# 9 CONNECTIONS

## 9.1 GENERAL

The structural properties of connections should conform to the global analysis method used and comply with the assumptions made in the design of members given in section 6. Connections should be designed on the basis of direct load paths and by taking account of both the stiffness and strength of the various parts of the connection. Connection design relies on the ductility of steel to relieve residual stresses, which may generally be ignored in normal cases.

The connections between members should be capable of withstanding the forces and moments to which they are subjected, within acceptable deformation limits and without invalidating the design assumptions. Clauses 6.11.1, 6.11.2 and 6.11.3 describe these assumptions for pinned, rigid and semi-rigid connections respectively.

Detailing of the connections should take into account possible dimensional variations caused by rolling tolerances and fabrication variations which may lead to a small degree of lack of fit.

Where possible in connection design, members should be arranged on the principle that their centroidal axes coincide with the action lines of the forces in the members. Where this is not practical, the effects of additional moments due to eccentricity shall be taken into account in the design of the connection. In the case of bolted connections consisting of angles and T-sections, the intersection of the setting-out lines of the bolts may be adopted instead of the intersections of the centroidal axis. The effects of fatigue need not normally be considered in the design of connections for building structures, unless there are members supporting cranes or heavy vibratory plant or machinery, see clause 2.3.3. In the latter cases, reference should be made to specialist fatigue literature or design codes, see Annex A1.10.

The practicability of carrying out fabrication and erection work shall be considered in the design and detailing of connections:

(a) **Shop fabrication**

- the requirements of the welding procedures for the materials and joint types concerned.
- accessibility for welding.
- welding constraints.
- effects of angular and length tolerances on fit-up, i.e. minimising residual stress and distortion.
- allowance for dimensional variation on rolled sections.
- inspection and testing, including considerations of access for testing welds of partially completed complex connections before such welds are inaccessible.
- surface treatment.

(b) **Site fabrication and erection**

- clearances necessary for installing and tightening fasteners.
- Allowance in terms of slotted holes, shim packing etc. for dimensional tolerances.
- need for access for field welding.
- effects of angular and length tolerances on fit-up, i.e. minimising residual stress and distortion.
- welding constraints.

## 9.2 WELDED CONNECTIONS

### 9.2.1 Through-thickness tension

Corner or T-joint welding details of rolled sections or plates involving transfer of tensile forces in the through-thickness direction should be avoided whenever possible.

If tensile stresses are transmitted through the thickness of the connected part, the connection detail, welding procedure and sequence of welding shall be designed to minimise constraints which can cause additional tensile stresses in the through-thickness direction from weld shrinkage. The through-thickness properties of the part should be such as to minimise the risk of lamellar tearing.

### 9.2.2 Types of welds

For the purposes of design, welds may be classified as:

(a) **Fillet welds**

- continuous welds.
- intermittent welds.
- plug welds on circular and elongated holes.
- slot welds.

(b) **Butt welds**

- full penetration butt welds.
- partial penetration butt welds.
- butt welds reinforced with fillet welds.

(c) **Flare groove welds**

### 9.2.3 Weldability and electrodes

Steel material shall have good weldability such that crack-free and sound structural joints can be produced without great difficulty, special or expensive requirements on the welding procedure. The welding procedure (including all parameters such as preheating or post-heating requirements, interpass temperature, AC/DC current value, arc speed etc.) shall take into account of the properties of the parent material including the carbon equivalent (CE) value, thickness and the welding consumable type. The chemical contents and mechanical properties of steel material shall conform to the requirements in section 3.1. The welding consumable shall conform to the acceptable standards given in Annex A1.4 with chemical contents matching the parent metal and mechanical properties not inferior to the parent metal.

### 9.2.4 Welded connections to unstiffened flanges

In a T-joint of a plate to an unstiffened flange of I, H or box section, a reduced effective breadth $b_e$ should be used for both the parent members and the welds.

(a) For I or H section, $b_e$ should be obtained from Figure 9.1 as

$$b_e = t_c + 2r_c + 5T_C \tag{9.1}$$

but $b_e \le t_c + 2r_c + 5\left(\dfrac{T_c^{\,2}}{t_p}\right)\left(\dfrac{p_{yc}}{p_{yp}}\right)$

where

$p_{yc}$ is the design strength of the member
$p_{yp}$ is the design strength of the plate
$r_c$ is root radius of rolled section or toe of fillet on welded section
$T_c$ is the thickness of connected flange
$t_c$ is the stem of connected structural member
$t_p$ is the thickness of the connected plate

(b) For box sections, $b_e$ should be obtained from

$$b_e = 2t_c + 5T_c \tag{9.2}$$

but $b_e \le 2t_c + 5\left(\frac{T_c{}^2}{t_p}\right)\left(\frac{p_{yc}}{p_{yp}}\right)$

If $b_e$ is less than 0.7 times the full breadth, the joint should be stiffened.

Using $b_e$ as the weld length, the capacity of the weld $P_x$ should be obtained from:

$$P_x = p_w\ a\ b_e \tag{9.3}$$

in which $p_w$ is the design strength of weld and $a$ is the throat thickness.

The strength of the connected plate at the weld, e.g. the flange of a beam, should be considered by using $b_e$ as the effective breadth.

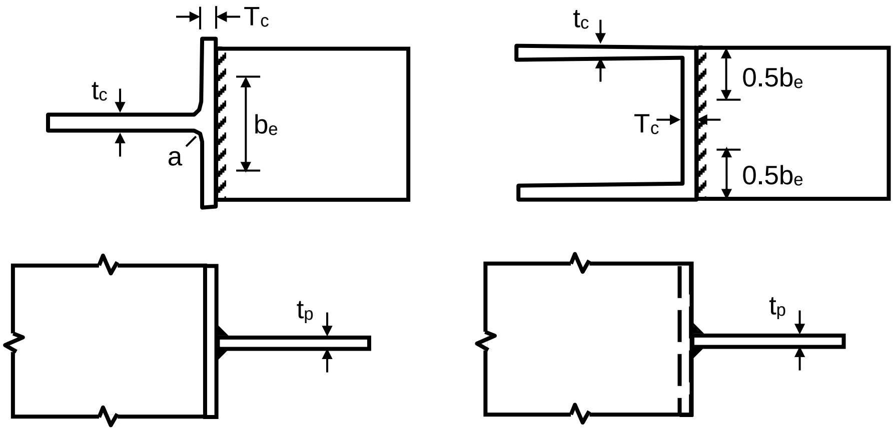



**Figure 9.1 - Effective breadth of an unstiffened Tee-joint**

## 9.2.5 Strength of welds

*9.2.5.1 Fillet welds*

9.2.5.1.1 Cross sectional geometry

The intersection angle between the fusion faces of fillet welds should be between 60° and 120°. The size of the fusion faces, referred to as leg length s, and the throat size a is defined in Figure 9.2.

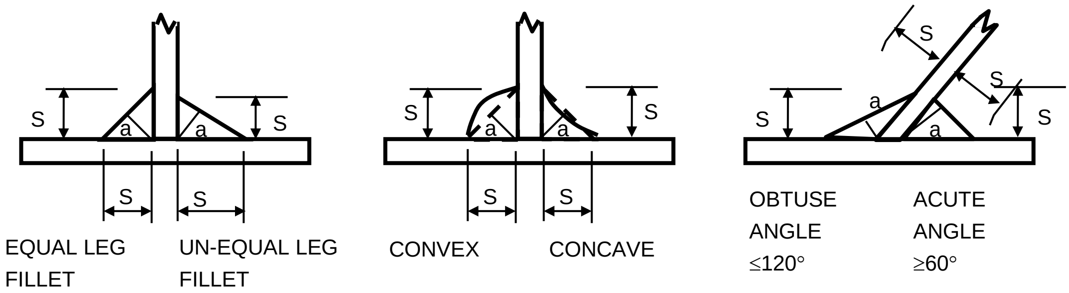



**Figure 9.2 - Geometry of fillet welds**

9.2.5.1.2 Weld size

(a) Leg length s

The minimum leg length of a fillet weld should not be less than that specified in Table 9.1 and should be able to transmit the calculated stress. For a T-joint the minimum leg length is not required to be greater than the thickness of the thinner part.

