

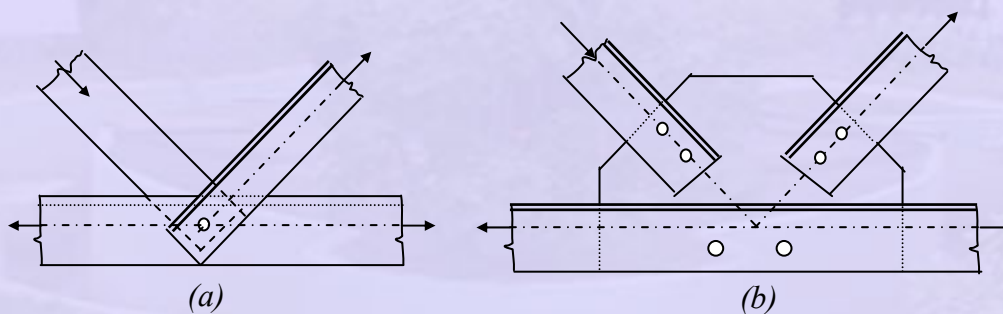
## 3.2 Bolted connections

### 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)].

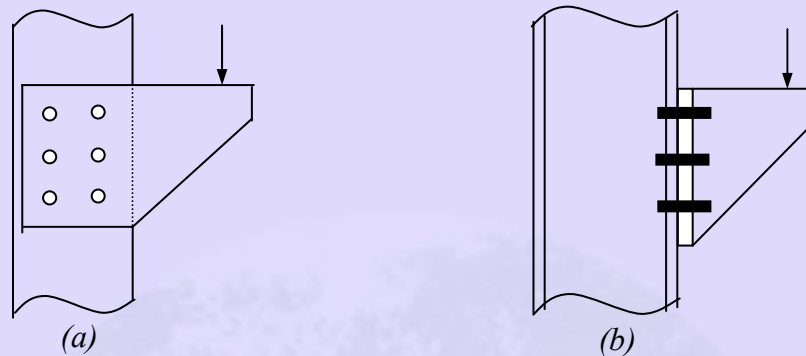
The Moment connections are more complex to analyse compared to the above two types and are shown in Fig.3.2 (a) and Fig.3.2 (b). The connection in Fig.3.2 (a) is also known as bracket connection and the resistance is only through shear in the bolts.



**Fig. 3.1 Concentric connections**

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.

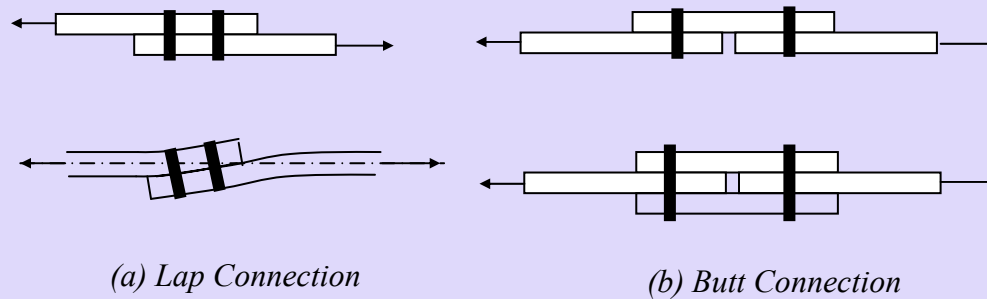
(b) *Classification based on the type of force experienced by the bolts:* The bolted connections can also be classified based on geometry and loading conditions into three types namely, shear connections, tension connections and combined shear and tension connections.



**Fig 3.2 Moment connections**

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.

