KIITPOLYTECHNIC

LECTURE NOTES

ON

STRUCTURAL DESIGN-II

Compiled by

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Chapter-1

Introduction

1.1 structural steel

Structural steels are commonly used in steel construction, namely, structural mild steel and high tensile structural steel.

All the structural steel used in general construction, coming under the purview of this standard shall before fabrication conform to IS 2062.

Structural steel other than those specified in 1.4.2 may also be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

Steel that is not supported by mill test result may be used only in unimportant members and details, where their properties such as ductility and weldability would not affect the performance requirements of the members and the structure as a whole. However, such steels may be used in structural system after confirming their quality by carrying out appropriate tests in accordance with the method specified in IS 1608.

Properties

Physical properties of structural steel irrespective of its grade may be taken as:Unit mass of steel, p = 7850 kg/m~

Modulus of elasticity, E = 2.0 x 10s N/mm2 (MPa)

Poisson ratio, p = 0.3

Modulus of rigidity, G = 0.769 x 10s N/mm2 (MPa)

Co-efficient of thermal expansion cx.= 12 x 10’ /“c

Mechanical properties of structural steel

The principal mechanical properties of the structural steel important in design are the yield stress, fy; the tensile or ultimate stress, fu; the maximum percent elongation on a standard gauge length and notch toughness. Except for notch toughness, the other properties are determined by conducting tensile tests on samples cut from the plates, sections, etc, in accordance with IS 1608. Commonly used properties for the common steel products of different specifications are summarized in Table 1.4

Table 1.4 Tensile Properties of Structural Steel Products

Sl No.	Indian Standard	Grade/Classification	Properties: Yield Stress MPa, *Min*			Ultimate Tensile Stress MPa, *Min*	Elongation, Percent, *Min*
(1)	(2)	(3)	(4)			(5)	(6)
			d or *t* < 20	20-40	> 40		
viii)	IS 2062	E 165 (Fe 290)	165	165	165	290	23
		E 250 (Fe 410 W) A	250	240	230	410	23
		E 250 (Fe 410 W) B	250	240	230	410	23
		E 250 (Fe 410 W) C	250	240	230	410	23
		E 300 (Fe 440)	300	290	280	440	22
		E 350 (Fe 490)	350	330	320	490	22
		E 410 (Fe 540)	410	390	380	540	20
		E 450 (Fe 570) D	450	430	420	570	20
		E 450 (Fe 590) E	450	430	420	590	20

2.1 **Concept of Design of steel structures**

Design problems are seldom amenable to solution by exact mathematical formulae. There is a considerable scope for exercising engineering judgement. Hence, there is no “correct solution” to a design problem, as there could be several so-called “correct solutions” to the same problem. This is because

- the designs are invariably subject to individual interpretation of Standards and Codes,
- the solutions are also subject to differing ideas about what is or what is NOT required from an engineering and environmental stand point, and
- the individual designers have ingrained ideas from their past experience, which may be valid to-day only to a limited extent, or may not be valid at all.

(Stress due to dead load + live load) < allowable stress
(Stress due to dead load + wind load) < allowable stress
(Stress due to dead load + live load + wind) < 1.33 times allowable stress.

In practice there are severe limitations to this approach. These are the consequences of material non-linearity, non-linear behaviour of elements in the post-buckled state and the ability of the steel components to tolerate high theoretical elastic stresses by yielding locally and redistributing the loads. Moreover the elastic theory does not readily allow for redistribution of loads from one member to another in a statically indeterminate structures.

3.1 Limit state design

An improved design philosophy to make allowances for the shortcomings in the “Allowable Stress Design” was developed in the late 1970’s and has been extensively incorporated in design standards and codes formulated in all the developed countries. Although there are many variations between practices adopted in different countries the basic concept is broadly similar. The probability of operating conditions not reaching failure conditions forms the basis of “Limit States Design” adopted in all countries.
“Limit States" are the various conditions in which a structure would be considered to

have failed to fulfil the purpose for which it was built. In general two limit states are considered at the design stage and these are listed in Table 3.1.

Table 3.1: Limit States

Limit State of Strength	Serviceability Limit State
Strength (yield, buckling)	Deflection
Stability against overturning and sway	Vibration
Fracture due to fatigue	Fatigue checks (including reparable damage due to fatigue)
Plastic collapse	Corrosion
Brittle Fracture	Fire

"Limit State of Strength" are: loss of equilibrium of the structure and loss of stability of the structure. "Serviceability Limit State" refers to the limits on acceptable performance of the structure.

Not all these limits can be covered by structural calculations. For example, corrosion is covered by specifying forms of protection (like painting) and brittle fracture is covered by material specifications, which ensure that steel is sufficiently ductile.

3.2 Partial safety factor

The major innovation in the new codes is the introduction of the partial safety factor format. A typical format is described below:

In general calculations take the form of verifying that

$S^* < R^*$

where S^* is the calculated factored load effect on the element (like bending moment, shear force etc) and R^* is the calculated factored resistance of the element being checked, and is a function of the nominal value of the material yield strength. S^* is a function of the combined effects of factored dead, live and wind loads. (Other loads – if applicable, are also considered)

In accordance with the above concepts, the safety format used in Limit State Codes is based on probable maximum load and probable minimum strengths, so that a consistent level of safety is achieved.

3.3 Limit state of serviceability

As stated in IS: 800: 2007, Serviceability Limit State is related to the criteria, governing normal use. Serviceability limit state is limit state beyond which service criteria, specified below, are no longer met:

a) Deflection Limit
b) Vibration Limit
c) Durability Consideration

d) Fire Resistance

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1]		WL/EL	AL	DL	LL[1]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+ WL/EL	1.2 1.2	1.2 1.2	1.05 0.53	0.6 1.2	—	1.0	0.8	0.8	0.8
DL+WL/EL	1.5 (0.9)[2]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.

[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

Abbreviations:

DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

Table 3.3: Partial safety factors for materials γ_m

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

Table 3.4 Deflection Limits

Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000

	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane+ wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter storey drift	—	Storey height/300

Table 3.5 Limiting Width to Thickness Ratio

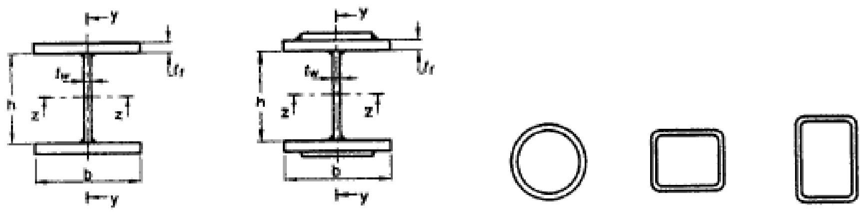
Compression Element (1)			Ratio (2)	Class of Section: Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\le 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\le 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\le 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε

4.1 Structural elements

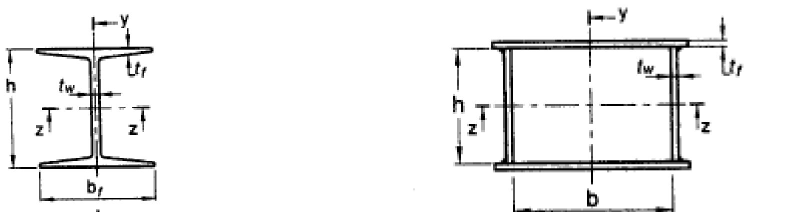
A building structure consisting of a steel frame work skeleton is made up of the following structural elements or members

i) Flexural members; beams or girders
ii) Tension members
iii) Compression members: columns, stanchions, struts
iv) Torsional members
v) Elements of foundation structure

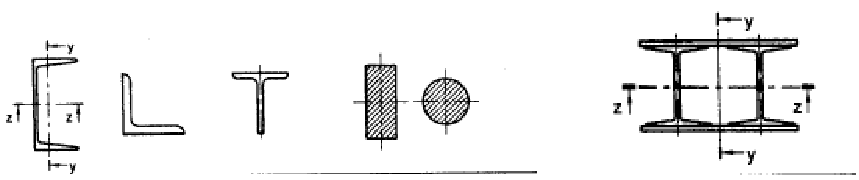
Some elements or members may be subjected to combined bending and axial loads. The members of steel frame are jointed together by riveted, bolted, pinned or welded connections or joints. No matter how complicated a structure may appear to be, it must consist of some combination of the basic members mentioned above. However, flexural members, (or beams) may, in some cases, appear as extremely heavy built-up girders and the compression members (or columns) and tension members (or ties) may be combined to form heavy trusses in an extensive frame work. The structural elements are made up of the following commonly used structural shapes and built-up sections as shown in Figure 4.1 below.



welded I-Section Hollow Section



Rolled I-Sections Welded Box Section



Channel, Angle, T and Solid Sections Built-upMember

Figure 4.1 Commonly used structural shapes

i) Angle section
ii) T-section
iii) Channel section
iv) I-section
v) Z-section
vi) Solid square section
vii) Square tube
viii) Circular section (solid)
ix) Hollow circular section
x) Plate section
xi) Compound and built-up sections

5.1 Structural steel sections

Structural steel is rolled into a variety of shapes and sizes. The shapes are designated by the shape and size of their cross-section. Following are various types of rolled structural steel sections commonly used:

i) Rolled steel beam sections (I-section)
ii) Rolled steel channel sections
iii) Rolled steel angle sections
iv) Rolled steel T-sections
v) Rolled steel bars
vi) Rolled steel plates
vii) Rolled steel sheets and strips
viii) Mild steel flats

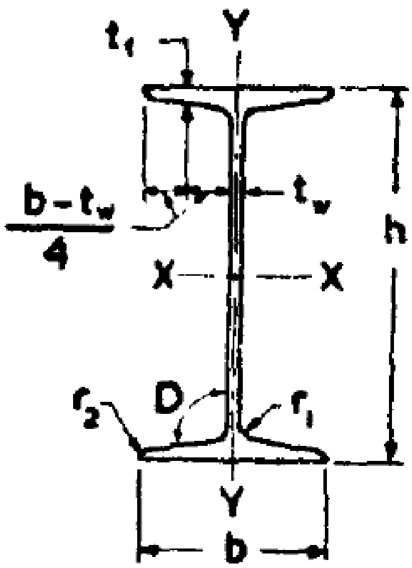


Figure 5.1 Rolled steel I beam

1. Rolled steel beam section (Figure 5.1)
   ISI Hand Book for Structural Engineers Vol 1: Structural steel sections gives five series of beam section

i) Junior beams, designated as ISJB (Indian Standard Junior Beams)
ii) Light Beams designated as ISLB (Indian Standard Light Beams)
iii) Medium Beams, designated as ISMB (Indian Standard Medium Weight Beams)
iv) Wide Flange Beams, designated as ISWB (Indian Standard Wide-Flange Beams)
v) H-Beams or column beams designated as ISHB (Indian Standard H-Beams)

Each beam section is designated by the series to which it belongs followed by the depth (in mm) of the section. For example, ISMB 400 means a beam section of medium weight and of depth equal to 400 mm. In some cases of wide flange beams and H-beams, more than one section is available for the same depth. For example, more than one section is available for the same depth. For example, there are two sections of ISWB600; these two sections are differentiated by writing the mass of the beams per m run. Thus we have ISWB 600 @133.7 kg/m and ISWB 600@145.1 kg/m both of these being two different sections having different properties. Similarly we have ISHB 300@58.8 kg/m and ISHB 300@63.0 kg/m giving two different sections having different geometrical properties.

2. Rolled Steel Channel Sections (Figure 5.2)

ISI hand book gives the following four series of channel sections:

i) Junior channels designated by ISJC (Indian Standard Junior Channels)
ii) Light Channels designated by ISLC (Indian Standard Light Channels)
iii) Medium channels designated by ISMC (Indian Standard Medium Weight Channels)
iv) Special channels designated by ISSC (Indian Standard Special Channels)

Each rolled steel section is designated by the series to which it belongs, followed by its depth (in mm) and then its mass per metre length. Thus we have ISLC 400@45.7 Kg/m, meaning thereby that it is a light channel, having depth equal to 400 mm and mass equal to 45.7 kg/m. A channel section has only one axis of symmetry. Due to this, it is subjected to twisting or torsion, along with bending, when used as a beam.

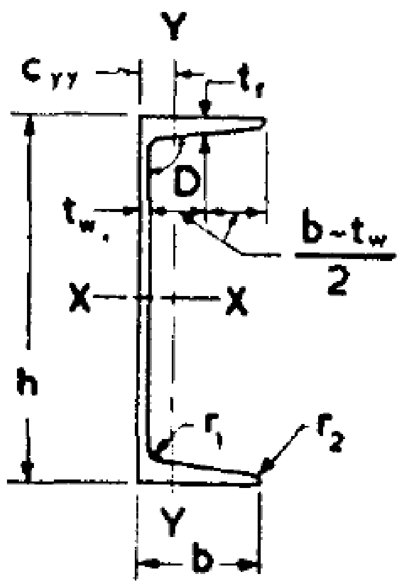


Figure 5.2 Rolled steel channel

3. Rolled Steel Angle Sections (Figure 5.3)

ISI hand book gives three series of angle sections:

i) Equal angles section designated by ISA (Indian Standard Equal Angles)
ii) Unequal angles section designated by ISA (Indian Standard Unequal Angles)
iii) Bulb angle section designated by ISBA (Indian Standard Bulb angles) (Figure 5.4)

Since the equal angle section and unequal angle section are designated by the same series, the width and height of the legs of the angle are also mentioned along with the series. Thus, ISA

4040 will mean an equal angle section, having width and depth equal to 40 mm. Similarly ISA 4025 will mean an unequal angle section having depth equal to 40 mm and width equal to 25 mm. A bulb angle has unequal legs and hence only its depth is mentioned along with the series designation. Thus we have ISBA 300 meaning there by that it is a bulb angle section having its depth equal to 300 mm. However, there may be two bulb sections of the same depth, and these are differentiated by mentioning their mass per metre length. Thus we have ISBA 300 @ 47.5 kg/m and ISBA 300 @52.6 kg/m giving two sections which have different properties. It should be noted that the angle sections mentioned above do not have any axis of symmetry and hence the principal axes u-u and v-v are inclined to x-x and y-y axes.

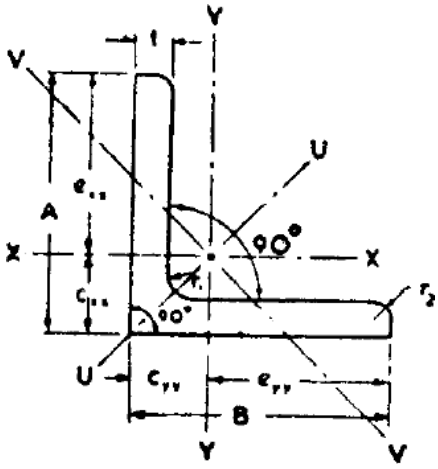


Figure 5.3 Rolled steel angle

4. Rolled Steel T-sections (Figure 5.5)

i) Normal Tee designated by ISNT, having width of flange, equal to the depth of section
ii) H-Tee or wide flange tee, designated by ISHT, having width of flange equal to twice the depth of the section.
iii) Short-Tee designated by ISST, having the width of flange shorter than the depth of the section
iv) Light-Tee, designated by ISLT, having light weight
v) Junior Tee designated by ISJT

Each T-section is designated by the series to which it belongs followed by the depth in mm. Thus we have ISNT 100, meaning there by that it is a normal Tee section, having its depth equal to 100 mm. Similarly ISLT 100 will mean a light Tee section having depth of section equal to 100 mm. It is always preferable to mention the mass per metre length also such as ISNT 100 @ 15.0 kg/m or ISST 200 @ 28.4 kg/m etc.

5. Rolled Steel Bar Sections (Figure 5.6)

ISI Hand book gives rolled steel bars of two types:

i) Round bars designated by ISRO
ii) Square bars designated by ISSQ

Each bar section is designated by the series to which it belongs along with the diameter or the width in mm. Thus ISRO 100 will mean a round bar having 100 mm diameter. Similarly ISSQ 80 will mean a square bar of size 80 mm.

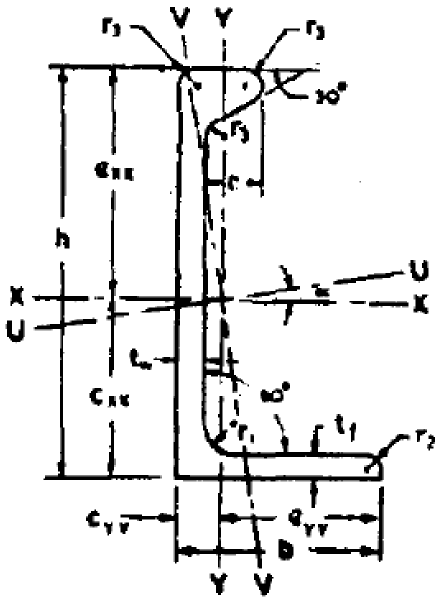


Figure 5.4 Rolled steel bulb angle

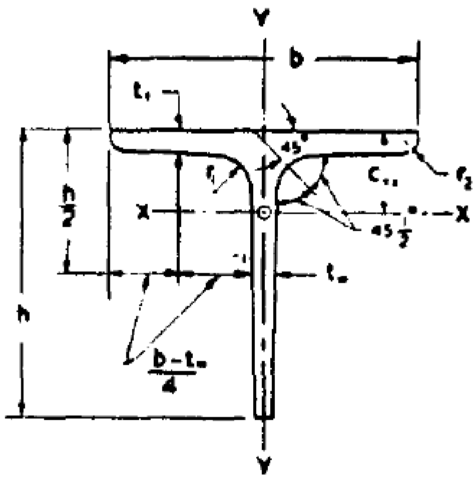


Figure 5.5 Rolled steel Tee bars

Figure 5.6 Square and round bars

Chapter-2

Structural steel fasteners and connections

A structure is an assembly of various elements or components which are fastened together through some type of connection. If connections are not designed properly and fabricated with care, there may be a source of weakness in the finished structure, not only in their structural action but also because they may be the focus of corrosion and aesthetically unpleasing. Where as the design of main members has reached an advanced stage, based upon theories which have been developed and refined, the behaviour of connections is often so complex that theoretical considerations are of little use in practical design. By their very nature, connections are a jumble of local effects. Most connections are highly inderminate, with the distribution of stress depending upon the deformation of fasteners and the detail material. Local restraints may prevent the deformation necessary for desirable stress redistribution.

Following are the requirements of a good connection in steelwork:

1. It should be rigid, to avoid fluctuating stresses which may cause fatigue failure
2. It should be such that there is the least possible weakening of the parts to be joined
3. It should be such that it can be easily installed, inspected and maintained.

The following are the common types of connections used for structural steel work;

1. Riveted connections
2. Bolted connections
3. Pinned connections
4. Welded connections

Rivets, bolts and welds are used extensively, and frequently the economic advantage of one over the other two is so small to be uncertain. However, at one time, riveting prevailed but it has been superseded in importance by welding and high-strength bolting.

