Comprehensive

DESIGN OF STEEL STRUCTURES

- Dr. B. C. PUNMIA
- ASHOK KUMAR JAIN
- ARUN KUMAR JAIN

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Contents

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SYMBOLS

A = Area of cross-section.
A_e = Equivalent area.
A_g = Gross-area of cross-section.
B = Width
a, b = Respectively the greater and lesser projections of the plate beyond the column.
b = Width
b_0 = Width of steel flange in encased member
C = The distance centre to centre of battens ; constant
C_m = Coefficient
c = Distance between vertical stiffeners.
c_1, c_2 = Respectively the lesser and greater distances from the N.A. to the extreme fibers.
D = Overall depth.
d = Depth of web
d' = Depth of girder – to be taken as the clear distance between flange angles or where there are no flange the clear distance between the flanges ignoring fillets.
d_e = Effective depth of Plate girder.
d_0 = Diameter of the reduced end of the column.
d_1 = (*i*) For the web of a column without horizontal stiff- eners – the clear distance between the flange, neglecting fillets or the clear distance between the inner toes of the flange angles as appropriate.
(*ii*) For the web of a beam with horizontal stiffeners – the clear distance between the horizontal stiffener and the toes of the tension flange angles as appropriate.
d_2 = Twice the clear distance from the N.A. of a beam to the compression flanges, neglecting fillets or the inner toes of the flange angles as appropriate.
E = The modulus of elasticity for steel, taken as 2×10^5 N/mm^2 (MPa) in this book.
e = Eccentricity.
f_y = Yield stress
f_{cb} = Elastic critical stress in bending
f_{cc} = Elastic critical stress in compression, also known as Euler critical stress.
f_c = Crushing stress
g = Gauge ; distance.
h = Outstand of the stiffener ; depth of section
h_1, h_2 = Depths
I = Moment of inertia.
K_b or K_c = Flexural stiffnesses.
k_1, k_2 = Coefficients
k = Distance from outer face of flange to web toe of filler of member to be stiffened.
L = Span/length of member.
l = Effective length of member
M = Bending moment.
N = No. of parallel planes of battens
n = Coefficient in the Merchant Rankine formula, assumed as 1.4.
P = Axial force (compressive or tensile)
P_{ac} = Calculated maximum load capacity of a strut

P_{at} = Calculated maximum load capacity as tension member.
P_e = Euler load.
P_y = Yield strength of axially loaded section.
P_c = Ultimate load for a strut
q_0 = Actual soil pressure
R = Reaction ;
r = Radius of gyration of the section.
s = Staggered pitch.
T = Mean thickness of compression flange.
t = Thickness of web.
V = Transverse shear
V_1 = Longitudinal shear
W = Total load
w = Pressure or loading on the underside of the base
Z = Section modulus.
β = Ratio of the smaller to the larger moment.
β_1, β_2 = Stiffness ratio
λ = Slenderness ratio of the member ; ratio of effective length (l) to the appropriate radius of gyration (r)
λ_0 = Characteristic slenderness ratio $= \sqrt{P_y/P_e}$
σ_{ac} = Max. permissible compressive stress in an axially loaded strut not subjected to bending.
σ_{at} = Max. permissible tensile stress on an axially loaded tension member not subjected to bending.
σ_{bs} = Max. permissible bending stress in slab base.
σ_{bc} = Max. permissible compressive stress due to bending in a member not subjected to axial force.
σ_{bt} = Max. permissible tensile stress due to bending in a member not subjected to axial force.
σ_c = Max. permissible stress in concrete in compression.
σ_e = Max. permissible equivalent stress.
σ_p = Max. permissible bearings stress in a member.
σ_{pf} = Max. permissible bearings stress in a fastener.
σ_{sc} = Max. permissible stress in steel in compression.
σ_{tf} = Max. permissible stress in axial tension in fastener.
$\sigma_{ac,cal}$ = Calculated average axial compressive stress.
$\sigma_{at,cal}$ = Calculated average stress in a member due to an axial tensile force.
$\sigma_{bc,cal}$ = Calculated compressive stress in a member due to bending about a principal axis.
$\sigma_{bt,cal}$ = Calculated tensile stress in a member due to bending about both principal axes.
τ_{va} = Max. permissible average shear stress in a member
τ_{vm} = Max. permissible shear stress in a member.
τ_{vf} = Max. permissible shear stress in fastener.
ψ = Ratio of total area of both the flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment.
ω = Ratio of moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flange each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment.

1

Introduction

1.1. STRUCTURAL DESIGN

A *structure* is a body, composed of several *structural elements* so assembled that it can set up *resistance* against deformation caused due to application of *external forces*. *Structural Engineering* is that branch of Civil Engineering which deals with both the *structural analysis* as well as *structural design*. The various *structural elements* that may be present in a structure are: (*i*) tension members (*ii*) compression members (*iii*) flexural members (*iv*) torsional members, and (*v*) foundation elements. The *structural analysis* deals with the determination of internal stresses in these members as well as the determination of reaction components, when the structure is subjected to external forces. The methods of analysis and principles involved in structural analysis do not normally depend upon the type of material used for various structural components. *Structural design* is taken up after the structural analysis is over. The *design* of a structure has two aspects: (*i*) functional aspect, and (*ii*) strength aspect. In the first aspect of design, called the *functional design*, a structure is so proportioned and constructed that it serves the needs efficiently for which it is constructed. In the second aspect, called *structural design*, the structure should be strong enough to resist external forces to which it is subjected during its entire period of service.

The following are the requirements that govern the *structural design* :

(*i*) It should have adequate strength.

(*ii*) It should have adequate stability and rigidity.

(*iii*) It should be durable.

(*iv*) It should not interfere with the functional requirements.

(*v*) It should be economical.

(*vi*) It should be readily adaptable to future extension.

The aim of the structural designer is to produce a *safe* and *economical* structure to meet certain functional and esthetic requirements. Structural design is to a great extent an art based on *creative ability*, *imagination* and *experience* of the designer. The designer must have a thorough knowledge of structural behavior, of structural analysis and of correlation between the layout and the function of a structure, along with the appreciation of esthetic values. The structural designer uses his knowledge of structural mechanics, the codes of practice and practical experience to produce a safe design.

Structures may also be classified on the basis of materials used for construction, as follows:

(*i*) Steel structures
(*ii*) Aluminum structures
(*iii*) Timber structures
(*iv*) Plastic structures
(*v*) Masonry structures
(*vi*) Concrete structures
(*vii*) Composite structures.

Another classification that applies to buildings, made wholly or in part, of steel is in accordance with the type of structural framework:

1. Wall-bearing structure
2. Beam and column structure
3. Long-span framing
   (*a*) Plate girders
   (*b*) Trusses and mill building frames.
   (*c*) Rigid frames.
   (*d*) Arches.
   (*e*) Suspension systems.

In the case of wall-bearing construction, columns are avoided, and the roof structure is supported directly on the walls. Wall-bearing construction has been almost completely superseded by the skeleton frame in large or heavily loaded buildings. Wall bearing buildings normally are not highly resistant to seismic loading. Almost all multi-storey skeleton-frame steel buildings (*tier building*) are of beam and column framing. Long span industrial buildings may employ girders or trusses supported on stanchions, or may use rigid frames. Long-span framing is used to obtain wide, unobstructed floor areas or to carry loads that are too heavy for rolled sections.