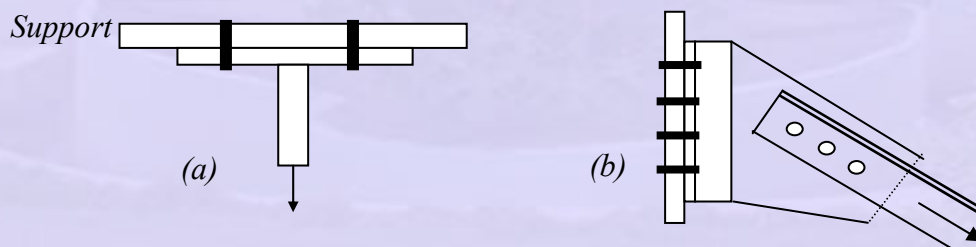
In the case of lap joint or a single cover plate butt joint, there is only one shearing plane, and so the bolts are said to be in *single shear*. In the case of double cover butt joint, there are two shearing planes and so the bolts will be in *double shear*. It should be noted that the single cover 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 case 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  $\text{kgf/mm}^2$  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  $40 \text{ kgf/mm}^2$  ( $392 \text{ Mpa}$ ) and minimum yield strength of 0.6 times 40, which is  $24 \text{ kgf/mm}^2$  ( $235 \text{ Mpa}$ ).

*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.

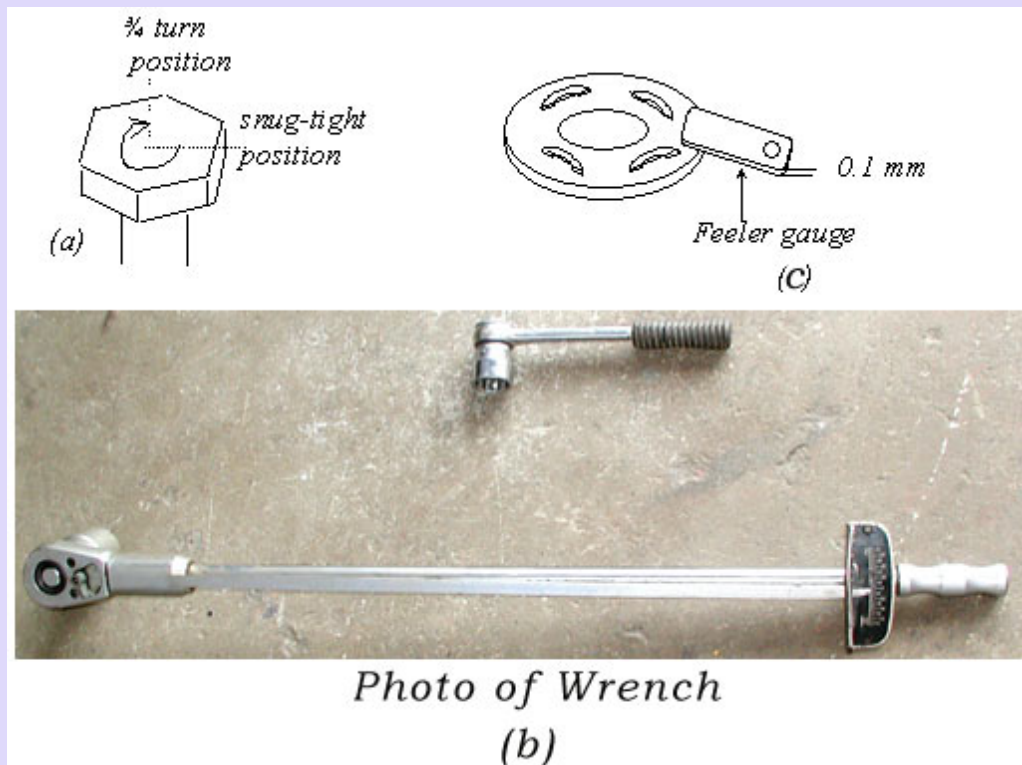
HSFG bolts are made from quenched and tempered alloy steels with grades from 8.8 to 10.9. The most common are the so-called, general grade of 8.8 and have medium carbon content, which makes them less ductile. The 10.9 grade have a much higher tensile strength, but lower ductility and the margin between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