6.2 Rivet and Riveting

Riveting is a method of joining together structural steel components by inserting ductile metal pins, called rivets, into holes of the components to be connected from coming apart. A rivet consists of (i) a shank of given length and diameter and (ii) a head known as manufactured head. The size of the rivet is defined by the diameter of the shank. Riveting is essentially a forging process during which a hot rivet is driven in its plastic state and a head isformed at the other end. The head so formed at the other end of the rivet with the help of a riveting hammer and a buckling bar is known as driven head.

Rivets driven in the field during the erection of a structure are known as field rivets. Rivets driven in the fabricating shop are known as shop rivets. Both these types are known as hot driven rivets since the rivets are heated to a temperature ranging between 1000^{o} F to 1950^{o}F before driving. Field rivets are driven by a hand operated pneumatic riveting hammer, while the shop rivets are driven by “bull” riveter. Some rivets are driven at atmospheric temperature. They are known as cold driven rivets which are squeezed or driven to fill the holes and to form the heads by application of large pressure. However, they are smaller in diameter, ranging from 12 mm to 22 mm. Strength of cold driven rivet is more than hot driven rivets. Rivets driven by hand operated riveting hammer are known as hand driven rivets while those driven by power operated equipment are known as power driven rivets. Some times, even the field rivets may also be power driven.

6.3 Types of Rivetd Joints

There are two types of riveted or bolted joints.

i) Lap joint: The first is the lap joint in which the plates to be connected overlap each other. (Figure 6.1)
ii) Butt joint: The second is the butt joint in which the plates are to be connected butt against each other and the connection is made by providing a cover plate on one or both sides of the joint. (Figure 6.2)

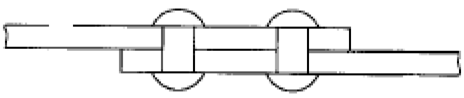


Figure 6.1 Single Riveted lap joint

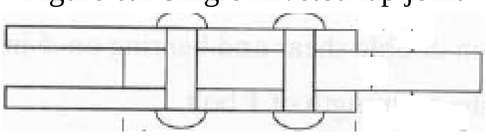


Figure 6.2 Double riveted butt joint

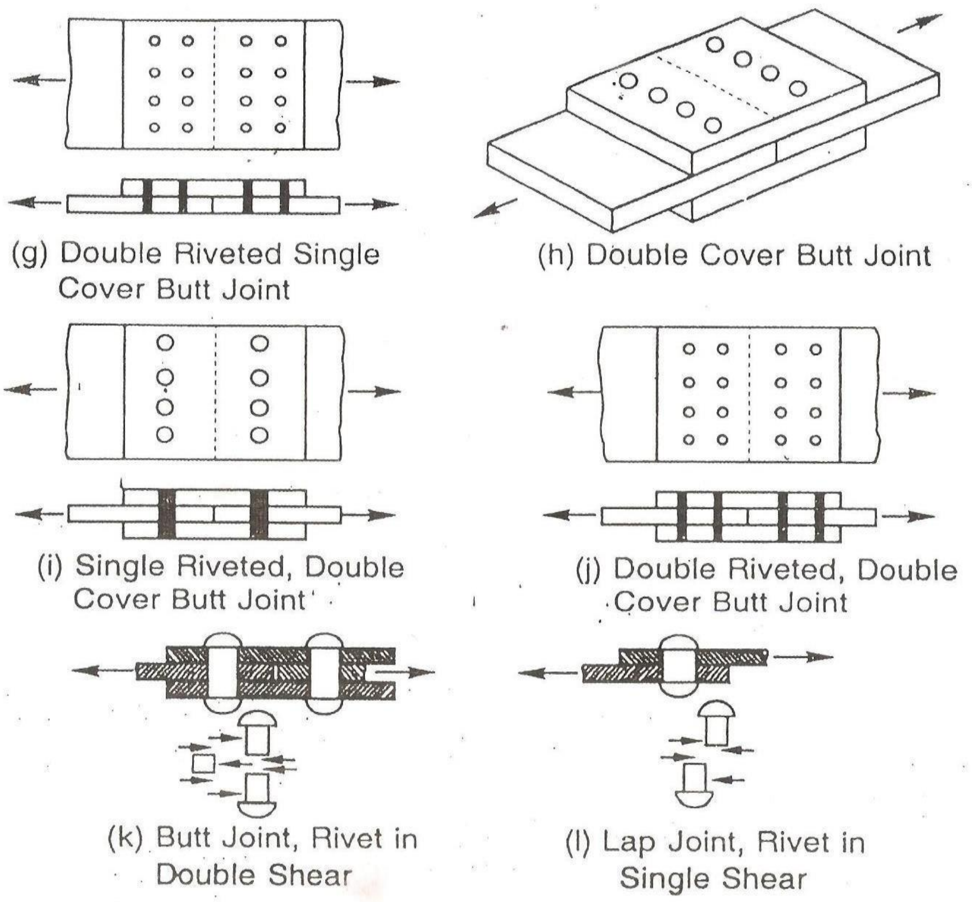


Fig. 2.2 Types of Riveted Joints

6.4 Definitions

The following definitions are used for riveted or bolted joints

i) Nominal diameter: The diameter of the shank of a rivet before riveting is called the nominal diameter.

ii) Effective diameter or gross diameter: The effective or gross diameter of a rivet is equal to the diameter of the hole it fills after riveting
iii) Gross area: The gross area of a rivet is given by its gross diameter.
iv) Pitch: The distance between centres of any two adjacent rivets is called the pitch.
v) Gauge: A row of rivets parallel to the direction of force is called a gauge line. The normal distance between two adjacent gauge lines is called the gauge distance.
vi) Edge distance; It is the distance between the edge of a member or cover plate and the centre of the nearest rivet hole.

The joint may fail in any of the following manners.

i) Tearing of the plate between rivet holes: The strength of the plate is reduced by rivet holes and the plate may tear off along the line of the rivet holes as shown in Figure b. This type of failure is for tension members only.
ii) Shearing of rivet: The rivets fail by shearing if the shearing stress exceeds their shearing strength. In lap joints and single cover butt joints, the rivets are sheared at one plane only. In a double cover butt joint, the rivets are sheared at two planes as shown in Figure c.
iii) Bearing of plate or rivet: The plate or rivet is crushed if the compressive stress exceeds the bearing strength of the plate or the rivet as shown in Figure d.

6.5 Assumptions in the theory of riveted joints

Certain assumptions are made while deriving expressions for the strength of riveted joints as follows:

i) The tensile stress is uniformly distributed on the portions of the plate between the rivets.
ii) The friction between the plates is neglected.
iii) The shearing stress is uniformly distributed on the cross-sections of the rivets.
iv) The rivets fills the holes completely.
v) The rivets in a group share the direct load equally
vi) Bending stress in rivets is neglected

6.8 Efficiency of a joint

The original strength of a section is reduced by rivet holes. The efficiency of a joint is the ratio of the joint and the original strength of the member without rivet holes. At the weakest critical section, the number of rivet holes should be minimum for maximum efficiency.

6.9 Design of riveted joints for axially loaded members

The diameter of a rivet is generally calculated by the following formula;

$d = 6\sqrt{t}$ where d is the rivet diameter in mm and t is the thickness of plate in mm

Number of rivets required for the joint = Load/Rivet value

The rivets are arranged bearing in mind the following points:

i) The arrangement should satisfy the gauge, pitch and edge distance requirements
ii) The strength of joint should be increased gradually towards the joint for uniform distribution of stress in the rivets.

iii) The cg of each rivet group should coincide ith the centreline of the connected members. It is not possible practically to follow this condition in some cases e.g the angle connection with gusset plate. The small eccentricities are usually neglected.

iv) The centreline of all members meeting at a joint should coincide at one point only otherwise the joint will twist out of position,

v) The strength of member reduces due to rivet holes. The reduction in area due to rivet holes is minimum if rivets are arranged in a zig-zag form.

Table 6.1 Rivet diameter pitch and edge distances

Nominal dia of rivets (mm)	12	14	16	18	20	22	24	27	30
Gross dia of rivets	13.5	15.5	17.5	19,5	21.5	23.5	25.5	29	32
Minimum edge distance for sheared or rough edge	19	25	29	32	32	38	44	51	57
Minimum edge distance for rolled or planed edge	17	22	25	29	29	32	38	44	51

Minimum pitch = 2.5 times the diameter of the rivet holeMaximum pitch = 32 t or 300 mm whichever is less

BOLTED CONNECTIONS

A bolt may be defined as a metal pin with a head at one end and a shank threaded at the other end to receive a nut as in Fig 1.0(a). Steel washers are usually provided under the bolt as well as under the nut to serve two purposes:

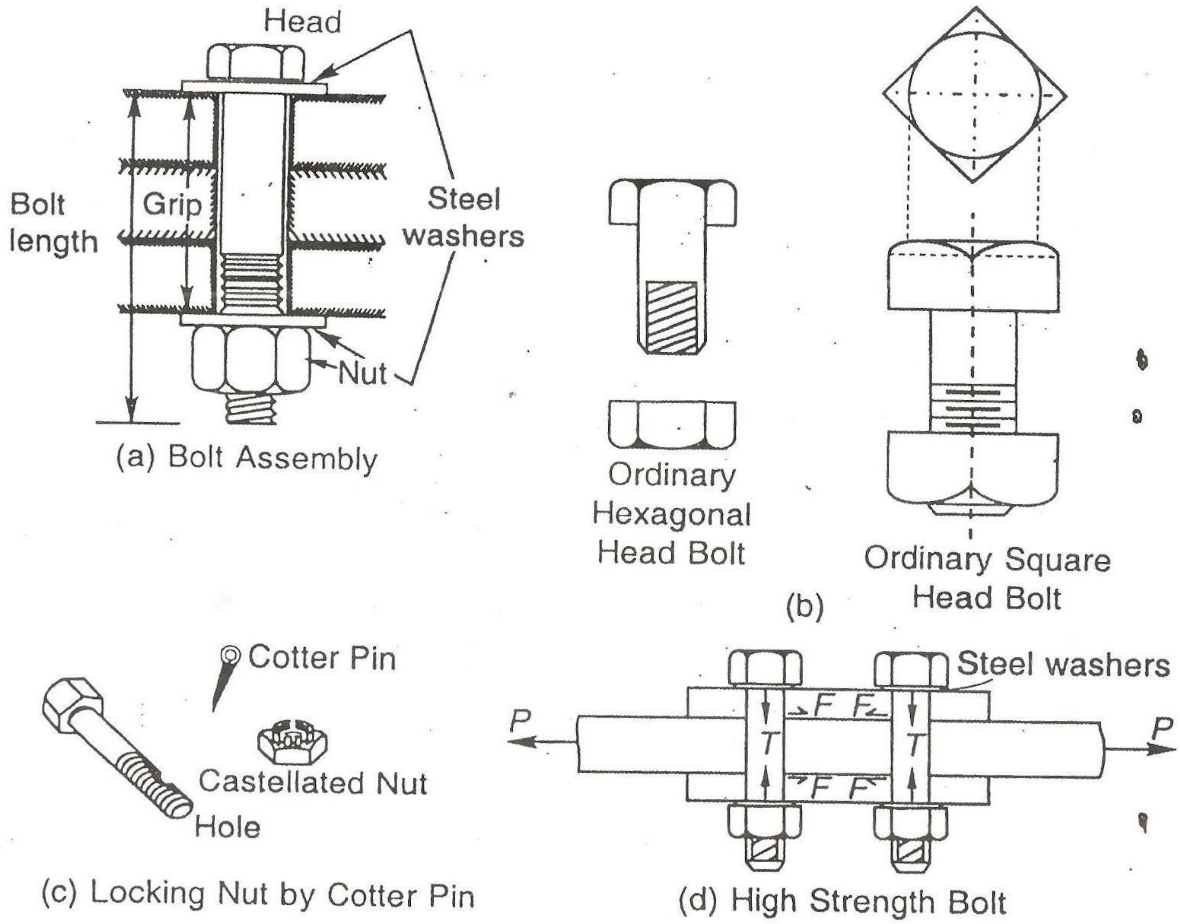


Fig. 2.12 Bolted Joints

1. To distribute the clamping pressure on the bolted member, and
2. To prevent the threaded portion of the bolt from bearing on the connecting pieces.

In order to assure proper functioning of the connection, the parts to be connected must be tightly clamped between the bolt between the bolt head and nut. If the connection is subjected vibrations, the nuts must be locked in position. Bolted connections are quit similar to riveted connections in behaviour but have some distinct advantages as follows:

1. The erection of the structure can be speeded up, and
2. Less skilled persons are required.

The general objections to the use of bolts are:

1. Cost of material is high: about double that of rivets.
2. The tensile strength of the bolt is reduced because of area reduction at the root of the thread and also due to stress concentration.
3. Normally these are of a loose fit excepting turned bolts and hence their strength is reduced.
4. When subjected to vibrations or shocks bolts may get loose.

Uses

1. Bolts can be used for making end connections in tensions and compression member.
2. Bolts can also be used to hold down column bases in position.
3. They can be used as separators for purlins and beams in foundations, etc.

Types

There are several types of bolts used to connect the structural elements. Some of the bolts commonly used are:

a) Unfinished bolts
b) Turned bolts

c) Ribbed bolts

d) High strength bolts

e) Interference bolts

UNFINISHED BOLTS

Unfinished bolts are also called ordinary, common, rough or black bolts. There are used for light structures (purlins, bracings, etc.) under static loads. They are not recommended for connections subjected to impact load, vibrations and fatigue. Bolts are forged from low carbon rolled steel circular rods, permitting large tolerances. Ordinary structural bolts are made from mild steel with square or hexagonal head, as shown in Fig 1.0(b). Square heads cost less but hexagonal heads give a better appearance, are easier to hold by wrenches and require less turning space. The bolt hole is punched about 1.6mm more than the bolt diameter. The nuts on bolts are tightened with spud wrenches, producing little tension. Therefore, no clamping force is induced on the sections jointed. Sometimes a hole is drilled in the bolt and a cotter pin with a castellated nut is used to prevent the nut from turning on the bolt, as shown in Fig 1.0(c). the connections with unfinished bolts are designed in a similar way as all the riveted connections except that the permissible stresses are reduced to account for tolerances provide on shank and threaded portion of the bolts. The requirements regarding pitch and edge distance are same as that for rivets. The permissible stresses are as given in Table 8.1 of I.S:800-1984.

TURNED BOLTS

These are similar to unfinished bolts, with the differences that the shank of these bolts is formed from a hexagonal rod. The surfaces of the bolts are prepared carefully and are machined to fit in the hole. Tolerances allowed are very small. These bolts have high shear and bearing resistance as compared to unfinished bolts. However, these bolts are obsolete nowadays. The specifications for turned bolts are given in I.S:2591-1969.

RIBBED BOLTS

These are also called fluted bolts. The head of the bolt is like a rivet head. The threaded and nut are provided on the other end of the shank. From the shank core longitudinal ribs project making the diameter of the shank more than the diameter of the hole. These ribs cut grooves into the connected members while tightening and ensure a tight fit. These bolts have more resistance to vibrations as compared to ordinary bolts. The permissible stresses for ribbed are same as that for rivets.

Table 10.1 Tensile Properties of Fasteners

Specification	Grade/ classification	Properties		
		Yield stress, MPa (Min)	Ultimate tensile stress, MPa (Min)	Elongation percentage (Min)
IS: 1367-1991	4.6	240	400	22
(ISO 898)	4.8	320	400	14
Specifications of	5.6	300	500	20
fasteners-threaded	5.8	400	500	10
steel for technical	8.8	640	800	12
supply conditions	10.9	900	1000	9

Nominal diameter of bolt, mm	12	14	16	18	20	22	24	27	30	Above 36
Diameter of hole, mm	13	15	18	20	22	24	26	30	33	Bolt diameter + 3 mm
Minimum edge distance,* mm										
(a) for sheared or rough edge	22	26	31	34	37	41	44	51	56	1.7 × hole diameter
(b) for rolled, sawn, or planed edge	18	23	27	30	33	36	39	45	50	1.5 × hole diameter

*The edge distances in this table, which are for standard holes, must be increased if oversize or slotted holes are used.

Max. edge distance = $12t\varepsilon$ where $\varepsilon = (250/f_y)^{0.5}$

Pitch (min.)	2.5 × nominal diameter of bolt
Pitch (max.)	$32t$ or 300 mm
(a) parts in tension	$16t$ or 200 mm, whichever is less
(b) parts in compression	$12t$ or 200 mm, whichever is less
(c) tacking fasteners	$32t$ or 300 mm, whichever is less $16t$ or 200 mm, whichever is less for plates exposed to weather

where t is the thickness of the thinner outside plate or angle.

For grade 4.6 bolts, nuts of grade 4 are used and for grade 8.8, nuts of grade 8 or 10 are used

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

Though square heads cost less, hexagonal heads give better appearance, are easier to hold by wrenches, require less turning space. Most of the connections with black bolts are made by inserting them in clearance holes of about 1.5mm to 2 mm more than the bolt diameter and by tightening them with nuts. They are produced in metric sizes ranging from 5-36 mm and designed as M5 to M36. In Structural steel work M16, M20, M24 and M30 bolts are often used. The bolts used in steel work have a course pitch of thread, that is, 2, 2.5, 3 and 3.5 mm for 16, 20, 24 and 30 mm diameter bolts respectively. The ratio of net tensile area at threadsto nominal plain shank area of the bolt is 0.78 as per IS 1367 (Part I). The other dimensions are given in Table 10.2

Table 10.2 Dimensions of Grade 4.6- Hexagon Head Bolts (IS 1364)

Bolt size (d), mm	Head diagonal (e), mm	Head thickness (k), mm	Thread* length (b), mm	Pitch of thread, mm	Washer (IS: 5370-1969) Outer diameter, mm	Inner diameter, mm	Thickness, mm
(12)	20.88	8	20	1.75	24	14	3
16	26.17	10	23	2.0	30	18	3
20	32.95	13	26	2.5	37	22	3
(22)	35.03	14	28	2.5	39	24	3
24	39.55	15	30	3.0	44	26	4
(27)	45.20	17	33	3.0	50	30	4
30	50.85	19	35	3.5	56	33	4
36	60.79	23	40	4.0	66	39	5

*For length $l \leq 125$ mm. For $125 < l \leq 200$, b is 6 mm more and for $l > 200$, b is 19 mm more.

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

Advantages of Bolted Connections

Black Bolted connections offer the following advantages over riveted or welded connections:

- Use of unskilled labour and simple tools
- Noiseless and quick fabrication
- No special equipment/process needed for installation
- Fast progress of work
- Accommodates minor discrepancies in dimensions
- The connection supports loads as soon as the bolts are tightened (in welds and rivets,cooling period is involved).

HIGH STRENGTH BOLT

These bolts are called friction grip bolts. These are made from bars of medium carbon steel. Their high strength is achieved through quenching and tempering processes or by alloying steel. Steel washers of hard steel or carburized steel are provided as shown in Fig1.0 (d), to evenly distribute the clamping pressure on the bolted member and to prevent the threaded portion of the bolt from bearing on the connecting pieces. If the bolts are tightened by the turn of nut method, the nut is made snug and is tightened a half turn more by hand wrenches, then the washers are not required. The vibrations and impact resistance of the joint is also improved. The nut and headof the bolts are kept sufficiently large to provide an adequate bearing area. The specifications for high strength bolts are laid in I.S:3757-1972 and I.S: 4000-1967. These bolts have a tensile strength several times that of the ordinary bolts. High strength bolts have replaced rivets and are being used in structures, such as high rise buildings, bridges, machines etc. Due to their distinct advantages and vital use, high strength bolts are discussed below in some detail.

