Arc Welding (SAW) etc. On the other hand pressure alone is used to make the bonding by plastic deformation, examples being cold welding, roll welding, ultrasonic welding etc. There are other welding methods where both pressure and heat are employed, such as resistance welding, friction welding etc. A flame, an arc or resistance to an electric current, produces the required heat. Electric arc is by far the most popular source of heat used in commercial welding practice.

3.3.2 Welding process

In general, gas and arc welding are employed; but, almost all structural welding is arc welding.
In gas welding a mixture of oxygen and some suitable gas is burned at the tip of a torch held in the welder’s hand or by an automatic machine. Acetylene is the gas used in structural welding and the process is called oxyacetylene welding. The flame produced can be used both for cutting and welding of metals. Gas welding is a simple and inexpensive process. But, the process is slow compared to other means of welding. It is generally used for repair and maintenance work.

The most common welding processes, especially for structural steel, use electric energy as the heat source produced by the electric arc. IS:816 in this process, the base metal and the welding rod are heated to the fusion temperature by an electric arc. The arc is a continuous spark formed when a large current at a low voltage is discharged between the electrode and the base metal through a thermally ionised gaseous column, called plasma. The resistance of the air or gas between the electrode and the objects being welded changes the electric energy into heat. A temperature of $3300^0$ C to $5500^0$ C is produced in the arc. The welding rod is connected to one terminal of the current source and the object to be welded to the other. In arc welding, fusion takes place by the flow of material from the welding rod across the arc without pressure being applied. The Shielded Metal Arc Welding process is explained in the following paragraph.

In Shielded Metal Arc Welding or SMAW (Fig.3.12), heating is done by means of electric arc between a coated electrode and the material being joined. In case bare wire electrode (without coating) is employed, the molten metal gets exposed to atmosphere and combines chemically with oxygen and nitrogen forming defective welds. The electrode coating on the welding rod forms a gaseous shield that helps to exclude oxygen and stabilise the arc.

The coated electrode also deposits a slag in the molten metal, which because of its lesser density compared to the base metal, floats on the surface of the molten metal pool, shields it from atmosphere, and slows cooling. After cooling, the slag can be easily removed by hammering and wire brushing.
The coating on the electrode thus: shields the arc from atmosphere; coats the molten metal pool against oxidation; stabilises the arc; shapes the molten metal by surface tension and provides alloying element to weld metal.

Fig.3.12 Shielded metal arc welding (SMAW) process

The type of welding electrode used would decide the weld properties such as strength, ductility and corrosion resistance. The type to be used for a particular job depends upon the type of metal being welded, the amount of material to be added and the position of the work. The two general classes of electrodes are lightly coated and heavily coated. The heavily coated electrodes are normally used in structural welding. The resulting welds are stronger, more corrosion resistant and more ductile compared to welds produced by lightly coated electrodes. Usually the SMAW process is either automatic or semi-automatic.

Fig.3.12 Shielded metal arc welding (SMAW) process

The term weldability is defined as the ability to obtain economic welds, which are good, crack-free and would meet all the requirements. Of great importance are the chemistry and the structure of the base metal and the weld metal. The effects of heating and cooling associated with fusion welding are experienced by the weld metal and the Heat Affected Zone (HAZ) of the base metal. The cracks in HAZ are mainly caused by high carbon content, hydrogen embrittlement and rate of cooling. For most steels, weld cracks become a problem as the thickness of the plates increases.
3.3.3 Types of joints and welds

By means of welding, it is possible to make continuous, load bearing joints between the members of a structure. A variety of joints is used in structural steel work and they can be classified into four basic configurations namely, Lap joint, Tee joint, Butt joint and Corner joint.

For lap joints, the ends of two members are overlapped and for butt joints, the two members are placed end to end. The T-joints form a Tee and in Corner joints, the ends are joined like the letter L. Most common joints are made up of fillet weld or the butt (also calling groove) weld. Plug and slot welds are not generally used in structural steel work. Fig.3.14 Fillet welds are suitable for lap joints and Tee joints and groove welds for butt and corner joints. Butt welds can be of complete penetration or incomplete penetration depending upon whether the penetration is complete through the thickness or partial. Generally a description of welded joints requires an indication of the type of both the joint and the weld.

Though fillet welds are weaker than butt welds, about 80% of the connections are made with fillet welds. The reason for the wider use of fillet welds is that in the case of fillet welds, when members are lapped over each other, large tolerances are allowed in erection. For butt welds, the members to be connected have to fit perfectly when they are lined up for welding. Further butt welding requires the shaping of the surfaces to be joined as shown in Fig. 3.15. To ensure full penetration and a sound weld, a backup plate is temporarily provided as shown in Fig.3.15

Butt welds:

Full penetration butt welds are formed when the parts are connected together within the thickness of the parent metal. For thin parts, it is possible to achieve full penetration of the weld. For thicker parts, edge preparation may have to be done to achieve the welding. There are nine different types of butt joints: square, single V,
double V, single U, double U, single J, double J, single bevel and double bevel. They are shown in Fig. 3.13. In order to qualify for a full penetration weld; there are certain conditions to be satisfied while making the welds.

