31 WELDS- STATIC AND FATIGUE STRENGTH – II

1.0 INTRODUCTION

In the previous chapter, a detailed account of various welding processes, types of welds, advantages of welded connections etc. were presented. It was seen that welded connections are continuous and more rigid when compared to bolted connections. It was also pointed out that fillet welds and butt welds constitute respectively 80% and 15% of all welds in the construction industry; the balance 5% is made up by plug, slot and spot resistance welds.

In this chapter, the behaviour and design of welded connections under various static loading conditions is considered. A typical connection design process is initiated with the design, which is followed by the welding operation and, concludes with inspection.

2.0 CONNECTION DESIGN

In the design of connections, due attention must be paid to the flow of the force through the connection. The transfer of forces should occur smoothly, without causing any stress concentration or cracks. The connections can be either concentric or eccentric. In concentric connections, the forces acting on the connections will essentially be axial in nature, whereas in eccentric connections, the axial forces will be coupled with bending or torsion. These types of connections are described in the following.

2.1 Concentric connections

Static strength of a welded joint depends upon the following factors

- Type and size of the weld
- Manner of welding, and
- Type of electrode used.

A primary responsibility of a designer is to select the type and size of the weld. A number of varieties of welds are available. When it is properly chosen with the correct electrode, it develops full strength of the parent material. The chosen type of weld should develop minimal residual stresses and distortions.

As stated in the introduction, butt and fillet welds are the usual forms of welds in practical building construction. Butt welds are used at an edge-to-edge junction or a tee junction. A butt weld connection is made by bringing the plates to be joined face to face edgewise and then filling the cavity formed by edge preparation or by just penetrating the unprepared junction. Butt welds can be either full penetration or partial penetration.

Partial penetration butt welds may be used for static loading, if reduced strength is acceptable. On the other hand, a fillet weld is made away from the edges of the abutting plates. The joint is formed by welding the members in an overlapped position or by using a secondary joining material. The main advantage of a fillet weld is that the requirements of alignment and tolerance are less rigorous when compared to butt welds. Fillet welding could be applied for lap joints, tee joints and corner joints. A detailed description of these two types of welds and their design requirements are presented in the following.

2.2 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, which has been discussed in the previous chapter.

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. 1. In order to qualify for a full penetration weld, there are certain conditions to be satisfied while making the welds. The more important ones are given below:

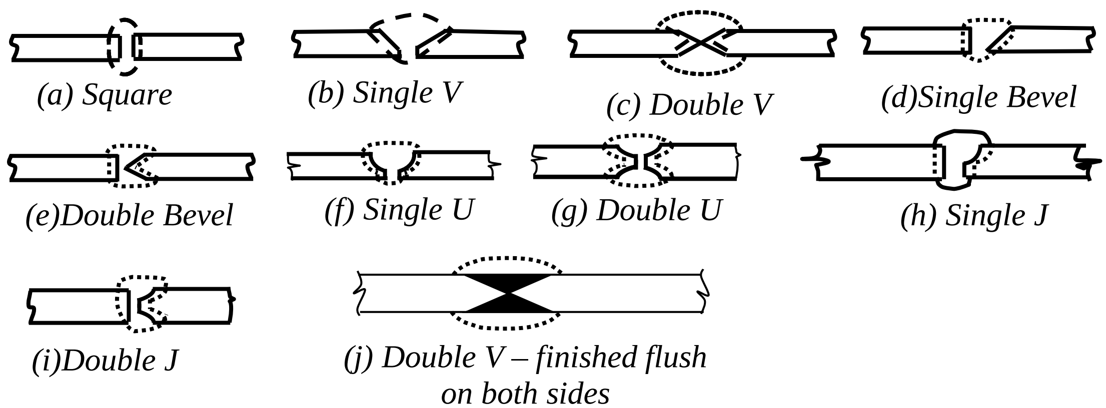


Fig. 1Different types of butt joints

2.2.1 Static behaviour of butt welds

For butt welds the most critical form of loading is tension applied in the transverse direction (Fig. 2). 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. During the application of the load, the welded portion and the HAZ (Heat Affected Zone of the parent metal) have less transverse contraction compared to the parent metal. 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 increase in yield stress in the HAZ is due to the quenching effect associated with rapid cooling after deposition of the weld.

The yield stress of the weld metal is also raised due to the quenching effect. The metal alloys constituting the electrode contribute to the increase in yield stress. These alloys penetrate the parent metal influencing its mechanical properties.

Due to the lower yield stress and higher transverse contraction in the parent metal, it experiences a higher true stress. As a consequence, the failure of the coupon always occurs away from the weld. The higher strength achieved by the welded joint adversely affects its toughness and ductility properties. These negative effects can be minimised by choosing proper electrodes.

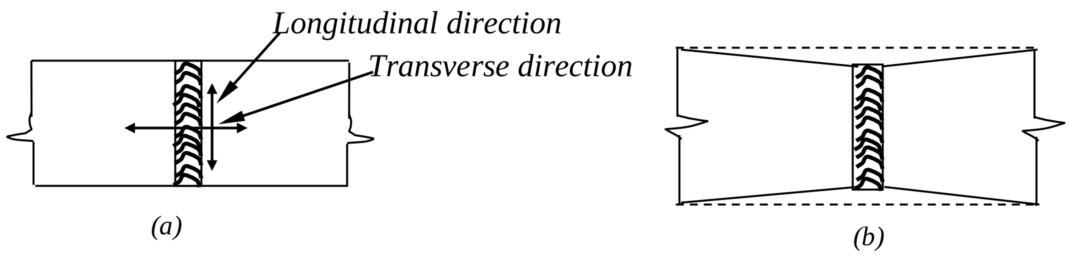


Fig.2 (a) Load applied in transverse direction (b) longitudinal shrinkage restraint

Partial penetration welds, shown in Fig. 3, differ in two ways from the full penetration welds: the reduction in cross section and the uncertainty of the weld root quality. Firstly, there is a reduction in the cross section at the joint resulting in overloading and severe plastic straining. Further, the weld root quality cannot be inspected and they cannot be repaired as may be done for full penetration welds.

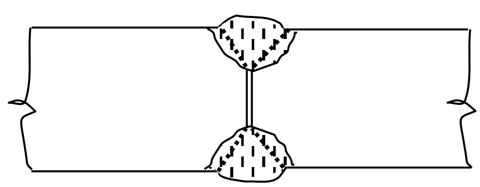


Fig.3 Partial penetration weld

2.2.2 Design

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.

Generally speaking, partial penetration welds must be avoided. Partial penetration groove welds are used in non-critical details, so as to avoid back-gouging. If they are considered essential, they should be designed with care. Some codes of practice do not recommend their use in tension. Others specify that they be designed in the same way as fillet welds. This is because the load transfer is not smooth and efficient with partial penetration welds. The effective throat thickness of a partial penetration weld is taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement. For stress calculation, a maximum value of reduced effective throat thickness equal to 5/8 of the thickness of the thinner part joined must be used. The unwelded portion in partial penetration butt welds, welded from both sides, shall not be greater than ¼ thickness of the thinner part joined, and should be in the central portion.

If the stresses are uniform across the throat thickness, the average stress concept may be applied to determine its strength. Connections with partial penetration welds with welding on only one side is generally avoided under tensile load due to the eccentric loading involved. Otherwise, the eccentricity effects should be considered in the design.

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.4 (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. 4(*c*)]. Where reduction of the wider part is not possible, the ends of the weld shall be returned to ensure full throat thickness.

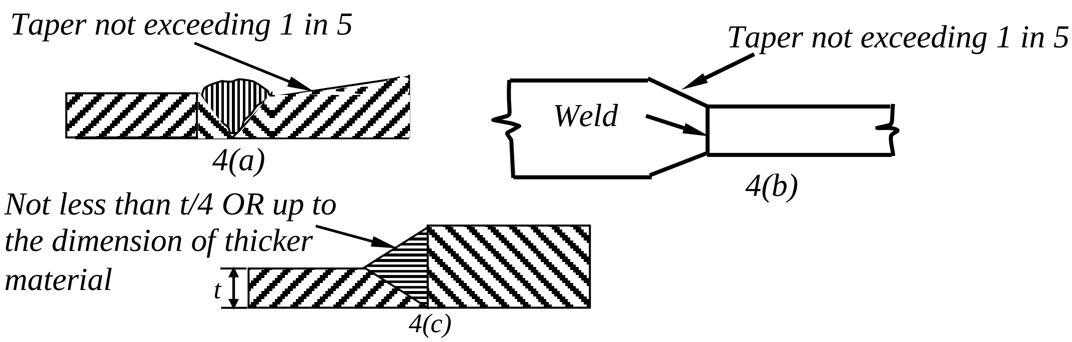


Fig. 4 Butt welding of members with (a)&(b) unequal thickness (c) unequal width

Permissible stresses for butt welds are assumed same as for the parent metal with a thickness equal to the throat thickness. For field welds, the permissible stresses in shear and tension may be reduced to 80% of the above value.