Advantages of high strength bolts

High strength friction grip (HSFG) bolts have replaced the rivets because of their distinct advantages as discussed below. However, the material cost is about 50% greater than that of ordinary bolts and special workmanship is required in installing and tightening these bolts.

1. These provide a rigid joint. There is no slip between the elements connected
2. Large tensile stresses are developed in bolts, which in turn provide large clamping force to the elements connected. High frictional resistances is developed providing a high static strength the joint.
3. Because of the clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing.

4. The frictional resistance is effective outside the hole and therefore lesser load is transmitted through the net section. Thus, the possibility of failure at the net section is minimized.
5. There are no stress concentrations in the holes; therefore, the fatigue strength is more.
6. The tension in bolts is uniform. Also the bolts are tensioned up to proof load hence; the nuts are prevented from loosening
7. Few persons are require to make the connections, thus cost is reduced.
8. Noise nuisance is not there as these bolts are tightened with wrenches.
9. The hazard of fire is not there and there is no danger of tossing of the bolt.
10. Alterations can be done easily.
11. For some strength, lesser number of bolts are required as compared to rivets which brings overall economy.

Diameter, *d* mm	M16	M20	(M22)	M24	(M27)	M30	M36
Head diagonal, *e* mm	29.56	37.29	39.55	45.20	50.85	55.37	66.44
Head thickness, *k* mm	10	12.5	14	15	17	18.7	22.5
Nut thickness, mm	13	16	18	19	22	24	26
Washer outer diameter, **D*	30	37	39	44	50	56	60
Washer inner diameter, *d* mm	18	22	24	26	30	33	36
Washer thickness, mm	4	4	4	4	5	5	5
Thread length, ***b* mm							
<100	31	36	38	41	44	49	56
>100	38	43	45	48	51	56	63

* The outside diameter of a washer is an important dimension when detailing, for example, to avoid overlapping an adjacent weld.

**The thread length depends on the length of the bolt, which is calculated as grip length plus the allowance for grip.

Design Strength of Ordinary Black Bolts

The nominal capacity, Vnsb, of a bolt in shear is given in the code as

$$V_{nsb} = (f_u / \sqrt{3})(n_n A_{nb} + n_s A_{sb})\beta_{lj}\beta_{lg}\beta_{pk}$$

where nn = number of shear planes with threads intercepting the shear plane,
ns = number of shear planes without threads intercepting the shear plane,

βlj = reduction factor which allows for the overloading of end bolts that occur in long connections
βlg = reduction factor that allows for the effect of large grip length,
βpk = reduction factor to account for packing plates in excess of 6mm.

The factored shear force V_{sb} should satisfy $V_{sb} \le V_{nsb} / \gamma_{mb}$ (γ_{mb} = 1.25)

Design Strength of Black Bolts (cont.)

Asb = Nominal shank area
Anb = Net tensile stress area through the threads
Anb = / 4 (d - 0.9382p)2 ≈ 0.78 Asb
p= pitch of thread, mm
Reduction Factor for Long Joints:
$\beta lj = 1.075 – lj\ (200\ d)$ with $0.75 \le \beta lj \le 1.0$
Reduction Factor for Large Grip Length:
$\beta lg = 8d / (3d + lg)$; $lg \le 8d$; $\beta lg \le \beta lj$
Reduction Factor for Packing plate:
$\beta pk = (1-0.0125\ tpk)$; tpk is the thickness of the thicker packing plate in mm

Bolts in Tension

The nominal capacity of a bolt in tension is:
$Tnb = 0.90\ fub\ Anb < fyb\ Asb\ (\gamma m1 / \gamma m0)$
where Asb = Shank area of bolt
Anb = Net Tensile Stress area of bolt
fyb = Yield stress of the bolt
$\gamma m1 = 1.25$; $\gamma m0 = 1.10$

The factored tension force Tb shall satisfy
$Tb \le Tnb / \gamma mb$; $\gamma mb = 1.25$

If any of the connecting plates is flexible, the additional prying forces must be considered
Bolts in Bearing

Bolts in Bearing

The nominal bearing strength of the bolt is :

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

f_u = Ultimate tensile stress of the plate in MPa

d = nominal diameter of the bolt in mm

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction (If the bolts are countersunk, the thickness of the plate minus one half of the depth of counter sinking)

k_b is smaller of $e/(3d_o)$, $p/(3d_o)-0.25$, f_{ub}/f_u and 1.0,

where f_{ub} is the ultimate tensile stress of the bolt, e is the edge distance, p is the pitch of the fastener along bearing direction, and d_o is the diameter of the bolt hole.

V_{npb} should be multiplied by a factor 0.7 for over size or short slotted holes and by 0.5 for long slotted holes.

The factor k_b takes into account inadequate edge distance or pitch and also prevents bearing failure of bolts.

If we adopt a minimum edge distance of 1.5 x bolt hole diameter and a minimum pitch of 2.5 x diameter of bolt, k_b may be approximately taken as 0.50.

The bolt bearing on any plate subjected to a factored shear force V_{sb}, shall satisfy

$$V_{sb} \leq V_{npb} / \gamma_{mb} \quad ; \gamma_{mb} = 1.25$$

Tables 10.6, 10.7 and 10.8 will aid the designer while designing joints using ordinary bolts.

Table 10.6 Strength of Bolts in Clearance Holes

	Bolt grade 4.6 N/mm^2	Bolt grade 8.8 N/mm^2	Other grades of bolts N/mm^2
Shear streng th, v_{nsb}	185	370	$f_u / (\sqrt{3} \times 1.25)$
Bearing strength, V_{npb}	400	800	$2.5 k_b f_u / 1.25 (k_b = 0.5)$
Tension strength, t_{nb}	272	576	$0.9 f_{bu} / (1.25)$ not greater than $f_{yb} / (1.25/1.1)$

Table 10.7 Bearing strength v_{npp} of connected parts for ordinary bolts in clearance holes in N/mm^2

Grade 410	Grade 540	Grade 570	Other grades
820	540	570	$2.5 k_b f_u / 1.25$

Bolt size, d(mm)	Thread stress area (mm^2)	Tension capacity, T_b(kN) t_{nb} = 272 MPa	Single shear capacity, V_{sb}(kN) v_{nsb} = 185 MPa	Minimum thickness of ply for bolt bearing v_{npb}= 400 MPa $t_{bb} = t_e$, mm
(12)	84.3	23.9	15.6	3.2
16	157	42.7	29.0	4.5
20	245	66.6	45.3	5.6
(22)	303	82.9	56.0	6.3
24	353	96.0	65.3	6.8
(27)	459	124.8	84.9	7.8
30	561	152.5	103.8	8.6
36	817	222.2	151.1	10.5

$V_{sb} = A_{nb}v_{nsb}$; $T_b = A_{nb}t_{nb}$; $t_{bb} = V_{npb}/(dv_{npb})$; Sizes in brackets are not preferred

Principles of high strength bolts

The shank of the high strength bolts does not fully fill the hole. So shear and bearing are not the criteria for load transmission as is in the case of rivets, which fill the hole completely. The nut is tightened to develop a clamping force on the plates which is indicated as the tensile force T in the Bolt. This tension should be about 90% of proof load. When a shear load is applied to the joint no slip will occur until the shear load exceeds the frictional resistance between the elementsjointed.

When shear load exceeds the frictional resistance a slip occurs. On further increase ofthis load, the gradual slipping brings the bolt in contact with the plate edges.

The horizontal frictional forces F, is induced in the joints which is equal to the tensile force T, as in Fig.1.0(d), in the bolts multiplied by the coefficient of friction.

$$F = \mu T$$

This frictional force F should exceed the applied force P on the member.

μ= Coefficient of friction or slip factor, is defined as ratio of the load per effective interface required to produce slip in a pure shear joint to the proof load induced in bolt. When the element surfaces are free from paint, dust, etc. its value is 0.45.

PIN CONNECTIONS

When two structural members are connected by means of a cylindrical shaped pin, the connection is called a pin connection. Pins are manufactured from mild steel bars with diameters ranging from 9 to 330 mm. Pin connections are provided when hinged joints are required, i.e.,for the connection where zero moment of free rotation is desired. Introduction of a hingesimplifies the analysis by reducing indeterminacy. These also reduce the secondary stresses. These connections cannot resist longitudinal tension. For satisfactory working it is necessary to minimize the friction between the and members connected. High grade machining is done to make the pin and pin hole surface smooth and frictionless. Pins are provided in the following cases:

FAILURE OF BOLTED JOINTS

The bolted joint may fail in any of the following six ways, out of which some failures can be checked by adherence to the specifications of edge distance. Therefore, they are not of much importance, whereas the others require due consideration.

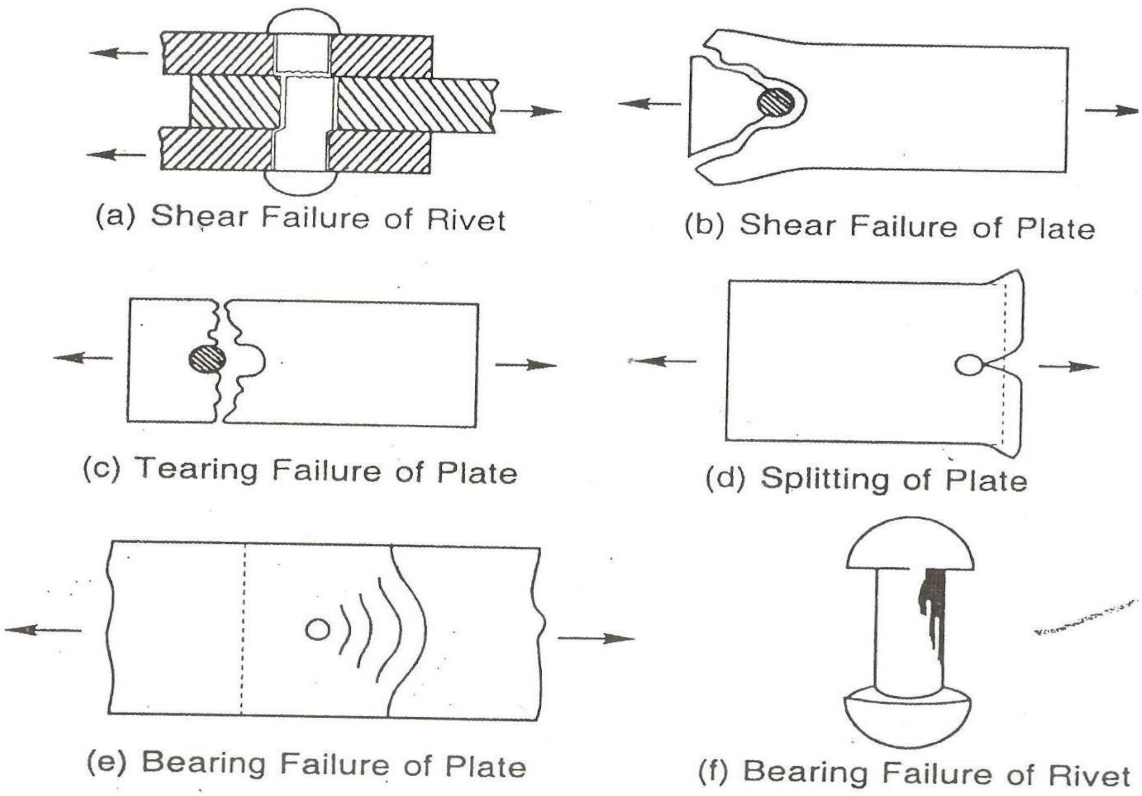


Fig. 2.3 Failure of Riveted Joints

Shear failure of bolts (Fig. 2.3 (a))

The shear stress in the bolt may exceed the working shear stress in the bolt. Shear stresses are generated because the plates slip due to applied forces.

Shear failure of plates (Fig. 2.3(b))

The internal pressure of overdriven (shank length more than the grip) bolts placed at a lesser edge distance than specified causes this failure. This can be checked by providing proper edge distance between the center of the hole and the end of the plate as specified by I.S.800.

Tension or tearing failure of plates (Fig. 2.3(c))

The tensile stress in the plate at the net cross-section may exceed the working tensile stress. Tearing failure occurs when bolts are stronger than the plates.

Splitting of plates (Fig. 2.3(d))

Bolts may have been placed at a lesser edge distance than required causing the plates to split or shear out.

Bearing failure of plates (Fig. 2.3(e))

The plate may be crushed when the bearing stress in the plate exceeds the working bearing stress.

Bearing failure of bolts (Fig. 2.3(f))

The bolt is crushed around the half circumference. The plate may be strong in bearing and the heaviest stressed plate may press the bolt.

CAPACITY OF HSFG SHEAR BOLTS

As stated in Fig, these are the bolts made of high tensile steel which are pretensioned and then provided with nuts. The nuts are clamped also. Hence resistance to shear force is mainly by friction.

There are two types of HSFG bolts. They are parallel shank and waisted shank type. Parallel shank type HSFG bolts are designed for no-slip at serviceability loads. Hence they slip at higher loads and slip into bearing at ultimate loads. Hence such bolts are checked for their bearing strength at ultimate load. Waisted shank HSFG bolts are designed for no slip even at ultimate load and hence there is no need to check for their bearing strength.

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_0$$

Where,

μ_f = Co-efficient of friction (Called slip factor) as specified in Table 3.1.

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

[Note: n_e = 1 for lap joints and 2 for double cover butt joints]

K_h = 1.0 for fasteners in clearance holes

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

=0.70 for fasteners in long slotted holes loaded parallel to the slot.

F_0 = Minimum bolt tension at installation and may be taken as $A_{nb}\, f_0$

A_{nb} = net area of the bolt at threads $= \left(0.78 \dfrac{\pi}{4} d^2 \right)$

f_0 = Proof stress = 0.70 f_{ub}

Table 3.1 Typical average value for coefficient of friction (μ_f) [Table 20 in IS 800-2007]

Sl. No.	Treatment of Surface	μ_f
1	Surface not treated	0.20
2	Surface blasted with shot or grit with any loose rust removed, no pitting	0.50
3	Surface blasted with shot or grit and hot-dip galvanized	0.7
4	Surface blasted with shot or grit and spray-metallized with zinc (thickness 50–70 µm)	0.25
5	Surfaces blasted with shot or grit and painted with ethylzinc silicate coat (thickness 30–60 µm)	0.30
6	Sand blasted surface, after light rusting	0.52
7	Surface blasted with shot or grit and painted with ethylzinc silicate coat (thickness 60–80 µm)	0.30
8	Surface blasted with shot or grit and painted with alkalizinc silicate coat (thickness 60–80 µm)	0.30
9	Surface blasted with shot or grit and spray metalled with aluminium (thickness > 50 µm)	0.50
10	Clean mill scale	0.33
11	Sand blasted surface	0.48
12	Red lead painted surface	0.1

The slip resistance should be taken as

$V_{sf} = V_{nsf} /1.10$

Where,

=1.10, if the slip resistance is designed at service load (Parallel shank HSFG)

=1.25, if the slip resistance is designed at ultimate load (Waisted shank HSFG).

It may be noted that the reduction factors specified (Fig. 3.11) for bearing bolts hold good for HSFG bolts also.

For commonly used HSFG bolts (Grade 8.8), yield stress f_{yb} =640 Mpa and ultimate stress f_{ub} =800 N/mm^2

Example 3.12

Determine the shear capacity of bolts used in connecting two plates as shown in Fig.3.30

1. Slip resistance is designated at service load

2. Slip resistance is designated at ultimate load

Given:

HSFG bolts of grade 8.8 are used.

Fasteners are in clearance holes

Coefficient of friction = 0.3

Solution:

For HSFG bolts of grade 8.8,

For fasteners in clearance holes K_h = 1.0

Coefficient of friction μ_f =0.3

.'. Nominal shear capacity of a bolt

$$V_{nsf} = \mu_f \, n_c \, K_h \, F_0$$

Where $F_0 = 0.7 \, f_{ub} \, A_{nb}$

$$= 0.7 \times 800 \times 0.78 \times \frac{\pi}{4} \times 20^2$$

n_e=2, since it is double cover butt joint

(i) Design capacity of one bolt, if slip resistance is designated at service load

V_{nsf} = 0.3 x 2 x1.0 x137225

= 82335 N

= 82335/1.1 =74850 N

Therefore design capacity of joint = 6 x 74850, since 6 bolts are used

= 449099 N

= 449.099 kN

(ii) Design capacity of one bolt, if the slip resistance is designated at ultimate load

= 82335/1.25 =65868 N

Therefore design capacity of joint = 6 x 65868, since 6 bolts are used

= 395208 N

= 395.208 kN

In case (i), bearing strength at ultimate load should be checked. If it is low that will be the governing factor.

TENSION RESISTANCE OF HSFG BOLTS

The expression for nominal tension strength of HSFG bolts is also as that for bearing bolts. i.e,

$$T_{nf} = 0.9\, X f_{ub}\, X A_n \leq f_{yb}\, A_{sb} \frac{\gamma_{mb}}{\gamma_m}$$

$$T_{df} = \frac{0.9\, f_{ub} A_n}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_m}$$

Where

A_n = net tensile area as specified in various parts of IS 1367, it may be taken as the area at the root of the thread $= \left(0.78 \frac{\pi}{4} d^2 \right)$

A_{sb} = shanke area.

γ_{mb} = 1.25, γ_m = 1.1

f_{ub} for bolts of grade 8.8 is 800 MPa and f_{yb} = 640 MPa.

Welded Connections

7.1 Introduction

Steel sections are manufactured and shipped to some standard lengths, as governed byrolling, transportation and handling restrictions. However, most of the steel structural members used in structures have to span great lengths and enclose large three dimensional spaces. Hence connections are necessary to synthesize such spatial structures from one- and two-dimensional elements and also to bring about stability of structures under different loads.Thus, connections are essential to create an integral steel structure using discrete linear and two-dimensional (plate) elements.

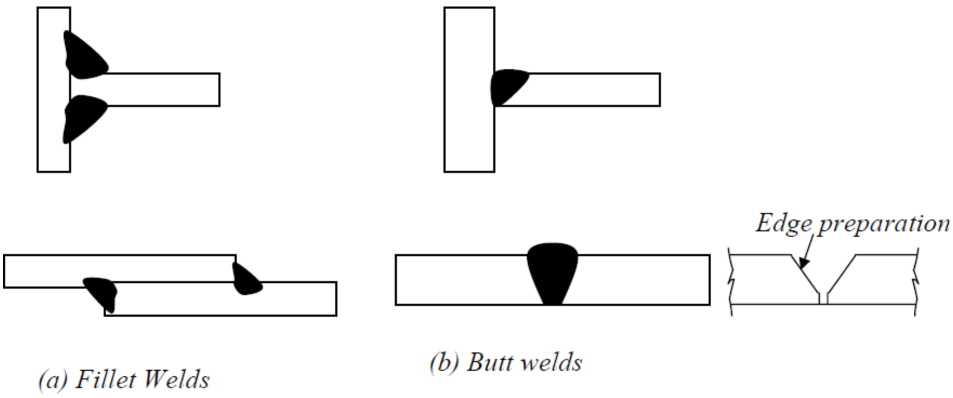


Figure 7.1 Typical welded connections

The merits of butt welds are:

- easily designed and fabricated to be as strong as the member,
- better fatigue characteristics, compared to fillet welds,
- better appearance, compared to fillet welds, and
- easy to detail and the length of the connection is considerably reduced.