Welds are also classified according to their position into flat, horizontal, vertical and overhead. Flat welds are the most economical to make while overhead welds are the most difficult and expensive.

![Fig. 3.13 Different types of butt welds](image)

The main use of butt welds is to connect structural members, which are in the same plane. A few of the many different butt welds are shown in Fig. 3.16. There are many variations of butt welds and each is classified according to its particular shape. Each type of butt weld requires a specific edge preparation and is named accordingly. The proper selection of a particular type depends upon: Size of the plate to be joined; welding is by hand or automatic; type of welding equipment, whether both sides are accessible and the position of the weld.

Butt welds have high strength, high resistance to impact and cyclic stress. They are most direct joints and introduce least eccentricity in the joint. But their major disadvantages are: high residual stresses, necessity of edge preparation and proper aligning of the members in the field. Therefore, field butt joints are rarely used.
To minimise weld distortions and residual stresses, the heat input is minimised and hence the welding volume is minimised. This reduction in the volume of weld also
reduces cost. Hence for thicker plates, double Butt welds and U welds are generally used. For a butt weld, the root gap, \( R \), is the separation of the pieces being joined and is provided for the electrode to access the base of a joint. The smaller the root gap the greater the angle of the bevel. The depth by which the arc melts into the plate is called the depth of penetration [Fig.3.17 (a)]. Roughly, the penetration is about 1 mm per 100A and in manual welding the current is usually 150 – 200 A. Therefore, the mating edges of the plates must be cut back if through-thickness continuity is to be established. This groove is filled with the molten metal from the electrode. The first run that is deposited in the bottom of a groove is termed as the root run [Fig.3.176 (c)]. For good penetration, the root faces must be melted. Simultaneously, the weld pool also must be controlled, preferably, by using a backing strip.

![Fig.3.17 Butt weld details](image)

**Fillet welds:**

Owing to their economy, ease of fabrication and adaptability, fillet welds are widely used. They require less precision in the fitting up because the plates being joined can be moved about more than the Butt welds. Another advantage of fillet welds is that special preparation of edges, as required by Butt welds, is not required. In a fillet weld the stress condition in the weld is quite different from that of the connected parts. A typical fillet weld is shown in Fig.3.18
Fig. 3.18 Typical fillet weld

The root of the weld is the point where the faces of the metallic members meet. The theoretical throat of a weld is the shortest distance from the root to the hypotenuse of the triangle. The throat area equals the theoretical throat distance times the length of the weld.

The concave shape of free surface provides a smoother transition between the connected parts and hence causes less stress concentration than a convex surface. But it is more vulnerable to shrinkage and cracking than the convex surface and has a much reduced throat area to transfer stresses. On the other hand, convex shapes provide extra weld metal or reinforcement for the throat. For statically loaded structures, a slightly convex shape is preferable, while for fatigue-prone structures, concave surface is desirable.

Large welds are invariably made up of a number of layers or passes. For reasons of economy, it is desirable to choose weld sizes that can be made in a single pass. Large welds scan be made in a single pass by an automatic machine, though manually, 8 mm fillet is the largest single-pass layer.
### 3.3.4 Weld symbols

The information concerning type, size, position, welding process etc. of the welds in welded joints is conveyed by standard symbols in drawings. The symbolic representation includes elementary symbols along with a) supplementary symbol, b) a means of showing dimensions, or c) some complementary indications. IS: 813 “Scheme of Symbols for Welding” gives all the details of weld representation in drawings.

Elementary symbols represent the various categories of the weld and look similar to the shape of the weld to be made. Combination of elementary symbols may also be used, when required. Elementary symbols are shown in Table 3.2.