2.3 Fillet welds

2.3.1 General

These are generally used for making lap joint splices and other connections where the connecting parts lap over each other. Though a fillet weld may be subjected to direct stresses, it is weaker in shear and therefore the latter is the main design consideration.

2.3.2 Behaviour

Fillet welds are broadly classified into side fillets and end fillets (Fig. 5). 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.

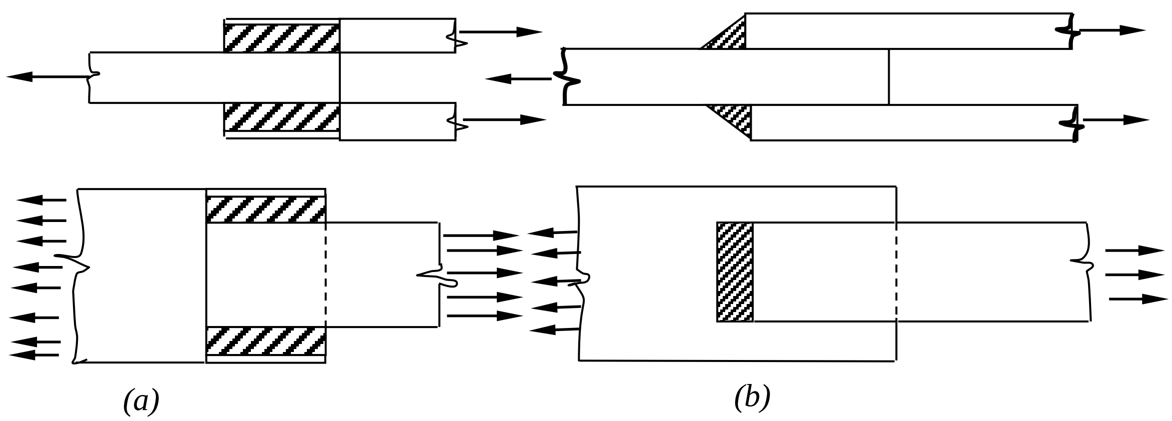


Fig.5 Fillet (a) side welds and (b) end welds

Actual distribution of stresses in a fillet weld is very complex. A rigorous analysis of weld behaviour has not been possible so far. Multiaxial stress state, variation in yield stress, residual stresses and strain hardening effects are some of the factors, which complicate the analysis.

In many cases, it is possible to use the simplified approach of average stresses in the weld throat (Fig. 6). 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 equal to ≤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.

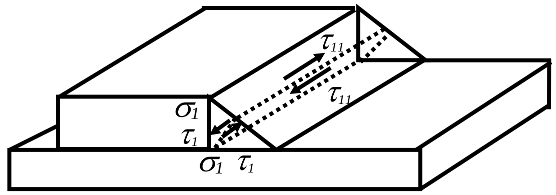


Fig.6 Average stress in the weld throat

2.3.3 Design

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. Alternatively, design strength can be different with direction of the load vector. This method is limited in usage to cases of pure shear, tension or compression (Fig.7). 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.

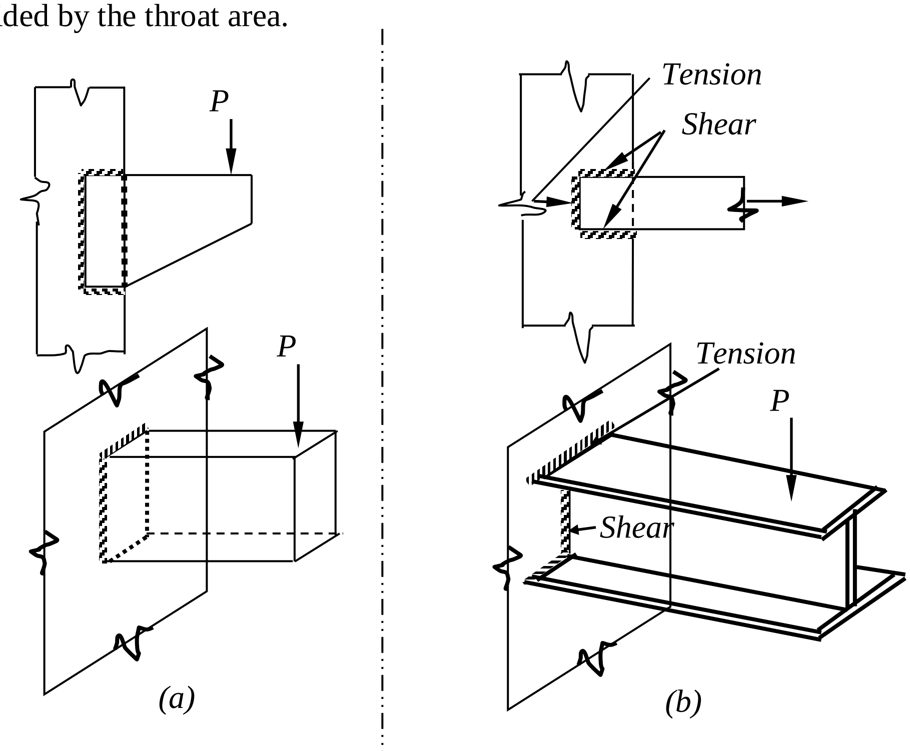


Fig.7 (a) connections with simple weld design,(b) connections with direction- dependent weld design

The size of a normal fillet should be taken as the minimum leg size (Fig. 8).

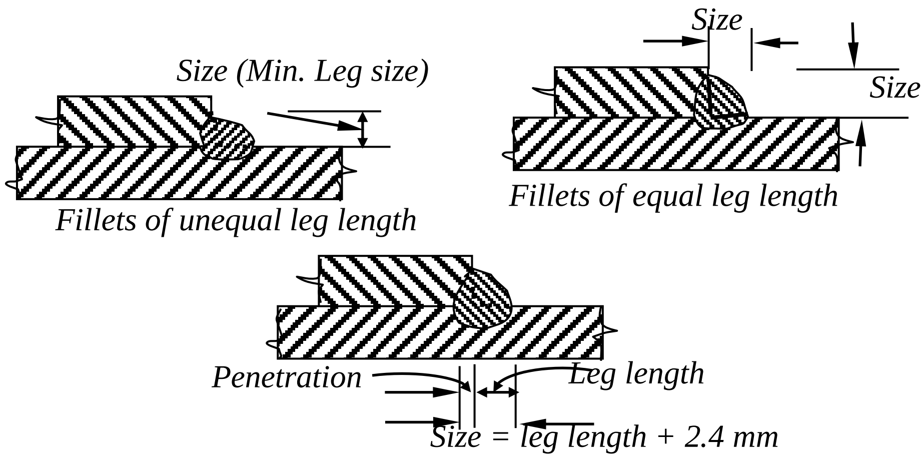


Fig. 8 Sizes of fillet welds

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 1. *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 1 Minimum size of first run or of a single run fillet weld

Thickness of thicker part		**Minimum size (mm)**
Over (mm)	Up to and including (mm)	
-	10	3
10	20	5
20	32	6
32	50	8 (First run) 10 (Minimum size of fillet)

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 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. 9(*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. 9(*b*)].

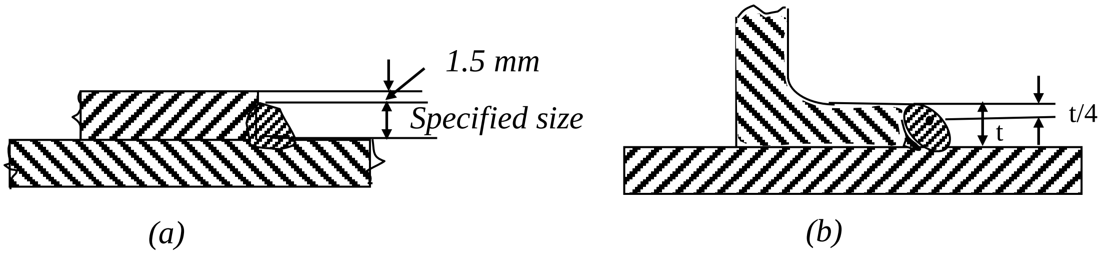


Fig.9 (a) fillet welds on square edge of plate, (b) fillet welds on round toe of rolled section

Table 2. Value of K for different angles between fusion faces

Angle between fusion faces	60° - 90°	91°-100°	101°-106°	107°-113°	114°-120°
Constant K	0.70	0.65	0.60	0.55	0.50

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.