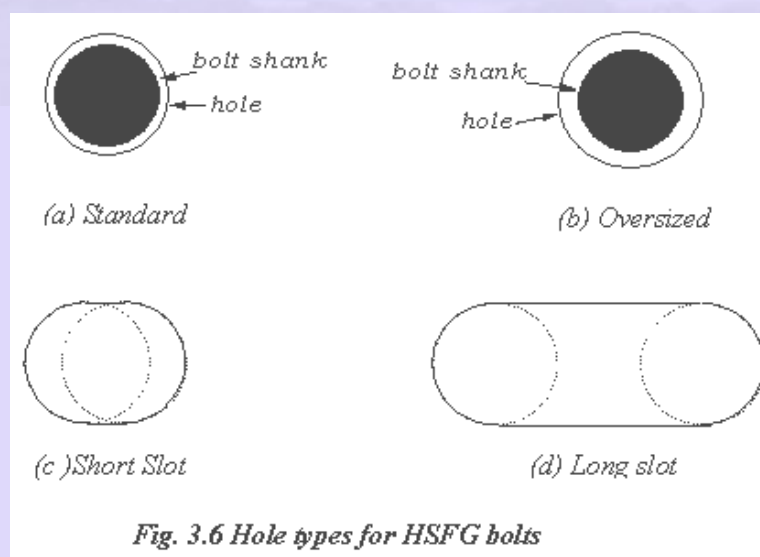
1. *Turn-of-nut tightening method:* In this method the bolts are first made **snug tight** and then turned by specific amounts (usually either half or three-fourth turns) to induce tension equal to the proof load (Fig 3.5(a)).
2. *Calibrated wrench tightening method:* In this method the bolts are tightened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





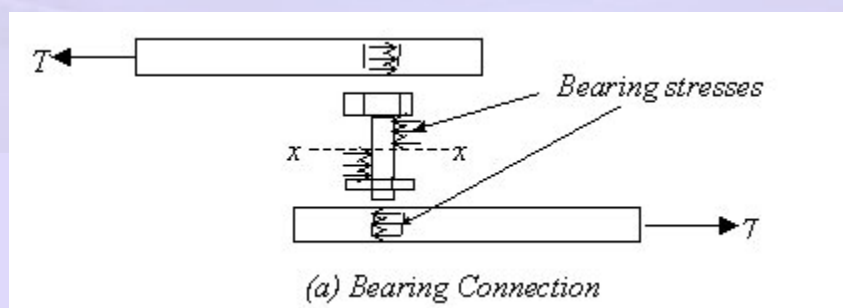
Holes must also satisfy **pitch and edge/end distance** criteria (Cl.10.2). A minimum pitch is usually specified for accommodating the spanner and to limit adverse interaction between the bearing stresses on neighbouring bolts. A maximum pitch criterion takes care of buckling of the plies under compressive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

### 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  $A_t$ , is taken as the shear area  $A_s$ . Since threads can occur in the shear plane, the area  $A_e$  for resisting shear should normally be taken as the net tensile stress area,  $A_n$ , 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

$V_{nsb}$  = nominal shear capacity of a bolt, calculated as follows:

$$\gamma_{mb} = 1.25$$

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

Where

$f_u$  = ultimate tensile strength of a bolt



$n_n$  = number of shear planes with threads intercepting the shear plane

$n_s$  = number of shear planes without threads intercepting the shear plane

$A_{sb}$  = nominal plain shank area of the bolt

$A_{nb}$  = 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,  $\gamma_{mb} = 1.25$

$V_{npb}$  = bearing strength of a bolt, calculated as

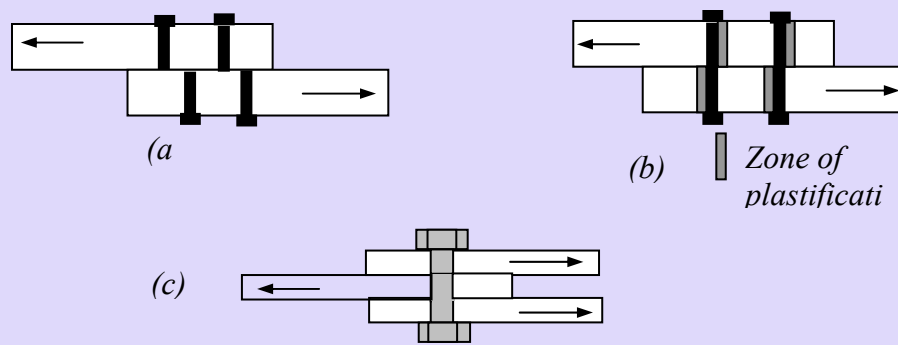
$$V_{npb} = 2.5dtf_u \quad (3.3)$$

Where

$f_u$  = smaller of the ultimate tensile stress of the bolt and the ultimate tensile stress of the plate