**Table 3.2 Elementary symbols**

<table>
<thead>
<tr>
<th>Illustration (Fig)</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Butt weld between plates with raised edges" /></td>
<td><img src="symbol" alt="Butt weld between plates with raised edges" /></td>
<td>Butt weld between plates with raised edges* (the raised edges being melted down completely)</td>
</tr>
<tr>
<td><img src="image" alt="Square butt weld" /></td>
<td><img src="symbol" alt="Square butt weld" /></td>
<td>Square butt weld</td>
</tr>
<tr>
<td><img src="image" alt="Single-V butt weld" /></td>
<td><img src="symbol" alt="Single-V butt weld" /></td>
<td>Single-V butt weld</td>
</tr>
<tr>
<td><img src="image" alt="Single-bevel butt weld" /></td>
<td><img src="symbol" alt="Single-bevel butt weld" /></td>
<td>Single-bevel butt weld</td>
</tr>
<tr>
<td><img src="image" alt="Single – V butt weld with broad root face" /></td>
<td><img src="symbol" alt="Single – V butt weld with broad root face" /></td>
<td>Single – V butt weld with broad root face</td>
</tr>
<tr>
<td><img src="image" alt="Single – bevel butt weld with broad root face" /></td>
<td><img src="symbol" alt="Single – bevel butt weld with broad root face" /></td>
<td>Single – bevel butt weld with broad root face</td>
</tr>
<tr>
<td><img src="image" alt="Single – U butt weld (parallel or sloping sides)" /></td>
<td><img src="symbol" alt="Single – U butt weld (parallel or sloping sides)" /></td>
<td>Single – U butt weld (parallel or sloping sides)</td>
</tr>
</tbody>
</table>
Supplementary symbols characterise the external surface of the weld and they complete the elementary symbols. Supplementary symbols are shown in Table 3.3. The weld locations are defined by specifying, a) position of the arrow line, b) position of the reference line, and c) the position of the symbol. More details of weld representation may be obtained from IS 813.
### Table 3.3. Supplementary symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Symbol" /></td>
<td>Flat (flush) single – V butt weld</td>
</tr>
<tr>
<td><img src="image2" alt="Symbol" /></td>
<td>Convex double – V butt weld</td>
</tr>
<tr>
<td><img src="image3" alt="Symbol" /></td>
<td>Concave fillet weld</td>
</tr>
<tr>
<td><img src="image4" alt="Symbol" /></td>
<td>Flat (flush) single – V butt with flat (flush) backing run</td>
</tr>
</tbody>
</table>

### Position of symbols in drawings:

Apart from the symbols as covered earlier, the methods of representation (Fig.3.19) also include the following:

- An arrow line for each joint
- A dual reference line, consisting of two parallel lines, one continuous and the other dashed.
- A certain number of dimensions and conventional signs

The location of welds is classified on the drawings by specifying:

Position of the arrow line, position of the reference line and the position of the symbol

![Fig. 3.19 Method of representation](image5)
The position of arrow line with respect to the weld has no special significance. The arrow line joins one end of the continuous reference line such that it forms an angle with it and shall be completed by an arrowhead or a dot. The reference line is a straight line drawn parallel to the bottom edge of the drawing.

The symbol is placed either above or beneath the reference line. The symbol is placed on the continuous side of the reference line if the weld is on the other side of the joint; the symbol is placed on the dashed line side.

### 3.3.5 Design of welds

**Design of butt welds:**

For butt welds the most critical form of loading is tension applied in the transverse direction. It has been observed from tests conducted on tensile coupons containing a full penetration butt weld normal to the applied load that the welded joint had higher strength than the parent metal itself. The yield stress of the weld metal and the parent metal in the HAZ region was found to be much higher than the parent metal.

The butt weld is normally designed for direct tension or compression. However, a provision is made to protect it from shear. Design strength value is often taken the same as the parent metal strength. For design purposes, the effective area of the butt-welded connection is taken as the effective length of the weld times the throat size. Effective length of the butt weld is taken as the length of the continuous full size weld. The throat size is specified by the effective throat thickness. For a full penetration butt weld, the throat dimension is usually assumed as the thickness of the thinner part of the connection. Even though a butt weld may be reinforced on both sides to ensure full cross-sectional areas, its effect is neglected while estimating the throat dimensions. Such reinforcements often have a negative effect, producing stress concentration, especially under cyclic loads.
Unsealed butt welds of V, U, J and bevel types and incomplete penetration butt welds should not be used for highly stressed joints and joints subjected to dynamic and alternating loads. Intermittent butt welds are used to resist shear only and the effective length should not be less than four times the longitudinal space between the effective length of welds nor more than 16 times the thinner part. They are not to be used in locations subjected to dynamic or alternating stresses. Some modern codes do not allow intermittent welds in bridge structures.

For butt welding parts with unequal cross sections, say unequal width, or thickness, the dimensions of the wider or thicker part should be reduced at the butt joint to those of the smaller part. This is applicable in cases where the difference in thickness exceeds 25 % of the thickness of the thinner part or 3.0 mm, whichever is greater. The slope provided at the joint for the thicker part should not be steeper than one in five [Figs. 3.20 (a) & (b)]. In instances, where this is not practicable, the weld metal is built up at the junction equal to a thickness which is at least 25 % greater than the thinner part or equal to the dimension of the thicker part [Fig. 3.20 (c)]. Where reduction of the wider part is not possible, the ends of the weld shall be returned to ensure full throat thickness.

Stresses for butt welds are assumed same as for the parent metal with a thickness equal to the throat thickness (Cl. 10.5.7.1). For field welds, the permissible stresses in shear and tension calculated using a partial factor $\gamma_{mw}$ of 1.5. (Cl. 10.5.7.2)

**Design of fillet welds:**

Fillet welds are broadly classified into side fillets and end fillets (Fig. 3.21). When a connection with end fillet is loaded in tension, the weld develops high strength and the stress developed in the weld is equal to the value of the weld metal. But the ductility is minimal. On the other hand, when a specimen with side weld is loaded, the load axis is parallel to the weld axis. The weld is subjected to shear and the weld shear strength is limited to just about half the weld metal tensile strength. But ductility is considerably
improved. For intermediate weld positions, the value of strength and ductility show intermediate values.