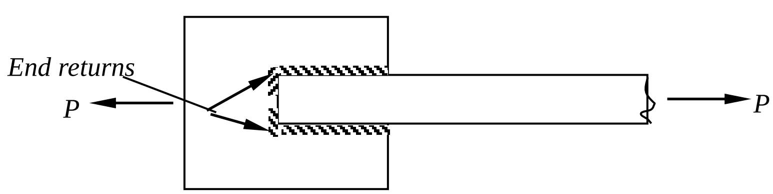


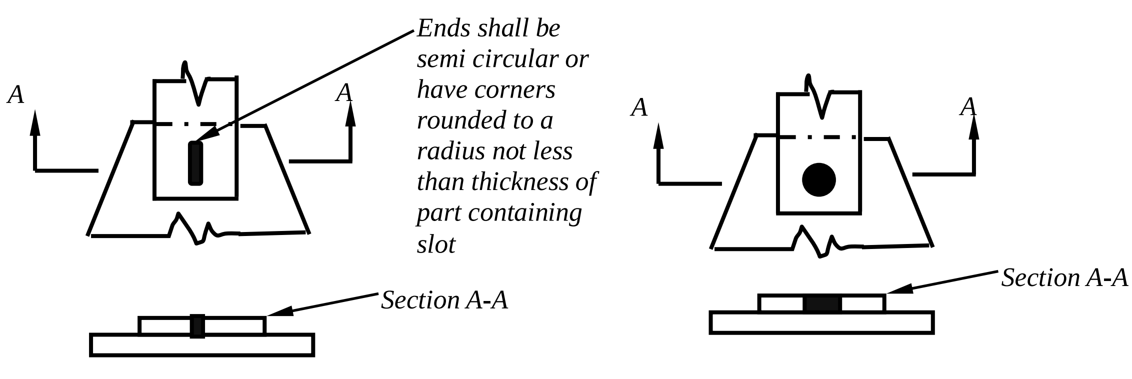
Fig. 10 End returns

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. 10. 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.

2.4 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. 11(a). Another way of developing the required strength is by providing slot or plug welds. Slot and plug welds [Fig. 11(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.



(a) Slot weld *(b) Plug weld*

Fig. 11 Slot and Plug welds

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:

$$L = \frac{Load}{(width)\ allowable\ stress} \tag{1}$$

3.0 ECCENTRIC JOINTS

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. 12. These two common cases are treated in this section.

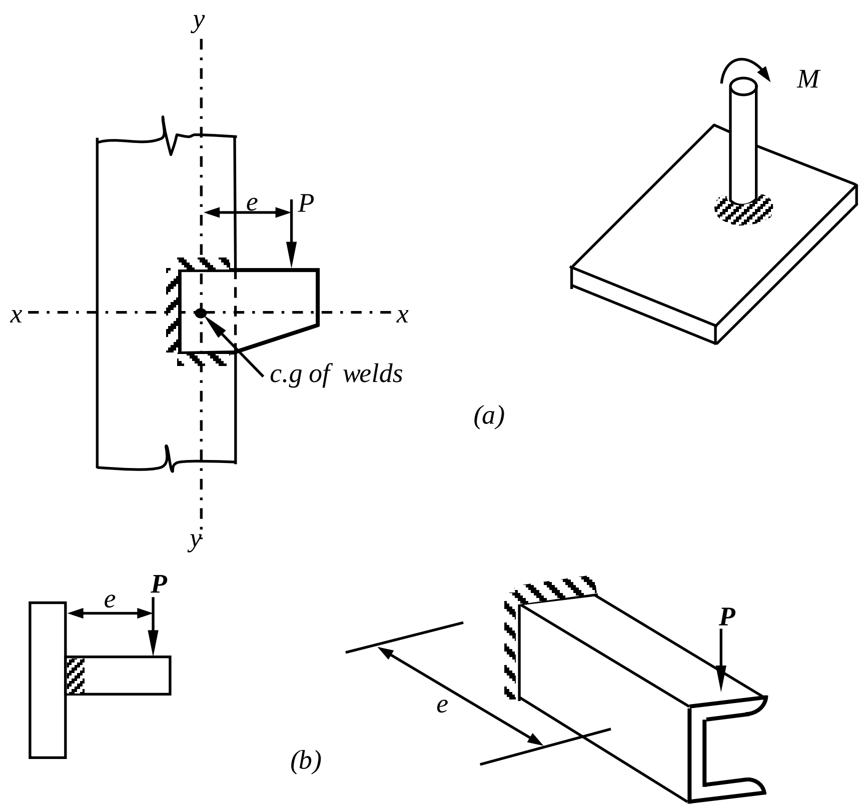


Fig. 12 (a) Welds subjected to shear and torsion,
(b) Welds subjected to shear and bending

3.1 Shear and torsion

Considering the welded bracket shown in Fig. 12(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

$$F = T.s/J = (Moment\ /\ Polar\ moment\ of\ inertia) \qquad (2)$$

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:

$$F_h = Tv/J \quad and \quad f_v = Th/J \qquad (3)$$

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

$$\tau = R/L \tag{4}$$

where, τ 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.

3.2 Shear and bending

Welds, which are subjected to combined shear and bending are shown in Fig. 12(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. 13.

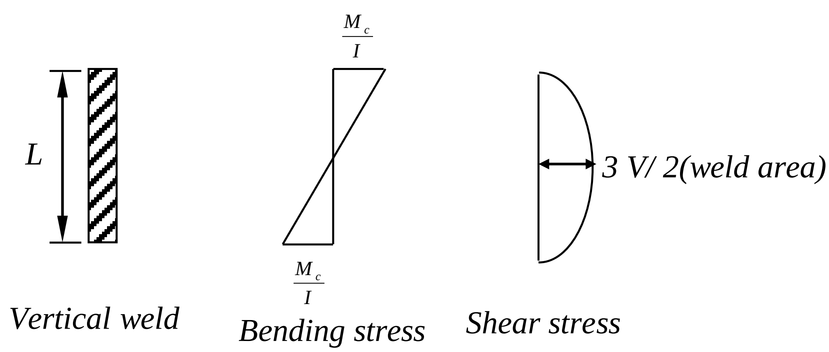


Fig. 13 Variation of bending and shear stress

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. 14, it can be safely assumed that the web welds would carry all the of the shear and the flange welds all of the moment.

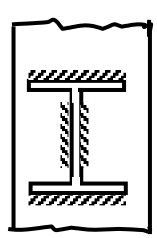


Fig.14 Weld provision for carrying shear and moment

4.0 TRUSS CONNECTIONS

Connections are very important in structural steelwork design. Therefore, the type of connections has to be finalized at the conceptual stage of the design to achieve maximum economy. This is especially true in the case of trusses. Efficient sections for the members alone do not result in economy unless suitable and economic connections are also designed.

Fabrication cost of connections would depend on the following factors.

- Precise cutting to length of sections(minimization of wasted length)
- Requirement of weld preparation
- Requirement of close control on weld root gaps
- Need for stiffening of the connection
- Chosen weld type

If some of the rigours in connections can be avoided, their cost can be reduced contributing to overall economy. Thus, for the economy of the entire truss, close attention has to be paid to connections.

4.1 Planar trusses

In a conventional riveted or bolted trusswork, gusset plates are generally provided at the connections. In a welded truss, it may be possible to omit the gusset plates. Tees or angles with unequal legs are normally used for the top and bottom chord members. The web member angles may be welded directly to the vertical sides of the chord members (Fig. 15).