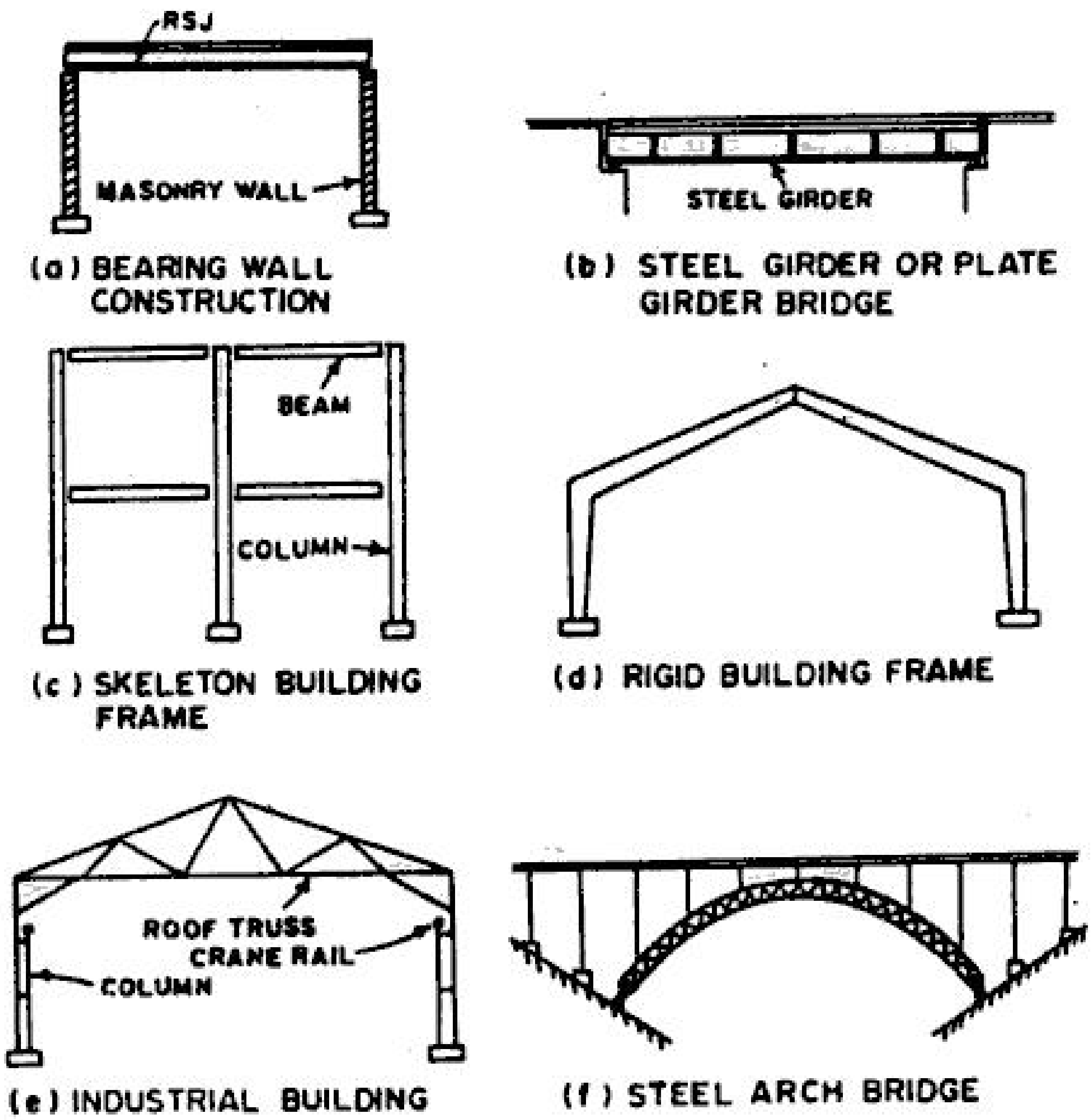


FIG. 1.1. TYPES OF STEEL STRUCTURES

1.3. 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 : ties.
(*iii*) Compression members : columns, stanchions, struts.
(*iv*) Torsional members.

and (*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.

Fig. 1.2 shows typical details of framing for multistorey building while Fig. 1.3 shows the components of an industrial building.

The *structural elements* are made up of the following commonly used structural shapes and built-up sections shown in Fig. 1.4.

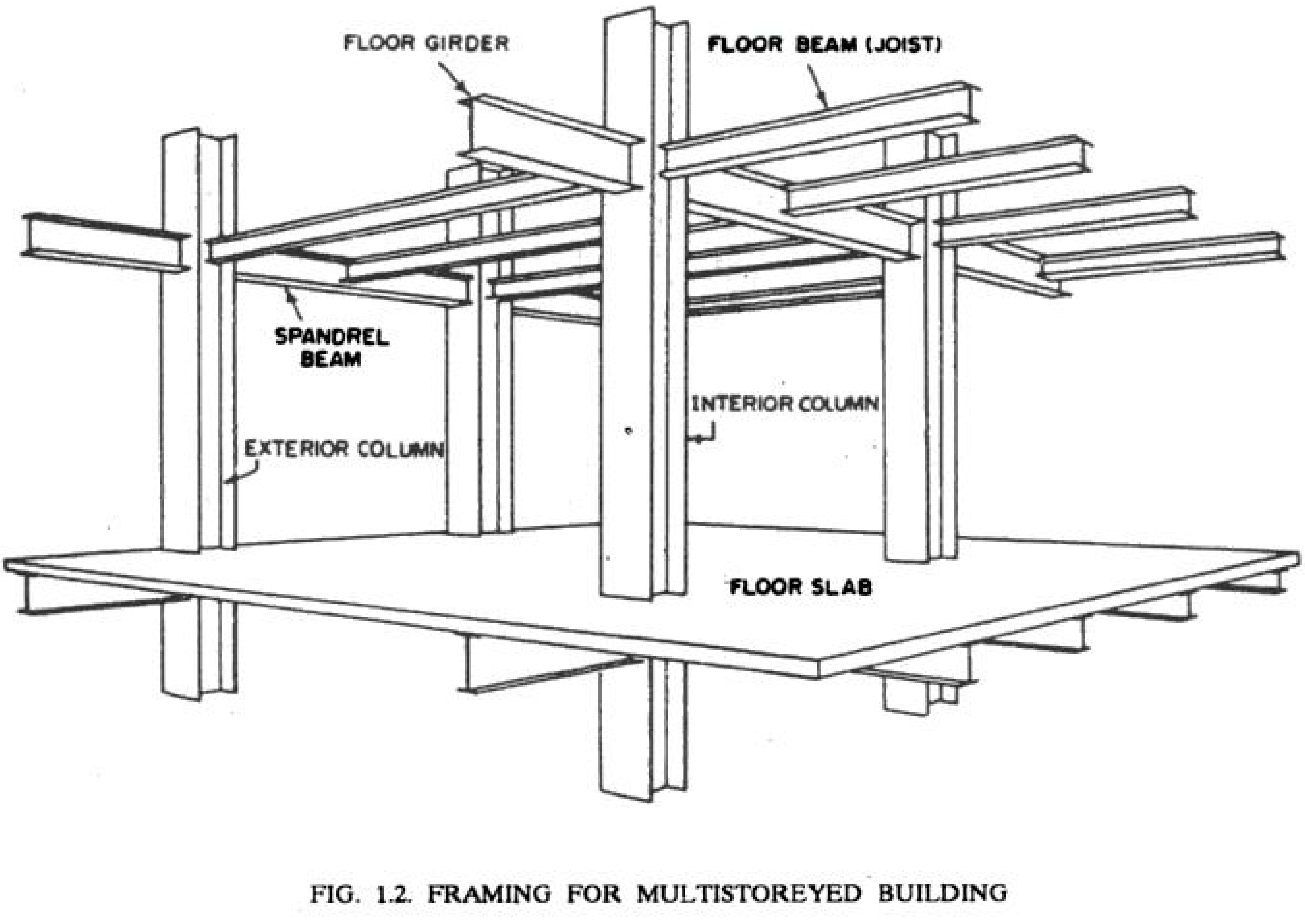


FIG. 1.2. FRAMING FOR MULTISTOREYED BUILDING

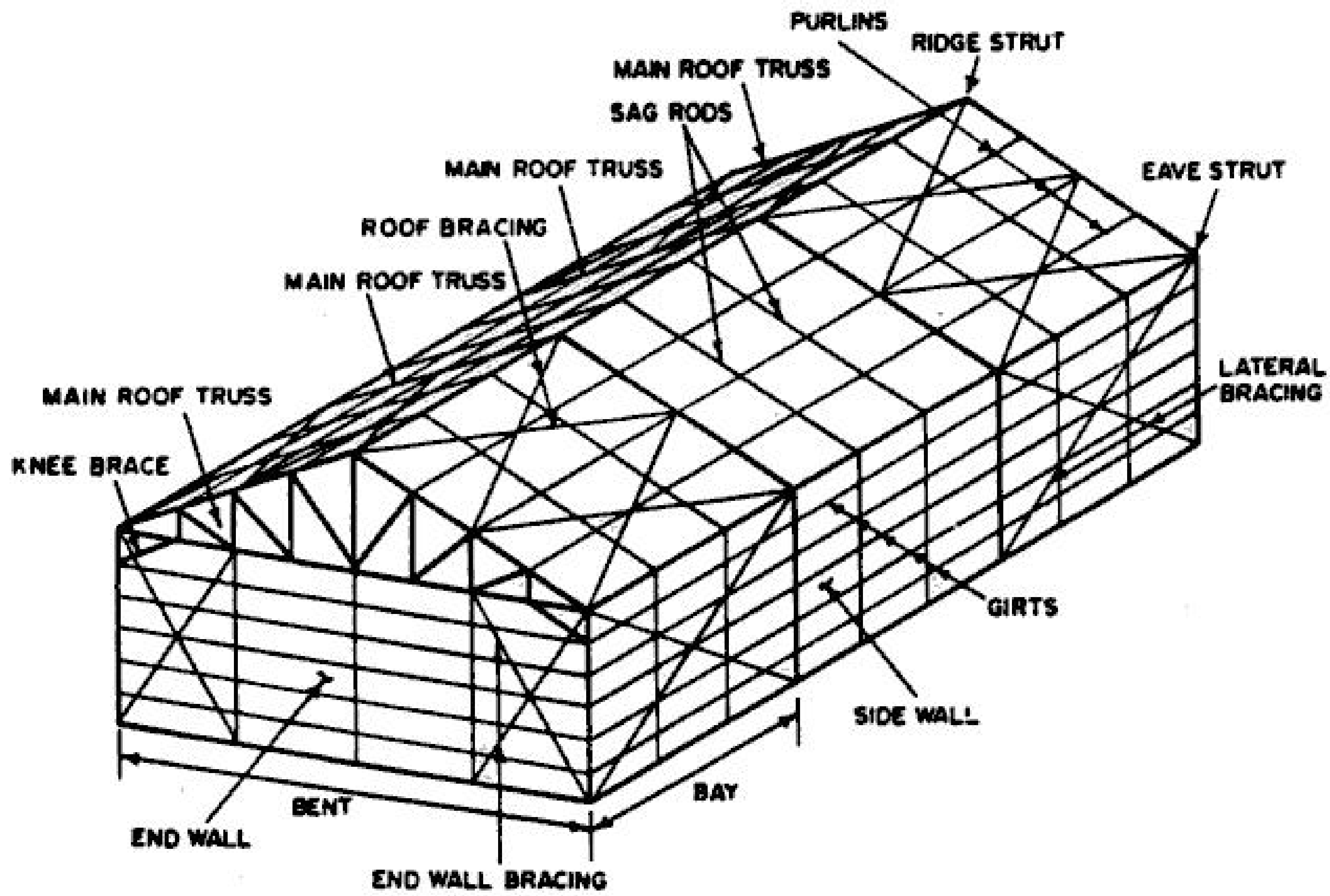


FIG. 1.3. FRAMING FOR AN INDUSTRIAL BUILDING

(*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.