![Diagram of butt welding with unequal thickness and width](image)

**Fig. 3.20 Butt welding of members with (a) & (b) unequal thickness (c) unequal width**

In many cases, it is possible to use the simplified approach of average stresses in the weld throat (Fig. 3.22). In order to apply this method, it is important to establish equilibrium with the applied load. Studies conducted on fillet welds have shown that the fillet weld shape is very important for end fillet welds. For equal leg lengths, making the direction of applied tension nearly parallel to the throat leads to a large reduction in strength. The optimum weld shape recommended is to provide shear leg $\leq 3$ times the tension leg. A small variation in the side fillet connections has negligible effect on strength. In general, fillet welds are stronger in compression than in tension.

![Diagram of fillet welds](image)

**Fig. 3.21 Fillet (a) side welds and (b) end welds**
Fig.3.22 Average stress in the weld throat

A simple approach to design is to assume uniform fillet weld strength in all directions and to specify a certain throat stress value. The average throat thickness is obtained by dividing the applied loads summed up in vectorial form per unit length by the throat size.

This method is limited in usage to cases of pure shear, tension or compression (Fig.3.23). It cannot be used in cases where the load vector direction varies around weld group. For the simple method, the stress is taken as the vector sum of the force components acting in the weld divided by the throat area.

Fig.3.23 (a) connections with simple weld design, (b) connections with Direction-dependent weld design

Stresses Due to Individual forces - When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by:
\[ f_a \text{ or } q = \frac{P}{t_t l_w} \]

Where

- \( f_a \) = calculated normal stress due to axial force in N/mm\(^2\)
- \( q \) = shear stress in N/mm\(^2\)
- \( P \) = force transmitted (axial force \( N \) or the shear force \( Q \))
- \( t_t \) = effective throat thickness of weld in mm
- \( l_w \) = effective length of weld in mm

**Fig. 3.24 End fillet weld normal to direction of force**

The design strength of a fillet weld, \( f_{wd} \), shall be based on its throat area (Cl.10.5.7).

\[ f_{wd} = f_{wn} / \gamma_{mw} \text{ in which } f_{wn} = \frac{f_u}{\sqrt{3}} \]

Where \( f_u \) = smaller of the ultimate stress of the weld and the parent metal and
\( \gamma_{mw} \) = partial safety factor (=1.25 for shop welds and = 1.5 for field welds)

The design strength shall be reduced appropriately for long joints as prescribed in the code.

The size of a normal fillet should be taken as the minimum leg size (Fig. 3.25)
For a deep penetration weld, the depth of penetration should be a minimum of 2.4 mm. Then the size of the weld is minimum leg length plus 2.4 mm. The size of a fillet weld should not be less than 3 mm or more than the thickness of the thinner part joined. Minimum size requirement of fillet welds is given below in Table 3.4 (Cl.10.5.2.3). Effective throat thickness should not be less than 3 mm and should not exceed 0.7 t and 1.0 t under special circumstances, where ‘t’ is the thickness of thinner part.

**Table 3.4 Minimum size of first run or of a single run fillet weld**

<table>
<thead>
<tr>
<th>Thickness of thicker part (mm)</th>
<th>Minimum size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>t ≤ 10</td>
<td>3</td>
</tr>
<tr>
<td>10 &lt; t ≤ 20</td>
<td>5</td>
</tr>
<tr>
<td>20 &lt; t ≤ 32</td>
<td>6</td>
</tr>
<tr>
<td>32 &lt; t ≤ 50</td>
<td>8 (First run) 10 (Minimum size of fillet)</td>
</tr>
</tbody>
</table>

For stress calculations, the effective throat thickness should be taken as K times fillet size, where K is a constant. Values of K for different angles between tension fusion faces are given in Table 3.5 (Cl.10.5.3.2). Fillet welds are normally used for connecting parts whose fusion faces form angles between 60° and 120°. The actual length is taken as the length having the effective length plus twice the weld size. Minimum effective length should not be less than four times the weld size. When a fillet weld is provided to square edge of a part, the weld size should be at least 1.5 mm less than the edge.
thickness [Fig. 3.26 (a)]. For the rounded toe of a rolled section, the weld size should not exceed 3/4 thickness of the section at the toe [Fig. 3.26 (b)] (Cl.10.5.8.1).