The demerits of the butt welds are:

- more expensive than fillet welds because of the edge preparation required, and
- require more skilled manpower, than that required for filled welds.

It has been pointed out that steel sections are linear elements produced in certain convenient lengths due to constraints on manufacturing and transportation. Therefore connections are necessary to provide continuity, where required, as well as to create three-dimensional steel structures

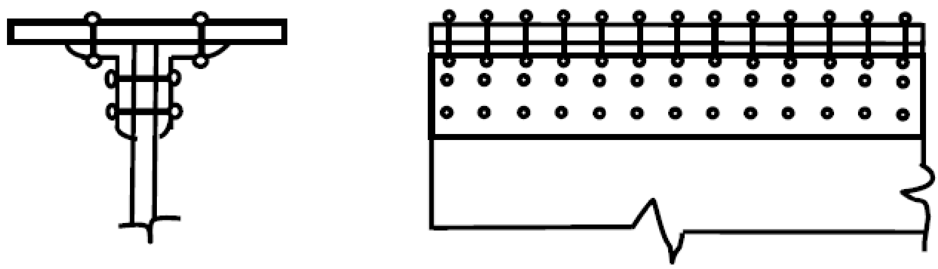
Advantages of welding

Welding offers many advantages over bolting and riveting. Some of the advantages are listed in the following.

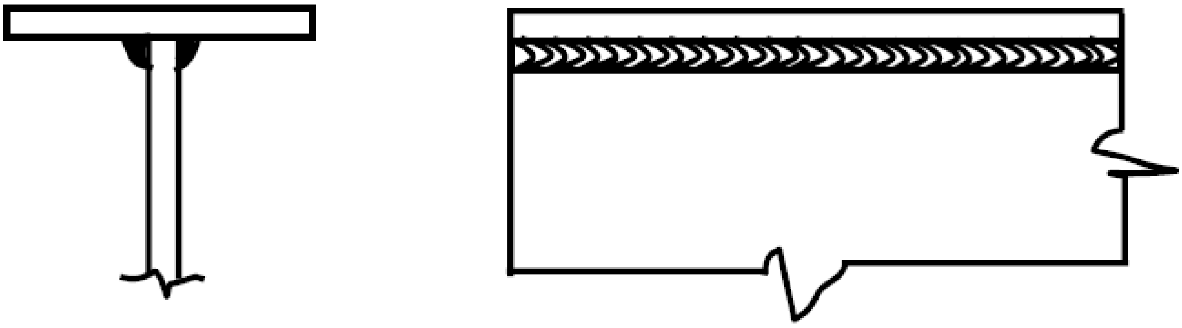
• Welding enables direct transfer of stress between members. Hence, the weight of the joint is minimum. Besides efficiency, design details are very simple. Less fabrication cost compared to other methods due to handling of fewer parts and elimination of operations like drilling, punching etc. The most striking advantage of welded structures is in the area of economy. Welded structures allow the elimination of a large percentage of the gusset and splice plates necessary for riveted or bolted structures. Time is saved in detailing, fabrication and field erection. In some bridge trusses it may be possible to save up to 15% of the steel weight by resorting to welding. Welding also requires considerably less labour for executing the work.

• Welding offers air tight and water tight joining of plates and hence ideal for oil storage tanks, ships etc.

• Welded structures usually have a neat appearance as against the cluttered surface of bolted or riveted connections. Fig. 7.2 shows a comparison of riveted plate girder and a welded plate girder. Further, welded connections offer the designer more freedom for innovation in his design concept. It enables him to use any cross section and the best configuration to transmit forces from one member to another.



a) Riveted plate girder



b). Welded plate girder
Figure 7.2

The range of application of welding is very wide. For example, connection of a steel pipe column to other members can be made very easily by welding whereas it is virtually impossible by bolting or riveting. Welding is practicable even for complicated shapes of joints.

• There is no need for holes in members connected by welding except possibly for erection purposes. This has direct influence in the case of tension members as the problem of determining the minimum net section is eliminated. This also results in a member with a smaller cross section.

• Welded structures are more rigid compared to structures with riveted and bolted connections. The rigidity of welded structures is due to the direct connection of members by welding. In bolted or riveted structures, the connection is established through angles or plates,which deflect under loads, making the structure flexible.

• It is easier to make design changes and to correct mistakes during erection, if welding is used. It is also a simple procedure to strengthen the existing structures with welding.

*. A truly continuous structure is formed by the process of fusing the members together. This gives the appearance of a one-piece construction. Generally welded joints are as strong or stronger than the base metal, thereby placing no restriction on the joints.

Due to this continuity advantage, a very large number of steel frames have been constructed all over the world.

• Stress concentration effect is considerably less in a welded connection. Some of the lesser important advantages of the welding processes are: relative silence of the process of welding and fewer safety precautions.

Some of the disadvantages of welding are:

• Welding process requires highly skilled manpower

• Experienced manpower is needed for inspection of welded connections. Also, non-destructive evaluation may have to be carried out to detect defects in welds

- Welded joints are highly prone to cracking under fatigue loading
- Costly equipment is essential to make welded connections
- Proper welding can not be done in the field environment
- Large residual stresses and distortion are developed in welded connections

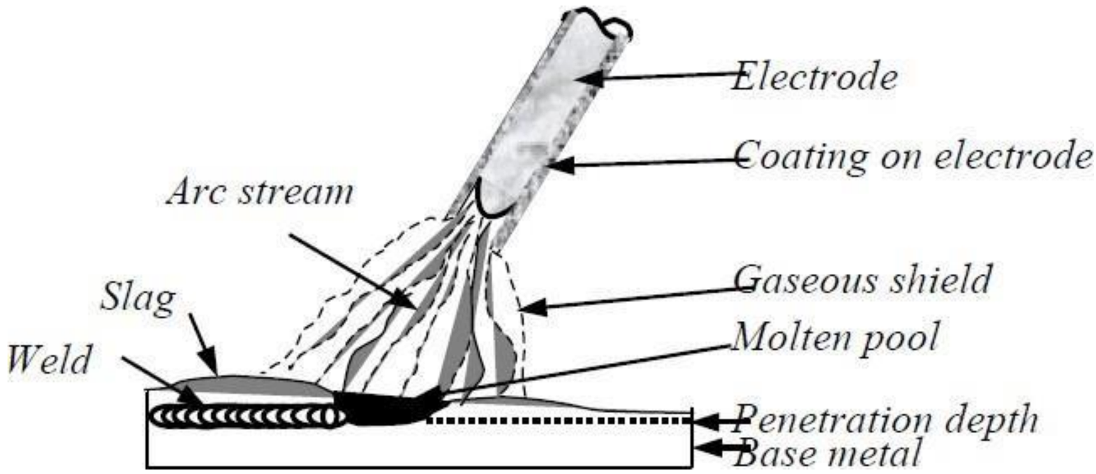


Figure 7.3 Shielded Metal Arc Welding (SMAW) process

Types of joints and welds

By means of welding, it is possible to make continuous, load bearing joints between the members of a structure. A variety of joints is used in structural steel work and they can be classified into four basic configurations as shown in Figure 7.7.

They are:

1. Lap joint
2. Tee joint
3. Butt joint, and
4. Corner joint

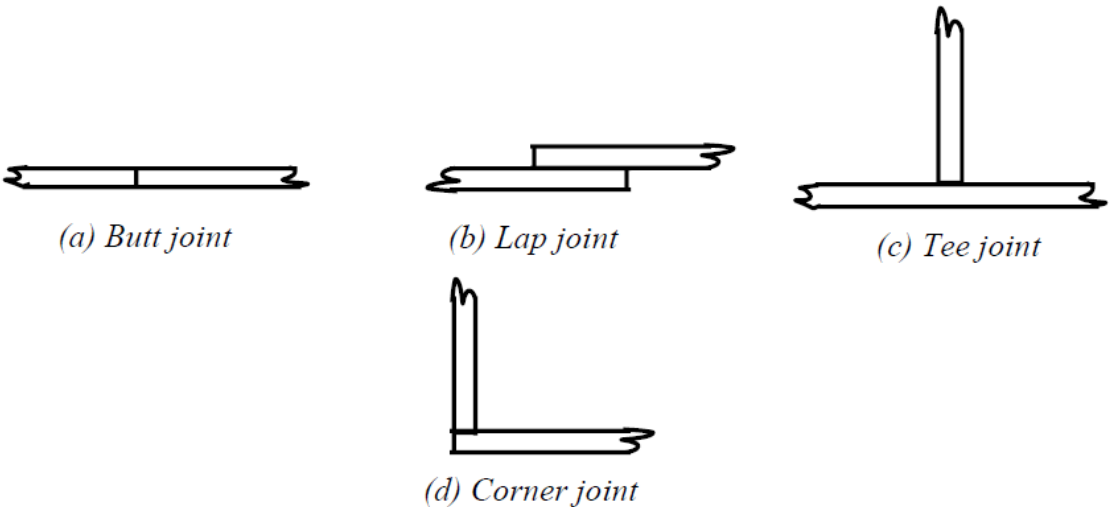


Figure 7.7 Types of joints

For lap joints the ends of two members are overlapped, and for butt joints the two members are placed end to end. The T- joints form a Tee and in Corner joints, the ends are joined like the letter L. The common types of welds are shown in Fig.7. 8. Most common joints are madeup of fillet weld and the groove weld. Plug and slot welds are not generally used in structural steel work. Fillet welds are suitable for lap joints and Tee joints and groove welds for buttand corner joints. Groove welds can be of complete penetration or incomplete penetration depending upon whether the penetration is complete through the thickness or partial.

Generally a description of welded joints requires an indication of the type of both the joint and the weld.

fillet welds are weaker than groove welds, about 80% of the connections are made with fillet welds. The reason for the wider use of fillet welds is that in the case of fillet welds, when members are lapped over each other, large tolerances are allowed in erection. For groove welds, the members to be connected have to fit perfectly when they are lined up for welding. Further groove welding requires the shaping of the surfaces to be joined as shown in Fig. 7.9. To ensure full penetration and a sound weld, a backup plate is temporarily provided as shownin Fig. 7.9.

Welds are also classified according to their position into flat, horizontal, vertical and overhead (Fig. 7.10). Flat welds are the most economical to make while overhead welds are the most difficult and expensive.

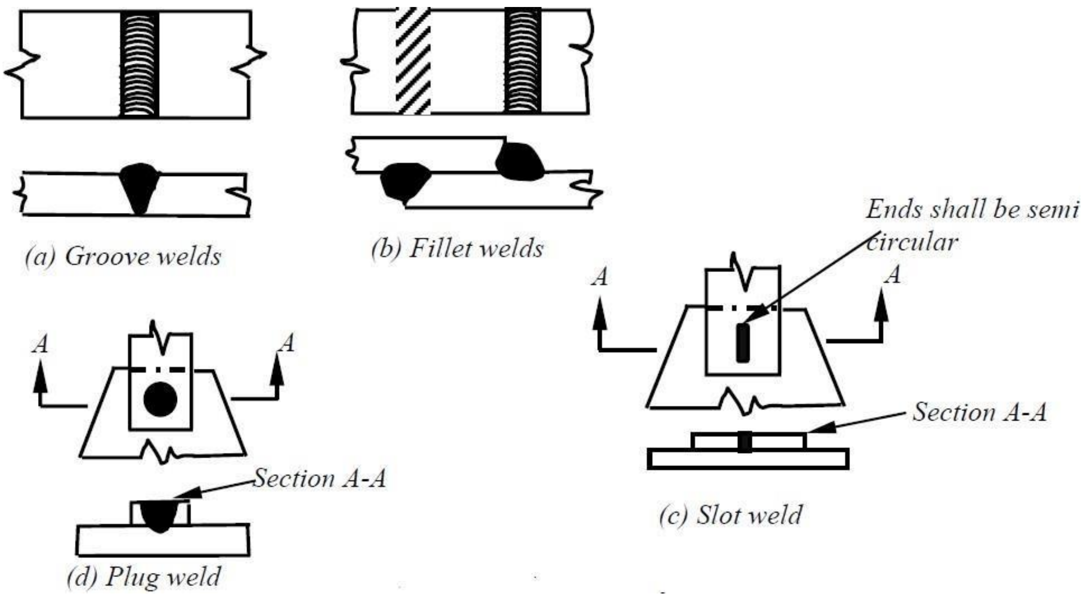


Figure 7.8 Common types of welds

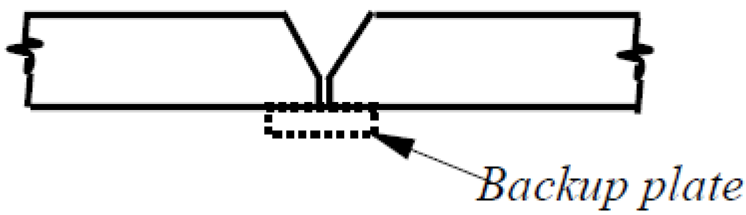


Figure 7.9 Shaping of surface and backup plate

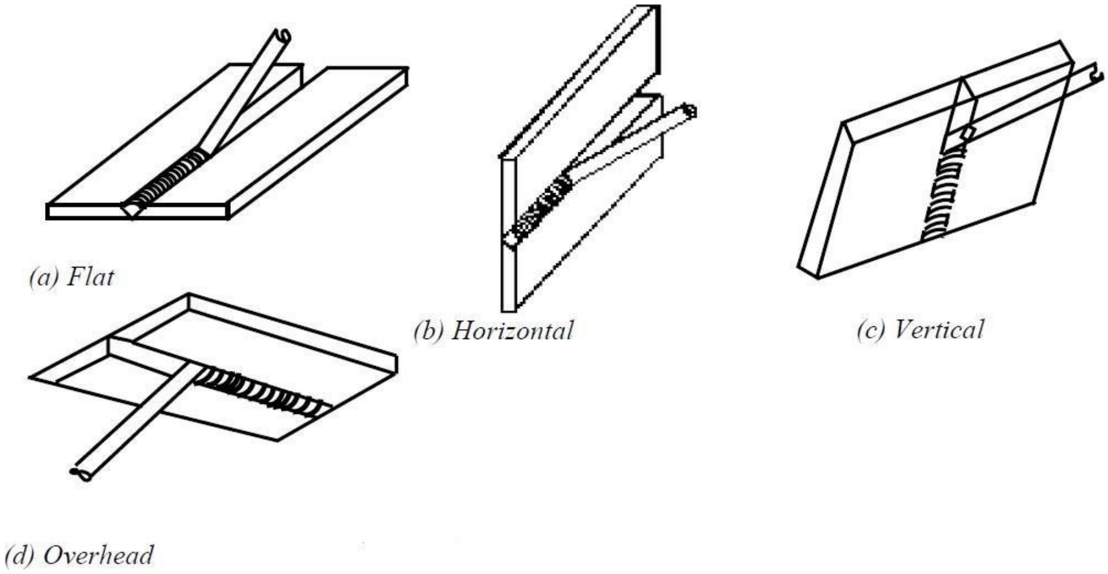


Figure 7.10 Classification based on position

Groove welds

The main use of groove welds is to connect structural members, which are in the same plane. A few of the many different groove welds are shown in Fig. 7.11. There are many variations of groove welds and each is classified according to its particular shape. Each type of groove weld requires a specific edge preparation and is named accordingly. The proper selection of a particular type depends upon

- Size of the plate to be joined.
- Welding is by hand or automatic.
- Type of welding equipment.
- Whether both sides are accessible.
- Position of the weld.

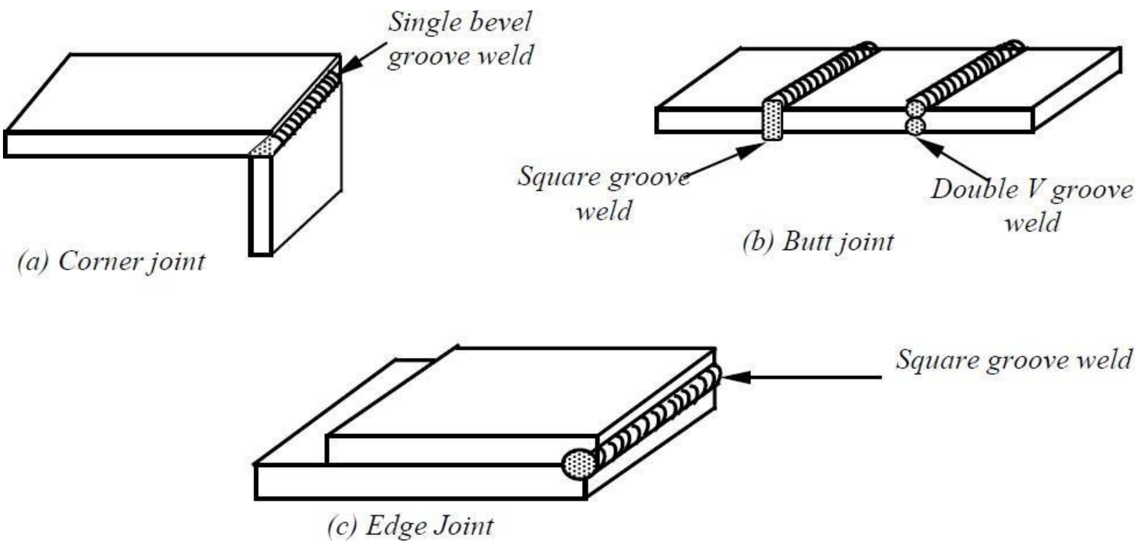


Figure 7.11 Typical connections with groove weld

The aim is to achieve the most economical weld of the requisite efficiency and strength. The butt weld whether of full penetration or partial penetration should attain the required strength of the joined parts. The size of the butt weld is defined by the thickness i.e. the thickness of the connected plate for complete penetration welds or the total depth of penetration for partial penetration welds.

Groove welds have high strength, high resistance to impact and cyclic stress. They are most direct joints and introduce least eccentricity in the joint. But their major disadvantages are: high residual stresses, necessity of edge preparation and proper aligning of the members in the field. Therefore, field butt joints are rarely used.

To minimise weld distortions and residual stresses, the heat input is minimised and hence the welding volume is minimised. This reduction in the volume of weld also reduces cost. Hence for thicker plates, double groove welds and U welds are generally used.