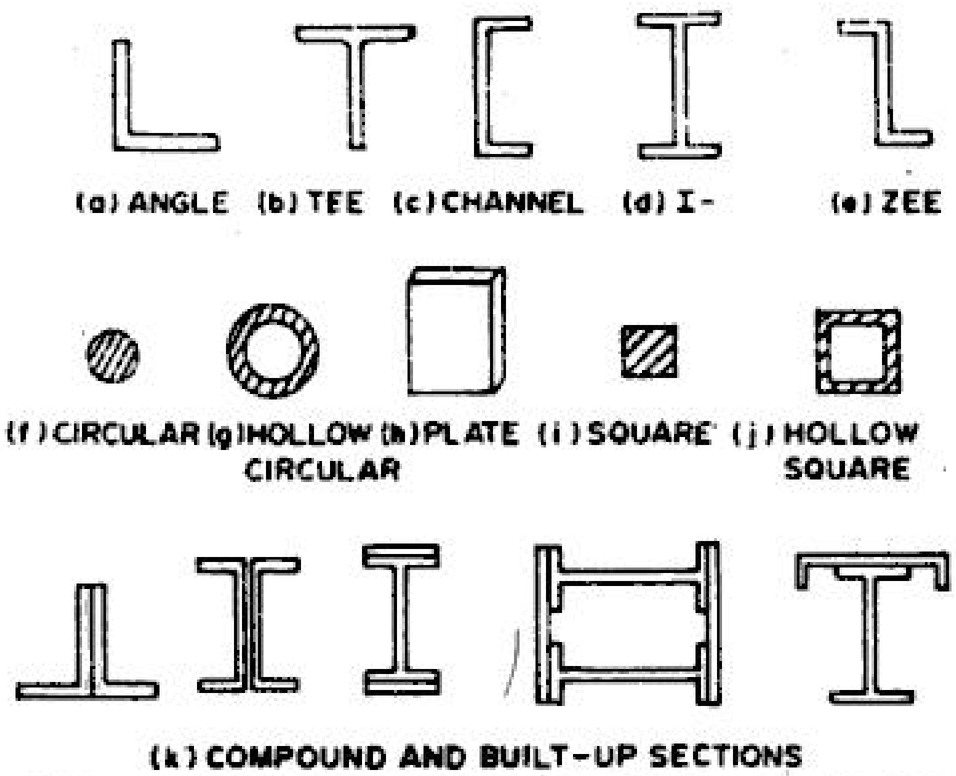


FIG. 1.4. COMMONLY USED STRUCTURAL SHAPES.

1.4. BEAMS AND GIRDERS

A *beam* is a structural member the primary function of which is to support loads normal to its axis. The word *beam* and the word *girder* are used more or less interchangeably. However, the word *girder* may mean either a built-up member (usually a heavy one) or a main beam (single rolled shaped or built-up) which supports other beams. In a beam, loads are resisted by bending and shear, but local stress conditions and deflection are also important considerations.

Beams in structures may also be referred to by typical names that suggest their function in the structure, as given below:

(*i*) ***Girder*** : Usually indicate a major beam frequently at wide spacing that supports small beams.

(*ii*) ***Joists*** : Closely spaced beams supporting the floors and roofs of buildings.

(*iii*) ***Purlins*** : Roof beams usually supported by trusses.

(*iv*) ***Rafters*** : Roof beams usually supported by purlins.

(*v*) ***Lintel*** : Beam over window or door openings that support the wall above.

(*vi*) ***Girts*** : Horizontal wall beams used to support wall covering on the side of an industrial building

(*vii*) ***Spandrel beam*** : Beam around the outside perimeter of a floor that support the exterior walls and the outside edge of the floor.

The term *beam-column* is used for that structural element that supports both transverse and axial loads.

Fig. 1.5 shows some of the commonly used beams sections. The optimum section for *flexural resistance* is the one in which the material is located as far as possible from the neutral axis. The *angle section* (Fig. 1.5 *a*) is not an efficient beam shape, though it may be good for short, lightly loaded spans where the flat leg may be used to support some other element of the structure, such as floor or roof deck. The *channel section* (Fig. 1.5 *b*) is also used for light loads, such as for *purlins* and *girts*. The *I-section* (Fig. 1.5 *c*), known as *universal beam*, is most commonly used for wall supported structures. Fig. 1.5 (*d*) shows a composite section, made of I-section with thin web and with flat plates attached to flanges. This gives higher percentage of material concentrated in the flange, resulting in higher elastic section modulus for the same mass per unit length. Fig. 1.5 (*e*) shows composite beam section commonly used as *gantry girders* for cranes. The *bar joist* shown in Fig. 1.5 (*f*) is a light, trussed beam very widely used for floor and roof framing in lightly loaded buildings. The flanges or chords of such a section can be commonly seen on railway platforms. Fig. 1.5 (*g*) shows a plate-girder used for heavy loads in buildings and bridges. Fig. 1.5 (*h*) shows *double-web box girders* particularly useful for heavy, flexural members subjected also to torsion or direct stress. Fig. 1.5 (*i*) shows a *castellated beam* giving an increased depth of the rolled beam by castellating. To obtain such a section, a zig-zag line is cut along the beam web by an automatic flame cutting machine. The two halves thus produced are rearranged so that the teeth match up and the teeth are then welded together.

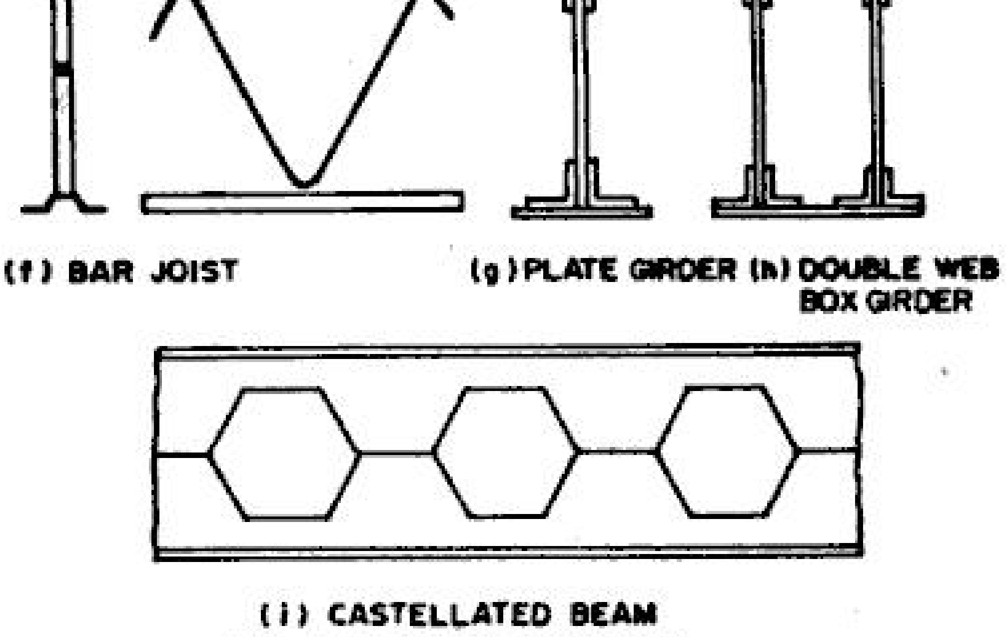


FIG. 1.5. BEAM SECTIONS.

1.5. TENSION MEMBERS

A tension member is the one which is intended to resist axial tension. Tension members are also called *ties* or *hangers*. The cross-sectional arrangement of material in axially stressed tension members is structurally unimportant. Cross-sections of some common tension members are shown in Fig. 1.6.

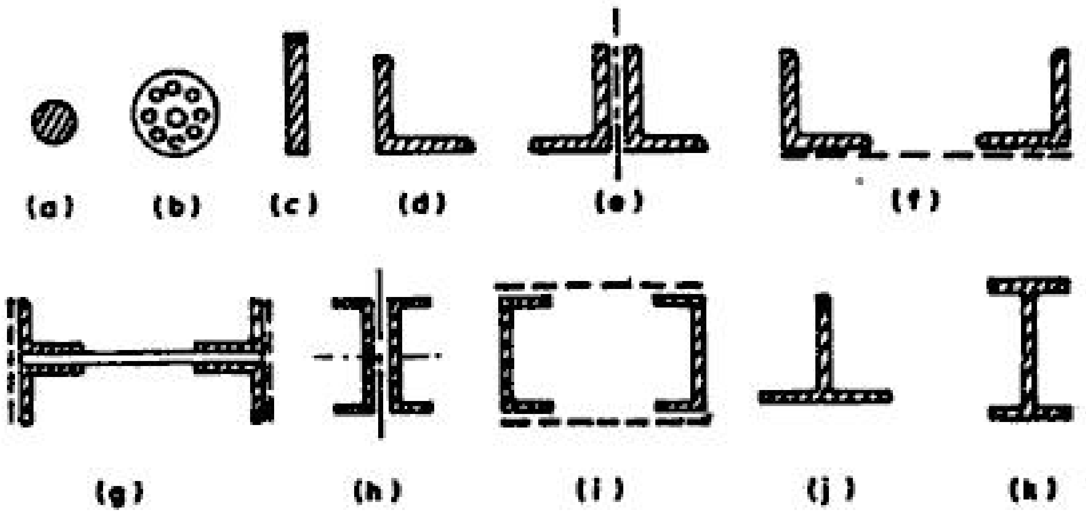


FIG. 1.6. TYPICAL TENSION MEMBERS.