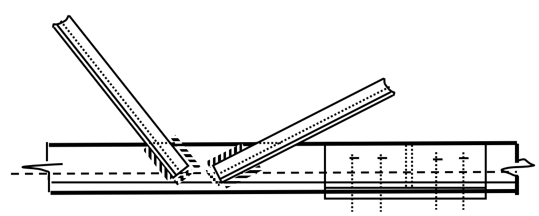


Fig.15 Direct connection of web members

When the trusses are very long, they may have to be fabricated in parts in fabrication shops. Such parts are assembled at site by bolting. Welded connection at the apex of a roof truss is shown in Fig. 16.

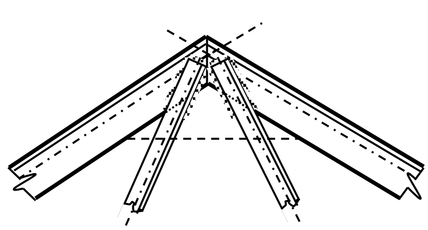


Fig.16 connection at the apex of a roof truss

The two rafters may either be butt welded together or a small plate is introduced at the connection facilitating fillet welding. If Tees are used for the rafters, the web members can be welded directly to the stalk of the Tees. Splices are sometimes provided in welded trusses for the purpose of transportation.

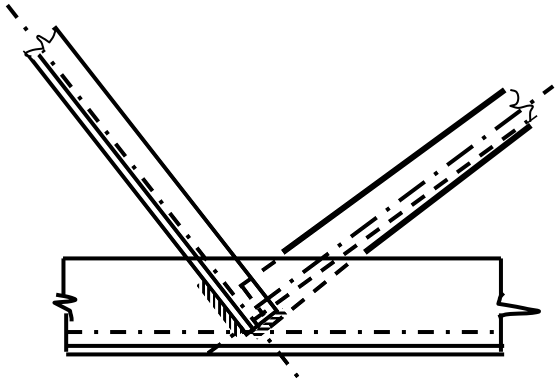


Fig. 17 typical welded truss connection

It is often possible to forego the use of gusset plates by joining the members eccentrically. This provision makes the trusses economical by making the connection simple; the resulting penalty on the member size is marginal. Alternative method to avoid eccentricity is to join the web members on opposite sides of the truss [Fig. 17]. A typical eaves level connection is shown in Fig. 18. For low-pitch roofs, this would produce a long connection. To avoid such a connection, often the connection is truncated and the resulting eccentricity is accounted for in the design.

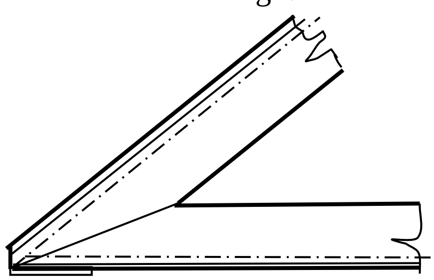


Fig.18 Eaves connection

4.2 Eccentricities in truss connections

Eccentricity in truss connections can occur due to two reasons: 1) element centroidal axes not intersecting at a point, and 2) connection centroid not coinciding with the element centroid.

Figure 19 shows the two cases of eccentricity. All the members joining the connection resist the moment caused by the eccentricity, *Pe*. It is distributed between them in proportion to their bending stiffness per unit length (*I/l*). Each member connecting to the joint should be designed to withstand the axial forces and its share of bending moment. If the connection centroids do not coincide with the member axes, they will be subjected to additional moments.

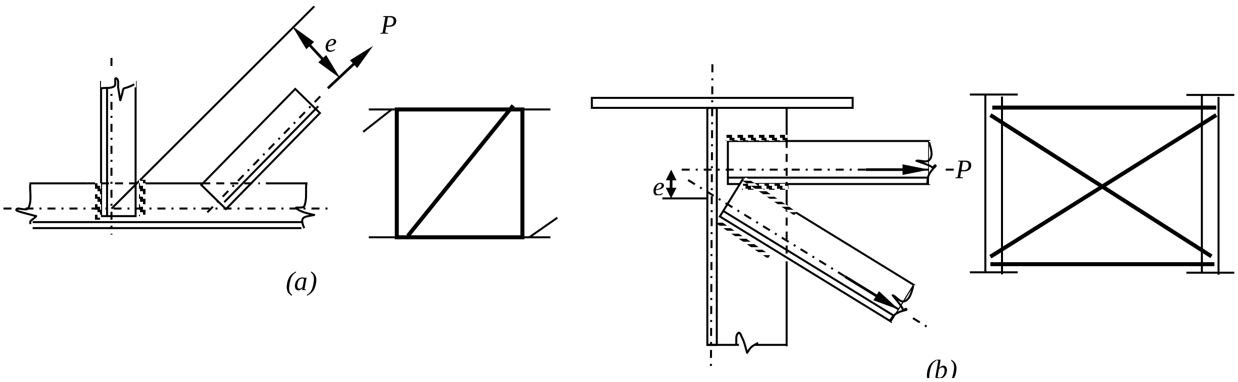


Fig.19 Eccentricities in truss connections (a) Pratt truss, (b) cross bracing between plate girders

A simple way to consider both moments due to eccentricity is to design for axial load and then use interaction diagrams for weld groups subject to combined loading to determine the increase in weld size required to include the moments. It is often observed that there is minimal reduction in axial capacity. This could be comfortably absorbed in the factor of safety usually assumed for connection design.

5.0 PORTAL FRAME CONNECTIONS

5.1 General

Portal frames are widely used for steel industrial buildings. They are used in various spans and heights. However, the frame spacing is often limited to the range between 4.5 m and 7.5 m. Lot of optimisation studies of frame connections have been carried out both in theory and practice and the results of the study have been incorporated in the design of connections.

5.2 Connection locations

Connections in frames are provided at the eaves and apex locations. Based on different types of analysis (elastic, plastic etc.), it is known that critical moment generally occurs at the eaves. An elastic analysis with rolled prismatic sections does not provide economy due to the poor utilisation of the rafter capacity. A plastic analysis leads to redistribution of moments and generally a lighter section for the rafter. However, design of connection has to provide the requisite rotation capacity. Due to the difficulty in meeting this requirement, haunched sections are sometimes introduced. The added benefit of using lighter sections for rafters lead to considerable economy. Presently, non-prismatic members, fabricated by automatic means provide large economy; they also possess the required strength.

5.3 Connection at eaves and apex

Figs. 20 and 21 show two principal ways of transferring moments around the corner. For ease of understanding, the moments are replaced by a pair of forces in the tension and compression flanges of the beam and column. In the first case, equilibrium is achieved by shear on the corner panel. If a diagonal stiffener is added, then the overall equilibrium is obtained by the system of forces, shown in Fig. 21.

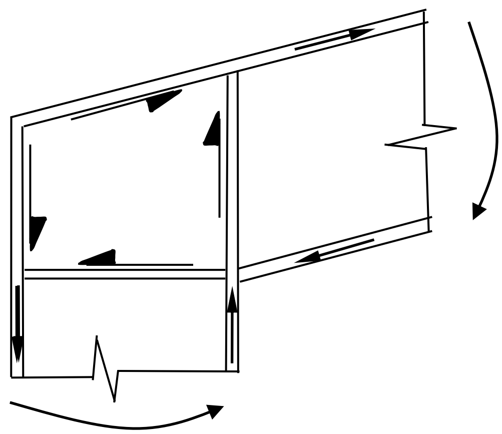


Fig.20 Moment transmission at corner using a shear panel

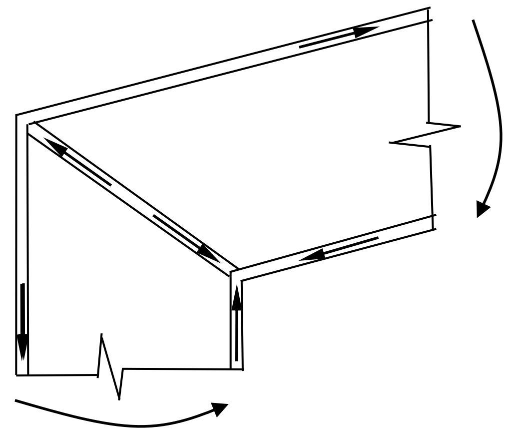


Fig.21 Moment transmission at corner using a diagonal stiffener

Various ways of providing welded eaves connection are shown in Fig. 22.