![Fig.3.26 (a) Fillet welds on square edge of plate, (b) Fillet Welds on round toe of rolled section](image)

### Table 3.5. Value of K for different angles between fusion faces

<table>
<thead>
<tr>
<th>Angle between fusion faces</th>
<th>60° - 90°</th>
<th>91°-100°</th>
<th>101°-106°</th>
<th>107°-113°</th>
<th>114°-120°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant K</td>
<td>0.70</td>
<td>0.65</td>
<td>0.60</td>
<td>0.55</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Intermittent fillet welds may be provided where the strength required is less than that can be developed by a continuous fillet weld of the smallest allowable size for the parts joined. The length of intermediate welds should not be less than 4 times the weld size with a minimum of 40 mm. The clear spacing between the effective lengths of the intermittent welds should be less than or equal to 12 times the thickness of the thinner member in compression and 16 times in tension; in no case the length should exceed 20 cm. Chain intermittent welding is better than staggered intermittent welding. Intermittent fillet welds are not used in main members exposed to weather. For lap joints, the overlap should not be less than five times the thickness of the thinner part.

For fillet welds to be used in slots and holes, the dimension of the slot or hole should comply with the following limits:

a) The width or diameter should not be less than three times the thickness or 25 mm whichever is greater

b) Corners at the enclosed ends or slots should be rounded with a radius not less than 1.5 times the thickness or 12 mm whichever is greater, and
c) The distance between the edge of the part and the edge of the slot or hole, or between adjacent slots or holes, should be not less than twice the thickness and not less than 25 mm for the holes.

![End returns for side welds](image)

**Fig. 3.27 End returns for side welds**

The effective area of a plug weld is assumed as the nominal area of the whole in the plane of the *faying* surface. Plug welds are not designed to carry stresses. If two or more of the general types of weld (butt, fillet, plug or slots) are combined in a single joint, the effective capacity of each has to be calculated separately with reference to the axis of the group to determine the capacity of the welds.

The high stress concentration at ends of welds is minimised by providing welds around the ends as shown in Fig. 3.27. These are called *end returns*. Most designers neglect end returns in the effective length calculation of the weld. End returns are invariably provided for welded joints that are subject to eccentricity, impact or stress reversals. The end returns are provided for a distance not less than twice the size of the weld.

**Design of plug and slot welds:**

In certain instances, the lengths available for the normal longitudinal fillet welds may not be sufficient to resist the loads. In such a situation, the required strength may be built up by welding along the back of the channel at the edge of the plate if sufficient space is available. This is shown in Fig. 3.28 (a). Another way of developing the required strength is by providing slot or plug welds. Slot and plug welds [Fig. 3.28 (b)] are generally used along with fillet welds in lap joints. On certain occasions, plug welds are used to fill the holes that are temporarily made for erection bolts for beam and...
column connections. However, their strength may not be considered in the overall strength of the joint.

The limitations given in specifications for the maximum sizes of plug and slot welds are necessary to avoid large shrinkage, which might be caused around these welds when they exceed the specified sizes. The strength of a plug or slot weld is calculated by considering the allowable stress and its nominal area in the shearing plane. This area is usually referred to as the faying surface and is equal to the area of contact at the base of the slot or plug. The length of the slot weld can be obtained from the following relationship:

\[
L = \frac{\text{Load}}{(\text{Width}) \text{ allowable stress}}
\]

(3.15)

\begin{figure}
\centering
\includegraphics[width=\textwidth]{Fig_3.28_Slot_and_plug_welds.png}
\caption{Fig. 3.28 Slot and plug welds}
\end{figure}
3.4 Analysis of bolt groups

In general, any group of bolts resisting a moment can be classified into either of two cases depending on whether the moment is acting in the shear plane or in a plane perpendicular to it. Both cases are described in this section.

3.4.1 Combined shear and moment in plane

Consider an eccentric connection carrying a load of P as shown in Fig. 3.29. The basic assumptions in the analysis are (1) deformations of plate elements are negligible, (2) the total shear is assumed to be shared equally by all bolts and (3) the equivalent moment at the geometric centre (point O in Fig. 3.29) of the bolt group, causes shear in any bolt proportional to the distance of the bolt from the point O acting perpendicular to the line joining the bolt centre to point O (radius vector).

Resolving the applied force P into its components $P_x$ and $P_y$ in x and y-directions respectively and denoting the corresponding force on any bolt $i$ to these shear components by $R_{xi}$ and $R_{yi}$ and applying the equilibrium conditions we get the following:

$$R_{xi} = P_x/n \quad \text{and} \quad R_{yi} = P_y/n$$

(3.16)

Where $n$ is the total number of bolts in the bolt group and $R_{xi}$ and $R_{yi}$ act in directions opposite to $P_x$ and $P_y$ respectively.