Edge Preparation for Butt Weld

Typical edge preparations are shown in Fig. 7.12

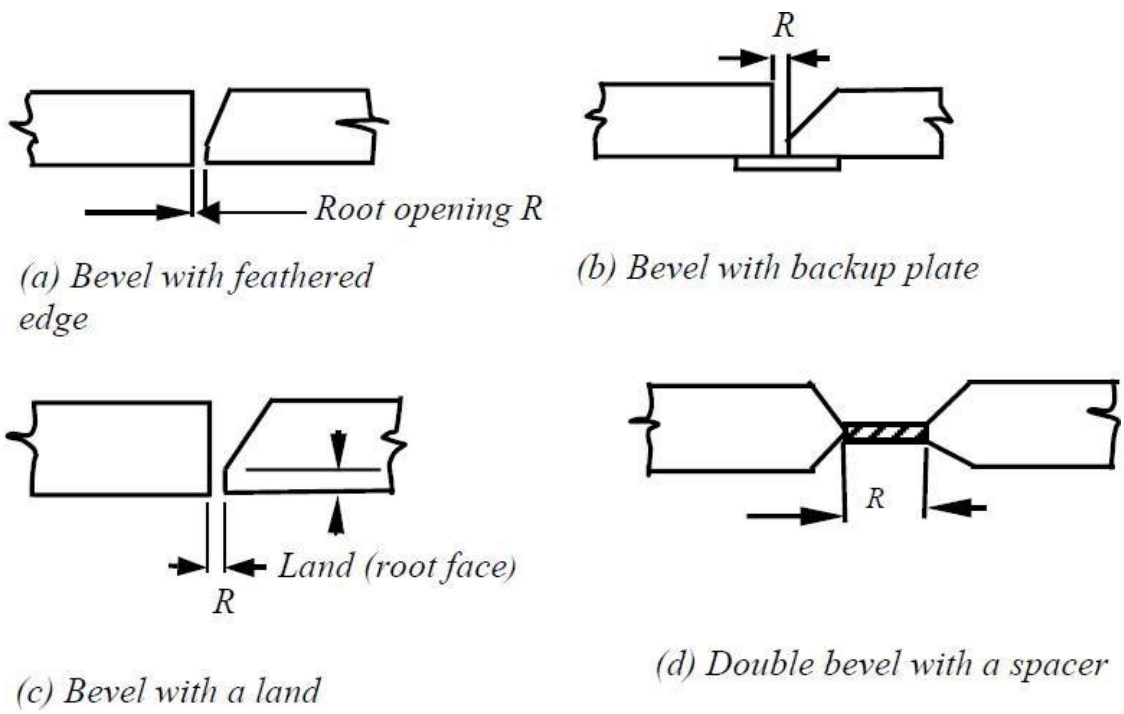


Figure 7.12 Typical edge preparation for butt weld

For a butt weld, the root opening, R, is the separation of the pieces being joined and is provided for the electrode to access the base of a joint. The smaller the root opening the greater the angle of the bevel. The depth by which the arc melts into the plate is called the depth of penetration [Fig. 7.13 (a)]. Roughly, the penetration is about 1 mm per 100A and in manual welding the current is usually 150 – 200 A.

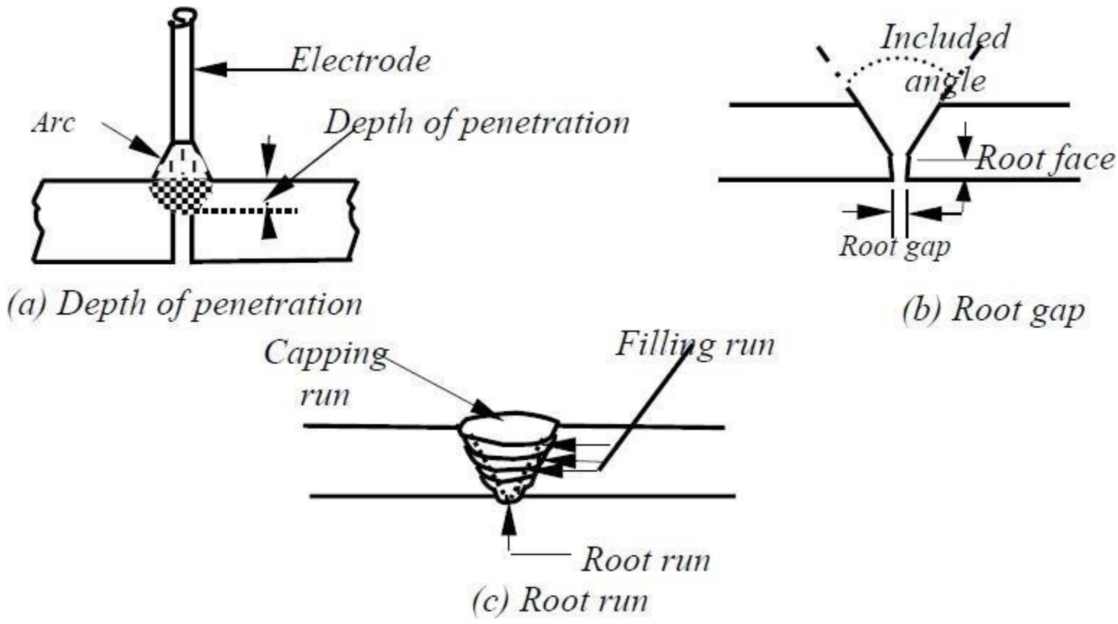


Figure 7.13 Groove weld details

Fillet welds

A typical fillet weld is shown in Fig. 7.14 (a).

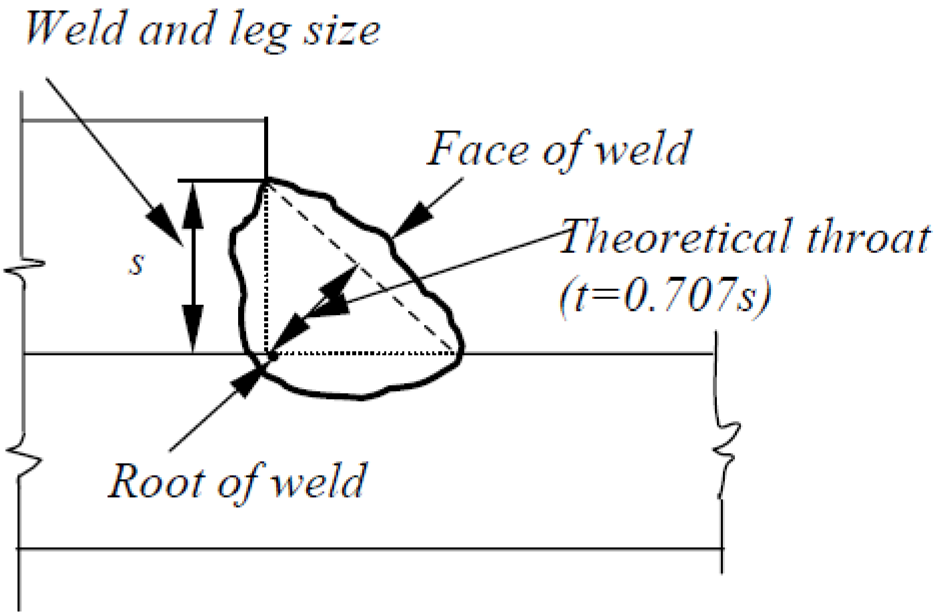
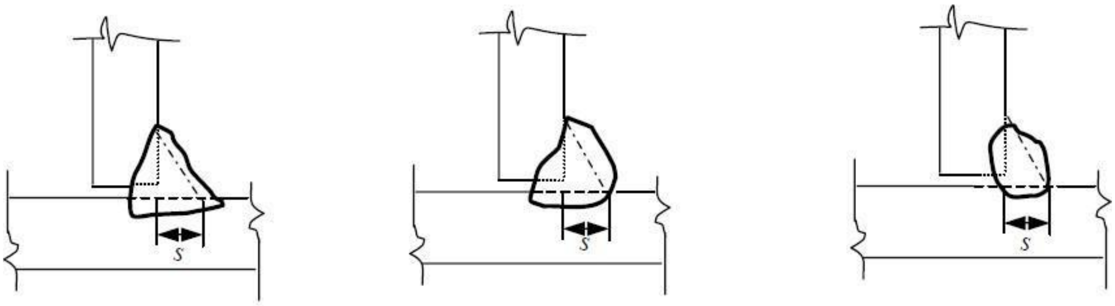


Figure 7.14 Typical fillet weld

The root of the weld is the point where the faces of the metallic members meet. Thetheoretical throat of a weld is the shortest distance from the root to the hypotenuse of the triangle. The throat area equals the theoretical throat distance times the length of the weld. Though a fillet weld is specified by defining the two sides of the inscribed triangle, its actual cross section will be quite complex. A fillet weld must penetrate the base metal and the interface of the weld is either concave or convex [Fig .7.17(a)&(b)].

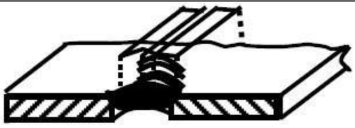
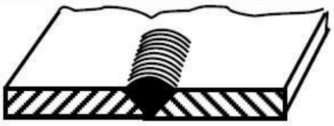
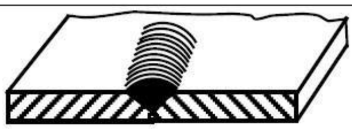
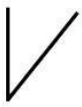
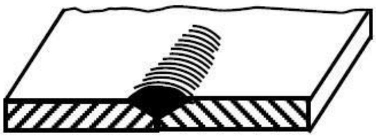
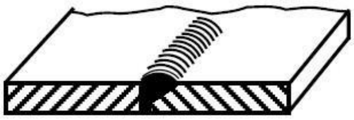
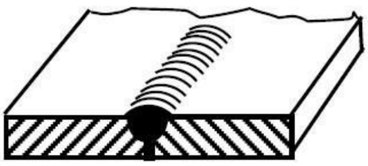


(a) Concave (b) Convex (c) Under cutting

Figure 7.17 Cross section of fillet weld

The concave shape of free surface provides a smoother transition between the connected parts and hence causes less stress concentration than a convex surface. But it is more vulnerable to shrinkage and cracking than the convex surface and has a much reduced throat area to transfer stresses. On the other hand, convex shapes provide extra weld metal or reinforcementfor the throat. But while making a convex surface there is always the possibility of causing undercut at the edges, which undermines the strength of the joint [Fig. 7.17(c)]. The stress concentration is higher in convex welds than in concave welds. It is generally recommended that for statically loaded structures, a slightly convex shape is preferable, while for fatigue – prone structures, concave surface is desirable. Large welds are invariably made up of a number of layers or passes. For reasons of economy, it is desirable to choose weld sizes that can be made in a single pass. Large welds scan be made in a single pass by an automatic machine, though manually, 8 mm fillet is the largest single-pass layer.

Table 7.1 Elementary Symbols

Illustration(Fig.)	Symbol	Description
		Butt weld between plates with raised edges*(the raised edges being melted down completely)
		Square butt weld
		Single-V butt weld
		Single-bevel butt weld
		Single – V butt weld with broad root face
		Single – bevel butt weld with broad root face
		Single – U butt weld (parallel or sloping sides)

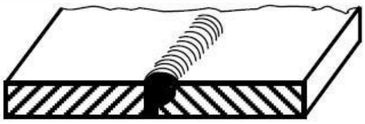
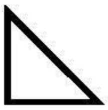
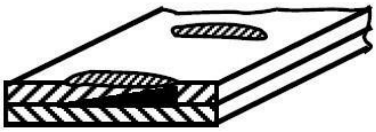
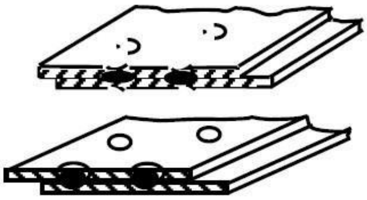
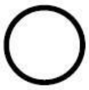
		Single – J butt joint
		Backing run; back or backing weld
		Fillet weld
		Plug weld; plug or slot weld
		Spot weld

Table 7. 2 Supplementary Symbols

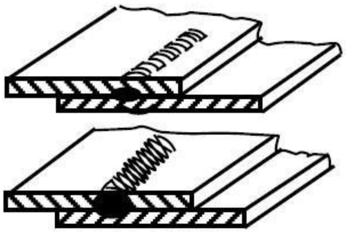
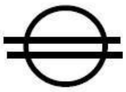
		Seam weld

Table 7. 3 Combination of Elementary and Supplementary Symbols

Shape Of Weld Surface	**symbol**
(a) flat (usually finished flush)	
(b) convex	
(c) concave	

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 lecture.

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. 8.1. 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:

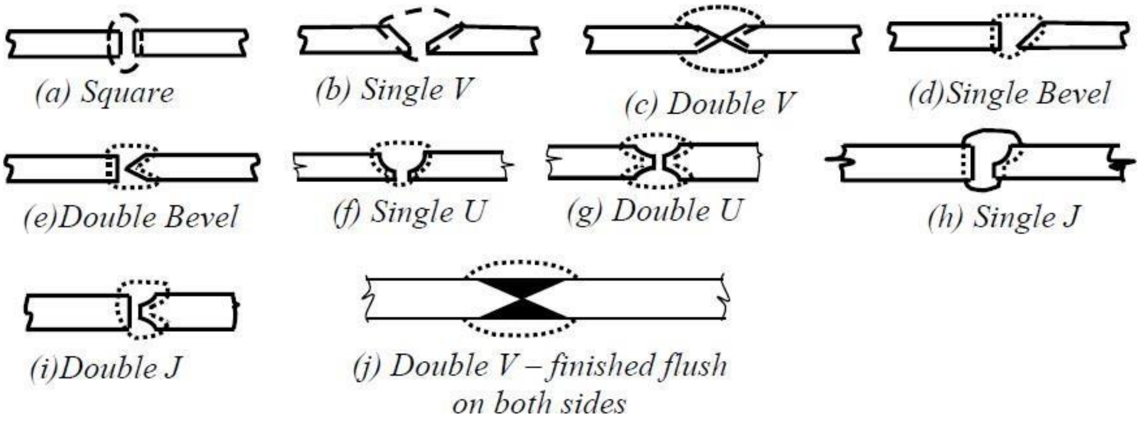


Figure 8.1 Different types of butt joints

Static behaviour of butt welds

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.

Effective length of Groove welds

The effective length of groove welds in butt joints is taken as the length of continuous full size weld, but it should not be less than four times the size of the weld.

Effective area of groove weld (Figure 8.5)

The effective area of groove weld is the product of the effective throat dimension t_e multiplied by the effective length of the weld. The effective throat dimension of a groove weld depends on the minimum width of expected failure plane

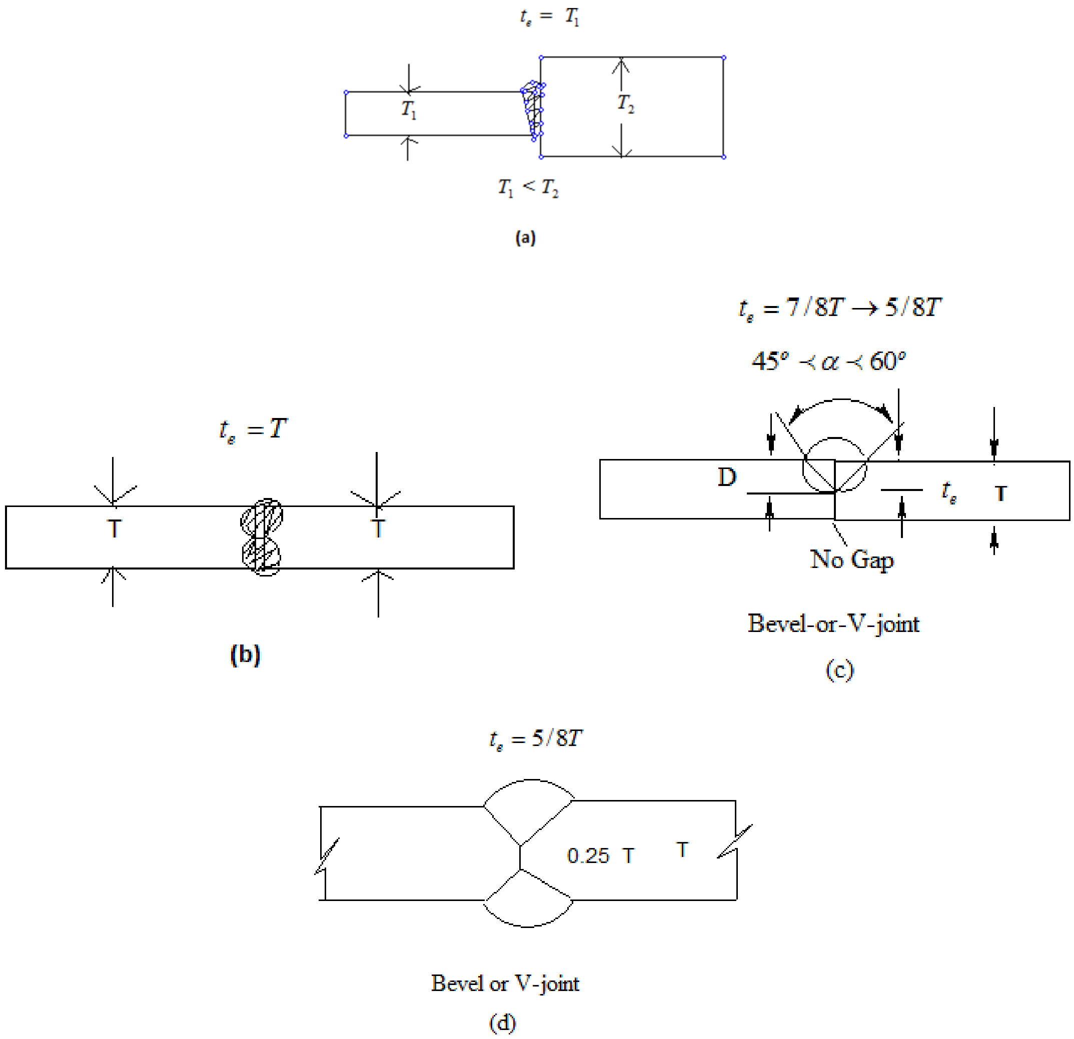


Figure 8.5 Effective throat dimensions for groove welds

The effective throat thickness of a complete penetration groove weld is taken as the thickness of the thinner part joined. The effective throat thickness of T or L joints are taken as the thickness of the abutting part. Reinforcement which is provided to ensure full cross-sectional area is not considered as part of the effective throat thickness.

The effective throat thickness of partial penetration joint weld is taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement. In unsealed single groove welds of V, U, J and bevel types and groove welds welded from one side only, the throat thickness should be at least 7/8th of the thickness of the thinner part joined. However, for the purpose of stress calculation, the effective throat thickness of 5/8th thicknessof the thinner member only should be used (IS816:1969). The unwelded portion in incomplete penetration welds welded from both sides should not be greater than 0.25 times the thickness of the thinner part joined and should be central in the depth of the weld. In this case also, a reduced effective throat thickness of 5/8th of the thickness of the thinner part should only be used in the calculations. Groove welds used in butt joints, where the penetration is less than those specified above, due to non-accessibility, should be considered as non-load carrying for the purposes of design calculations.