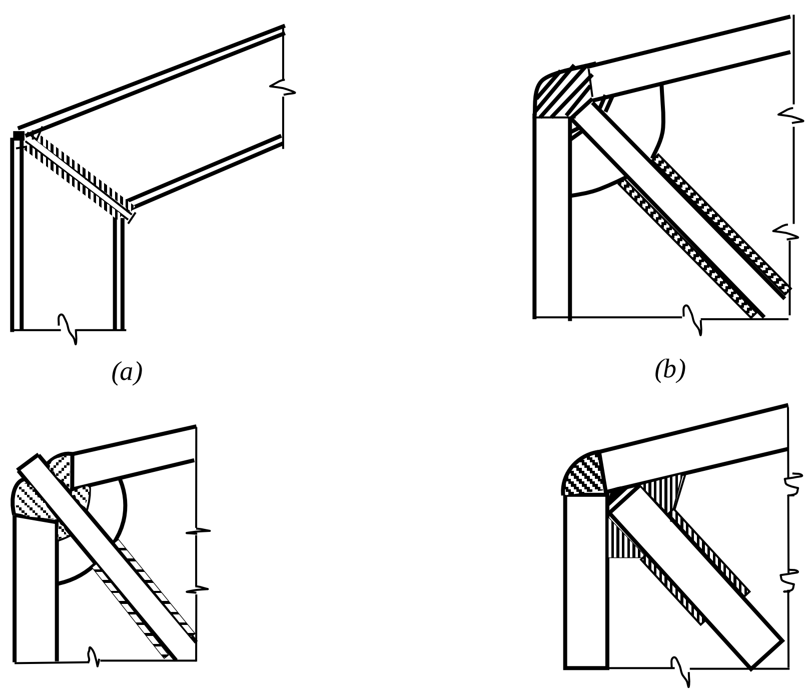


Fig.22 Various eaves connections of portal frames

The most efficient method is shown in Fig. 22(*a*). Here, a diagonal stiffening element, 10-15 mm thickness, is provided. The butt weld configuration for the tension flange is shown in Fig. 22(*b*). For thicker dimension plates, the weld configuration in Fig. 22(*c*) may be used. Figure 22(*d*) shows a welded connection where the butt joint is made initially and then the stiffeners are added.

Welded apex connections are made very similar to eaves connections and the various considerations are also same. Various butt- and fillet- welded connections are shown in Fig. 23.

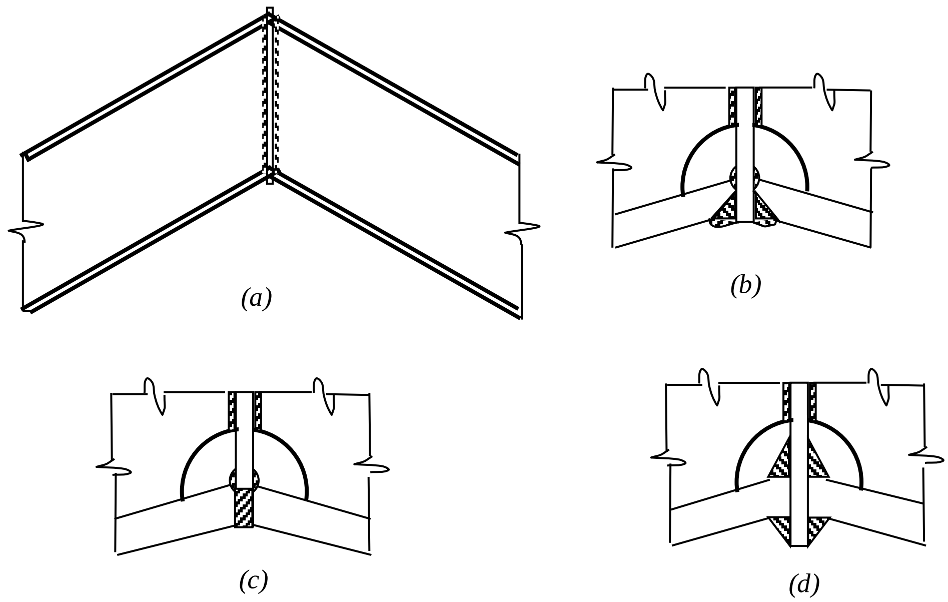


Fig. 23 Welded apex connections. (a) General arrangement (b) – (d) different approaches to the tension flange connection

6.0 COLUMN AND BEAM SPLICES

6.1 General

The need for splices arises due to the reason that structural sections are available only in specific lengths. Splices are provided, as far as possible, at locations away from critical sections. In the cases of beams and columns, the section at which the bending moment is minimum is chosen for locating splices. For beams, the section at splice location should have a higher capacity than the shear force at the location. If the member being spliced is subject to instability, such as in a column, the splice is necessarily to be located close to a point of restraint. If, for any reason, the splice needs to be provided away from a restraint, it may require special consideration.

6.2 Column splice

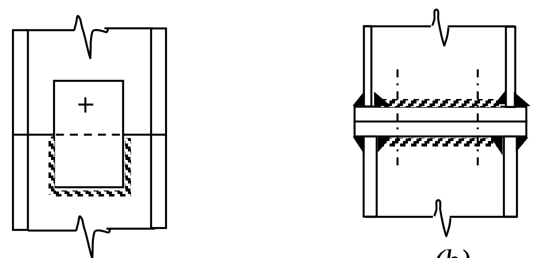
If a compression member is loaded concentrically, then theoretically that member does not need splicing. Compression will be transmitted from a top member to the bottom one by direct bearing. Column sections could be kept one on top of another and they function satisfactorily. However, this ideal case cannot be adopted in practice due to various reasons. Firstly the load is never truly axial; secondly the bearing surfaces of adjacent sections, however well prepared, are not perfect. Also, a real column is subjected occasionally to either laterally applied loads or eccentric loads. Hence, adjacent members of the column should be definitely connected.

6.2.1 Butt welded splices

Full penetration welds can be provided; however, it is economical to make the column ends flush with each other and apply partial penetration welds [Fig. 24(*a*)]. Direct bearing will transfer most of the load if the ends are faced.

6.2.2 Welded splice plate joint

It is possible to make use of welded splice plates for making column splices. They are used at column ends, which are faced for bearing [Fig. 24(b)].



(a) *(b)*

Fig. 24 (a) butt-welded column splices, (b) welded splice plate connection

6.3 Beam and girder splices

Rolled beams are fabricated in one piece and they do not generally need splicing. But, in very long beams, splices may be required. In such cases, splices are located in regions of the beam where the bending moment is low. The structural requirements of these regions are also not very stringent. Requirements for rolled beams are simpler compared to those for plate girders.

Splicing of webs is very common in plate girders. This may become necessary due to several reasons. Few of them are: 1) the required length of plate is not available, 2) the girder may be cambered at the splice, and 3) the thickness of the girder may be varied. When splicing is done in the shop, the web can be spliced independently of the flanges. In other words, flanges need not be spliced at the point of web splicing.

In practice, there are two main methods for the design of beam splices. In one case, the splice is designed to resist the calculated moment and shear at the point of splice. The capacity of the designed splice may be much less than that of the full capacity of the section. In the second case, the splice may be designed to develop full resistance of the section in both shear and flexure, though at the location of the splice, the actual moment and shear will be much lower.

6.3.1 Types of beam splice

6.3.1.1 Butt-welded connections

The simplest, but the most expensive, form of connection is by connecting the elements with full-strength butt welds. Due to the high cost, its use is limited to cases where aesthetics are important. Proper arrangement is necessary for temporary support during welding. The staggered form as shown in Fig. 25 is often recommended. For good alignment, welds are omitted for a short distance on either side of the connection. They are completed after the main welding is over. In order to minimise distortion effects due to transverse weld shrinkage, flange welds are done before web welds.

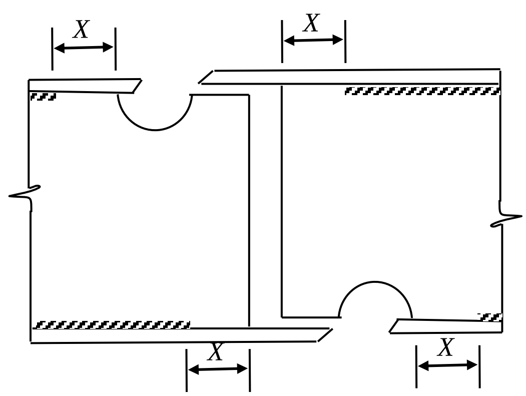


Fig.25 Staggered form of arrangement for temporary support

6.3.1.2 Welded splice plate connection

For lightly loaded beams, splice plates can be used for connections (Fig. 26). This would overcome the necessity of good alignment for butt-welded connections. But provision of double splice plates for this method of connection is difficult and hence limited to light beams.