Fig. 29 Bolt group eccentrically loaded in shear
The moment of force $P$ about the centre of the bolt group (point O) is given by

$$M = P_x y' + P_y x' \quad (3.17)$$

where $x'$ and $y'$ denote the coordinates of the point of application of the force $P$ with respect to the point $O$. The force in bolt $i$, denoted by $R_{mi}$, due to the moment $M$ is proportional to its distance from point $O$, $r_i$, and perpendicular to

$$R_{mi} = k r_i \quad (3.18)$$

Where, $k$ is the constant of proportionality. The moment of $R_{mi}$ about point $O$ is

$$M_i = k r_i^2 \quad (3.19)$$

Therefore the total moment of resistance of the bolt group is given by

$$MR = \sum k r_i^2 = k \sum r_i^2 \quad (3.20)$$

For moment equilibrium, the moment of resistance should equal the applied moment and so $k$ can be obtained as $k = M/\sum r_i^2$, which gives $R_{mi}$ as

$$R_{mi} = M r_i/\sum r_i^2 \quad (3.21)$$

Total shear force in the bolt $R_i$ is the vector sum of $R_{xi}$, $R_{yi}$ and $R_{mi}$

$$R_i = \sqrt{\left(R_{xi} + R_{mi} \cos \theta_i\right)^2 + \left(R_{yi} + R_{mi} \sin \theta_i\right)^2} \quad (3.22)$$

After substituting for $R_{xi}$, $R_{yi}$ and $R_{mi}$ from equations (3.16) and (3.21) in (3.22), using $\cos \theta_i = x_i/r_i$ and $\sin \theta_i = y_i/r_i$ and simplifying we get

$$R_i = \sqrt{\left(\frac{P_x}{n} + \frac{M_{yi}}{\sum (x_i^2 + y_i^2)}\right)^2 + \left(\frac{P_y}{n} + \frac{M_{xi}}{\sum (x_i^2 + y_i^2)}\right)^2} \quad (3.23)$$
The \( x_i \) and \( y_i \) co-ordinates should reflect the positive and negative values of the bolt location as appropriate.

### 3.4.2 Combined shear and moment out-of-plane

In the connection shown in Fig. 3.30, the bolts are subjected to combined shear and tension. The neutral axis may be assumed to be at a distance of one-sixth of the depth \( d \) above the bottom flange of the beam so as to account for the greater area in the compressed portions of the connection per unit depth.

The nominal tensile force in the bolts can be calculated assuming it to be proportional to the distance of the bolt from the neutral axis \( l_i \) in Fig. 3.30. If there exists a hard spot on the compressive load path such as a column web stiffener on the other side of the lower beam flange, the compressive force may be assumed to be acting at the mid-depth of the hard spot as shown in Fig. 3.30c. In such a case, the nominal tensile force in the bolts can be calculated in proportion to the distance of the bolt from the compressive force (\( l_i = L_i \)).

\[
T_i = k l_i \quad \text{where} \quad k = \text{constant} \quad (3.24)
\]

\[
M = \Sigma T_i L_i = k \Sigma l_i L_i \quad (3.25)
\]

\[
T_i = M l_i / \Sigma l_i L_i \quad (3.26)
\]

**Fig. 30 Bolt group resisting out-of-plane moment**
In the case of extended end plate connections, the top portion of the plate behaves as a T-stub symmetric about the tension flange. For calculating the bolt tensions in the rows immediately above and below the tension flange, \( l_i \) can be taken as the distance of the tension flange from the neutral axis to the line of action of the compressive force, as the case may be. If the end plate is thin, prying tension is likely to arise in addition to the nominal bolt tension calculated as above.

The shear can be assumed to share equally by all the bolts in the connection. Therefore, the top bolts will have to be checked for combined shear and tension as explained before.
3.5 Analysis of weld group

3.5.1 Eccentric welded connections

In some cases, eccentric loads may be applied to fillet welds causing either shear and torsion or shear and bending in the welds. Examples of such loading are shown in Fig. These two common cases are treated in this section.

Shear and torsion:

Considering the welded bracket shown in Fig. 3.31 (a), an assumption is made to the effect that the parts being joined are completely rigid and hence all the deformations occur in the weld. As seen from the figure, the weld is subjected to a combination of shear and torsion. The force caused by torsion is determined using the formula

\[ F = \frac{T \cdot s}{J} = \frac{(\text{Moment} / \text{Polar moment of inertia})}{3.27} \]

Where, \( T \) is the tension, \( s \) is the distance from the centre of gravity of the weld to the point under consideration, and \( J \) is the polar moment of inertia of the weld. For convenience, the force can be decomposed into its vertical and horizontal components:

\[ F_h = \frac{T \cdot v}{J} \quad \text{and} \quad f_v = \frac{T \cdot h}{J} \]

Where, \( v \) and \( h \) denote the vertical and horizontal components of the distance \( s \). The stress due to shear force is calculated by the following expression

\[ \tau = \frac{R}{L} \]

Where, \( \tau \) is the shearing stress and \( R \) is the reaction and \( L \) is the total length of the weld. While designing a weld subjected to combined shear and torsion, it is a usual practice to assume a unit size weld and compute the stresses on a weld of unit length. From the maximum weld force per unit length the required size of the fillet weld can be calculated.
**Shear and bending:**

Welds, which are subjected to combined shear and bending, are shown in Fig. 3.31 (b). It is a common practice to treat the variation of shear stress as uniform if the welds are short. But, if the bending stress is calculated by the flexure formula, the shear stress variation for vertical welds will be parabolic with a maximum value equal to 1.5 times the average value. These bending and shear stress variations are shown in Fig. 3.32.