Design of Butt Welds as per IS 800: 2007

The following assumptions are usually made in the analysis of welded joints

a) The welds connecting the various parts are homogeneous, isotropic and elastic
b) The parts connected by the welds are rigid and their deformation is therefore neglected
c) Only stresses due to external forces are considered. The effects of residual stresses, stress concentrations and shape of the weld are neglected.

As per IS 800: 2007 the grooved welds in butt joints will be treated as parent metal with a thickness equal to the throat thickness and the stresses shall not exceed those permitted in the parent metal.

a) For tension or compression normal to effective area and tension and compression parallel to the axis of the weld,

$$T_{dw} = \frac{f_y L_w t_e}{\gamma_{mw}}$$

Where T_{dw} is the design strength of the weld in tension, f_y is the smaller of yield stress of the weld and the parent metal in MPa, t_w is the effective throat thickness of the weld in mm, L_w is the effective length of the weld in mm, and γ_{mw} is the partial safety factor taken as 1.25 for shop welding and as 1.5 for site welding.

b) For shear on effective area

$$V_{dw} = \frac{L_w t_e f_{yw}}{\sqrt{3}\gamma_{mw}}$$

Where V_{dw} is the design strength of the weld in shear.

As discussed previously, in the case of complete penetration groove weld in butt joints, design calculations are not required as the weld strength of the joint is equal to or even greater than the strength of the member connected. In the case of incomplete penetration groove weld in butt joints, the effective throat thickness is computed and the requiredeffective length is determined and checked whether the strength of the weld is equal to or greater than the strength of the member connected to the applied external force.

Problem 1: Two plates of thickness 14 mm and 12 mm are to be jointed by a groove weld as shown in Figure 8.6 below.

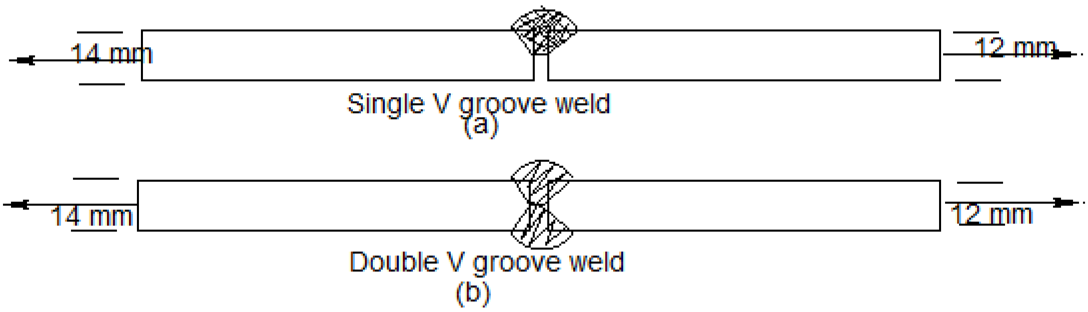


Figure 8.6

The joints are subjected to a factored tensile force of 350 KN. Assuming an effective length of 150 mm, check the safety of the joint for i) single V-groove weld joint ii) double V-groove weld joint

Assume that Fe410 grade steel plates are used and that the welds are shop welded.

Solution

Case (i)

Single V-groove weld: In this case, incomplete penetration results due to single-V groove weld

Hence throat thickness,

$t_e = 5t/8 = 5\times12/8 = 7.5mm$

Effective length of weld $L_e = 150mm$

Strength of weld = $L_e t_e f_y/\gamma_{mw} = 7.5\times150\times250/(1.25\times1000) = 225KN < 350KN$

Hence the joint is not safe

Case (ii)

In the case of double-V groove weld, complete penetration takes place

Throat thickness = thickness of thinner plate = 12 mm

Strength of weld = $12\times150\times250/(1.25\times1000) = 360KN > 350KN$ hence the joint is safe.

Case (i)

Sample Matlab Program is provided for calculating strength of butt weld (incomplete penetration)

```
% Calculation of the strength of the weld
% t = thickness of the plate
% te=5/8t (for incomplete penetration)
%te=t (for complete penetration)
% Le= effective length of the weld
% fy=yield strength of the plate
```

```
% vmw= partial safety factor for weld material
```

```
t=12;
te=5*t/8;
Le=150;
fy=250;
vmw=1.25;
P=Le*te*fy/vmw
% P in KN
P=P/1000
```

Case (ii)

Sample Matlab Program is provided for calculating strength of butt weld (complete penetration)

```
% Calculation of the strength of the weld
% t = thickness of the plate
% te=5/8t (for incomplete penetration)
%te=t (for complete penetration)
% Le= effective length of the weld
% fy=yield strength of the plate
% vmw= partial safety factor for weld material

t=12;
te=1*t/1;
Le=150;
fy=250;
vmw=1.25;
P=Le*te*fy/vmw
% P in KN
P=P/1000
```

Problem 2 : The tie member of a truss is made of ISA 65 x 65 x 6 mm is subjected to a factored tension load of 90 KN. The length of the angle is not enough to go from end to end and hence a splice has to be provided. Design a groove welded joint.

Solution:

Provide a single V groove weld

The effective throat thickness = $t_e = 5t / 8 = 5\times 6 / 8 = 3.75mm$

Perimeter length of angle available for welding = 65 + 65 = 130 mm

Strength of weld = $L_e t_e f_y / \gamma_{mw} = 3.75\times 130 \times 250 / (1.25\times 1000) = 97.5KN \text{ f } 90KN$

Area of angle = 744 mm^2

Design strength of the member = $Af_y / \gamma_{mo} = 744 \times 250 / (1.1\times 1000) = 169KN \text{ f } 90KN$

Fillet welds

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.

Fillet welds are broadly classified into side fillets and end fillets (Fig. 8.7). 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.

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.

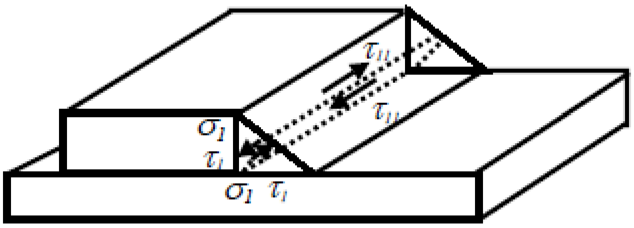


Figure 8.8 Average stress in the weld throat

In many cases, it is possible to use the simplified approach of average stresses in the weld throat (Fig. 8.8). In order to apply this method, it is important to establish equilibrium withthe 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 variationin the side fillet connections has negligible effect on strength. In general, fillet welds are stronger in compression than in tension.

Table 8.1 Minimum size of first run or of a single run fillet weld

(*Clause* 10.5.2.3)

Sl No.	Thickness of Thicker Part mm		Minimum Size mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

2 Where the thicker part is more than 50 mm thick, special precautions like pre-heating should be taken.

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 8.2. 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. 8.11(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. 8.11(b)].

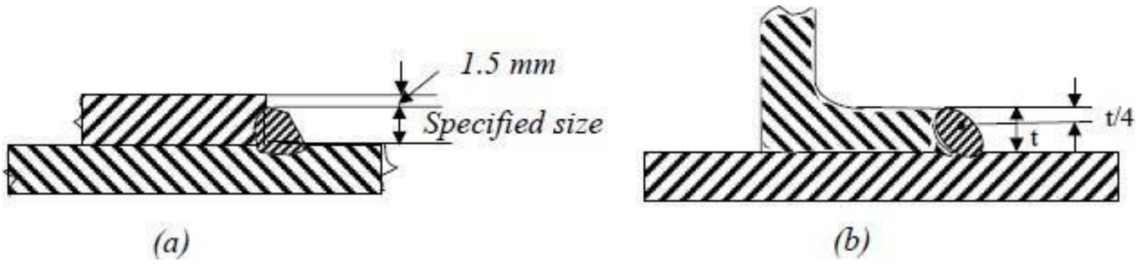


Figure 8.11 (a) fillet welds on square edge of plate, (b) fillet welds on round toe of rolled section

Table 8.2 Value of K for different angles between fusion faces

(*Clause* 10.5.3.2)

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 thethickness 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 mmwhichever 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.

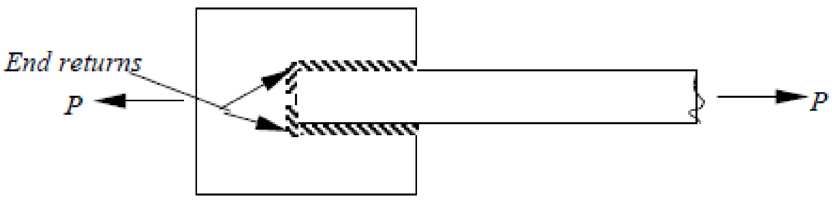


Figure 8.12 End returns

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. 8.12. 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.

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. 8.13(a). Another way of developing the required strength is by providing slot or plug welds. Slot and plug welds [Fig. 8.13(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.

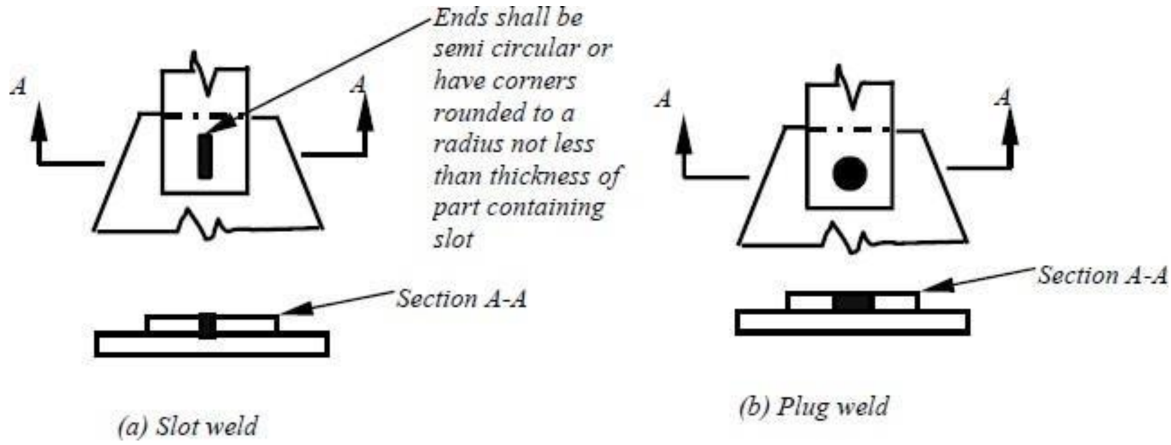
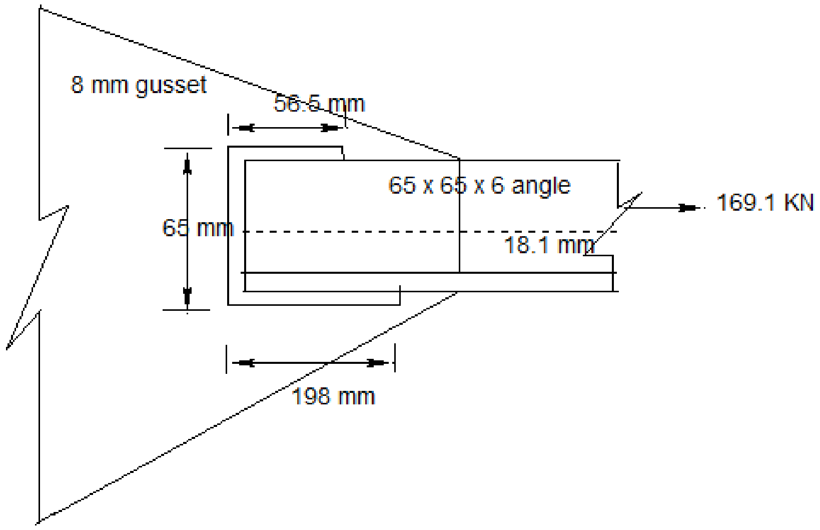


Figure 8.13 Slot and Plug welds

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 = Load/ (width) allowable stress

Example 4: Design a joint according to the instructions given in Example 3. If the welding is done on three sides of the angle as shown below.



Solution:

Strength of 4 mm weld = 2.8 x 410 / (sqrt(3) x 1.25) = 530 N/mm

P_2 =530 x 65/1000 = 34.45 KN

P_1 = Ty/d – P_2/2 = 169.1 x 18.1 / 65 – 34.45 /2
= 29.86 KN

P_3 = T-P_1 - P_2 = 169.1 -34.45 – 29.86 = 104.79 KN

L_{w1} = 29.86 x 1000/ 530 = 56.3 mm say 56.5 mm

L_{w3} = 104.79 x 1000/530 = 197.7 mm, say 198 mm

Total length of weld = 65 + 56.5 + 198 = 319.5 mm

Check for block shear failure

Since the member is welded to the gusset plate, no net areas are involved and hence A_{vn} and A_{tn} in the equation for T_{db} should be taken to be the corresponding gross areas. Using the weldment with L_1 = 198 mm L_2 = 56.5 mm and 65 mm at the end of the angle yields

T_{db1} = [8 x (198 x 2) x 250 /(sqrt(3)x1.1)+0.9 x 410 x 8x 65/1.25]/1000 = 569.2 KN

T_{db2} = [0.9 x 410 x (198 x 2/(sqrt(3) x 1.25) + 250 x 8x 65/1.1)/ 1000 = 658.1 KN

Hence

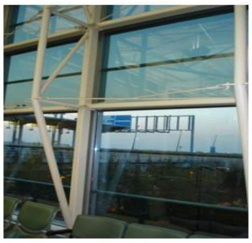
T_{db} = 569.2 KN > 169.1 KN

Hence, the thickness of gusset plate is adequate.

Note: L_2 does not enter into this calculation because a shear rupture of the gusset plate along the toe of the angle runs for the full length of the contact with the toe, 198 mm in stead of only the length L

Chapter-3

Design of Tension Members



Introduction

Tension members are linear members in which axial forces act so as to elongate (stretch) the member. A rope, for example, is a tension member. Tension members carry loads most efficiently, since the entire cross section is subjected to uniform stress. Unlike compression members, they do not fail by buckling (see chapter on compression members). Ties of trusses [Fig 9.1(a)], suspenders of cable stayed and suspension bridges [Fig. 9.1 (b)], suspenders of buildings systems hung from a central core [Fig. 9.1(c)] (such buildings are used in earthquake prone zones as a way of minimising inertia forces on the structure), and sag rods of roof purlins [Fig 9.1(d)] are other examples of tension members.

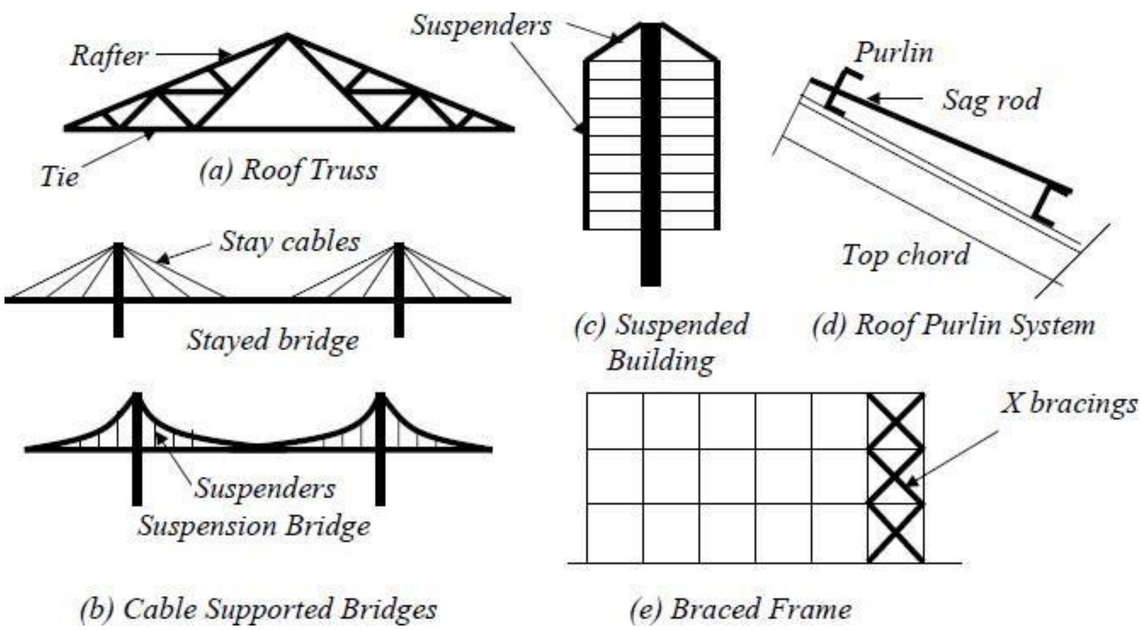


Figure 9.1 Tension Members in Structures

Tension members are also encountered as bracings used for the lateral load resistance. In X type bracings [Fig.1 (e)] the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice versa. Hence, such members may have to be designed to resist tensile and compressive forces.

The tension members can have a variety of cross sections. The single angle and double angle sections [Fig 9.2(a)] are used in light roof trusses as in industrial buildings. The tension members in bridge trusses are made of channels or I sections, acting individually or built-up [Figs. 9.2(c) and 9.2(d)]. The circular rods [Fig. 9.2 (d)] are used in bracings designed to resist loads in tension only. They buckle at very low compression and are not considered effective. Steel wire ropes [Fig. 9.2 (e)] are used as suspenders in the cable suspended bridgesand as main stays in the cable-stayed bridges.

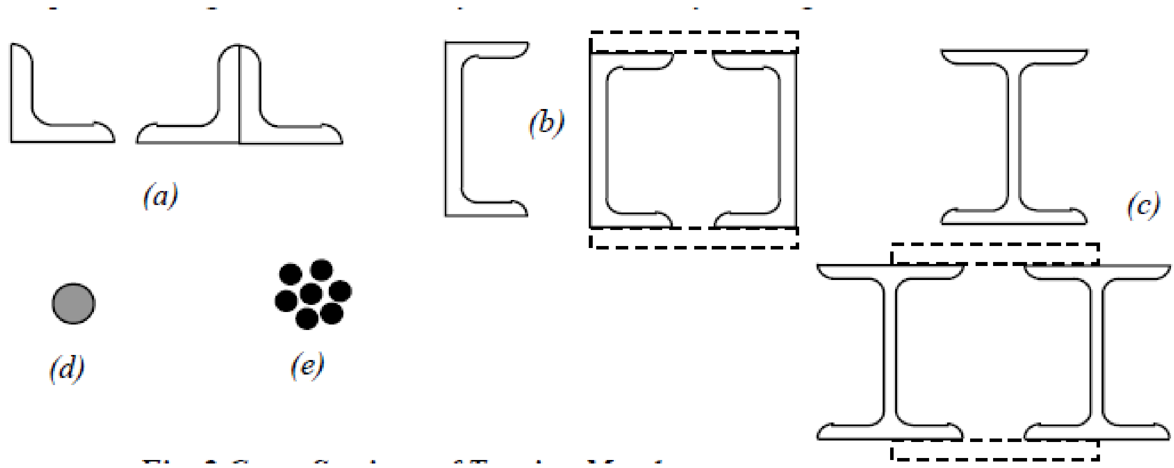


Figure 9.2 Cross section of tension members

Slenderness Ratio (Table 9.1)

Although stiffness is not required for the strength of a tension member, a minimum stiffness is stipulated by limiting the maximum slenderness ratio of the tension member.