Fig. 1.6 (*a*) shows an ordinary rod frequently used as tension member in bracing buildings, and as tension member in timber trusses. *Wires, ropes, bridge strand* and *cables*, shown in Fig. 1.6 (*b*) are extremely versatile, mostly used for suspension structures. Fig 1.6 (*c*) shows a *flat* or *rectangular bar* once used extensively as *eyebars* (with enlarged head containing a hole through which pin may pass) in pin connected bridges. Fig. 1.6 (*d*) and (*e*) show *single angle* and *double angle* members extensively used in *single-plane* (*single gusset*) trusses. Fig. 1.6. (*f*) and (*g*) show *two angle* and *four angle* members frequently used in light *double plane* (*double gusset*) rivetted trusses. Fig 1.6 (*h*) shows a form of tension member sometimes used for single plane trusses when bending must also be resisted in addition to axial tension. Form shown in Fig. 1.6 (*i*) is used for heavier trusses, with their open sides provided with intermittent tie plates or lattice bars as shown by horizontal dotted lines. *Structural Tee* shown in Fig 1.6 (*j*) make excellent chord members for lightly loaded welded trusses, since the stem may serve as a gusset for the attachment of single-angle or double angle web members. The *I-members* (Fig. 1.6 *k*) are used as tension members in heavier building or bridge trusses with double-plane construction.

1.6. COMPRESSION MEMBERS

Compression members, also called *columns*, *struts*, *posts* or *stanchions* are intended primarily to resist compressive stress. The requirements for compression members are more demanding than those for tension members, since in this case the carrying capacity is a function of shape as well as of area and material properties. The buckling of the column in any possible direction becomes a governing criterion. Some of the commonly used compression sections are shown in Fig. 1.7.

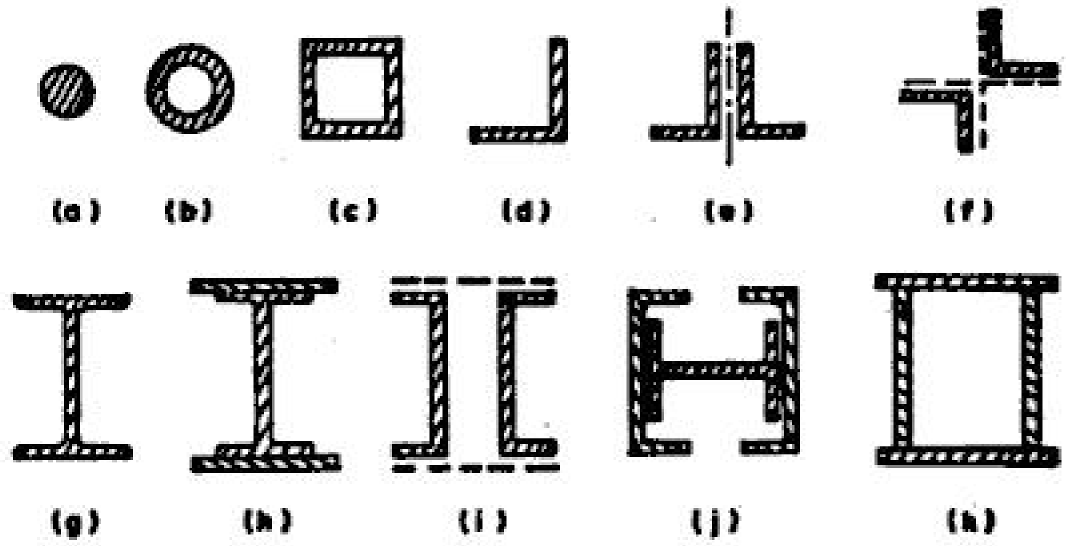


FIG. 1.7. TYPICAL COMPRESSION MEMBERS.

Fig. 1.7 (*a*) shows *solid circular section*, which is used as compression member in machines and special structures such as legs of tall, guyed transmission towers. The *cylindrical tube* (or hollow circular section) shown in Fig. 1.7 (*b*) is the optimum section for a column with equal unbraced lengths in each direction. Such sections are extensively used in *tubular trusses*. However, there are connection problems in such a section. Fig. 1.7 (*c*) shows a *square* or *rec-*

tangular tube, where the efficiency of a circular tube may be approached and at the same time the connection problem can be made simpler. The *single angle section*, shown in Fig. 1.7 (*d*) are useful only for truss members and compression legs of towers.

The double angle section (Fig 1.7 *e*) is commonly used in trusses, while the *cruciform arrangement* of the double angles (Fig 1.7 *f*) gives approximately equal radii of gyration in two directions. The I–section shown in Fig 1.5 (*g*) is commonly used in buildings because of its easy availability in different sizes and ease with which it can be spliced and connected. If the requirement of area is in excess of available section, additional plates can be attached to the flanges as shown in Fig 1.7 (*h*). Fig. 1.7 (*i*), (*j*) and (*k*) show built-up column sections, to carry heavy loads.

1.7. STRUCTURAL STEEL

Steel is probably the most *versatile* commonly used structural material. Not only is its versatility apparent in great variety of structures for which it is used but also in many different forms possible in a single building structure or a complex of structures. Many of the properties of structural steel of interest to the designer can be described by the behavior of steel during a simple tension test.

The essential elements in *steel* are *metallic iron* and the element *non-metallic carbon*, with small quantities of other elements such as silicon, nickel, manganese, chromium and copper. It is thus an *alloy*. Though steel is usually more than 98% iron, with other elements present in small quantities, these other elements have pronounced effect on the properties of steel.

Various *iron-carbon alloys*, used as structural material are of three types: (*i*) *cast iron*, (*ii*) *wrought iron*, and (*iii*) *steel*. *Cast iron* has a low carbon content, while *wrought iron* has high carbon content. In many ways, *steels* are intermediate in carbon content, between cast iron and wrought iron. The approximate limits for carbon in *steel* are between 0.04 to 2.25 percent, though the limits for carbon in *structural steel* are between 0.15 to 1.7 percent.

Cast iron: Cast iron has low carbon percentage, which makes it very brittle. The first use of cast iron as *structural material* was on a 100 ft. span bridge over Severn River at Coolbrookdate, England built in 1779, and which is *still in service*. During the period 1780 to 1820, many more cast iron bridges were built. However its use declined thereafter, because of (*i*) failures due to brittle facture in tension, and (*ii*) availability of wrought iron shapes commercially.

Wrought iron : Wrought iron has high carbon content, imparting it the ability to permit large deformations without fracture. Due to this quality, it replaced cast iron. The wrought iron could be formed into plates which can, in turn, be cut and shaped into structural members. The early example of use of wrought iron was the Britannia Bridge across the Menai straits in Wales completed in 1850.

Steel : Steel has carbon content intermediate between cast iron and wrought iron. With the development of *Bessemer process* in 1856 and *open-hearth* steel-making furnace in 1864, large quantities of steel became available for the first time. Many rolled sections, such as rolled bars and I-shapes were made available by 1870. This resulted in the replacement of wrought iron by steel, in construction industry, by 1890.

Depending upon the chemical composition, different types of steels are classified as (*i*) mild steel (*ii*) medium carbon steel (*iii*) high carbon steel (*iv*) low alloy steel and (*v*) high alloy steel. Out of these, the first three types of steels are known as *structural steel*, commonly used in steel structures. Indian standard IS:800-1984 (Code of practice for general construction in steel) is applicable to the types of structural steels covered by the following Indian Standards:

1.	IS :	226-1975	Structural steel (standard quality)
2.	IS :	1977-1975	Structural steel (ordinary quality)
3.	IS :	2062-1984	Weldable structural steel
4.	IS :	961-1975	Structural steel (high tensile)
5.	IS :	8500-1977	Weldable structural steel (medium and high strength qualities)

Structural Steel (standard quality) IS : 226-1975

Steel in this quality is known as *mild steel*, designated as St 44-S while the one in *copper bearing quality* is designated as St 44-SC with copper content varying from 0.2 to 0.35%. *Mild steel* is used for manufacture of *rolled steel sections*, rivets and bolts. Steel conforming to IS : 226-1975 is suitable for all types of structures subjected to static, dynamic and cyclic loading, and is suitable for welding upto 20 mm thickness. The chemical composition of this steel is given in Table 1.1.

TABLE 1.1 CHEMICAL COMPOSITION OF STEEL IS:226-1975

Constituents	*Max. percent*
Carbon (for thickness/dia. upto 20 mm)	0.23
Carbon (for thickness/dia. over 20 mm)	0.25
Sulphur	0.055
Phosphorus	0.055

The physical properties of mild steels are as under:

(*i*) *Mass* : 7.85 kg/cm^3 (7850 kg/m^3)

(*ii*) *Young's modulus of elasticity* (E) : 2.04×10^5 MPa (or N/mm^2)

(*iii*) *Modulus of Rigidity* (G) : 0.785×10^5 MPa (or N/mm^2)

(*iv*) *Poisson's Ratio* (μ) : 0.3 (in elastic range)

(*iv*) *Coefficient of thermal expansion or contraction* : 12×10^{-6} per °C or 6.7×10^{-6} per °F

1.8. 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.