**Table 9.1 - Minimum leg length of a fillet weld**

| Thickness of the thicker part (mm) | Minimum leg length (for unequal leg weld, the smaller leg length should be considered) (mm) |
|---|---|
| up to and including 6 | 3 |
| 7 to 13 | 5 |
| 14 to 19 | 6 |
| over 19 | 8 |

For a weld along the edge of a plate:

(i) If the thickness of plate is less than 6 mm, the maximum leg length should be the thickness of the plate.

(ii) If the thickness of the plate is equal to or greater than 6 mm, the maximum leg length should be the thickness of the plate minus 2 mm.

(b) Throat size *a*

(i) As shown in Figure 9.2, the throat size *a* is defined as the perpendicular length from the point of intersection of the connected plates to the inclined weld surface.

(ii) Slight concavity or convexity of weld profile is allowed but the throat size should be determined according to Figure 9.2.

(iii) If the welds are on both sides, the throat size on each side should not be greater than thickness of the thinner part with both sides welded.

9.2.5.1.3 Effective length

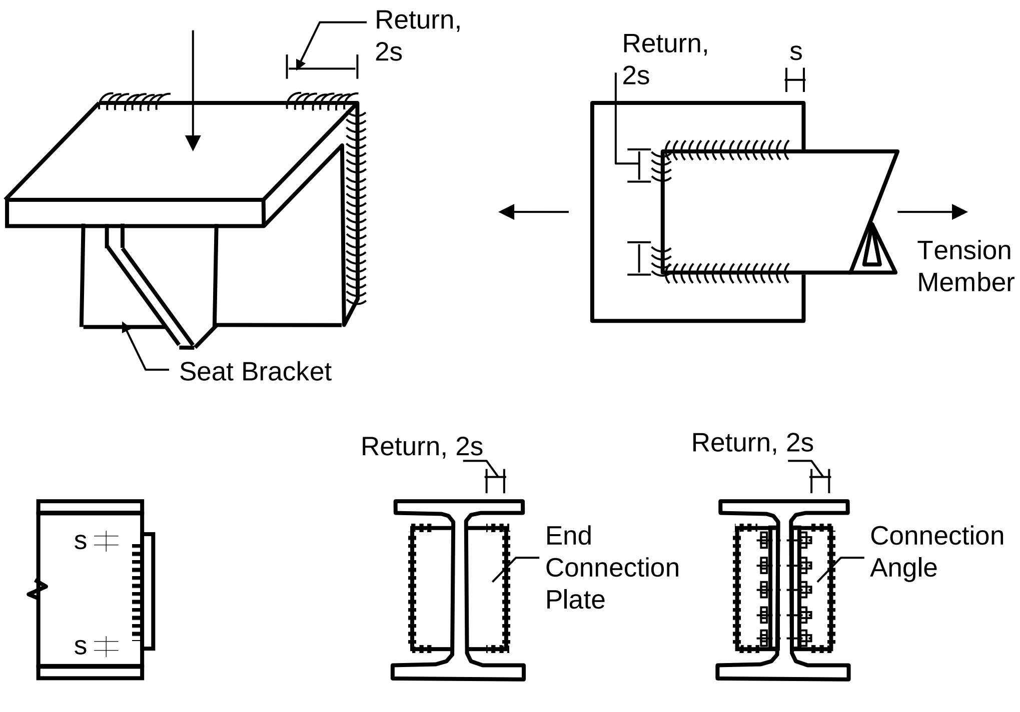
The effective length of a fillet weld should be its full length of weld less 2*s* with its end return excluded, but should not be less than 40 mm. A fillet weld with an effective length less than 4*s* or 40mm should not be used to carry load.

For a flat-bar tension member, if only longitudinal fillet welds are used for end connections, the length of each fillet weld should not be less than the width of the flat bar.

9.2.5.1.4 End returns

Fillet welds should not be terminated at the extreme ends or edges of members. They should either be returned continuously around the ends or edges for a length of not less than 2*s* or, if a return is impracticable, terminated not less than *s* from the ends or edges. The return and termination should not be included for calculation of the effective length of the weld.

In the case of fillet welds on the tension side or the tension end of a connection when subjected to significant moment from brackets, beam seats, framing angles, simple end plates, etc., the connection should be detailed with considerations of the practicality of making end returns as illustrated in Figure 9.3.




Beam End Connection

**Figure 9.3 - End return of welds**

**Table 9.2a - Design strength of fillet welds $p_w$ for BS-EN and American Welding Society (AWS) Standards**

| Steel grade | Electrode Grade | | | | | |
|---|---|---|---|---|---|---|
| | AWS standards | | | | | For other types of electrode and/or steel grades:<br>$p_w = 0.5U_e$ but $p_w \leq 0.55\ U_s$<br>where<br>$U_e$ is the minimum tensile strength of the electrode specified in the relevant product standard;<br>$U_s$ is the specified minimum tensile strength of the parent metal. |
| | ER60 | ER70 | ER80 | ER90 | ER110 | |
| | EN ISO standards | | | | | |
| | 35 | 42 | 50 | 55 | 69 | |
| S275 | 220 | (220)[a] | (220)[a] | (220)[a] | (220)[a] | |
| S355 | (220)[b] | 250 | (250)[a] | (250)[a] | (250)[a] | |
| S460 | (220)[b] | (250)[b] | 280 | (280)[a] | (280)[a] | |
| S550 | (220)[b] | (250)[b] | (280)[b] | 320 | (320)[a] | |
| S690 | (220)[b] | (250)[b] | (280)[b] | (320)[a] | 385 | |

a) Over-matching electrodes.
b) Under-matching electrodes.

**Table 9.2b - Design strength of fillet welds $p_w$ for GB or other Standards**

| Steel grade | Electrode classification | Design strength N/mm$^2$ | For other types of electrode and/or steel grades:<br>$p_w = 0.38U_e$ $U_e \geqslant U_S$<br>where<br>$U_e$ is the minimum tensile strength of the electrode specified in the relevant product standard;<br>$U_s$ is the specified minimum tensile strength of the parent metal. |
|---|---|---|---|
| Q235 | E43 | 160 | |
| Q345, Q355 | E50 | 200 | |
| Q390,Q420 | E55 | 220 | |
| Q550 | E60 | 225 | |
| Q690 | E85 | 335 | |

Note :- The ultimate strength of electrodes shall be greater than or equal to the tensile strength of the parent metal.

9.2.5.1.5 Strength of fillet welds

The strength of a fillet weld $p_w$ using electrodes or other consumables with chemical contents and mechanical properties not inferior to the parent metals and complying with the acceptable standards given in Annex A1.4 can be obtained from Tables 9.2a and 9.2b. When two different grades of parent materials are joined by fillet welds, the lower grade should be considered in the design.

Single sided fillet or partial penetration butt welds should not be used to transmit a bending moment about the longitudinal axis of the weld.

9.2.5.1.6 Capacity of fillet welds

The capacity of a fillet weld should be calculated using the throat size $a$, see clause 9.2.5.1.2(b), and the following methods:-

(a) Simplified method

Stresses should be calculated from the vector sum of forces from all directions divided by the weld throat area i.e. longitudinal and transverse forces and moment to ensure that it does not exceed the design strength of weld $p_w$.