$d$  = nominal diameter of the bolt

$t$  = 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  $15d$  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)

$$\beta_{lj} = 1.075 - l_j / (200 d) \quad \text{but } 0.75 \leq \beta_{lj} \leq 1.0$$

Where,  $d$  = nominal diameter of the bolt

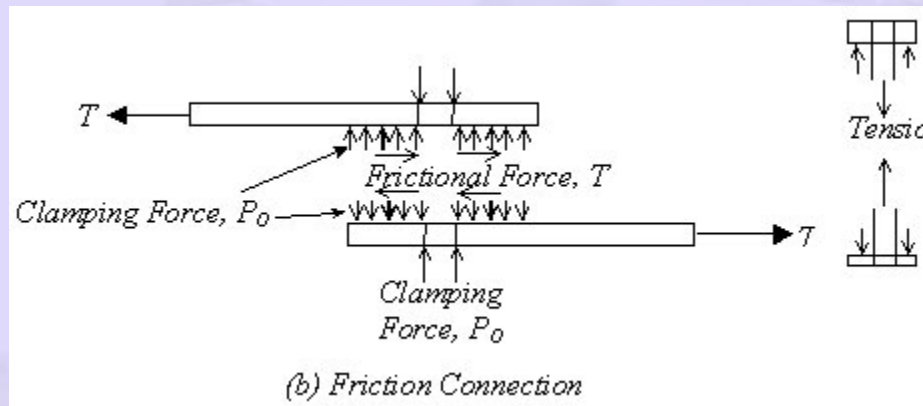
Similarly, if the **grip length** exceeds five times the nominal diameter, the strength is reduced as specified in IS 800. In multibolt connections, due to hole mismatch, all the bolts may not carry the same load. However, under ultimate load, due to high bearing ductility of the plates considerable 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.



**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:

$$V_{nsf} = \mu_f n_e K_h F_o \quad (3.4)$$

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 for fasteners in oversized and short slotted holes, and for fasteners in long slotted holes loaded perpendicular to the slot

= 0.7 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.

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

Treatment of surface	Coeff. of friction ( $\mu_f$ )
Clean mill scale	0.33
Sand blasted surface	0.48
Surfaces blasted with shot or grit and hot-dip galvanized	0.10

2. **Bearing strength:** The design for friction type bolting, in which bearing stress in the ultimate limit state is required to be limited, ( $V_{ub}$ =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:

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

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.*

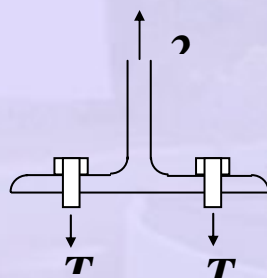
### 3.2.5 Tension connections with bearing and HSFG bolts

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

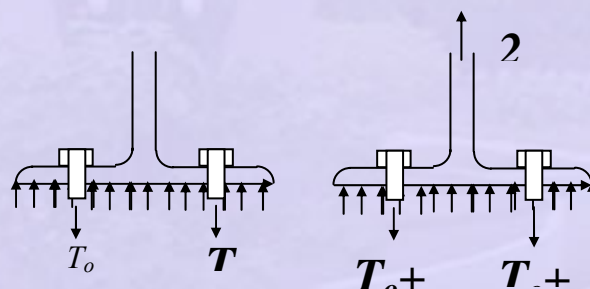
The free body diagram of the tension transfer in a bearing type of bolted connection is shown in Fig. 3.10(a). The variation of bolt tension due to externally applied tension is shown in Fig.3.10(c). It is seen that before any external tension is applied, the force in the bolt is almost zero, since the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

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)].