The *dimensions* and *properties* of all these sections are given in 'ISI Hand Book for Structural Engineers', Vol. 1, *Structural Steel Sections*'.

1. Rolled Steel Beam Sections

ISI hand book gives five series of beam sections:

(*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)

and (*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, there are two sections of ISWB 600; 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.

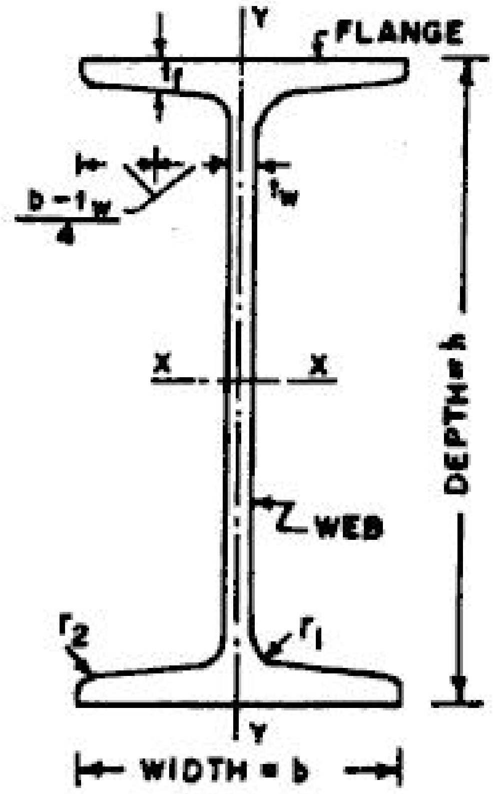


FIG. 1.8. ROLLED STEEL BEAM SECTION

2 Rolled Steel Channel Sections

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 channel 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.

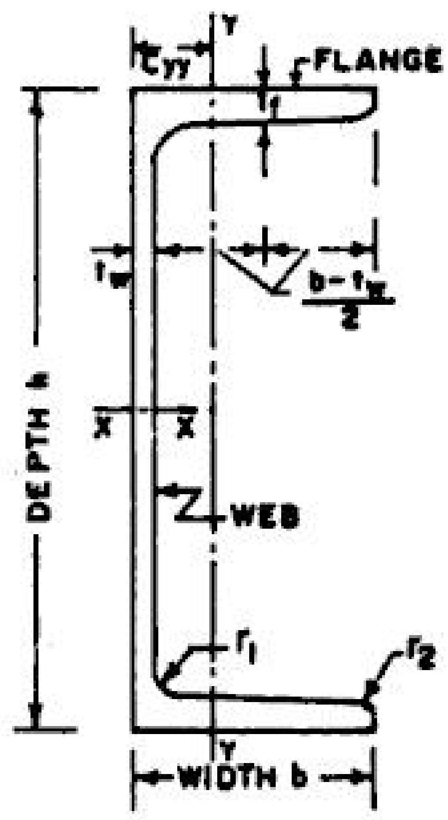


FIG. 1.9. ROLLED STEEL CHANNEL SECTION

3. Rolled Steel Angle Sections

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)

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,

1.9. METHODS OF DESIGN

Structural design is a process by which a structure required to perform a given function is proportioned to satisfy certain performance criterion in a safe and economic way. To any given structural performance specification, there may be a large number of solutions which will atleast satisfy the safety criterion, although many will clearly be uneconomic. However, one important point that should be noted is that solutions aimed at using the smallest amount of material (*i.e.* minimum weight designs) are very often not the most economical.

There are two philosophies for design of structures

(*i*) Elastic design or Working stress design (*ii*) Plastic design

The basic premise of the *elastic design method* is that the attainment of the yield stress at any point in the structure marks the end of acceptable behavior. The structural components are designed on the basis of *working stress* defined by the relation:

$$\frac{\text{Working stress}}{(\text{or safe permissible stress})} \leq \frac{\text{Yield stress}}{\text{Factor of safety}}$$

In the elastic design, the computed stresses are well within the elastic range, i.e. stresses are proportional to strains. In terms of a beam, for example, the safety criterion in working stress design may be expressed as

$$\left[f_b = \frac{M \cdot y_{max}}{I}\right] \leq \left[\sigma_b = \frac{f_y}{F_s}\left(\text{or } \frac{f_{cr}}{F_s'}\right)\right]$$

where f_b is the unit stress at the extreme fibre of the beam cross-section, caused by the maximum service load moment M and computed under the assumption that the beam is elastic, $\sigma_b =$ allowable stress obtained by dividing the limiting stress, such as yield stress (f_y) or the buckling stress f_{cr} by a factor of safety (F_s or F_s').

In the *plastic design method*, the usefulness of the material is limited upto ultimate load (or collapse load). The method, while taking account of the plastic extension of steel, is able to predict the load which should just cause the structure to collapse. This method has its main application in the analysis and design of statically indeterminate structures. The *plastic design* in *limit state terms* is, in fact, based on the ultimate limit state of collapse. This method provides the margin of safety in terms of *load factor*. Thus, the *working load* is determined in terms of *collapse load* and appropriate load factor by the following relationship :

$$\text{Working load} = \frac{\text{Collapse load}}{\text{Load factor}}$$

Indian Standard Code IS:800-1984 (Code of practice for general construction of steel) recommends the following methods of design of *steel frame work*:

1. Simple design
2. Semi-rigid design,

and 3. Fully rigid design.

1. Simple Design

This method applies to structures in which the end connections between the members are such that they will not develop restraint moments adversely affecting the members and the structure as a whole and in consequence the structure may, for the purpose of design, be assumed to be *pin-jointed*.

The *method of simple design* involves the following assumptions (IS : 800-1984):

(*i*) The beams are simply supported.

(*ii*) All connections of beams, girders or trusses are virtually flexible and are proportioned for the reaction shears applied at the appropriate eccentricity.

(*iii*) Members in compression are subjected to forces applied at the appropriate eccentricities with the appropriate effective length.

(*iv*) Members in tension are subjected to longitudinal force applied over the *net area* of the section.

2 Semi Rigid Design

This method assumes *partial fixidity* at the ends and hence as compared to the simple design method, permits a reduction in the maximum bending moment in the beams suitably connected to their supports. It thus provides a *degree of direction fixidity*, and in the case of triangulated frames, it permits account being taken of the rigidity of the connections and the moment of interaction of members. In cases where this method of design is employed, calculations based on general or particular experimental evidence shall be made to show that the stresses in any part of the structure are not in excess of those laid down in the code. Stress investigations may also be done on the finished structure for assurance that the actual stresses under specific design loads are not in excess of those laid down in IS : 800-1984.

3. Fully Rigid Design

This method, as compared with the methods of simple and semi-rigid designs gives the greatest rigidity and economy in the weight of steel used when applied in appropriate cases. The end connections of the members of the frame shall have sufficient rigidity to hold the original angles between such members and the members they connect virtually unchanged. Unless otherwise specified, the design shall be based on theoretical methods of elastic analysis and the calculated stresses shall conform to the relevant provisions of IS : 800-1984. Alternatively, it shall be based on the principles of plastic design.

The end connection behavior which distinguishes the three types of construction (and hence the three types of design) are illustrated in Fig. 1.13.

In the strict sense of term, all connections are *semi-rigid*, complete flexibility and complete rigidity being ideal conditions which cannot be attained in practice. The lightest *framing*