(b) Directional method

In consideration of more accurate behaviour, the force per unit length should be resolved into a longitudinal shear $F_L$ parallel to axis of the weld and a resultant transverse force $F_T$ perpendicular to this axis. The corresponding capacities per unit length should be:-

$$P_L = p_w\ a \quad \text{(longitudinal direction)} \tag{9.4}$$

$$P_T = K\ P_L \quad \text{(transverse direction)} \tag{9.5}$$

The coefficient $K$ should be obtained from:

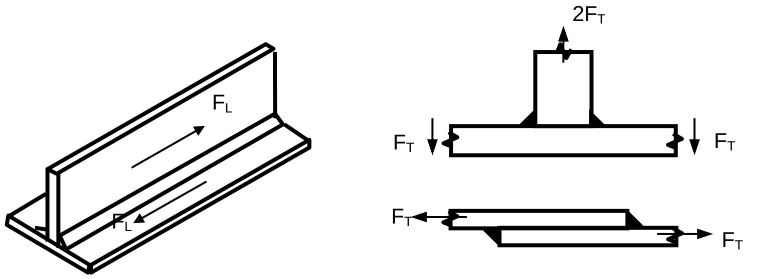
$$K = 1.25\sqrt{\frac{1.5}{1+\cos^2\theta}} \tag{9.6}$$

in which $\theta$ is the angle between the force $F_T$ and the throat of weld, see Figure 9.4.

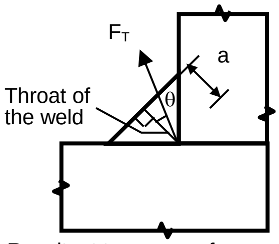
The stress resultants should satisfy the following relationship:

$$\left(\frac{F_L}{P_L}\right)^2 + \left(\frac{F_T}{P_T}\right)^2 \le 1 \tag{9.7}$$

in which $P_L$ and $P_T$ are the respective permissible capacities per unit length of weld in the longitudinal and the transverse directions.




a) Welds subject to longitudinal shear    b) Welds subject to transverse force




c) Resultant transverse force on weld

**Figure 9.4 - Fillet welds – directional method**

9.2.5.1.7 Intermittent fillet welds

Intermittent fillet welds may be used to transmit forces across a joint or faying surface.

(a) The longitudinal spacing along any one edge of the element between effective lengths of weld should not exceed the least of

(i) 300 mm;

(ii) 16 times the thickness of the thinner part of compression elements; and

(iii) 24 times the thickness of the thinner part of tension elements.

(b) A continuous fillet weld with a length of 3/4 of the width of narrower plate should be provided on each side of plate at both ends.

(c) In staggered intermittent fillet welds, the clear unconnected gap should be measured between the ends of welds on opposing sides.

(d) Intermittent fillet welds should not be used in corrosive conditions or to resist fatigue loads.

9.2.5.1.8 Plug welds

(a) Plug welds are welds that fill up circular or elongated holes. Plug welds should not be used to resist tension. They may be used in the following situations:

- to transmit shear on lap joints;
- to prevent buckling or separation of lapped parts; or
- to interconnect the components of built-up members.

(b) The diameter of a circular hole, or width of an elongated hole, should be at least 8 mm larger than the thickness of the element containing the hole.

(c) The effective shear area of a plug weld can be taken as the nominal area of the hole on the plane of the faying surface.

(d) The thickness of a plug weld in an element up to 16 mm thickness should be equal to the thickness of the element. In material over 16 mm thick, thickness of a plug weld should be at least half of the thickness of the element but not less than 16 mm.

(e) The minimum centre to centre spacing of plug welds should be 4 times their diameter but not greater than the distance necessary to prevent local buckling.

9.2.5.1.9 Slot welds

(a) Slot welds are fillet welds in a slot. Slot welds may be used to transmit shear or to prevent buckling or separation of lapped parts. It should not be used to resist tension.

(b) Slot welds should satisfy the requirements of plug welds in clause 9.2.5.1.8.

(c) The length of the slot should not exceed 10 times the thickness of the weld.

(d) The width of the slot should not be less than thickness of the element containing it plus 8 mm.

(e) The ends of the slot should be semi-circular, except for those ends which extend to the edge of the element concerned.

9.2.5.1.10 Lap joints

The minimum lap should be *5t* or 25 mm whichever is the greater, where *t* is the thickness of the thinner part joined.

For lap joints longer than 100*s*, a reduction factor $\beta_{LW}$ should be taken to allow for the effects of non-uniform distribution of stress along its length.

$$L_{eff} = \beta_{LW} \, L_j \tag{9.8}$$

where

$$\beta_{LW} = 1.2 - 0.002\left(\frac{a}{L_j}\right) \leq 1.0 \tag{9.9}$$

$a$ = fillet weld leg size

$L_j$ = actual length of weld

$L_{eff}$ = effective length

A single fillet weld should not be used for lap joints unless the parts are restrained to prevent opening of the joint and eccentric moments.

*9.2.5.2 Penetration welds*

9.2.5.2.1 Full penetration welds

The design strength of a full penetration weld, or a butt weld, can be taken as equal to the parent metal if all the following conditions are satisfied:

(a) A full penetration weld should have complete penetration and fusion of weld with parent metal throughout the thickness of the joint.

(b) Welding consumables should possess mechanical properties not inferior to those specified for the parent metal.

(c) The backing material should be not inferior to parent material.

The welding of single V, U, J, bevel or square butt welds should follow a proper procedure by depositing a sealing run of weld metal on the back of the joint. When welding is on one side only, facilitating this process by the use of temporary or permanent backing material or by using an approved specialist method without the need of using backing material is acceptable.

Precautionary measures against residual stresses should be taken to mitigate the adverse effect when welding plates with thickness greater than 40mm or in constrained or congested locations.

9.2.5.2.2 Partial penetration welds

(a) Throat size of partial penetration welds

The throat size of a single-sided partial penetration weld, see Figures 9.5(a) and 9.5(c), or the size of each throat of a double-sided partial penetration weld, see Figures 9.5(b) and 9.5(d), should be the minimum depth of penetration from that side of the weld.

The minimum throat size of a longitudinal partial penetration weld should be $2\sqrt{t}$ where $t$ is the thickness of the thinner part joined.

(b) Capacity of partial penetration welds

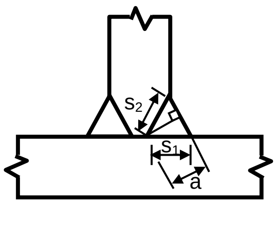
A single-side partial penetration butt weld should not be used to resist a bending moment about its longitudinal axis to avoid possible tension at the root of the weld, nor to transmit forces perpendicular to the longitudinal axis that would significantly produce such a bending moment, see Figure 9.6.

The capacity of a partial penetration weld in a butt joint, see Figures 9.5(a) and 9.5(b), or a corner joint, see Figure 9.5(c), should be taken as sufficient if the stress does not exceed the relevant strength of the parent material throughout the weld.