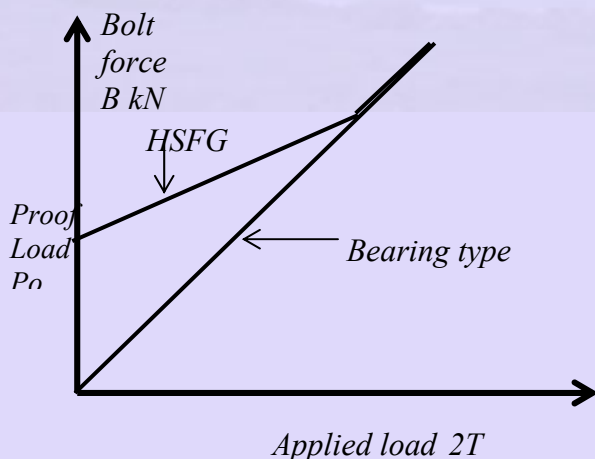
The free body diagram of an HSFG bolted connection is shown in Fig. 3.10(b). It is seen that even before any external load is applied, the force in the bolt is equal to proof load. Correspondingly there is a clamping force between the plates in contact. When the external load is applied, part of the 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.



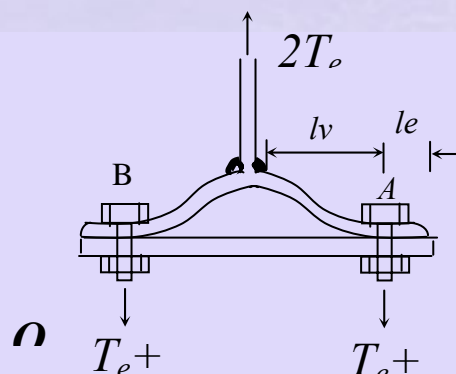
(a) Bearing type connection



(b) HSFG Connection



(c) External Tension  
versus bolt force



(d) Prying Effect



### 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).

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

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

$$l_e = 1.1 t \sqrt{\frac{\beta f_o}{f_y}} \quad (3.8)$$

Where,

$\beta = 2$  for non pre-tensioned bolt and 1 for pre-tensioned bolt

$\gamma = 1.5$

$b_e$  = effective width of flange per pair of bolts

$f_o$  = proof stress in consistent units

$t$  = 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,

$$M_A = Qn; \quad M_B = Tb - Qn \quad (3.9)$$

$$M_A = M_B = \frac{Tb}{2} = M_p \quad (3.10)$$

taking  $M_p$  as

$$M_p = \frac{f_y}{1.15} \frac{wt^2}{4} \quad (3.11)$$

the minimum thickness for the end plate can be obtained as

$$t_{\min} = \sqrt{\frac{1.15 \times 4 \times M_p}{f_y \times w}} \quad (3.12)$$

The corresponding prying force can then be obtained as  $Q = M_p/n$ . 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 (Cl.10.3.4)

$$T_b \leq T_{nb} / \gamma_{mb} \quad \gamma_{mb} = 1.25$$

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

$$T_{nb} = 0.90 f_{ub} A_n < f_{yb} A_{sb} (\gamma_{m1} / \gamma_{m0}) \quad \gamma_{m0} = 1.10 \text{ and } \gamma_{mf} = 1.25$$

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

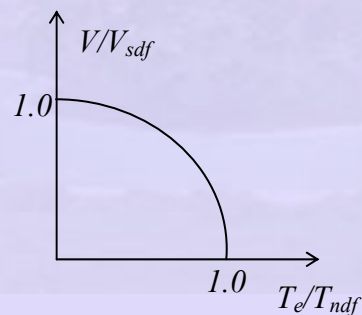
$$\left(\frac{V}{V_{sd}}\right)^2 + \left(\frac{T_e}{T_{nd}}\right)^2 \leq 1.0 \quad (3.13)$$

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**