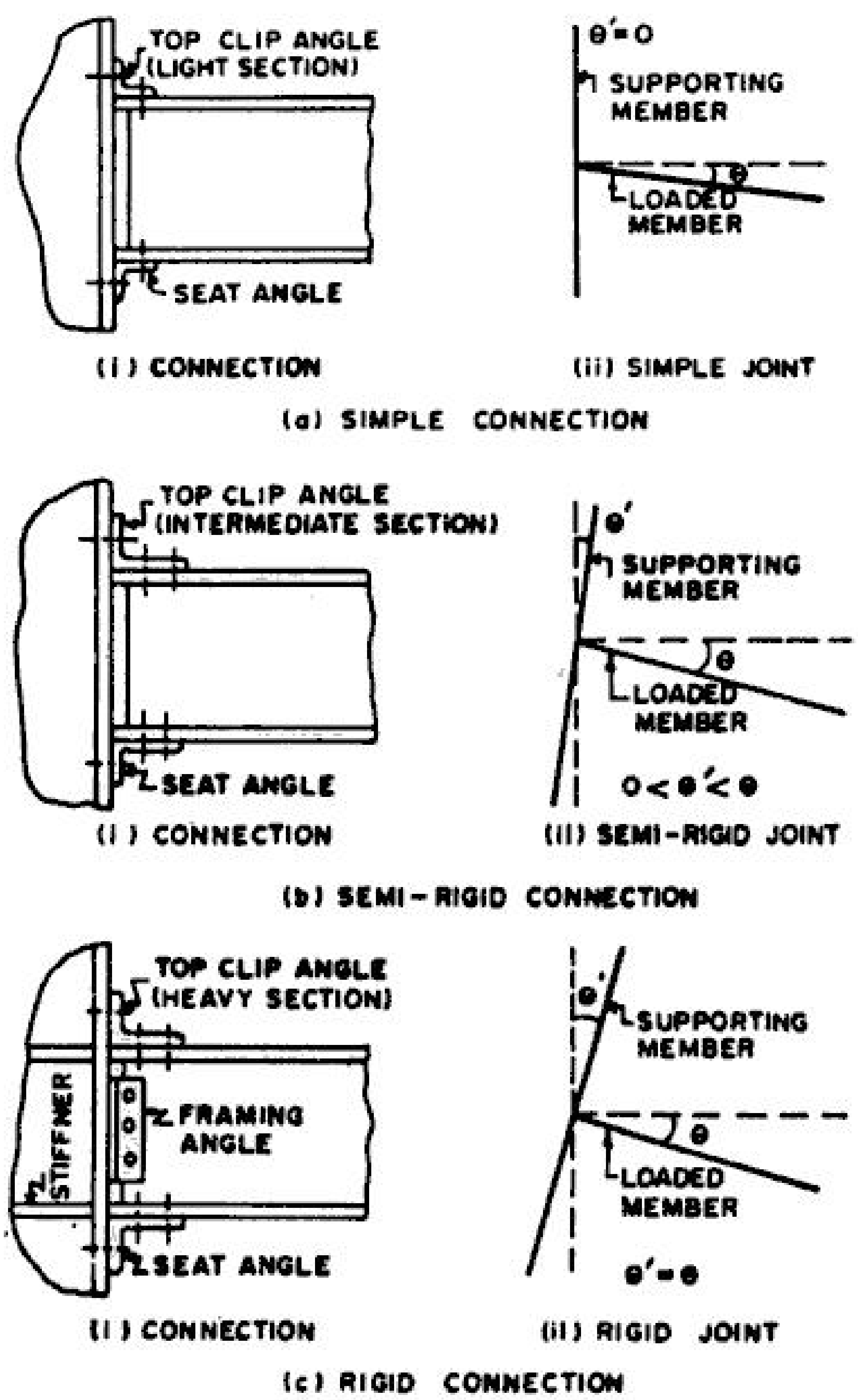
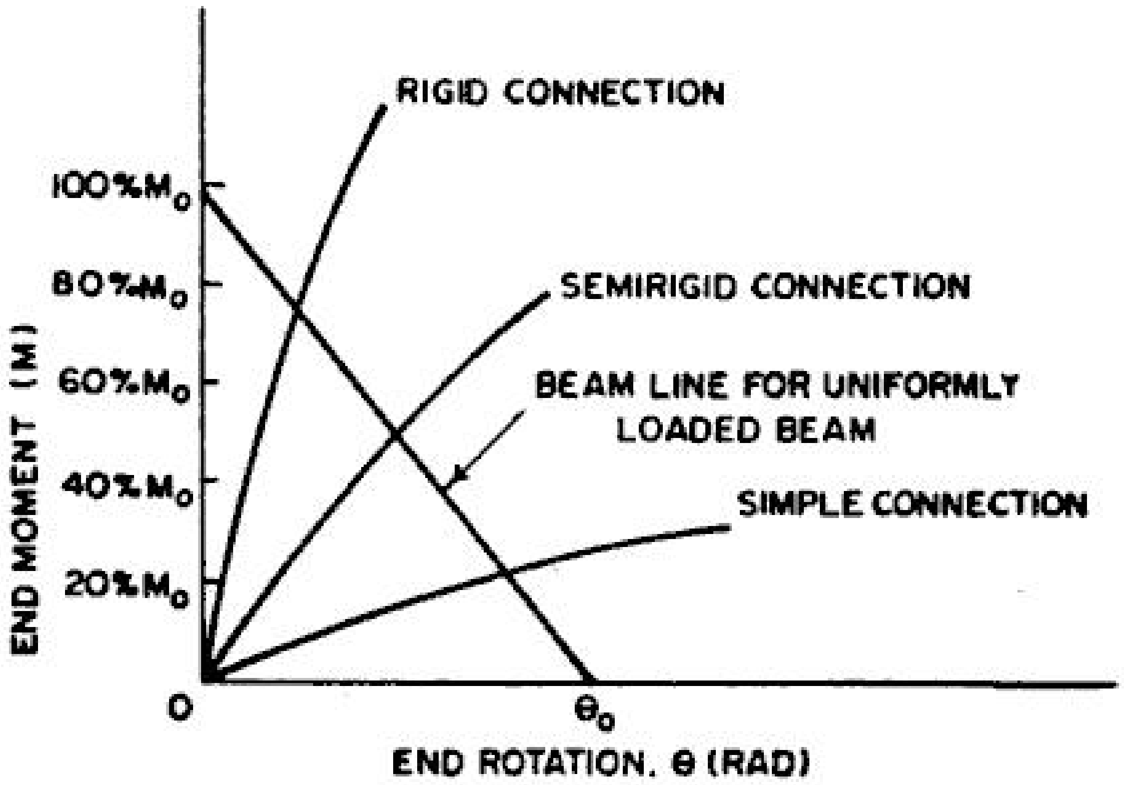


FIG. 1.13. TYPICAL CONNECTIONS AND TYPES OF JOINT BEHAVIOUR.

angles (*i.e.* top clip angle and bottom seat angle shown in Fig 1.13 *a*) have some *moment of resistance*, and due to this the *supporting member* (*i.e.* column) rotates *slightly*. Similarly, in rigid connection (such as shown in Fig.1.13 *c*), elastic deformation of the connecting material, rivet or bolt, *slip* or deformation of elements of supporting members (such as column flanges), permit some *difference* in angular rotation ($\theta' \neq \theta$) of the joined members.



M_0= Fixed-end Moment for U.D.L.= $\frac{wL^2}{12}$

θ_0= End Rotation for Simply Supported Beam with U.D.L.= $\frac{wL^3}{24EI}$

FIG. 1.14. TYPICAL END MOMENT-ROTATION CURVES FOR VARIOUS TYPES OF CONNECTIONS.

Fig. 1.14 shows typical end moment-rotation curves for the three types of connections discussed above.

Experimentally Based Design : IS 800-1984 recommends that where the structure is non-conventional or of complex nature, the design may be based on full scale or model tests subject to the following conditions :

(*a*) A full scale test of prototype structure may be done. The protoype shall be accurately measured before testing to determine the dimensional tolerance in all relevant parts of the structure ; the tolerances then specified on the drawing shall be such that all successive structures shall be in practical conformity with the prototype. Where the design is based on failure loads, a load factor of not less than 2.0 on the loads or load combinations shall be used. Loading devices shall be previously calibrated and care shall be exercised to ensure that no artificial restraints are applied to the prototype by the loading systems. The distribution and duration of forces applied in the test shall be representative of those to which the structure is deemed to be subjected.

(*b*) In the case where the design is based on the testing of a small scale model structure, the model shall be constructed with due regard for the principles of dimensional similarity. The thrusts, moments and deformations under working loads shall be determined by physical measurements made when the loadings are applied to simulate the conditions assumed in the design of actual structure.

1.10. STANDARD SPECIFICATIONS AND CODES OF PRACTICE

A steel structure, before completion, passes through three phases: (*i*) analysis and design (*ii*) fabrication, and (*iii*) erection or construction. Thus, apart from the owner, other parties that are involved in the construction of a structure are (*a*) the designers (*b*) fabricators, and (*c*) the erectors. The design is based on the *strength* of available materials, *i.e* structural steel, rivets, bolts, pins, weld and other connecting components. An economical design can not be produced unless uniform data about material strength are available. For this it is essential that the used materials are manufactured according to certain *standards*. The presence of *standard* *specifications* and widely accepted *codes of practice* for the use of structural steel would give rise of most economical design. Many advanced countries like U.S.A., U.K., U.S.S.R, Japan, etc have envolved their own *standards* and *code of practice*. India is also one of the leading country in this aspect and *Bureau of Indian Standards* (BIS), formerly known as *Indian Standard Institution* (ISI) is doing this job of preparing standards and codes of practices in India. ISI published the first code of practice entitled *Use of Structural Steel in General Building Construction*', and designated as IS : 800 was published in 1956. The code was subsequently revised in 1962, and its latest revision was made in 1984. The following is the list of useful IS *Codes*:

1.	IS : 226-1975	Structural steel (standard quality) (*fifth revision*)
2.	IS : 456-1978	Code of practice for plain and Reinforced concrete (*Third revision*)
3.	IS : 723-	Mild steel wire nails.
4.	IS : 800-1984	Code of practice for General Construction in steel (*second revision*)
5.	IS : 806-	Use of steel tubes in General construction in steel
6.	IS : 808-1989	Hot Rolled steel Beams, channels and angle sections
7.	IS : 812-1957	Glossary of Terms relating to welding and Cutting of Metals
8.	IS : 813-1961	Scheme of symbol for Welding
9.	IS : 816-1969	Code of Practice for use of Metal Arc Welding for General Construction in mild steel (*first revision*)
10.	IS : 819-1957	Code of Practice for resistance spot welding for light assemblies in mild steel.
11.	IS : 875-1964	Code of Practice for structural safety of Buildings ; Loading standards (*revised*)
12	IS : 961-1975	Structural steel (High tensile) *second revision*)
13	IS : 1024-1979	Code of practice for use of welding in bridges and structures subject to dynamic loading (*first revision*)
14.	IS : 1148-1973	Hot-rolled steel rivet-bars (up to 40 mm dia.) for structural purposes (second revision)
15	IS : 1149-1982	High tensile steel rivet bars for structual purposes.
16.	IS : 1161-	Steel tubes for structural purposes
17.	IS : 1173-	Rolled Steel section, Tee-Bars
18.	IS : 1252-	Rolled steel sections, Bulb-Angles
19.	IS : 1261-1959	Code of Practice for seam welding in Mild steel.
20.	IS : 1278-1972	Filler Rods and wires for Gas Welding (*second revision*)
21.	IS : 1323-1962	Code of practice for oxy-acetylene Welding for structural Work in Mild steel (*revised*)
22.	IS : 1363-1967	Black hexagon bolts, nuts and lock nuts (dia. 6 to 39 mm) and Black Hexagonal screws (dia. 6 to 24 mm) (*first revision*)
23.	IS : 1364-1967	Precision and Semi-precision hexagon bolts, screws, nuts and lock nuts (dia. range 6 to 39 mm) (first revision)
24.	IS : 1730-	Dimensions for steel plates, sheets and strips for structural and General Engineering purposes.