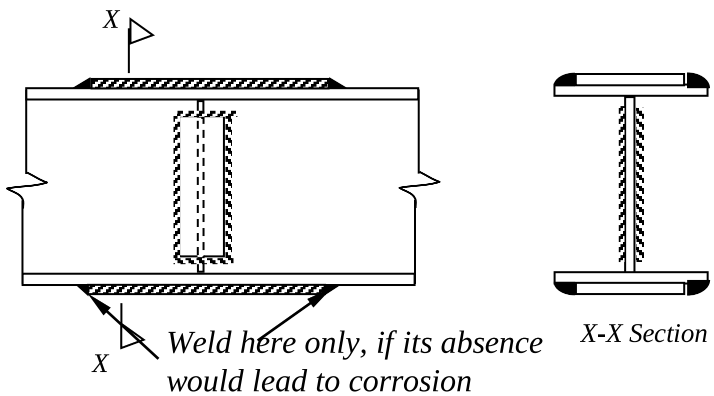


Fig.26 Welded splice plate beam connection

6.4 Hybrid connections

In this form of connection with splice plates (Fig. 27), the plates are attached to half of the splice by welding (mostly done in fabricating shops) and field bolting makes the other half of the connection. HSFG bolts are generally used for this connection to provide good stiffness. Since only single plates can be used, this type of connection has to be restricted to light beams. As both welding and drilling of holes are necessary at the fabrication shops, this type of connection is not very common.

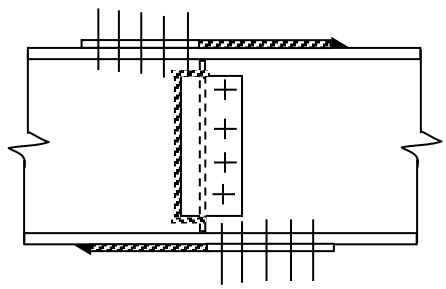


Fig.27 Hybrid connection

7.0 WELDED BEAM-TO-BEAM AND BEAM-TO-COULMN CONNECTIONS

This section deals with beam-to-beam and beam-to-column connections commonly used in steel buildings.

7.1 Types of beam connections

According to their rotational characteristics under load, beam-to-column connections are broadly divided into three classes: simple, semirigid and rigid. A connection, which is free to rotate and has no moment resistance, is called a simple connection. On the other hand, a rigid connection has complete moment resistance and does not rotate at all. A semirigid type connection has moment resistance, whose value falls in between the simple and rigid types.

In reality, connections are never completely rigid or flexible. They are always in between these two extreme cases. The classification of connections, as presented above, are made on the basis of a percentage of moment developed to complete moment resistance. An approximate rule for classification is:

- 0-20% moment resistance – simple connections
- 20-90% ,, – semirigid connections
- > 90% ,, – rigid connections

Rotational characteristics of connections cannot be obtained theoretically; experimental studies are conducted and moment-rotation relationship curves are plotted for each type of connection. Each of the three types of connections is briefly described in the following.

7.1.1 Simple connections

These are very flexible and under loads, the beam tends to rotate freely downwards by a large amount. Although this assumption of rotation is generally made, there is some amount of moment resistance, which is neglected. They are assumed to resist shear only. Examples of flexible connections are shown in Fig. 28. A common practice is to shop-weld the web angles to the beam web and field-bolt them to the column.

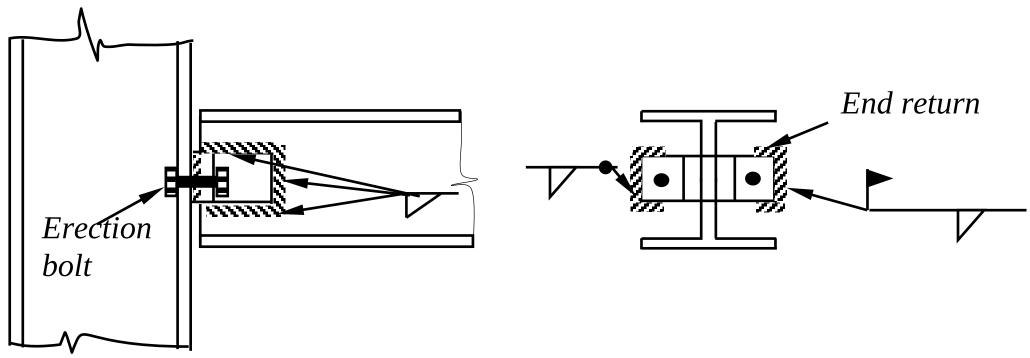


Fig. 28 Framed simple connection

7.1.2 Semirigid connections

These types of connection offer considerable resistance to end rotation and hence develop end moments (Fig. 29). In design practice, the designer assumes that the connections are either fully rigid or fully flexible in order to simplify the analysis. If the designer were to consider the actual nature of connection, namely semirigid, he could bring about considerable amount of moment reduction as shown in Fig. 30. As shown in the figure, the beam with different percentages of rigidity is loaded uniformly.

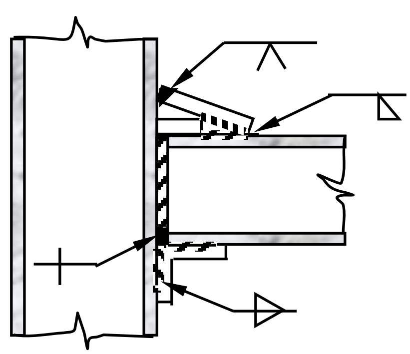


Fig. 29 Welded semirigid connection

Though most of the connections are semirigid, no moment reduction is assumed in calculations. Two main reasons for doing so are:

- it is very difficult to show how much moment reduction a connection is capable of providing, and
- There is no easy method of analysis to consider the varying percentages of moment restraint.

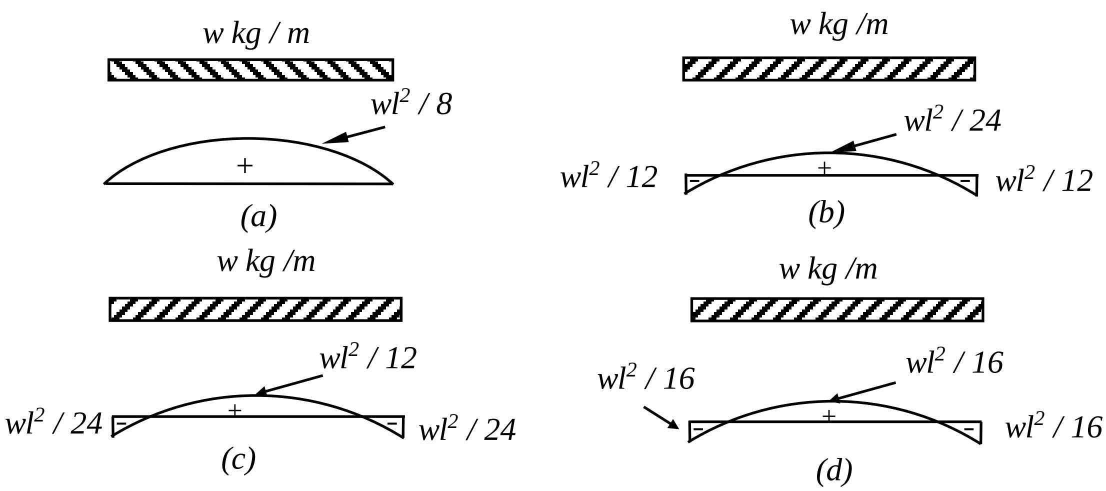


Fig.30 (a) simple connections(0%) (b) rigid connections (100%) (c) semi rigid connections (50%) (d) semirigid connections (75%)

7.1.3 Rigid connections

Rigid connections do not allow rotation at the beam ends and hence transfer the moment fully of a fixed end. This type of connection is usually provided in tall buildings, for which wind resistance is developed by the continuity of members provided by the rigidity of the connection. A welded rigid joint is shown in Fig. 31.