The capacity of a T-butt joint comprising a pair of partial penetration butt welds with additional fillets, see Figure 9.5(d), should be determined by treating the weld as:

- a butt weld, if $a > 0.7s$;
- a fillet weld, see 9.2.5.1.6, if $a \leq 0.7s$;

in which $a$ is the effective throat size and $s$ is the length of the smaller fusion face, see Figure 9.5(d).




a) Butt joint with single-sided partial penetration butt weld

b) Butt joint with double-sided partial penetration butt weld

c) Corner joint with single-sided partial penetration butt weld

d) Tee-butt joint with a pair of partial penetration butt welds with external fillets

**Figure 9.5 - Partial penetration butt welds**

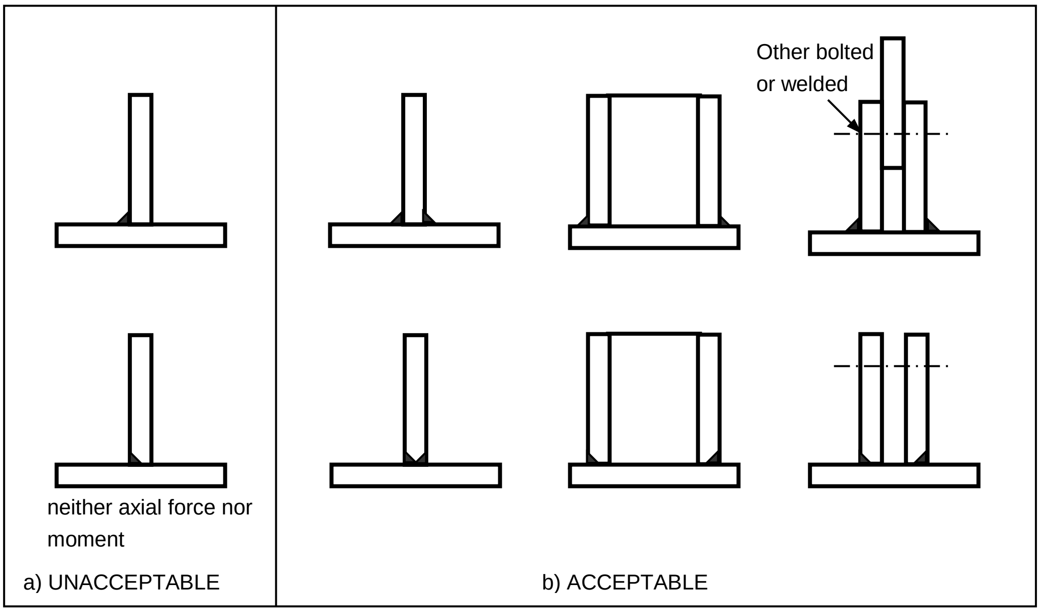



**Figure 9.6 - Single and double sides of penetration welds**

## 9.3 BOLTED CONNECTIONS

### 9.3.1 Bolt spacing

*9.3.1.1 Minimum spacing*

(a) The spacing between centres of bolts in the direction of load transfer should not be less than *2.5d*, where *d* is the nominal diameter of the bolts. This spacing can be increased as necessary for bearing capacity.

(b) The spacing between centres of standard holes measured perpendicular to the direction of load transfer should normally be *3d*. This spacing may be reduced to not less than *2.5d* provided that the bearing on the bolt is not more than 2/3 of $P_{bb}$ (see clause 9.3.6.1.2).

(c) The spacing for slotted holes should be measured from the centres of its end radius or the centreline of the slot.

*9.3.1.2 Maximum spacing*

The maximum spacing between centres of standard holes measured either parallel or perpendicular to the direction of load transfer should be limited to the lesser of *12t* or 150 mm, where *t* is the thickness of the thinner connected plate.

### 9.3.2 End and edge distances

The end distance is the distance from the centre of a hole to the adjacent edge in the direction in which the fastener bears. The end distance shall be sufficient to provide adequate bearing capacity. The edge distance is the distance from the centre of a hole to the adjacent edge at right angles to the direction of stress.

The distance from the centre of a standard hole to the adjacent edge or end of any part, measured either parallel or perpendicular to the direction of load transfer, should be not less than those listed in Table 9.3.

**Table 9.3 - Minimum end and edge distances of holes (for standard holes)**

| Bolt Size | At sheared and hand flame cut edge(mm) | At rolled edges of plates, shapes, bars or gas cut edges (mm) |
|---|---|---|
| M12 | 22 | 18 |
| M16 | 28 | 22 |
| M18 | 32 | 24 |
| M20 | 34 | 26 |
| M22 | 38 | 28 |
| M24 | 42 | 30 |
| M27 and over | 1.75*d* | 1.25*d* |

The maximum end and edge distance should not be greater than $11t\varepsilon$, in which $\varepsilon$ is the material constant equal to $\sqrt{275/p_y}$ and *t* is the thickness of the connected thinner plate. This does not apply to bolts interconnecting the components of back-to-back tension or compression members. For those exposed to a highly corrosive environment, the end and edge distance should not exceed *40 + 4t*.

Limiting the edge distance ensures adequate resistance against end shear, assists to exclude moisture and to prevent corrosion. More restriction should be exercised in severe conditions of exposure.

### 9.3.3 Hole dimensions

To compensate for shop fabrication tolerance and to provide some freedom for adjustment during erection, four types of holes are permitted as shown in Table 9.4.

For oversize holes, spacing, end and edge distances should be increased. The increment should be half of the difference between the diameters of an oversize hole and a standard hole.

**Table 9.4 - Nominal hole dimensions**

| Bolt size | Standard hole<br>Diameter<br>d (mm) | Oversize hole<br>Diameter<br>d (mm) | Short slot hole<br>Width × Length<br>(mm) | Long slot hole<br>Width × Length<br>(mm) |
|---|---|---|---|---|
| M12 | 14 | 16 | 14 × 18 | 16 × 30 |
| M16 | 18 | 20 | 18 × 22 | 18 × 40 |
| M18 | 20 | 22 | 20 × 24 | 20 × 45 |
| M20 | 22 | 25 | 22 × 26 | 22 × 50 |
| M22 | 24 | 27 | 24 × 28 | 24 × 55 |
| M24 | 26 | 30 | 26 × 32 | 26 × 60 |
| M27 and over | d + 3 | d + 8 | (d + 3) × (d + 10) | (d + 3) × ( 2.5d) |

### 9.3.4 Sectional area of connected parts

*9.3.4.1 Gross area*

The gross area $a_g$ should be computed as the products of the thickness and the gross width of the element, measured normal to its axis.

*9.3.4.2 Net area*

The net area $a_n$ should be the gross area less the deductions for bolt holes. See Figure 9.7.

*9.3.4.3 Deduction for bolt holes*

(a) Holes not staggered

For bolts aligned perpendicular to the direction of force, the deduction should be the sum of sectional areas of the bolt holes.

(b) Staggered holes

Where the bolt hole are staggered, the deduction should be the greater of

(i) The deduction for non-staggered holes, see Figure 9.7 line (1).

(ii) The sum of the sectional area of all holes lying on diagonal or zig-zag line less a justification factor of $0.25S^2t/g$ for each gauge g that it traverses diagonally. See Figure 9.7 lines (2) and (3).

(iii) For angles with bolts on both legs, the gauge length g should be the sum of the gauge lengths on each leg $g_1$ and $g_2$ measured from the heel minus the thickness t of the angle, see Figure 9.8.