Table 9.1 Slenderness ratio for tension members

Member	Maximum effective slenderness ratio (L/r)
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compressive forces resulting only from a combination of wind/earthquake actions, provided the deformation of such a member does not adversely affect the stresses in any part of the structure	250
A member normally acting as a tie in a roof truss or a bracing member, which is not considered effective when subject to reversal of stress resulting from the action of wind or earthquake forces	350
Members always in tension (other than pre-tensioned members)	400

Behaviour of tension members

Since axially loaded tension members are subjected to uniform tensile stress, their load deformation behaviour (Fig. 9.3) is similar to the corresponding basic material stress strain behaviour. Mild steel members (IS: 2062) exhibit an elastic range (a-b) ending at yielding (b). This is followed by yield plateau (b-c). In the Yield Plateau the load remains constant as the elongation increases to nearly ten times the yield strain. Under further stretching the material shows a smaller increase in tension with elongation (c-d), compared to the elastic range. This range is referred to as the strain hardening range. After reaching the ultimate load (d), the loading decreases as the elongation increases (de) until rupture (e). High strength steeltension members do not exhibit a well-defined yield point and a yield plateau (Fig.9.3). The 0.2% offset load, T, as shown in Fig. 9.3 is usually taken as the yield point in such cases.

9.1 Design Of Tension Members

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads up to the ultimate load, at which stage they mayfail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non- functional due to excessive elongation. Plates and other rolled sections in tension may alsofail by block shear of end bolted regions

The factored design tension T, in the members shall satisfy the following requirement.

$$T \ \mathrm{p}\, T_d \qquad (9.1)$$

Where T_d = design strength of the member under axial tension, T_d is the lowest of the design strength due to the yielding of cross-section, T_{dg}, rupture of critical section T_{dn} and block shear failure, T_{db}.

9.2 Design Strength Due To Yielding Of Gross-Section

Tension yielding of the members at the gross cross-section is given by

$$T_{dg} = f_y A_g / \gamma_{mo} \qquad (9.2a)$$

Where f_y is the yield stress of material in MPa, and A_g is the gross area of cross-section γ_{mo} is the partial safety factor for failure in tension by yielding

9.3 Design Strength Due To Rupture Of Critical Section

9.3.1 Plates

Tension rupture of the plate at the net cross-section is given by

$$T_{dn} = 0.9\, f_u A_n / \gamma_{m1} \tag{9.2b}$$

Where f_u is the ultimate stress of the material in MPa and A_n is the net effective area of the member given (as shown in the Figure 9.4 for the definition of variables). γ_{m1} is the partial safety factor for failure at ultimate stress

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

Where

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

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

g=. gauge length between the bolt holes, as shown in Fig. 9.4,

p_s=. staggered-pitch length between line of bolt holes, as shown in Fig. 9.4,
n=. number of bolt holes in the critical section, and
i= subscript for summation of all the inclined legs

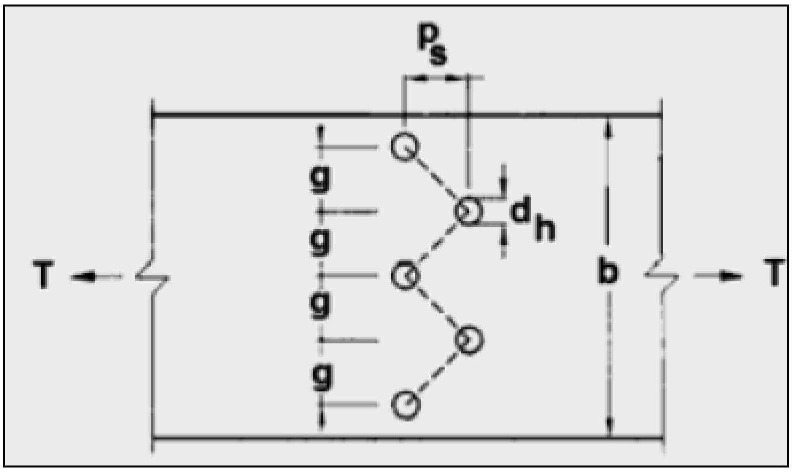


Figure 9.4 plate with the staggered holes in tension

9.3.2 The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by
$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$
Where
A_n is the net root area at the threaded section

9.3.3 Single Angles

For angle members connected through one leg, the design rupture strength T_{dn}, is calculated as:
$T_{dn} = 0.9 A_{nc} f_u / \gamma_{m1} + \beta A_{go} f_y / \gamma_{mo}$
Where
$\beta = 1.4 - 0.076 (w/t)(f_y / f_u)(b_s / L_c) \leq (f_u \gamma_{mo} / f_y \gamma_{m1}) \geq 0.7$

w=outstand leg width
b_s = shear lag width, as shown in Fig. 9.5, and
L_c = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

For preliminary sizing, the rupture strength of net section may be approximately taken as:

$T_{dn} = \alpha A_n f_u / \gamma_{m1}$

Where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length;

A_n = net area of the total cross-section;

A_{nc} = net area of the connected leg;

A_{go} = gross area of the outstanding leg; and

t= thickness of the leg.

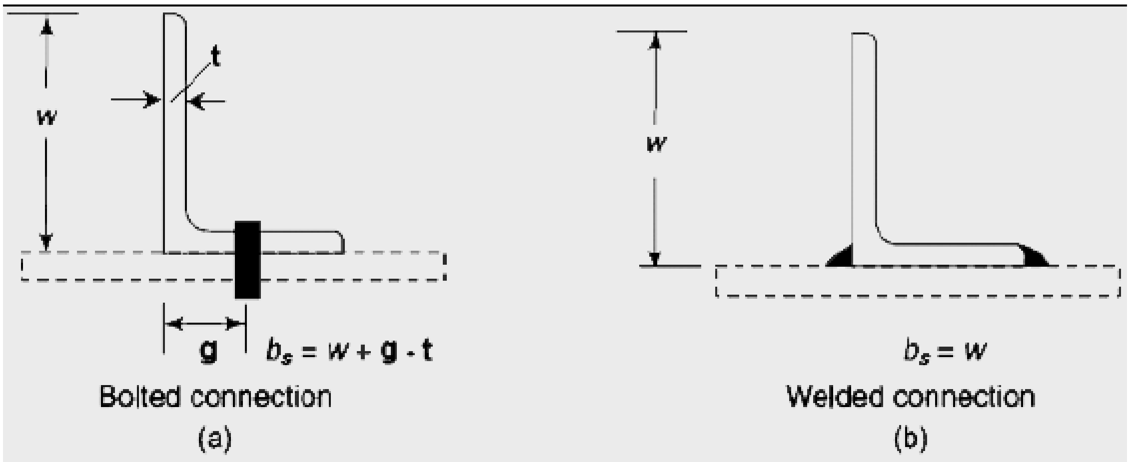


Figure 9.5 Angles with single leg connections

9.3.4 Other Sections

The rupture strength, Tdn,of the double angles, channels, I-sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in 9.4.3, where β is calculated based on the shear lag distance, b,, taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross-section.

9.4 Design Strength Due to Block Shear

Block shear failure was recognized as a failure mode first in 1978, when Birkemoe and Gilmor conducted tests on coped beams with bolted web connections, and incorporated in AISC specifications in 1978. Block shear failure in angles were investigated after the failure of Hartford Civic Center roof, Connecticut in 1978. Block shear failure in bolted / welded connections is characterized by a condition, where a "block" of material, in a pattern surrounding the bolted region, reaches its capacity through a combination of tension and shear. If the connection is loaded further, the block is eventually displaced from the connection region (see Fig. 9.6). Block shear is usually initiated with tension fracture. The block shear strength, Tdb of the connection shall be smaller of

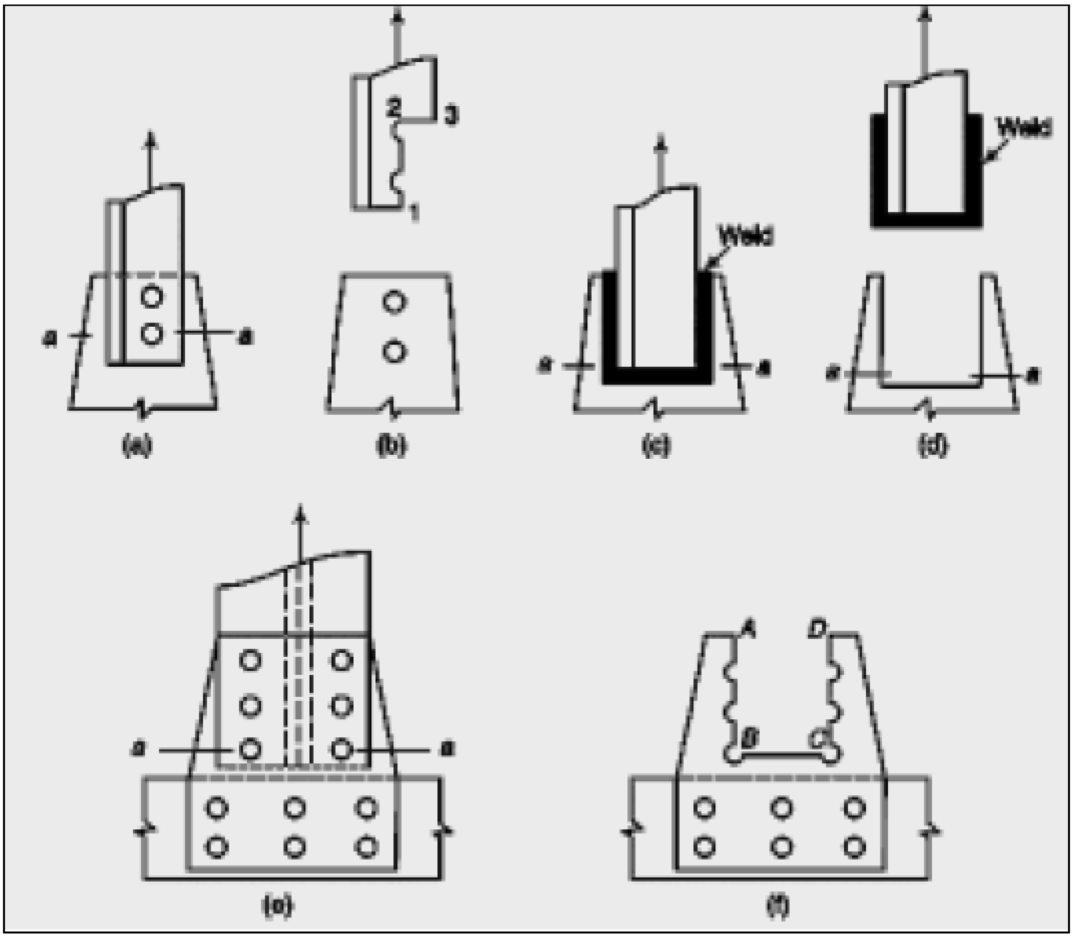


Figure 9.6 Examples Of Block Shear Failures

$$T_{db} = \left[A_{vg}\, f_y / \left(\sqrt{3}\gamma_{mo}\right) + 0.9 A_{tn}\, f_u / \gamma_{m1}\right]$$

Or

$$T_{db} = \left[0.9 A_{vn}\, f_u / \left(\sqrt{3}\gamma_{m1}\right) + A_{tg}\, f_y / \gamma_{mo}\right]$$

A_{vg} and A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively (along 1-2 in Fig.9. 6a or along A-B and D-C in Fig 9.6f), A_{tg} and A_{tn} = minimum gross and net area in tension from the hole to the toe of the angle, or next last row of bolts in plates perpendicular to the line of force, respectively (along 2-3 in Fig. 9.6a or along B-C in Fig. 9.6f).

It may be of interest to note that the American code has adopted the following block shear formula for angles, with a resistance factor of $\varphi = 0.75$

$$\varphi T_n = \varphi\left[0.6\, f_y A_{vg} + 0.5\, f_n A_{tn}\right]$$

with

$$0.6\, f_u\, A^{vg} = 0.6\, f_u\, A_v$$

10.2 Design of a tension Member

The following example is given to explain the application of tension member design provisions of the code.

A tie member in a bracing system consists of two angles 75 x 75 x 6 bolted to 10 mm gusset, one on each side using single row of bolts (Fig. 10.17a) and tack bolted. Determine the tensile capacity of the member and the number of bolts required to develop full capacity ofthe member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted (Fig. 10.17 b)? What is the effect on tensile strength if the members are not tack bolted?

Solution
a).. Two angles connected to opposite side of the gusset as in Fig. 10.17a

(i) Design strength due to yielding of gross section
$T_{dg} = f_y(A_g/\gamma_{m0})$
$A_g = 866\ mm^2$ (for a single angle)
$T_{dg} = 250 \times 2 \times (866/1.10) \times 10^{-3}$
$T_{dg} = 393.64$ kN

(ii) The design strength governed by tearing at net section
$T_{dn} = \alpha A_n(f_u/\gamma_{m1})$
Assume single line of four numbers of 20mm diameter

bolts (α=0.8)
An = [(75 - 6/2 - 22) 6 + (75 - 6/2) 6]2
An = (300 + 432)2 = 1464 mm2
Tdn =(0.8 x 1464 x 410/1.25)/1000 = 384.15 kN
Therefore Tensile capacity = 384.15 kN

Design of bolts:
Choose edge distance = 35 mm
Capacity of bolt in double shear (Table 10.8)
= 2 x 45.3 = 90.6 kN
Bearing capacity of the bolt does not govern as per
Table 10.7

Hence strength of a single bolt = 90.6 kN
Provide 5 bolts
Total strength of the bolt = 5 x 90.6= 453 kN > 384.15 kN
Hence safe.
Minimum spacing = 2.5 t = 2.5 x 20 = 50 mm
Provide a spacing of 50 mm
The arrangements of bolts are shown in Fig. 10.17c

Check for block shear strength: (clause 6.4)
Block shear strength Tdb of connection shall be taken
as the smaller of,
Tdb1 = [Avgfy / (v3m0) + 0.9Atn fu / λm1]
Tdb2 = [0.9fu Avn /(v3γm1) + fy Atg / γm0]
Avg =(4 x 50 + 35) 6 = 1410 mm2
Avn = (4 x 50 + 35 − 4.5 x 22) 6 = 816 mm2
Atn = (35.0 −22/2)6 = 144 mm2
Atg = (35 x 6) = 210 mm2
Tdb1 = [1410 x 250 / (v3 x 1.10) + 0.9 x 144 x 410 /1.25] x 10-3 = 227.5 kN
Tdb2 = [0.9 x 410 x 816/ (v3 x 1.25)] + 250 x 210 /1.10] x 10-3 = 186.8 kN

For double angle block shear strength = 2 x186.8= 373.6 kN
Therefore Tensile capacity = 373.6 kN (smallest of 393.64 kN, 384.15 kN and 373.6 kN)

b) Two angles connected to the same side of the gusset plate (Fig. 10.17b)
i. Design strength due to yielding of Gross section= 393.64 kN
ii. Design strength governed by tearing at net section= 384.15 kN
Assuming 10 bolts of 20 mm diameter, five bolts in each connected leg
Capacity of M20 bolt in single shear = 45.3 kN
Total strength of bolts = 10 x 45.3 = 453 kN> 393.64 kN

Hence the connection is safe.
The arrangement of bolts is shown in Fig. 10.12d. Since it is similar to the arrangement in Fig. 10.17c, the block shear strength well be same, i.e. 373.6 kN.
Hence the tensile capacity = 373.6 kN

The tensile capacities of both the arrangements (angles connected on the same side and connected to the opposite side of gusset) are same as per the code though the load application is eccentric in this case. Moreover, the number of bolts is 10 whereas in case (a) we used only 5 bolts since the bolts were in double shear.

c) If the angles are not tack bolted, they behave as single angles connected to gusset plate.

In this case also the tensile capacity will be the same and we have to use 10 numbers of M20 bolts. This fact is confirmed by the test and FEM results, which states that "the net section strength of double angles on opposite sides of the gusset and tack connected adequately over the length is nearly the same as that of two single angles acting individually. Current design provisions indicating greater efficiency of such double angles are not supported by test and FEM results".

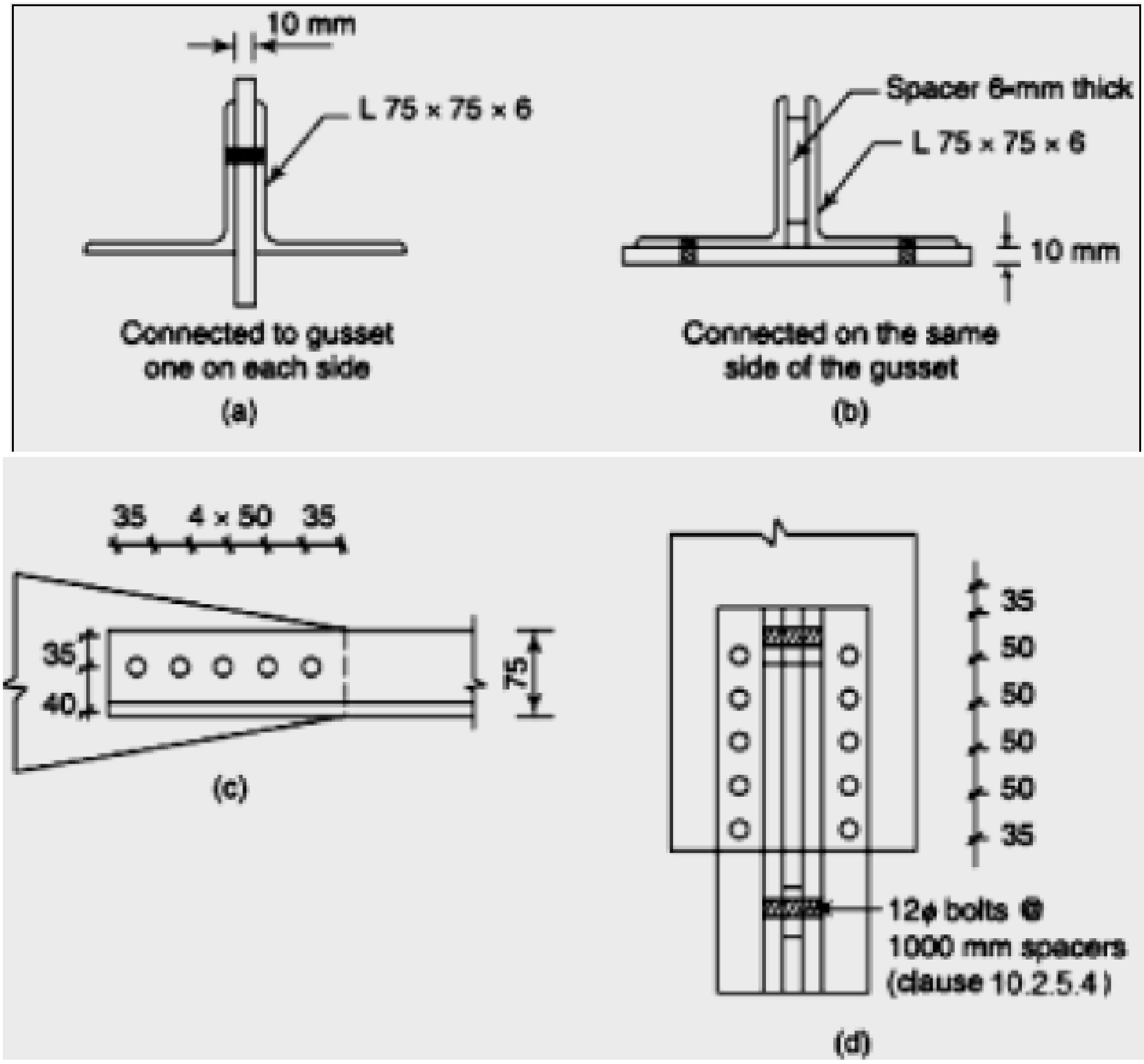


Figure 10.17 Example problem

Sample Questions in Steel Structures

Q1. A builtup steel beam comprises of two flange plates 250 mm wide x 10 mm thick and web plate 330 mm deep x 6 mm thick. The moment of inertia of the section is 162.5 x 10^6 mm^4. The allowable normal stress in the extreme fibres of flange plates is 140 N/mm^2. And the average allowable shear stress in the web plate is 100 N/mm^2. Compute the flexural and shear capacities of beam. The flange plates are welded to web plate by 6 mm intermittentfillet welds. The allowable shear stress in weld material is 100 N/mm^2. Determine thepercentage of weld length for intermittent welding at the section of maximum shear. At a section of maximum bending moment, the flange plates are to be spliced using 12 mm thick plates. The maximum allowable normal stress in splice plate is 150 N/mm^2. Determine the minimum width B of flange splice plate. The total length of flange splice plate is L and it is welded all around using 6 mm fillet weld. Determine length L.