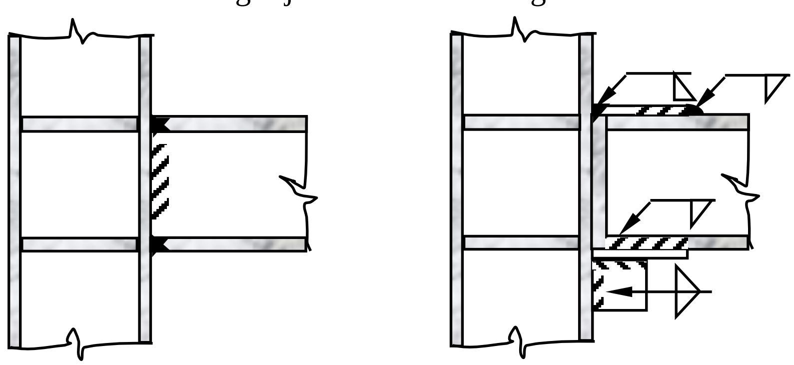


Fig.31 Welded moment - resisting connection

7.2 Types of welded beam connections

There are several methods of making welded connections between beams and girders, and between beam and columns. Some of them are:

- Web angles
- Beam seats
- Stiffened beam seats, and
- Moment resistant connections

While the first three are designed as simple connections with only shear transfer (known as shear connections) the last category transfers moment also (referred to as moment connections)

For designing a connection, a designer has to clearly understand the force paths in the structure and their transmission through the connections. Two of the very common examples are:

- Bending forces are resisted mostly by beam flanges and to transfer these forces the welds must be provided at the beam flanges
- To transfer shear forces in the beam, which occur primarily in the webs, the weld has to be positioned on the webs.

While designing welded connections, the designer has an option to weld the member directly to the other member without using connecting plates or angles. Direct connection of members is often difficult due to lack of proper fit in the field. Therefore, members are connected using connecting plates or angles, if necessary by clamping. Further, direct connections may not offer flexibility necessary in shear connections.

7.2.1 Welded web angles

Beams, which are connected to either columns or girders by means of web angles (Fig.32) are assumed to have simple end supports. These web angle connections are designed to transmit shear only and do not provide any resistance to rotation. Generally, these angles are shop-welded to the beam and then field-connected to the columns or girders with high strength bolts. The distance by which the angles project out from the beam web (usually about *12 mm*) is known as setback.

Erection bolts are used to erect these beams and often they are located at the bottom of the angle (Fig. 32) so that they induce least rigidity to the angles. For some reason, if they are provided at the top, they can be removed after the erection of the beam.

In order to achieve simply supported end conditions, the beam ends should be capable of rotating; to facilitate this only thin angles (e.g ISA 90×90×6), which can deflect easily are used for making the connection. Generally, *100 mm* size legs are used for connecting the beam and the leg size at the column or girder side is a little longer. The length of the angle is usually kept equal to the beam depth, which must also be sufficient for welding. The weld size will be 1 to 3 mm smaller than the web angle thickness. The required maximum weld length is the beam depth minus its width.

Though it is often assumed that the web angles are subject to only shear force, which is equal to the end reaction, it is not so. There will be also moment due to eccentricity of the reaction forces with respect to both shop and field welds, as shown in Fig. 33.

Due to this rotation effect, the field welds cause web angles to press against the beam web at the top and tear apart at the bottom, thus inducing horizontal shear in the fillet weld. General practice is to assume the neutral axis is located at the distance *1/10 L* from the top of the angle. The horizontal shear at *1/10* height is taken as zero and maximum at the bottom of the angle.

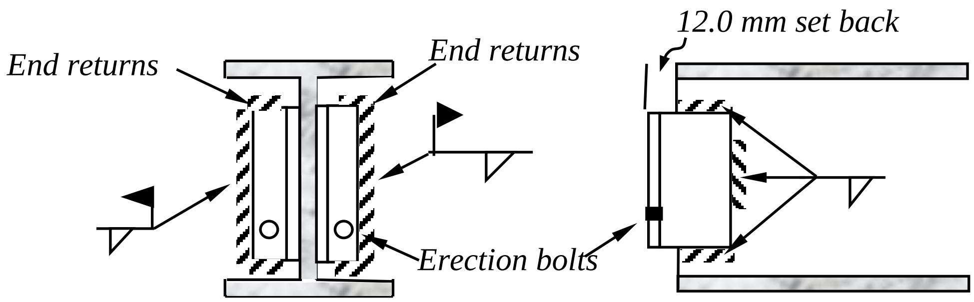


Fig. 32 Welded web angles

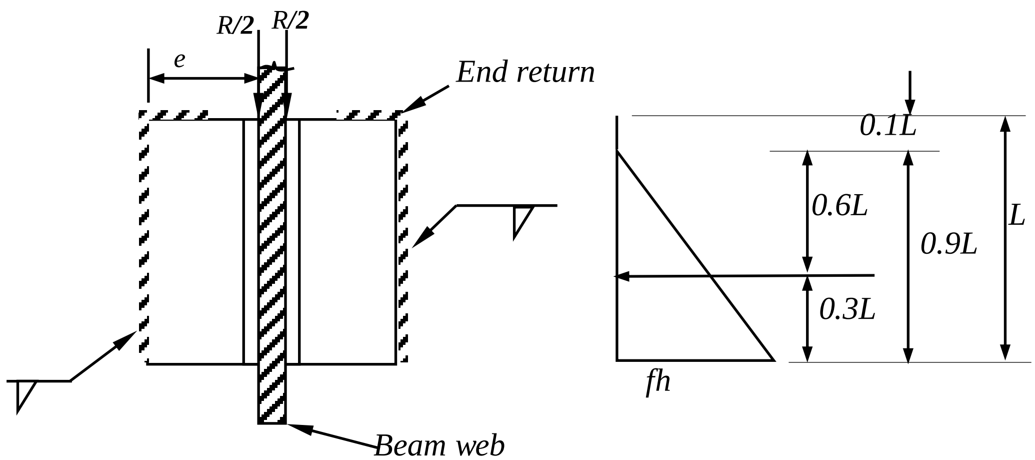


Fig. 33 eccentricity of reaction forces

The top pressure acts at the *1/10 th* point from the top and the centre of gravity of the horizontal shear is located at *0.7 L* from the top of the angle. Equating the moment produced by these forces and the externally applied moment, the value of the horizontal shear f_h can be determined.

$$0.5 \times 0.9\, L \times f_h \times 0.6\, L = R/2 \times e \tag{5}$$

$$f_h = Re/0.54\, L^2 \tag{6}$$

The vertical shear stress f_s in the weld will be *R/2* divided by the height of the weld. The weld size can be determined after calculating the maximum stress on the weld by the equation

$$F_r^{\,2} = f_h^2 + f_s^2 \tag{7}$$

7.2.2 Welded seated beam connections

This is a flexible type of connection (Fig. 34). The beam seat is normally shop-welded to the column and after seating the beam properly it is field-welded or bolted. Seat angles, also known as shelf angles, are provided with bolt holes for erection purposes. Normally, an angle is welded at the top of the beam also to provide the required lateral support; no load is carried by this angle. Flexible angles (say 100 x 100 x 6 mm) are provided at the top to allow the beam to rotate under the imposed load.

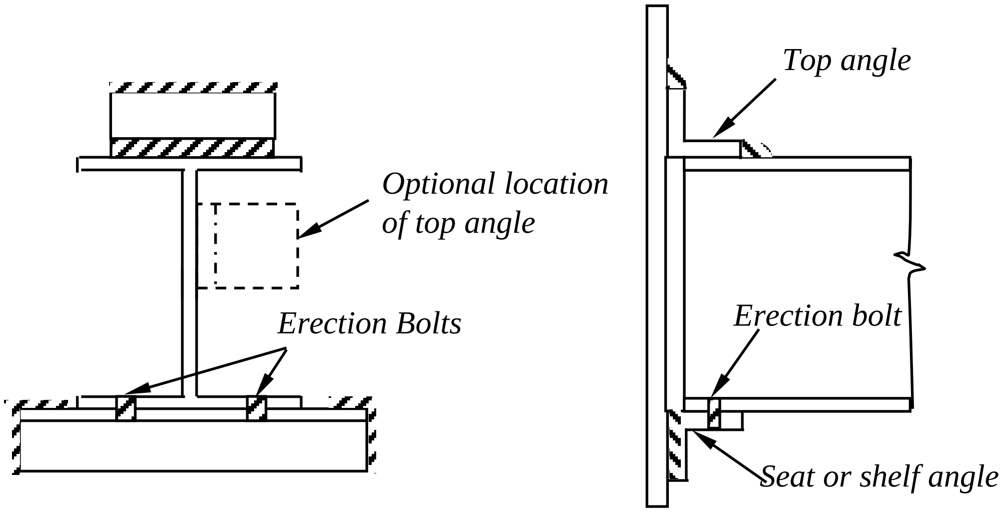


Fig. 34 Welded seated- beam connections

For relatively light loads, only the two vertical ends of the seat angle need be welded. For allowing the beam to rotate, the top angle is welded only on its toes. The bearing length normally provided is 75 to 100 mm. The horizontal leg of the angle is subjected to a moment due to the beam reaction. The critical section for bending is assumed to be at the toe of the fillet weld, which is approximately t+ 10 mm from the back of the vertical leg. The length of the vertical leg can be calculated from the weld size required. This is often done by trial and error after assuming an initial depth of the weld. The stress variation in the vertical welds is normally assumed such that the neutral axis occurs at mid depth.