25.	IS : 1731-	Dimensions for steel flats for structural and General Engineering purposes
26.	IS : 1732-	Dimensions for Round and square steel Bars for structural and General Engineering purposes.
27.	IS : 1911-	Schedule of unit weight of materials
28.	IS : 1929-1961	Rivets for General Purposes (12 to 48 mm dia.)
29.	IS : 1977-1975	Structural steel (ordinary quality) *(second division)*
30.	IS : 2062-1984	Weldable Structural steel (third revision)
31.	IS : 2155-1962	Rivets for General Purposes (below 12 mm dia.)
32.	IS : 2585-	Black square Bolts and Nuts and Black square screws
33.	IS : 3640-1967	Hexagon fit Bolts.
34.	IS : 3757-1972	High-tensile friction Grip Bolts *(first revision)*
35.	IS : 4000-1967	Code of Practice for assembly of structural joints using High Tensile friction Grip fasteners
36.	IS : 6623-1972	High Tensile Friction Grip Nuts.
37.	IS : 6639-1972	Hexagon bolts for steel structures
38.	IS : 6649-1972	High Tensile Friction Grip Washers
39.	IS : 7205-1974	Safety Code for Erection of structural steel work
40.	IS : 8500-1977	Weldable structural steel (medium and high strength qualities)
41.	IS : 9595-1980	Recommendations for Metal Arc Welding of carbon and Carbon Manganese steels.

In addition to the above, ISI has published the following hand books:

1. Hand book for structural Engineers : 1 (Structural Steel Sections)
2. Hand book for structural Engineers : 2 (Steel Beams and Plate Girders)
3. Hand book for structural Engineers : 3 (Steel Column and Struts)

Apart from ISI, the *Indian Road Congress* (IRC), and Ministry of Railways and Railway Board have also prepared and published *specifications* for construction of structures in structural steel.

For design of *Railway Bridges*, the following publications are useful :

(*i*) Bridge Rules
(*ii*) Steel Bridge Code

For design of *High Way Bridges*, Indian Road Congress has specified general features of design, loads and stresses and detailed specifications in their publication: *Standard Specification and Code of Practice for Road Bridges, Sections I to IX.*

In the United States of America, specifications for the design, fabrication and erection of structural steel buildings have been prepared by American Institute of Steel Construction (AISC) through its popular publication 'Manual of Steel Construction' (AISCM). For designs of specific nature, the following specifications are followed:

AISI	:	American Iron Steel Institute Specifications
AWS	:	American Welding Society Specifications
AASHTO	:	American Association of State Highway and Transportation Official Specification.
AREA	:	American Railway Engineering Association Specifications

SJI	:	Steel Joist Institute Specifications
USASI	:	U.S. American Standard Institute Specifications
USBPR	:	U.S. Bureau of Public Roads Specifications

In Britian, Australia and USSR, the following publications are widely followed by the designers:

BS 449 (Part II)	:	Specifications for use of structural steel in Building ; Part II : Metric Units. British Standard Insitution.
AS 1250	:	SSA Steel Structure Code. Standards Association of Australia.
SNIP-II-V3	:	Code of Practice for design of steel structures of the USSR State Committee for construction.

1.11. MERITS AND DEMERITS OF CONSTRUCTION IN STRUCTURAL STEEL

Construction in Structural Steel has the following *merits*:

1. ***High strength*** : Structural Steel has high strength per unit weight. Due to this, the self weight constitutes very small part of the load that can be supported by the steel structure. Due to this, steel members are slender or small in size (in comparison to R.C.C. members), resulting in *more available space* in the structure. This property is important in design of structures such as tall buildings, long-span bridges and air-plane hangers.

2. ***Easy Transportation*** : Because of its small size and self weight, the steel member can be easily transported.

3. ***Easy Fabrication, Erection and Replacement***: Steel structural components possess ease of fabrication and speed of erection, and they can be readily diassembled or replaced. They can also be used as pre-fabricated members.

4. ***Elasticity***: Steel follows Hooke's law upto high values of stress, in both tension and compression, and its behaviour can be predicted quite accurately, in contrast to reinforced cement concrete.

5. ***Ductility***: Because of its ductility (*i.e.* ability to undergo large deformations without fracture), steel is able to resist sudden collapse. Many of the simplifying assumptions used in structural steel design can be justified because of the ductility of steel.

6. ***Uniformity***: Greater control is exercised in the manufacture of steel. Hence both the properties of steel and uniformity of structural shapes can be assured. This eliminates the need to over-design a member because of uncertainty about the steel.

7. ***Strengthening of existing structure***: Existing structures can be easily strengthened by connection of additional plates or sections.

8. ***Gas and Water Tightness***: Because of high density of steel, and because of improved welding processes, steel structures can be made water-tight and gas tight.

9. ***Easy Inspection and Maintenance*** : Steel structures, can be easily inspected, and hence its maintenance is easy.

10. ***Longer Life***: Steel structures are known to have long service life.

11. ***Scrap Value*** : Even at the end of its useful life it has scrap value.

However, inspite of the above numerous merits, steel structures have the following ***demerits***.

1. ***High Cost of Construction***: Steel structures have relatively higher cost of construction in comparison to R.C.C. structures.

2. ***High Maintenance cost***: Exposure to water and air has a devastating effect, because of *corrosion*. This requires painting of steel structures on a periodic basis, resulting in high maintenance cost. However, in the case of *weathering steel*, a *protective layer* having an attractive red-brown colour is formed when it *oxidizes* under exposure to air and water, and hence painting is not required.

3. ***Poor fireproofing***: In case of fire, the rise in temperature results in drastic reduction in the strength of steel. Steel at 1000° F has about 65% of the strength at normal temperature, and at 1600°F, it is about 15% only. Such a drastic reduction results in collapse of structure. Hence steel should be covered by insulating material.

4. ***Buckling***: Because of the slender size, compression members of steel tend to fail due to buckling rather than to a lack of material strength. Hence additional steel is required to stiffen the member and prevent buckling.

5. ***Fatigue***: A structural member subject to many stress reversals or even large changes in either tension or compression may fracture due to fatigue.

2

Loads and Stresses

2.1. INTRODUCTION

The basic requirement of any structure or structural component is that it should be strong enough to carry or support all possible types of loads to which it is likely to be subjected. Loads coming on a structure may be divided into three *categories*:

1. Dead loads
2. Live loads and forces (super-imposed loads)
3. Wind loads

In a building, the *dead load* consists of weight of all permanent construction (such as walls, partitions, floors etc), including fixed equipment such as plumbing, air conditioning and *stationary* machines or other mechanical devices. In the case of a bridge, dead load includes weight of deck, tracks, side walks, railings, lighting fixtures etc, and of course the main structural frame.

Live loads and forces are the one which are always dynamic, atleast in principle. In a broad sence, live loads and forces include the following :

1. Loads due to occupants
2. Movable machinery, equipment, furniture, merchandise
3. Snow load
4. Fluid pressure
5. Earth pressure
6. Earthquake forces
7. Blast forces
8. Thermal forces

and 9. Wind forces.

The first three of the above forces are generally called as *super-imposed loads* in buildings. Since wind forces are quite prominent in tall structures (such as buildings, chimneys, water tanks, towers etc), they are normally grouped in a separate category.

2.2. DEAD LOADS

The dead load in a building shall comprise the weight of all walls, partitions, floors and roofs and shall include the weights of all other permanent construction in the building. Such

TABLE 2.2. IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES (IS : 875 -1987)

S.N.	*Occupancy Classification*	*Uniformly Distributed Load UDL* (kN/m^2)	*Concentrated Load (kN)*
1.	***Residential Buildings***		
	(*i*) Dwelling Houses	2.0 – 3.0	1.8 – 4.5
	(*ii*) Hotels, hostels, boarding houses, lodging houses, dormitories, residential clubs	2.0 – 4.0	1.8 – 4.5
	(*iii*) Boiler rooms and plant rooms	5.0	6.7
	(*iv*) Store rooms	5. 0	4.5
	(*v*) Garrages	2.5 – 5	9.0
	(*vi*) Balconies	3	1.5 kN/m at the outer edge
2.	***Educational Buildings***		
	(*i*) Class rooms, restaurants, offices, staff rooms, kitchens, toilets	2 – 3.0	2.7
	(*ii*) Store rooms etc.	5	4.5
	(*iii*) Libraries and archives	6.0 kN/m^2 for a min. height of 2.2 m + 2.0 kN/m^2 per m additional height	4.5
	(*iv*) Reading rooms	3.0 – 4.0	4.5
	(*v*) Corridors, lobbies, staircases	4.0	4.5
	(*vi*) Boiler rooms and plant rooms	4.0	4.5
	(*vii*) Balconies	Same as for rooms with a min. of 4	1.5 kN/m at the outer edge.
3.	***Institutional buildings***		
	(*i*) Bed rooms, wards, dormitories, lounges	2.0	1.8
	(*ii*) Kitchens, laundaries, laboratories, dining rooms, cafeteria, toilets,	2.0 – 3.0	2.7 – 4.5
	(*ii*) Corridors, passages, lobbies, staircases	4.0	4.5
	(*v*) Office rooms and OPD rooms	2.5	2.7
	(*vi*) Boiler rooms and plant rooms	5.0	4.5
	(*vii*) Balconies	Same as for (2 *vii*)	Same as for (2 *vii*)
4.	***Business and office buildings***		
	(*i*) Rooms with separate store	2.5	2.7
	(*ii*) Banking halls	3.0	2.7
	(*iii*) Vaults and strong rooms	5.0	4.5
	(*iv*) Record rooms/store rooms	5.0	4.5
5.	***Mercantile Buildings***		
	(*i*) Retail shops	4.0	3.6
	(*ii*) Wholesale shops	6.0 (min.)	4.5 (min.)
	(*iii*) Dining rooms, restaurants, cafeteria	3.0	2.7
	(*iv*) Corridors, passages, staircases	4.0	4.5
	(*v*) Office rooms	2.5	2.7

horizontal forces. The values given in Table 2.4 are minimum values and where values for actual loadings are available, they shall be used instead.