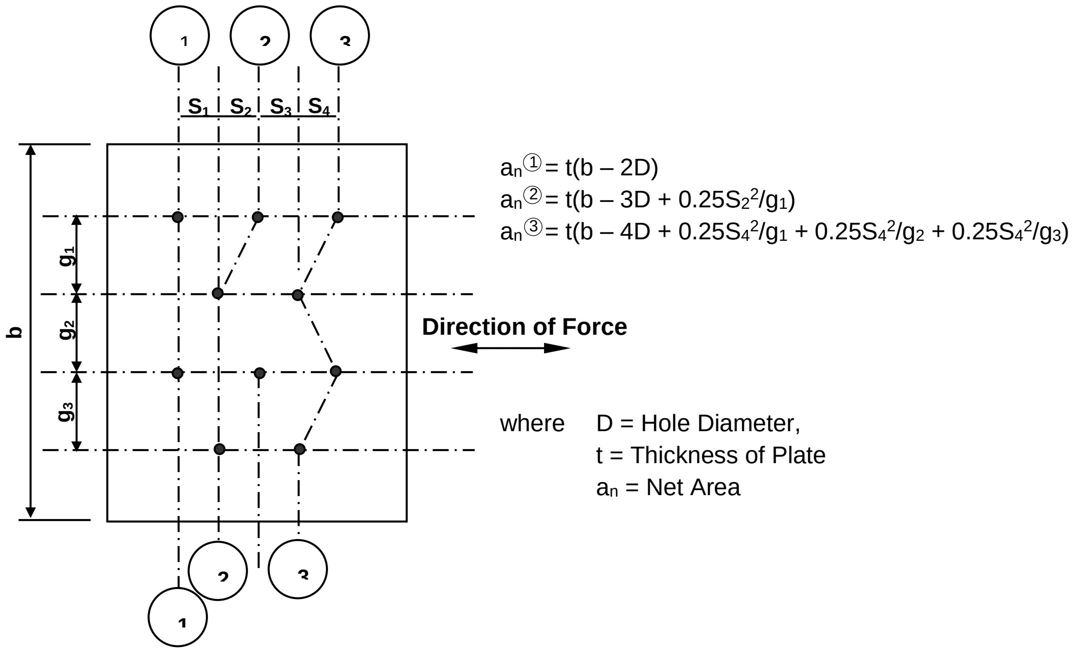



**Figure 9.7 - Staggered holes**

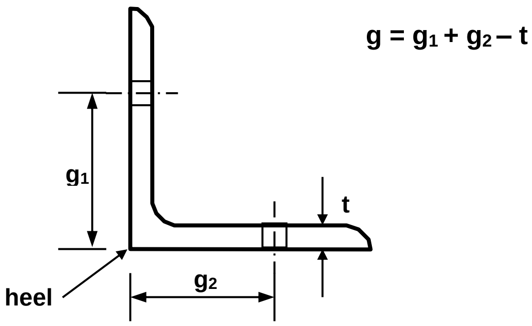



**Figure 9.8 - Angles with hole in legs**

9.3.4.4 *Effective area for tension*

The effective area $a_e$ perpendicular to the force direction should be determined from

$$a_e = K_e \, a_n \leq a_g \tag{9.10}$$

where the effective net area coefficient $K_e$ is given by

| | | |
|---|---|---|
| $K_e$ | = 1.2 | for steel of grade S275 |
| | = 1.1 | for steel of grade S355 |
| | = 1.0 | for steel of grade S460 |
| | = 0.84 | for steel of grade S550 |
| | = 0.80 | for steel of grade S690 |

$a_n$ is the net cross sectional area of the leg deduced for hole openings.
$a_g$ is the gross sectional area without reduction for openings.

9.3.4.5 *Effective area for shear*

Bolt holes need not be allowed for in the shear area provided that:

$$A_{v.net} \geq 0.85 \, A_v \, / \, K_e \tag{9.11}$$

where $A_v$ = gross shear area before hole deduction
$A_{v.net}$ = net shear area after deducting bolt holes
$K_e$ = effective net area coefficient from clause 9.3.4.4

Otherwise the net shear capacity should be taken as

$$0.7 \, p_y \, K_e \, A_{v.net} \tag{9.12}$$

## 9.3.5 Block shear

Block shear failure at a group of bolt holes near the end of web of a beam or bracket should be prevented by proper arrangement of a bolt pattern as shown in Figure 9.9. This mode of failure generally consists of tensile rupture on the tension face along the bolt line accompanied by gross section yielding in shear at the row of bolt holes along the shear face of the bolt group. On the tension side, the block shear capacity should be taken as,

$$P_r = (1/\sqrt{3}) p_y \; A_{v.eff} \qquad (9.13)$$

where $A_{v.eff}$ is the effective shear area defined as,

$$A_{v.eff} = t \left[ L_v + K_e \left( L_t - k \, D_t \right) \right] \qquad (9.14)$$

$t$ = thickness of connected part

$L_v$ = length of shear face, see Figure 9.9 below

$L_t$ = length of tension face

$K_e$ = effective net area coefficient see clause 9.3.4.4

$k$ = 0.5 for single row of bolts or 2.5 for two rows of bolts

$D_t$ = is the hole diameter for the tension face, but for slotted holes the dimension perpendicular to load direction should be used

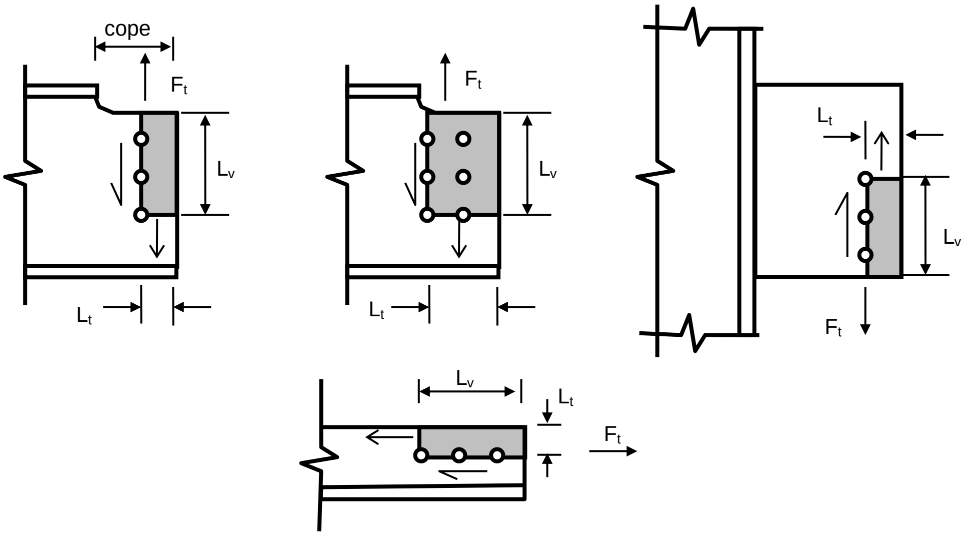



**Figure 9.9 - Block shear and effective shear area**

## 9.3.6 Design strength of bolts under shear and bearing

*9.3.6.1 Shear capacity of non-preloaded bolts*

This category of connection may be referred to as a “Bearing Type Connection”. For this type of connection, no preload or special treatment on contact surfaces is required. Ordinary bolts of ISO grade 4.6 or equivalent manufactured from low carbon steel to high strength bolts of ISO grade 10.9 or equivalent may be used.

9.3.6.1.1 Shear capacity of bolts

The shear capacity $P_s$ of a bolt should be taken as

$$P_s = p_s \; A_s \qquad (9.15)$$

where

$p_s$ is the design shear strength obtained from Table 9.5

$A_s$ is the effective shear area

- for the case of the bolt thread occurring in the shear plane, $A_s$ is taken as the tensile area $A_t$.
- for the case of the bolt thread not occurring in the shear plane, $A_s$ is taken as the cross sectional area of the shank.

The shear strength $p_s$ should be reduced due to the effect of bolting conditions, see clauses 9.3.6.1.4 to 9.3.6.1.6 below.

**Table 9.5 - Design shear strength of bolts**

| Bolt grade | | | Design shear strength $p_s$ (N/mm²) |
|---|---|---|---|
| ISO | 4.6 | | 160 |
| | 6.8 | | 240 |
| | 8.8 | | 375 |
| | 10.9 | | 400 |
| | 12.9 | | 480 |
| BS | General grade HSFG | ≤ M24 | 400 |
| | | ≥ M27 | 350 |
| | Higher grade HSFG | | 400 |
| ASTM | A307 | | 124 |
| | A325 | | 248 |
| | A490 | | 311 |
| GB50017 | 8.8 | | 250 |
| | 10.9 | | 310 |
| Other grades ($U_b$ ≤ 1200 N/mm²) | | | $0.4U_b$ |
| *Note:* $U_b$ is the specified minimum tensile strength of the bolt. | | | |