7.2.3 Welded stiffened beam seat connections

For relatively heavy beam connections (above 200 kN to 250 kN), stiffened beam seats are used. Stiffened beam seats are generally made of Tee sections or plates welded to the shape of a Tee (Fig. 35). The stiffened beam seat is designed to provide sufficient bearing length considering the web crippling of the beam. As the beam is loaded and it rotates, the centre of gravity of the beam reaction moves away towards the outer edge of the seat. As an approximation, the centre of gravity of the reaction is assumed at the centre of the bearing length. The stems of the Tee sections are provided a thickness equal to the web thickness of the beam. The depth of the stem is arrived at depending upon the weld length required. The flange of the beam seat is kept a little wider than the beam flange to provide simple field welds. A minimum of two weld size extra width may be provided on either side of the beam flange.

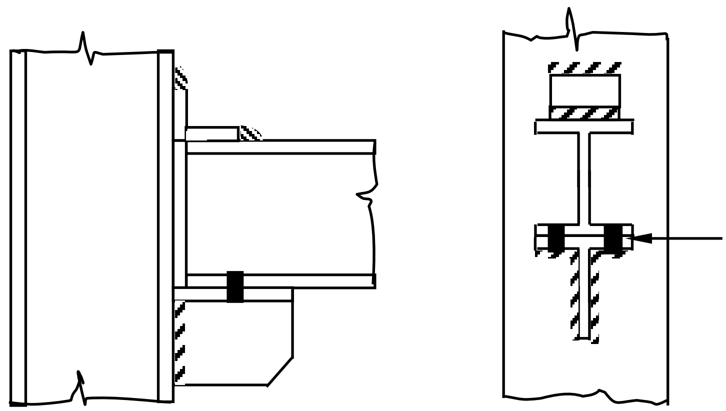


Fig. 35 Stiffened beam seat connection

The weld for the seat must be designed to resist the applied shear and the bending moment. The vertical welds being very close to each other cannot resist flexure. Therefore, the bottom side of the flange is welded to a distance of ¼ to ½ of the web depth. These horizontal welds significantly improve the resistance of the connection to twisting. No weld can be provided at the top of the flange because of the possible interference with beam.

7.2.4 Moment-resistant connections

Fig.36 shows a moment-resistant connection. Such connections are used for fully continuous structures, where the connections are designed to resist full moments. The efficiency of a welded connection is fully utilized in this type of connection. Firstly, the negative moments at the supports tend to reduce the positive moments at midspan resulting in usage of smaller size members. Secondly, in case of overloading of structures, the plastic redistribution is possible in continuous structures.

For the connection in Fig. 36(*a*), the tensile force (caused by hogging moments) at the top flange of the beam is transferred to the top flange plate by fillet welds and from the plates to the column by groove welds. The top plate is often provided a taper for ease in welding [Fig. 36(*b*)]. Under gravity loads, the force on the welds will be tension T in the top flange and compression C in the bottom flange. In case lateral forces are considered, the welds must be designed to resist both tension and compression. While providing

moment-resisting welded connections, it is usual to butt weld the beam flanges flush with the column at one end and connect the beam at the other end with the connection details as described above.

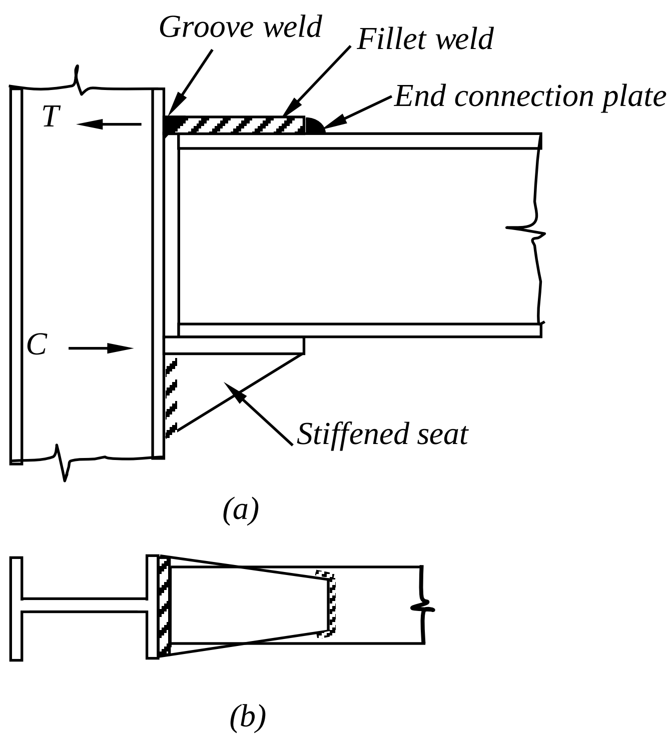


Fig.36 (a)&(b) Moment resisting connections

Two possibilities may arise whereby the moment resistance of the connection maybe reduced. The bending of the column at the connection point would reduce the moment resistance. The top connection plate while pulling away from the column tries to bend the column flange [Fig.37 (*a*)] and the middle part of the weld is overstressed. For these reasons, the column flange is normally stiffened with plates opposite to the beam flange [Fig. 37(*b*)]. The connection between different parts of a built-up beam or plate girder should be proper in order to meet to the demand.

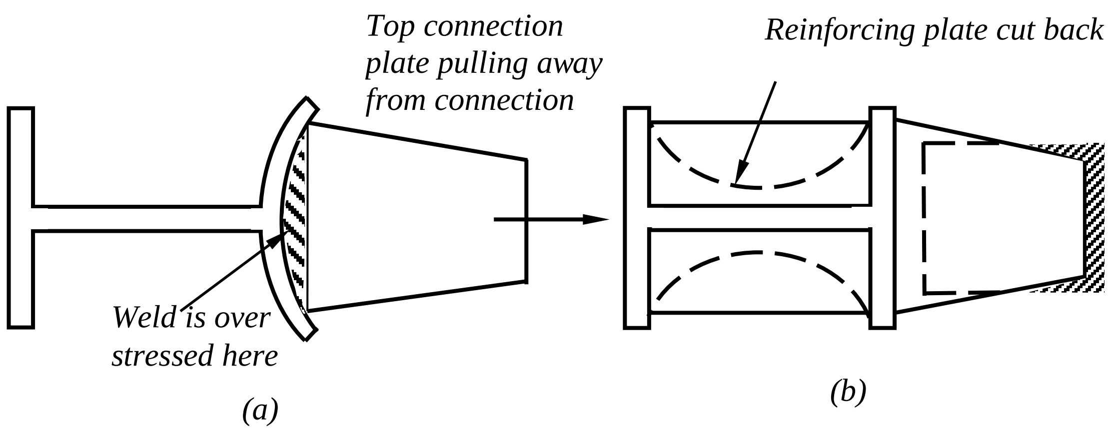


Fig. 37 (a) Overstressing of the weld, (b) Column flange Stiffened with plates

8.0 INSPECTION

As pointed out in a previous chapter, it is vital that all welded connections must be inspected to ensure that they conform to specifications. Good welding procedures can be formed from the relevant codal provisions and the guidelines from the manufacturers of welding supplies and equipment. The recommended procedures, if followed, would ensure sound welds.

The inspection and control should start just from the start of welding and continue through the welding procedure. If essential, a pretest of the connection should be done to ensure required performance.

9.0 CONCLUSIONS

In this chapter, behaviour and design of major types of welding, namely butt and fillet are explained in detail. The necessity and methods of providing beam and column splices are described. Truss connections are presented. Types of beam-to-beam and beam-to-column welded connections are described. Practical methods of providing welded connections are also presented. Worked examples are included for clarity.

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