It may be observed here that the locations of maximum bending and shearing stresses are not the same. Hence, for design purposes the stresses need not be combined at a point. It is generally satisfactory if the weld is designed to withstand the maximum bending stress and the maximum shear stress separately. If the welds used are as shown in Fig. 3.33 it can be safely assumed that the web welds would carry all the of the shear and the flange welds all of the moment.

![Fig. 3.31 (a) Welds subjected to shear and torsion, (b) Welds subjected to shear and bending](image-url)
When fillet welds are subjected to a combination of normal and shear stress, the equivalent stress $f_e$ shall satisfy the following

$$f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3} \gamma_{mw}}$$  \hspace{1cm} (3.30)

Where, $f_a$ is the normal stresses, compression or tension, due to axial force or bending moment and $q$ is the shear stress due to shear force or tension.

However, check for the combination of stresses need not be done:

i) for side fillet welds joining cover plates and flange plates, and

ii) for fillet welds where sum of normal and shear stresses does not exceed $f_{wd}$.

Similarly, the check for the combination of stresses in butt welds need not be done if:

i) butt welds are axially loaded, and
ii) in single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 percent of the design shear stress.

**Combined bearing, bending and shear:**

Where bearing stress, $f_{br}$ is combined with bending (tensile or compressive) and shear stresses under the most unfavorable conditions of loading, the equivalent stress, $f_e$, shall be obtained from the following formula

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2} \tag{3.31}$$

Where, $f_e$ is the equivalent stress; $f_b = \text{calculated stress due to bending in N/mm}^2$; $f_{br} = \text{calculated stress due to bearing in N/mm}^2$ and $q = \text{shear stress in N/mm}^2$. However, the equivalent stress so calculated shall not exceed the values allowed for the parent metal.
3.6 Beam and column splices

It is often required to join structural members along their length due to the available length of sections being limited and also due to transportation and erection constraints. Such joints are called splices. Splices have to be designed so as to transmit all the member forces and at the same time provide sufficient stiffness and ease in erection. Splices are usually located away from critical sections. In members subjected to instability, the splice should be preferably located near the point of lateral restraint else the splice may have to be designed for additional forces arising due to instability effects. In all cases, the requirements of the code should be satisfied.

3.6.1 Beam splices

Beam splices typically resist large bending moments and shear forces. If a rolled section beam splice is located away from the point of maximum moment, it is usually assumed that the flange splice carries all the moment and the web splice carries the shear. Such an assumption simplifies the splice design considerably. Where such simplification is not possible, as in the case of a plate girder, the total moment is divided between the flange and the web in accordance with the stress distribution. The web connection is then designed to resist its share of moment and shear (CI.G.3).

A typical bolted splice plate connection is shown in Fig. 3.34 (a). To avoid deformation associated with slip before bearing in bearing bolts, HSFG bolts should be used. Usually double-splice plates are more economical because they require less number of bolts. However, for rolled steel sections with flange widths less than 200 mm, single splice plates may be used in the flange. End-plate
connections may also be used as beam splices [Fig. 3.34(b)] although they are more flexible.

**Fig. 3.34 Bolted beam splice: (a) Conventional splice (b) End-plate splice**

### 3.6.2 Column splice

Column splices can be of two types. In the bearing type, the faces of the two columns are prepared to butt against each other and thus transmit the load by physical bearing. In such cases only a nominal connection needs to be provided to keep the columns aligned. However, this type of splice cannot be used if the column sections are not prepared by grinding, if the columns are of different sizes, if the column carries moment or if continuity is required. In such cases, HSFG bolts will have to be used and the cost of splice increases. When connecting columns of different sizes, end plates or packing plates should be provided similar to the beam splice shown in Fig. 3.34(b) (CI.G.3).
3.7 Summary

Different types of bolted connections were described and classified. The bearing and friction grip bolts were introduced and their installation procedures described. The force transfer mechanisms were explained and the failure modes and corresponding strength calculations were given. This will help in the design of simple bolted connections as in the worked examples. Simple analysis methods for bolt groups resisting in-plane and out-of-plane moments were described. Beam and column splices as well as various types of beam-to-column connections were described and their general behaviour as well as points to be kept in mind during their design was explained.
3.8 References


2) IS 812-1957 Glossary of terms relating to welding and cutting of metals

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6) IS 1395-1982 Low and medium alloy steel covered electrodes for manual metal arc welding (third revision)

7) IS 3640-1982 Specification for Hexagon fit bolts (first revision)

8) IS 3757-1985 Specification for high strength structural bolts (second revision)

9) IS 4000-1992 Code of practice for high strength bolts in steel structures (first revision)

10) IS 6610-1972 Specifications for heavy washers for steel structures
11) IS 6623-1985 Specifications for high strength structural nuts (first revision)

12) IS 6639-1972 Specifications for hexagonal bolts for steel structures

13) Teaching resources for structural Steel Design (Volume 1 to 3), INSDAG publication, Calcutta2000.