9.3.6.1.2 Bearing capacity of bolts

The bearing capacity $P_{bb}$ of a bolt bearing on connecting parts should be taken as

$$P_{bb} = d\ t_p\ p_{bb} \tag{9.16}$$

where $d$ is the nominal diameter of the bolt

$t_p$ is the thickness of the thinner connecting part

$p_{bb}$ is the bearing strength of the bolt obtained from Table 9.6

**Table 9.6 - Design bearing strength of bolts**

| Bolt grade | | | Design bearing strength $p_{bb}$ (N/mm²) |
|---|---|---|---|
| ISO | 4.6 | | 460 |
| | 6.8 | | 900 |
| | 8.8 | | 1000 |
| | 10.9 | | 1300 |
| | 12.9 | | 1600 |
| BS | General grade HSFG | ≤ M24 | 1000 |
| | | ≥ M27 | 900 |
| | Higher grade HSFG | | 1300 |
| ASTM | A307 | | 400 |
| | A325 | | 450 |
| | A490 | | 485 |
| GB50017 | 8.8 | | 720 |
| | 10.9 | | 930 |
| Other grades ($U_b$ ≤1000 N/mm²) | | | $0.7(U_b + Y_b)$ |
| *Note*: $U_b$ is the specified minimum tensile strength of the bolt.<br>$Y_b$ is the specified minimum yield strength of the bolt. | | | |

9.3.6.1.3 Bearing capacity of connected parts

The bearing capacity $P_{bs}$ of the connected parts should be the least of the followings

$$P_{bs} = k_{bs}\ d\ t_p\ p_{bs} \tag{9.17}$$

$$P_{bs} = 0.5\ k_{bs}\ e\ t_p\ p_{bs} \tag{9.18}$$

and

$$P_{bs} = 1.5\, l_c\, t_p\, U_s \leq 2.0\, d\, t_p\, U_b \qquad (9.19)$$

in which

$e$ is the end distance, measured in the same direction of load transfer

$p_{bs}$ is bearing strength of connected parts

- for steel of grade S275, $p_{bs}$ = 460 MPa
- for steel of grade S355, $p_{bs}$ = 550 MPa
- for steel of grade S460, $p_{bs}$ = 670 MPa
- for steel of grade S550, $p_{bs}$ = 770 MPa
- for steel of grade S690, $p_{bs}$ = 940 MPa
- for steel of other grades, $p_{bs} = 0.67(U_s + Y_s)$ (9.20)
  (refer to section 3 for other grades of steel)

$k_{bs}$ is hole coefficient taken as

- for standard holes $k_{bs}$ = 1.0
- for over size holes $k_{bs}$ = 0.7
- for short slotted holes $k_{bs}$ = 0.7
- for long slotted holes $k_{bs}$ = 0.5

$l_c$ is net distance between the bearing edge of the holes and the near edge of adjacent hole in the same direction of load transfer.

9.3.6.1.4 Long joints

In a lapped joint of bearing type, when the distance $L_j$ between the centres of two end bolts measured in the direction of load transfer is larger than 500 mm as shown in Figure 9.10, the shear capacity $P_s$ of all the bolts calculated from clause 9.3.6.1.1 should be reduced by multiplying a reduction factor $\beta_L$ as

$$\beta_L = \left(\frac{5500 - L_j}{5000}\right) < 1.0 \qquad (9.21)$$

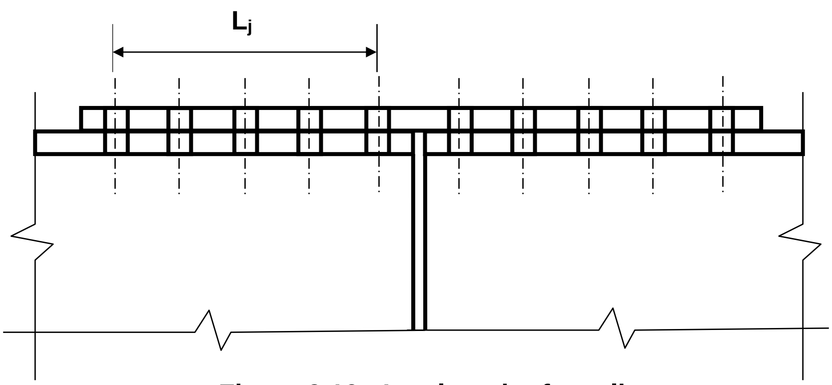



**Figure 9.10 - Lap length of a splice**

Note that this provision does not apply to a full length connection with uniform load distribution, e.g. the bolted connection of flanges to web of a plate girder.

9.3.6.1.5 Long grip length

Where the grip length $T_g$ (i.e. the total thickness of the connected plies) is greater than *5d*, the shear capacity $P_s$ should be reduced by multiplying a reduction factor $\beta_g$ given by,

$$\beta_p = \frac{8d}{3d + T_g} \qquad (9.22)$$

9.3.6.1.6 Bolts through packing

When a bolt passes through packing with thickness $t_{pa}$ greater than one-third of the nominal diameter $d$, its shear capacity $P_s$ should be reduced by multiplying a reduction factor $\beta_p$ obtained from:

$$\beta_p = \left( \frac{9d}{8d + 3t_{pa}} \right) \leq 1.0 \tag{9.23}$$

For double shear connections with packing on both sides of connecting member, $t_{pa}$ should have the same thickness; otherwise, the thicker $t_{pa}$ should be used.

This provision does not apply to preloaded bolt (friction-type) connections when working in friction, but does apply when such bolts are designed to slip into bearing.

*9.3.6.2 Shear capacity of preloaded bolts*

Only high strength friction grip bolts of ISO grade 8.8 or above should be used for preloaded bolts. High strength friction grip bolts complying with the acceptable references in Annex A1.3 should be used as such bolts have stronger heads and nuts to ensure failure occurs in the bolt shank. The bolts should be preloaded to the required tension with controlled tightening to generate the required gripping forces so that slip between the connected parts will not occur at ultimate limit state. The factored load on each bolt should not exceed the slip resistance $P_{SL}$ taken from the following equation:

$$P_{SL} = 0.9\, K_S\ \mu\, P_o \tag{9.24}$$

where

$P_o$ is the minimum proof loads of bolts specified in relevant international or local standards.

$\mu$ is the slip factor between connected parts. It may be obtained from Table 9.7 or determined from the results of test as specified to relevant standards.

$K_s$ is the coefficient allowing for type of hole

- for standard holes $K_s = 1.0$
- for oversize holes $K_s = 0.85$
- for slotted holes, loaded perpendicular to slot $K_s = 0.85$
- for slotted holes, loaded parallel to slot $K_s = 0.7$

**Table 9.7 - Slip factors for preloaded bolts**

| Class | Condition of faying surfaces | | Slip factor |
|---|---|---|---|
| | Preparation | Treatment | $\mu$ |
| A | Blasted with slot or grit | Loose rust removed, no pitting | 0.5 |
| | | Spray metallized with aluminium | |
| | | Spray metallized with a zinc based coating that has been demonstrated to provide a slip factor of at least 0.5 | |
| B | Blasted with shot or grit | Spray metallized with zinc | 0.4 |
| C | Wire brushed | Loose rust removed, tight mill scale | 0.3 |
| | Flamed cleaned | | |
| D | Untreated | Untreated | 0.2 |
| | Galvanized | | |

Should slip between the connected parts occur the bolts should be designed as bearing type.

### 9.3.7 Design strength of bolts in tension

*9.3.7.1* *Tension capacity of bolts*

The tension capacity $P_t$ of a bolt should be taken as

$$P_t = A_s \ p_t \quad (9.25)$$

where

$A_s$ is the tensile stress area

$p_t$ is tension strength obtained from Table 9.8.

**Table 9.8 - Design tension strength of bolts**

| Bolt grade | | | Design tension strength $p_t$ (N/mm²) |
|---|---|---|---|
| ISO | 4.6 | | 240 |
| | 6.8 | | 480 |
| | 8.8 | | 560 |
| | 10.9 | | 700 |
| | 12.9 | | 810 |
| BS | General grade HSFG | ≤ M24 | 590 |
| | | ≥ M27 | 515 |
| | Higher grade HSFG | | 700 |
| ASTM | A307 | | 310 |
| | A325 | | 620 |
| | A490 | | 780 |
| GB50017 | 8.8 | | 400 |
| | 10.9 | | 500 |
| Other grades ($U_b$ ≤ 1200 N/mm²) | | | 0.7 $U_b$ but ≤ $Y_b$ |
| Note: $U_b$ is the specified minimum tensile strength of the bolt.<br>$Y_b$ is the specified minimum yield strength of the bolt. | | | |

*9.3.7.2* *Prying force*

(a) Design against prying force is not required provided that all the following conditions are satisfied.