Q2. a) Details of a double angle web member of a steel roof truss are given below. Check to see if the member has the capacity to carry tensile force of 240 KN and compressive force of 165 KN at service load. The member consists of 2-ISA 90x90x6 connected to opposite sides of 8 mm gussets using 20 mm bolts.
Effective length for buckling about xx axis = 1.7 m
Effective length for buckling about yy axis = 4.0 m
Yield strength of steel used = 250 MPa
Properties of single ISA 90x90x6 are
Area = 1047 mm^2, $\bar{y} = 24.2mm$, $I_{x'x'} = 801000mm^4$

Allowable compressive stress = $\dfrac{0.60\, f_y\, f_{cc}}{\left(f_y^{1.5} + f_{cc}^{1.5}\right)^{1/1.5}}$

Where f_{cc} = Euler buckling stress

b) Calculate the number of bolts required at the end connections of the angle to the gussets using 20 mm bolts given the allowable shear stress in the bolt = 80 MPa and allowable bearing stress = 250 MPa

Q3. Two plates are proposed to be jointed by welding. Determine the size and length of the weld required to develop the full strength of the smallest plate which is 8 cm x 1.2 cm. Assume permissible tension in plate is 1500 Kg/cm^2 and permissible shear in fillet weld is 1025 Kg/cm.

Q4. Design a compression member when effective length is 1.5 m and which carries a load of 12000 Kg. Use angle sections and data given below

Angle	Sectional area cm^2	Radious of gyration r_{min} (cm)	l/r	Allowable stress in axial compression kg/cm^2
ISA 65x65x10	12.0	1.25	70	1075

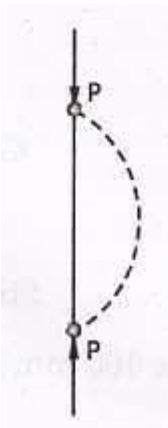
ISA 70x70x10	13.02	1.35	80	1007
ISA 75x75x10	14.02	1.45	90	928
ISA 80x80x10	17.81	1.64	100	840
ISA 90x90x10	20.12	1.64	110	753

Q5. Check the adequacy of the rafter member of a roof truss made of 2-ISA 90x90x6 double angles back to back made of the rafter member of a roof truss made of 2-ISA 90x90x6 double angles back to back made of mild steel for tension and compression given below. The length of the rafter member between nodes of the truss is 1.5 m. The effective length factor for buckling of the member in the plane of the truss is 0.85 and that for out of plane of the truss is 1.0. The member is connected to the end gussets 6 mm thick using 20 mm diameter bolts
Area of ISA 90x90x6 = $1047mm^2$, $I_{x'x'} = 801000mm^4$, distance of centre of gravity from the back = $24.2mm$, The compression in the member due to dead + live loads = 300 KN
The tension in the member due to dead load + wind uplift = 250 KN
The allowable stress in compression are as follows:

(kl/r)	30	40	50	60
σ_{ac} in MPa	145	145	139	122

Chapter-4
Compression member

DESIGN OF STEEL COMPRESSION MEMBERS A structural member loaded axially in compression is generally called a compression member. Vertical compression members in buildings are called columns, posts or stanchions. A compression member in roof trusses is called struts and in a crane is called a boom. Columns which are short are subjected to crushing and behave like members under pure compression. Columns which are long tend to buckle out of the plane of the load axis.



various end conditions

	Type	Effective length of member l
1	Effectively held in position and restrained in direction at both ends.	0.67 L
2	Effectively held in position at both ends restrained in direction at one end.	0.85 L

3	Effectively held in position at both ends but not restrained in direction.	L
4	Effectively held in position and restrained in direction at one end and at the other end effectively restrained in direction but not held in position.	L
5	Effectively held in position and restrained in direction at one end and the other end partially restrained in direction but not held in position.	1.5 L
6	Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end.	2.0 L

MAXIMUM SLENDERNESS RATIO:

According to Indian Standard IS 800, the slenderness ratio should not exceed the values given in the table below:

No.	**Type of Member**	**Slenderness ratio** $\lambda = \frac{l}{r}$
1	A member carrying compressive loads resulting from dead and superimposed loads.	180
2	A member subjected to compressive loads resulting from wind/earthquake forces provided the deformation of such members does not adversely affect the stress in any part of the structure.	250
3	A member normally carrying tension but subjected to reversal of stress due to wind or earthquake forces.	350
4	Tension member (other than pre-tensioned member)	400

Example 11.1 A single angle discontinuous strut ISA 150 mm x 150 mm x 12 mm (ISA 150 150,@0.272 kN/m) with single riveted connection is 3.5 m long. Calculate safe load carrying capacity of the section.

Solution:

Step 1: Properties of angle section

ISA 150 mm x 150 mm x 12 mm (ISA 150 150,@0.272 kN/m) is used as discontinuous strut. From the steel tables, the geometrical properties of the section are as follows:

Sectional area $A = 3459\ mm^2$

Radius of gyration $r_{xx} = r_{yy} = 149.3$ mm

Radius of gyration $r_{uu} = 58.3$ mm, $r_{vv} = 29.3$ mm

Step 2: Slenderness ratio,

Minimum radius of gyration $r_{min} = 29.3$ mm

Effective length of strut l= 3.5 m

Slenderness ratio of the strut

Step 3: Safe load

From IS:800-1984 for l/r=119.5 and the steel having yield stress, $f_y = 260\ N/mm^2$, allowable working stress in compression $\sigma_{ac} = 64.45\ N/mm^2$ (MPa)

For single angle discontinuous strut with single riveted connection, allowable working stress

$0.80\ \sigma_{ac} = (0.80 \times 64.45) = 51.56\ N/mm^2$.

The safe load carrying capacity $P = (\sigma_{ac} A) = \left(\frac{51.56 \times 3459}{1000}\right) = 178.346\ kN$

Example 11.2 In case in Example 11.1, a discontinuous strut 150 x 150 x 15 angle section is used, calculate the safe load carrying capacity of the section.

Solution:

Step 1: Properties of angle section

Angle section 150 mm x 150 mm x 15 mm is used as discontinuous strut. From the steel tables, the geometrical properties of the section are as follows:

Sectional area A = 4300 mm^2

Radius of gyration $r_{xx}= r_{yy}=45.7$ mm

Radius of gyration $r_{uu}= 57.6$ mm, $r_{vv}=29.3$ mm

Step 2: Slenderness ratio,

Minimum radius of gyration $r_{min}= 29.3$ mm

Effective length of strut l= 3.5 m

Slenderness ratio of the strut $\frac{l}{r_{min}} = \left(\frac{3.5\times1000}{29.3}\right) = 119.5$

Step 3: Safe load

From IS:800-1984 for l/r=119.5 and the steel having yield stress, $f_y=260$ N/mm^2, allowable working stress in compression σ_{ac} =64.45 N/mm^2 (MPa)

For single angle discontinuous strut with single riveted connection, allowable working stress

0.80 σ_{ac} = (0.80 x 64.45) = 51.56 N/mm^2.

The safe load carrying capacity $P = (\sigma_{ac} A) = \left(\frac{51.56 \times 4300}{1000}\right) = 221.708\ kN$

Chapter-5

Design of Steel Beams

LATERALLY SUPPORTED BEAM

•When the lateral support to the compression flange is adequate, the lateral buckling of the beam is prevented and the section flexural strength of the beam can be developed.

•The strength of I-sections depends upon the width to thickness ratio of the compression flange.

•When the width to thickness ratio is sufficiently small, the beam can be fully plastified and reach the plastic moment, such sections are classified as compact sections.

•However provided the section can also sustain the moment during the additional plastic hinge rotation till the failure mechanism is formed.Such sections are referred to as plastic sections.

•When the compression flange width to thickness ratio is larger, the compression flange may buckle locally before the complete plastification of the section occurs and the plastic moment is reached.
•Such sections are referred to as non-compact sections.

•When the width to thickness ratio of the compression

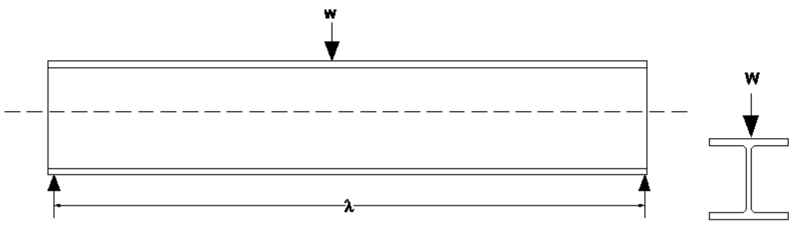
flange is sufficiently large, local buckling of compression flange may occur even before extreme fibre yields.
•Such sections are referred to as slender sections.

LATERALLY UNSUPPORTED BEAMS

•Under increasing transverse loads, a beam should attain its full plastic moment capacity.

Two important assumptions have been made therein to achieve the ideal beam behaviour.
They are:
•The compression flange of the beam is restrained from moving laterally; and

•Any form of local buckling is prevented.



1.Design a continuous beam of spans 4.9 m, 6 m, and 4.9 carrying a uniformly distributed load of **32.5 kN/m and the beam is laterally supported.**

Factored load calculation

Factored uniformly distributed load = 1.5 x 32.5 = 48.75 kN/m

The bending moment and shear force distribution are shown below

Maximum bending moment = 146.25 kNm

Maximum shear force = 146.25 + 146.25 = 292.5 kN

Plastic section modulus required

zp= = = 643.5x103mm3$M\ x\ \gamma\ m\ o\ fy$ 146.25x106x
1.10 250

Selection of suitable section

Choose a trial section of ISLB 350 @0.495 kN/m.

Overall depth *(h) = 350 mm*

Width of flange *(b)= 165 mm*

Thickness of flange *(tf) = 11.4 mm*

Depth of web (d) = *h -2(tf+ R) = 350 -2(11.4 + 16) = 295.2 mm*

Thickness of web *(tw) = 7.4 mm*

Moment of inertia about major axis *I, = 13158.3 x lo4 mm4*

Elastic section modulus *(Ze) = 75 1.9 x 103mm3*

Plastic section modulus *(Zp) = 851.11 x 103mm3*

Section classification

=$b/\ tf$+=82.5/ 11.4

$b/\ tf$ =295.2/7.4=39.9<84

Hence the section is plastic.

Check for shear capacity of section

=

Vd = fy mo x √3 x h x tw

250 1.1 x √3 x 350 x 7.4 =340kN

0.6 vd= 204 kN< 292.5 kN

This shows a high shear condition.

Check for moment capacity of the section [Eqn6.8(a)]

Mdv=Md-β(Md-Mfd) ≤ 1.09 x *Ze*x *fy*

where Mfdis the plastic design strength of the area of cross section excluding the shear area.

β = [2 x (v vd) x 1]2

= [2 x (292.5 340) x 1]2
Calculation of section modulus of flange
Zfd = Zp - Aw yw
= 851.11 x 103-(350 x 7.4 x) 350/4
= 624.485 x 103mm3

Chapter-6
Design of Tubular steel structures

Hollow Section is **a type of metal profile with a hollow tubular cross section**. HSS is only composed of structural steel per code. During the manufacturing process flat steel plate is gradually changed in shape to become round where the edges are presented ready to weld.

hollow structural section (**HSS**) is a type of metal profile with a hollow cross section. The term is used predominantly in the United States, or other countries which follow US construction or engineering terminology.

HSS members can be circular, square, or rectangular sections, although other shapes such as elliptical are also available. HSS is only composed of structural steel per code.

HSS is sometimes mistakenly referenced as *hollow structural steel*. Rectangular and square HSS are also commonly called *tube steel* or *box section*. Circular HSS are sometimes mistakenly called *steel pipe*, although true steel pipe is actually dimensioned and classed differently from HSS. (HSS dimensions are based on exterior dimensions of the profile; pipes are also manufactured to an exterior tolerance, albeit to a different standard.) The corners of HSS are heavily rounded, having a radius which is approximately twice the wall thickness. The wall thickness is uniform around the section.

In the UK, or other countries which follow British construction or engineering terminology, the term *HSS* is not used. Rather, the three basic shapes are referenced as CHS, SHS, and RHS, being circular, square, and rectangular hollow sections. Typically, these designations will also relate to metric sizes, thus the dimensions and tolerances differ slightly from HSS.

Chapter-7
Design of Masonry Structures

Masonry structures must have system of walls with cross walls at regular interval(about 3.0m to 3.5m c/c). In case cross walls are not available, need for provision of RCC columns to laterally support the long wall arises. Opening in masonry structures must be properly planned in consultation with Architect. Zones of stress /load concentrations must be identified. Introduction of RCC columns may be resorted where stress is exceeding.
All beams must rest on PCC bed blocks/or extended bearing length on masonry to keep the stress on brickwork within limit.

Mortars for brick work:
This shall be provided as per design requirements. Very approximately:

CM 1:6 - single storeyed (can be 3rd class brick in exceptional cases if design permit)

CM 1:5 - Ground storey of Two storied(with min 2nd class brick)

CM 1:4 - Ground storey of Three storied(with First class brick)

CM 1:3 - Ground storey of Four storied(with First class brick)

Curing of brick work - This is very neglected area and 10 to 14 days curing must be emphasized.
Provision of plasters also help in strengthening the walls Free standing Boundary walls must be in CM 1:5 minimum.

SP20 of Indian Standard (IS) Code provides all the guidelines for design and construction of brick masonry.

ADVANTAGES AND DEVELOPMENT OF LOADBEARING MASONRY

The basic advantage of masonry construction is that it is possible to use the same element to perform a variety of functions, which in a steelframed building, for example, have to be provided for separately, with consequent complication in detailed construction. Thus masonry may, simultaneously, provide structure, subdivision of space, thermal and acoustic insulation as well as fire and weather protection. As a material, it is relatively cheap but durable and produces external wall finishes of very acceptable appearance. Masonry construction is flexible in terms of building layout and can be constructed without very large capital expenditure on the part of the builder.

In the first half of the present century brick construction for multistorey buildings was very largely displaced by steel- and reinforcedconcrete-framed structures, although these were very often clad in brick. One of the main reasons for this was that until around 1950 loadbearing walls were proportioned by purely empirical rules, which led to excessively thick walls that were wasteful of space and material and took a great deal of time to build. The situation changed in a number of countries after 1950 with the introduction of structural codes of practice which made it possible to calculate the necessary wall thickness and masonry strengths on a more rational basis. These codes of practice were based on research programmes and building experience, and, although initially limited in scope, provided a sufficient basis for the design of buildings of up to thirty storeys. A considerable amount of research and practical experience over the past 20 years has led to the improvement and refinement of the various structural codes. As a result, the structural design of masonry buildings is approaching a level similar to that applying to steel and concrete.

1.2 BASIC DESIGN CONSIDERATIONS

Loadbearing construction is most appropriately used for buildings in which the floor area is subdivided into a relatively large number of rooms of small to medium size and in which the floor plan is repeated on each storey throughout the height of the building. These considerations give ample opportunity for disposing loadbearing walls, which are continuous from foundation to roof level and, because of the moderate floor spans, are not called upon to carry unduly heavy concentrations of vertical load. The types of buildings which are compatible with these requirements include flats, hostels, hotels and other residential buildings.

The form and wall layout for a particular building will evolve from functional requirements and site conditions and will call for collaboration between engineer and architect. The arrangement chosen will not usually be critical from the structural point of view provided that a reasonable balance is allowed between walls oriented in the principal directions of the building so as to permit the development of adequate resistance to lateral forces in both of these directions. Very unsymmetrical arrangements should be avoided as these will give rise to torsional effects under lateral loading which will be difficult to calculate and which may produce undesirable stress distributions.

Stair wells, lift shafts and service ducts play an important part in deciding layout and are often of primary importance in providing lateral rigidity.

The great variety of possible wall arrangements in a masonry building makes it rather difficult to define distinct types of structure, but a rough classification might be made as follows:

Types of Mortar

Mortar is produced by mixing a binding material (cement or lime) with fine aggregate (sand, surki, etc) with water. For construction purpose, different types of mortar are used. Depending upon the materials used for mortar mixture preparation, the mortar could be classified as follows.

1. Cement Mortar
2. Lime Mortar
3. Surki Mortar
4. Gauged Mortar
5. Mud Mortar

Cement Mortar

Cement mortar is a type of mortar where cement is used as binding material and sand is used as fine aggregate. Depending upon the desired strength, the cement to the sand proportion of cement mortar varies from 1:2 to 1:6.

Lime Mortar

Lime mortar is a type of mortar where lime (fat lime or hydraulic lime) is used as binding material and sand is used as fine aggregate. The lime to the sand proportion of cement mortar is kept 1:2. The pyramids at Giza are plastered with lime mortar.

Gauged Mortar

Gauged mortar is a type of mortar where cement and lime both are used as binding material and sand is used as fine aggregate. Basically, it is a lime mortar where cement is added to gain higher strength. The process is known as gauging. The cement to the lime proportion varies from 1:6 to 1:9. Gauged mortar is economical than cement concrete and also possess higher strength than lime mortar.

Surki Mortar

Surki mortar is a type of mortar where lime is used as binding material and surki is used as fine aggregate. Surki mortar is economic.

Mud Mortar

Mud mortar is a type of mortar where mud is used as binding material and sawdust, rice husk or cow-dung is used as fine aggregate. Mud mortar is useful where lime or cement is not available.

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