Design Example 1:

Design a Lap joint between plates 100? 8 so as to transmit a factored load of 100 kN using black bolts of 12mm diameter and grade 4.6. The plates are made of steel of grade ST-42-S.

Solution:

1) Strength Calculations:

Nominal diameter of bolt d = 12 mm

For grade 4.6 bolt, \( f_u = 40 \text{ kgf/mm}^2 = 392.4 \text{ MPa}, \quad mb = 1.25 \)

Assuming threads in the shear plane, \( n_n = 1, \quad n_s = 0 \)

Shear Area of one bolt \( A_{nb} = 0.8 A_{sb} = 0.8 \times 113.1 = 90.5 \text{ mm}^2 \)

Design shear strength per bolt \( V_{nsb} = f_u A_{nb} / \gamma_{mb} \sqrt{3} = 16.4 \text{ kN} \) (Cl. 10.3.2)

Design bearing strength per bolt \( V_{npb} = 2.5 d t f_u \)

\[ = 2.5 \times 12 \times 8 \times 392.4 \times 10^{-3} = 75.2 \text{ kN} \] (Cl. 10.3.3)

Therefore, bolt value = 16.4 kN

No. of bolts required = 100 / 16.4 = 6.1 say 7 bolts

2) Detailing:

Minimum pitch = 2.5 d = 30 mm (Cl. 10.2.1)

Minimum edge distance = 1.4 D = 16.8 mm say 20 mm (Cl. 10.2.3)

Provide 8 bolts as shown in Fig. E1.

![Fig. E1](image_url)
**Design Example 2:**

Design a hanger joint along with an end plate to carry a downward load of $2T = 330$ kN. Use end plate size $240 \text{ mm} \times 160 \text{ mm}$ and appropriate thickness and 2 nos of M25 Gr.8.8 HSFG bolts ($f_o = 565 \text{ MPa}$).

**Solution**

Assume 10mm fillet weld between the hanger plate and the end plate.

Distance from center line of bolt to toe of fillet weld $l_v = 60 \text{ mm}$

1) For minimum thickness design, $M = T l_v / 2 = 165 \times 60 / 2 = 4950 \text{ N-m}$

$$t_{\text{min}} = \sqrt{ \frac{1.15 \times 4 \times 4950 \times 10^3}{236 \times 160} } = 24.56 \text{ say } 25 \text{ mm}$$

$$M_p = Z_p f_y = \frac{W_t^2 f_y}{4} \gamma m_0$$

$$t = \sqrt{\frac{4M_p \gamma m_0}{f_y W_t}}$$

2) Check for prying forces distance’ $l_e$’ from center line of bolt to prying force is the minimum of edge distance or $1.1t$

$$\sqrt{(\beta_p / f_y)} = 1.1 \times 25 \sqrt{(2 \times 565 / 236)} = 60 \text{ mm} \quad \text{(Cl. 10.4.7)}$$

$$l_e = 40 \text{ mm}$$

prying force $Q = M / l_e = 4950 / 40 = 123.75 \text{ kN}$

bolt load = $165 + 123.75 = 288.75 \text{ kN} \quad \text{(Cl. 10.4.5)}$

tension capacity of 25 mm dia HSFG bolt = $0.9F_{u}A_{nb}/\gamma m_b = 222 \text{ kN} << 288.75$

Load carrying Capacity $<<$ Required load Capacity

---

*Fig E2*
In order to reduce the load on bolt to a value less than the bolt capacity, a thicker end plate will have to be used.

Allowable prying force \( Q = 222 - 165 = 57 \text{ kN} \)

Trying a 36 mm thick end plate gives \( l_e = 40 \text{ mm} \) as before

Moment at toe of weld = \( T l_v - Q l_e = 165 \times 60 - 57 \times 40 = 7620 \text{ N-m} \)

Moment capacity = \( (236 / 1.10) \times (160 \times 36^2/4) \times 10^3 = 11122 \text{ N-m} > 7620 \text{ OK} \)

Minimum prying force

\[
Q = \frac{l_v}{2l_e} \left[ T - \frac{\beta \gamma p_o b_i t^4}{27 l_v^2} \right] = \frac{60}{2 \times 40} \left[ 165 - \frac{2 \times 1.5 \times 0.565 \times 160 \times 36^4}{27 \times 40 \times 60^2} \right] \quad \text{(Cl. 10.4.7)}
\]

\[
= 36 \text{ kN} < 57 \text{ kN} \text{ safe!}
\]

Therefore, 36 mm end plate needs to be used to avoid significant prying action.