(i) Bolt tension capacity $P_t$ is reduced to

$$P_{nom} = 0.8\ A_t\ p_t \quad (9.26)$$

in which $P_{nom}$ is the nominal tension capacity of the bolt.

(ii) The bolt gauge G on the flange of UB, UC and T sections does not exceed 0.55B, in which B is total width of the flange, see Figure 9.11.

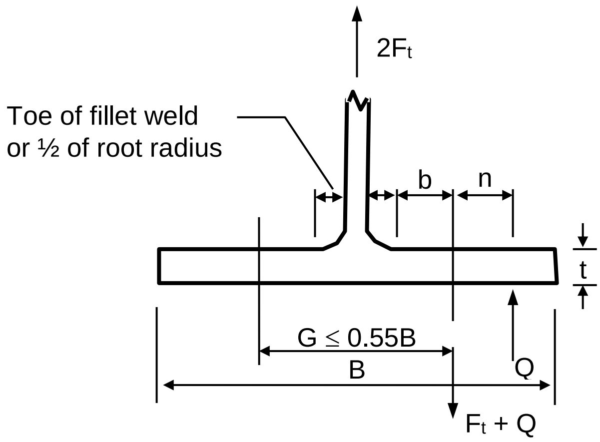



**Figure 9.11 - Prying force**

(b) If the conditions described in (a) above cannot be satisfied, the prying force $Q$ should be calculated and taken into account and $F_{tot}$ should be calculated as follows:

$$F_{tot} = F_t + Q < P_t \quad (9.27)$$

$F_{tot}$ is the total applied tension in the bolt including the prying force, and $F_t$ is the tension force in the bolt.

## 9.3.8 Combined shear and tension

*9.3.8.1 Tension combined with non-preloaded bolts*

Both shear and tension will directly act to the bolts.

(a) For bolts without consideration of prying force

$$\frac{F_s}{P_s} + \frac{F_t}{P_{nom}} \leq 1.4 \quad (9.28)$$

(b) For bolts with consideration of prying force

$$\frac{F_s}{P_s} + \frac{F_{tot}}{P_t} \leq 1.4 \quad (9.29)$$

*9.3.8.2 Tension combined with preloaded bolts*

It is still assumed that there is no slip between the connected parts under shear force. However, the gripping force will be reduced by the tension force. The combined effect of tension and shear should be

$$\frac{F_s}{P_{SL}} + \frac{F_{tot}}{0.9P_0} \leq 1.0 \quad (9.30)$$

in which $P_{SL}$ is the slip resistance of a preloaded bolt, $P_0$ is the specified minimum proof load and $F_s$ is the applied shear.

## 9.3.9 Bolts combined with welds

Only preloaded bolts designed to be non-slip and tightened after welding may share load with welds.

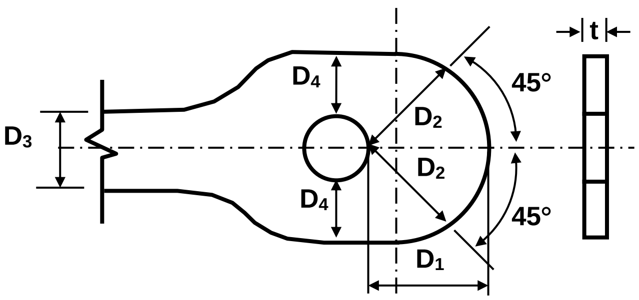
In alteration and addition works, if the existing bolted connections are HSFG type or load reversal is not expected, the existing bolts are permitted to carry loads present at the time of the alteration and any new welds shall be designed to resist the additional design loads.

## 9.3.10 Pin connections

*9.3.10.1 Pin connected tension members*

In pin connected tension members and their connecting parts, the thickness of an unstiffened element containing a hole for a pin should be not less than 25% of the distance from the edge of the hole to the edge of the element, measured perpendicularly to the axis of the member, see Figure 9.12. Where the connected elements are clamped together by external nuts, this limit on thickness need not be applied to internal plies.

The net cross-sectional area beyond a hole for a pin, in all directions within 45° of the member axis shown in Figure 9.12, should not be less than the net cross-sectional area $A_r$ required for the member. When measured perpendicular to the member axis, the net cross-sectional area on each side of the hole should not be less than $2A_r/3$.




**$A_r = F_t/p_y$**
**$D_1 \geq A_r/t$**
**$D_2 \geq A_r/t$**
**$D_3 \geq A_r/t$**
**$D_4 \geq 0.67A_r/t$**
**$t \geq 0.25D_4$**

**Figure 9.12 - Pin-ended tension member**

*9.3.10.2 Design of pins*

9.3.10.2.1 General

All pins shall be provided with locking devices to ensure that the pin does not move away from its position in service. The capacity of a pin connection should be determined from the shear capacity of the pin, see clause 9.3.10.2.2, and the bearing capacity on each connected part, see clause 9.3.10.2.3, taking due account of the distribution of load between the various parts. The moment in the pin should also be checked, see clause 9.3.10.2.4.

9.3.10.2.2 Shear capacity

The shear capacity of a pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable:

$$0.6\, p_{yp}\, A \qquad (9.31)$$

b) if rotation is required or if the pin is intended to be removable:

$$0.5\, p_{yp}\, A \qquad (9.32)$$

where

$A$ is the cross-sectional area of the pin

$p_{yp}$ is the design strength of the pin

9.3.10.2.3 Bearing capacity

The bearing capacity of a pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable:

$$1.5\, p_y\, d\, t \qquad (9.33)$$

b) if rotation is required or if the pin is intended to be removable:

$$0.8\, p_y\, d\, t \qquad (9.34)$$

where

$d$ is the diameter of the pin

$p_y$ is the smaller of the design strengths of the pin and the connected part

$t$ is the thickness of the connected part

9.3.10.2.4 Bending

The bending moment in a pin should be calculated on the assumption that the connected parts form simple supports. It should generally be assumed that the reactions between the pin and the connected parts are uniformly distributed along the contact length on each part, see Figure 9.13.

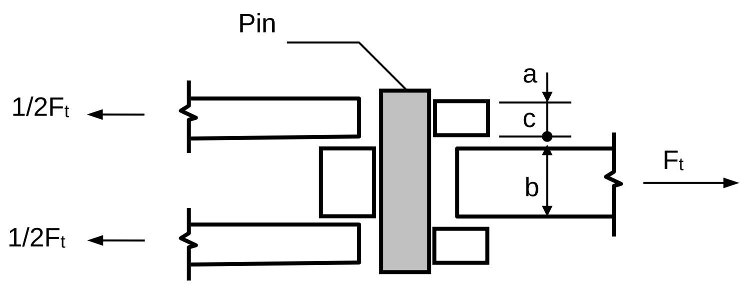



**Figure 9.13 - Bending action in pinned connection**

The moment capacity of the pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable:

$$1.5\, p_{yp}\, Z \tag{9.35}$$

b) if rotation is required or if the pin is intended to be removable:

$$1.0\, p_{yp}\, Z \tag{9.36}$$

where

$p_{yp}$ is the design strengths of the pin

$Z$ is the elastic section modulus of the pin

### 9.3.11 Connections for hollow sections of lattice girders

The design of connections of hollow sections members of trusses should be based on the following criteria as relevant.

(a) Chord face failure
(b) Chord web (or wall) failure by yielding or instability
(c) Chord shear failure
(d) Chord punching shear failure
(e) Local buckling failure
(f) Braces (web members) failure with reduced effective width
(g) Load eccentricities

Their design can be complex and good guidance and rules may be found from specialist literature, see Annex A1.3.

## 9.4 BASEPLATE AND ANCHOR CONSTRUCTION

### 9.4.1 Column base plates

Steel columns should be provided with a steel base plate of sufficient size, stiffness and strength to distribute the forces due to axial, bending and shear effects as appropriate from the column to the support without exceeding the load carrying capacity of the support. The nominal bearing pressure between the base plate and the support should be determined by assuming a linear distribution. The maximum stress induced on concrete foundation should not be larger than $0.6f_{cu}$ in which $f_{cu}$ is the lesser of the concrete or the bedding grout 28-day cube strength.

Steel base plate with design strength not larger than 275 N/mm$^2$ should be used for transmitting moments, and its thickness should be designed to prevent from brittle fracture if the base plate is exposed to external weather conditions.

(a) Base plates with axial forces applied concentrically may be designed by the effective area method. As shown on Figure 9.14, only the shaded portion of the base plate is considered to be effective to transmit the full design pressure of $0.6f_{cu}$ to the support. The thickness $t_p$ of the base plate can be determined by

$$t_p = c\sqrt{\frac{3w}{p_{yp}}} \tag{9.37}$$

where

$c$ is the largest perpendicular distance from the edge of the effective portion of the base plate to the face of the column cross-section, see Figure 9.14.

$w$ is the pressure under the effective portions of the base plate. It can be assumed as uniform distribution over entire effective portion but should be limited to $0.6\, f_{cu}$.

$p_{yp}$ is the design strength of the base plate.

$t, T$ are respectively the thickness of the thinner and thicker parts of the column section including the stiffeners, if any.

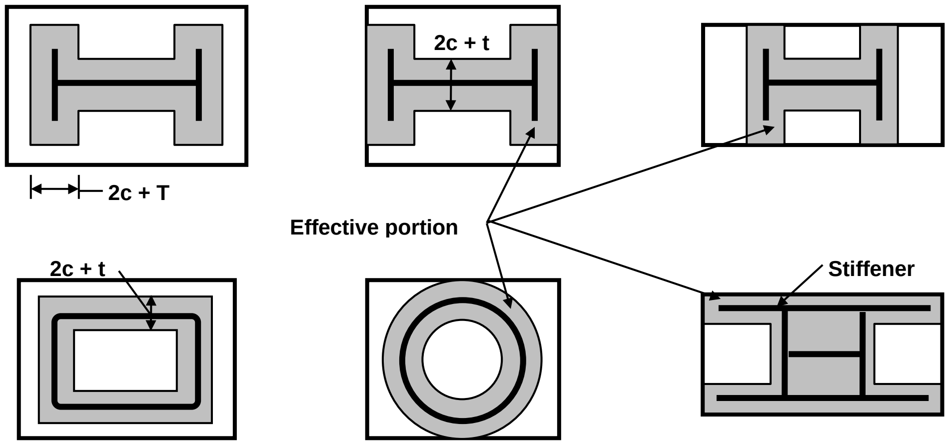



**Figure 9.14 - Typical base plates**

(b) If moments are applied to the base plate through the column, either caused by eccentric axial loads or direct moments, the base plate should be analysed by elastic linear analysis. The pressure under the compression zone may be considered as uniform or linearly distributed.

Stiffeners may be used to reinforce the base plate. These stiffeners should resist the total moment due to the design anchor tension or concrete pressure on the base plate and should not exceed

$$M_s \leq p_{ys} Z_s \tag{9.38}$$

where

$p_{ys}$ = design strength of stiffener

$Z_s$ = section modulus of stiffener

(c) Alternatively to (a) and (b) above, the Responsible Engineer may justify the design of a base plate by other means using acceptable structural principles.

The column may be connected to the base plate, either with or without gussets, by welding or bolting. The welds or bolts should be provided to transmit the total forces and moments unless tight bearing contact can be ensured such that welds or bolts transmit only shear and tension forces.

### 9.4.2 Anchor bolts for base plates, wall plates and hangers

Anchor bolts shall be required to attach steel plates to concrete structure, typically for column base plates, beam wall support plates, hangers and cladding steelwork.

Anchor bolts should be able to resist forces from the most severe design load combination of wind, imposed, permanent, construction and other loads. They should be designed to take the tension due to uplift forces and bending moment as appropriate. If no special element for resisting shear force is provided, such as shear key on the bottom of a column base plate or other restraint system, the anchor bolts should be designed to resist the shear forces in addition to the tension forces. Friction between the base plate and concrete or grout should not normally be considered to provide shear resistance.

The long-term durability of the anchorage system should be considered. Where corrosion is possible, materials with good corrosion resistance such as stainless steel or hot-dip galvanized steel may be used or a heavy-duty paint system employed.

**(a) Cast-in anchor bolts**

The cast-in anchor bolts may be fabricated from high yield reinforcing bars, plain bars of grade S275 or other grades of steel.

(i) Straight or hooked bars

This type of bolts is anchored by bond stress between steel and concrete. The design anchorage bond stress is assumed to be constant over the bond length. The minimum bond length should be calculated by

$$l = \frac{F_t}{\pi\, d\, f_{bu}} \qquad (9.39)$$

where

$F_t$ = total factored tension in bolt

$d$ = diameter (nominal) of bolt

$f_{bu}$ = design ultimate anchorage bond stress

= $\beta\sqrt{f_{cu}}$

$\beta$ = coefficient dependent on bar type and equal to
0.28 for plain bar in tension;
0.35 for plain bar in compression;
0.50 for deformed bar in tension; and
0.63 for deformed bar in compression

$f_{cu}$ = concrete cube strength on 28 days

The design of the bolt should be the same as ordinary bearing type bolted connections. Net area at thread portion of the anchor bolt should be used for design calculation. Interaction for combining tension and shear should be the vector sum of the ratio of design to capacity and should not be greater than unity as,

$$\left(\frac{V}{V_c}\right)^2 + \left(\frac{F_t}{P_t}\right)^2 \le 1 \qquad (9.40)$$

where

$V$, $F_t$ are factored shear and tension respectively.

$V_c$, $P_t$ are shear and tension capacities of anchor bolt respectively.

(ii) Headed bolts

This type of bolts is associated with washer plate (bearing plate) fixed at the embedded end of the bolt by either welding or nuts. Bond stress should not be taken into account for anchorage force. The total tension resisted by the washer plate should be calculated using the theory of shear cone on concrete.

**(b) Drilled-in anchor bolts**

Drilled-in anchors should only be used in existing concrete and in situations where they are required to resist significant tension, their uses should be avoided whenever possible. Their design and installation should be strictly in accordance with manufacturer's specification.

**(c) Anchor bolts for hangers**

There is often no redundancy in hanger systems and the Responsible Engineer should assess the consequences of failure of a particular system and, if it is considered necessary an additional partial safety factor should be used to account for this.

Anchor bolts for attaching steel hangers to new concrete beams or slabs should be mechanically locked around top reinforcement using hooks or top plates embedded in the concrete. Anchor bolts for attaching steel hangers to existing concrete beams or slabs should take the form of through bolts with substantial top plates. The use of expansion or chemical anchors in this situation should be avoided if at all possible.

## 9.5 STEEL CASTINGS AND FORGINGS

Steel castings and forgings may be used in bearings, connection or other parts for architectural aesthetics or because of joint node complexity. They are particularly well suited for nodes of structural hollow section structures with several members meeting at a single point. For details of the use of castings in buildings see Annex A2.2 and see Annex A1.2 for acceptable references for materials for castings and forgings.

Connections of castings or forgings may be designed by means of testing (see section 16), analysis or other rational methods.

Performance tests may be required to demonstrate the safety and functionality of castings or forgings in addition to valid mill certificates.