(*b*) ***Gradstands and the like***. Grandstands, stadia, assembly platforms, reviewing stands and the like shall be designed to resist a horizontal force applied to seats of 0.35 kN per linear metre along the line of seats and 0.15 kN per liner metre perperdicular to the line of seats. These loadings need not be applied simultaneously. Platforms without seats shall be designed to resist a minimum horizontal force of 0.25 kN/m^2 of plan area.

TABLE 2.3 IMPOSED LOADS ON VAROUS TYPES OF ROOFS

Sl. No.	*Type of Roof*	*Uniformly distributed imposed load measured on plan area*	*Minimum imposed load measured on plan*
(*i*)	Flat, sloping or curved roof with slopes up to and including 10 degrees		
	a) Access provided	1.5 kN/m^2	3.75 kN uniformly distributed over any span of one metre width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall
	b) Access not provided except for maintenance	0.75 kN/m^2	1.9 kN uniformly distributed over any span of one metre width of the roof slab and 4.5 kN uniformly distributed over the span of any beam or truss or wall
(*ii*)	Sloping roof with slope greater than 10 degrees	For roof membrane sheets or purlins : 0.75 kN/m^2 less 0.02 kN/m^2 for every degree increase in slope over 10 degrees	Subject to a minimum of 0.4 kN/m^2
(*iii*)	Curved roof with slope of line obtained by joining springing point to the crown with the horiozontal, greater than 10 degrees	$(0.75 - 0.52\gamma^2)$ kN/m^2, where $\gamma = h/l$ h = the height of the highest point of the structure measured from its springing; and l = chord width of the roof if singly curved and shorter of the two sides if doubly curved Alternatively, where structural analysis can be carried our for curved roofs of all slopes in a simple manner applying the laws of statistics, the curved roof shall be divided into minimum 6 equal segments and for each segment imposed load shall be calculated appropriate to the slope of the chord of each segment as given in (*i*) and (*ii*) above	Subject to a minimum of 0.4 kN/m^2

NOTE :

1. The loads given above do not include loads due to snow, rain, dust collection, etc. The roof shall be designed for imposed loads given above or for snow/rain load, whichever is greater.

2. For special types of roofs with highly permeable and absorbent material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

TABLE 2.4. HORIZONTAL LOADS ON PARAPETS, PARAPET WALLS AND BALUSTRADES

Sl. No.	*Usage area*	*Intensity of horizontal load, kN/m run*
(*i*)	Light access stairs gangways and the like not more than 600 mm wide	0.25
(*ii*)	Light access stairs, gangways and the like, more than 600 mm wide; stairways, landings, balconies and parapet walls (private and part of dwellings)	0.35
(*iii*)	All other stairways, landings and balconies, and all parapets and handrails to roofs except those subject to overcrowding covered under (*iv*)	0.75
(*iv*)	Parapets and balustrades in place of assembly, such as theatres, cinemas, churches, schools, places of entertainment, sports, buildings likely to be over-crowded	2.25

NOTE : In the case of guard parapets on a floor of multi-storeyed car park or crash barriers provided in certain buildings for the escape, the value of imposed horizontal load (together with impact load) may be determined.

4. Loading effects due to impact and vibration

(*a*) ***Impact allowance for lifts, hoists and machinery.*** The imposed loads specified in (1) above shall be assumed to include adequate allowance for ordinary impact conditions. However, for structures carrying loads which induce impact or vibration, as for as possible, calculation shall be made for increase in imposed load, due to impact or vibration. In absence of sufficient data for such calculation, the increase in the imposed loads shall be as follows :

	Structure	*Impact Allowance (min.)*
(*i*)	For frames supporting lifts and hoists	100 percent
(*ii*)	For foundations, footings and piers supporting lifts and hoisting apparatus	40 percent
(*iii*)	For supporting structures and foundations for light machinery, shaft or motor units	20 percent
(*iv*)	For supporting structures and foundations for reciprocating machinery or power units	50 percent

(*b*) ***Concentrated imposed loads with impact and vibration*** : Concentrated imposed loads with impact and vibration which may be due to installed machinery shall be considered and provided for in the design. The impact factor shall not be less than 20 percent which is the amount allowable for light machinery. Provision shall also be made for carrying any concentrated equipment loads which the equipment is being installed or moved for servicing and repairing.

(*c*) ***Impact allowances for crane girders*** : For crane gantry girders and supporting columns, to following allowances shall be deemed to cover all forces set up by vibration, shock from slipping or slings, kinetic action of acceleration, and retardations and impact of wheel loads (Table 2.5).

TABLE 2.5. IMPACT ALLOWANCES FOR CRANE GIRDERS.

	Type of load	Additional load
(a)	*Vertical loads for electric overhead cranes*	(i) 25% of maximum static loads for crane girders for all classes of cranes (ii) 25% for columns supporting class III and class IV cranes (iii) 10% for columns supporting class I and class II cranes (iv) No addional load for design of foundations
(b)	*Vertical loads for hand operated cranes*	10% of max. wheel loads for crane girders only
(c)	*Horizontal forces transverse to rails* 1. For electric over head cranes with trolley having rigid mast for suspension of lifted weight (such as soaker crane, stripper crane etc.)	10% of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst all the wheels on one side of rail track. For frame analysis this force shall be applied on one side of the frame at a time in either direction.
	2. For all other electric over head cranes and hand operated cranes	5% of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst the wheels on one side of rail track. For the frame analysis, this force shall be applied on one side of the frame at a time in either direction.
(d)	*Horizontal traction forces along the rails* For over head cranes, either electrically operated or hand operated.	5% of all static wheel loads

Note : See IS : 807-1976 for classification (classes 1 to 4) of cranes.

Forces specified in (c) and (d) shall be considered as acting at the rail level and being appropriately transmitted to the supporting system. Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces in (c) and (d) may act at the same time as the vertical load.

2.4. WIND LOADS

1. General

Wind is the air in motion relative to the surface of the earth. Since the vertical components of atmospheric motion are relatively small, specially near the surface of the earth, the term 'wind' denotes almost exclusively to horizontal wind. *Wind pressure*, therefore, acts horizontally on the exposed vertical surfaces of walls, columns, chimneys, towers etc and inclined roof surfaces.

The primary cause of wind is traced to differences in solar and terrestrial radiations setting up irregularities in temperature which give rise to convection either upwards or downwards. Gravity is the operative force working in some cases through the agency of pressure difference. The wind velocities are assessed with the aid of anemometers or anemographs which are installed at meteorological observations at heights generally varying from 10 to 30 metres.

All exposed structures are affected to some degree by wind forces. The liability of a building to high wind pressures depends not only upon the geographical location and proximity of other obstructions to air flow but also upon the characteristics of the structure itself.

The effect of wind on the structure as a whole is determined by combined action of *external* and *internal pressures* acting upon it.

2. Basic wind pressures

In the majority of structures, it is satisfactory to treat wind as a *static load*. The factors which determine the proper *equivalent static pressure* (p_e) are best under stood through the following equation presented by Davenport (1960) :

$$p_e = C_s \,.\, C_\alpha \,.\, C_g \,.\, q \qquad ...(2.1)$$

where C_s = a coefficient depending upon the shape of the structure

C_α = a coefficient dependant upon nearby topographic features

C_g = a gust coefficient dependent upon the magnitude of gust velocities and size of the structure

q = dynamic-pressure intensity, given by

$$q = \frac{1}{2}\rho\, v_H^2 \qquad ...(2.2)$$

where ρ = air density

v_H = design wind velocity at height H (the height above ground at which p_e is evaluated, or a characteristic height of the structure).

Also,

$$v_H = v_h \left(\frac{H}{h}\right)^{1/\alpha} \qquad ...(2.3)$$

where v_h = basic design wind velocity at height h (the height selected as standard for the measurement of wind velocities).

α = an exponent for the velocity increase with height determined by the surface roughness in the vicinity of the site and other influences.

Combining Eqs. 2.1 through 2.3, we get

$$p_e = \frac{1}{2} C_s \,.\, C_\alpha \,.\, C_g \,.\, \rho\, v_h^2 \left(\frac{H}{h}\right)^{2/\alpha} \qquad ...(2.4)$$

3. Design wind speed as per IS : 875-1987

The design wind speed (V_z) is obtained by multiplying the *basic wind speed* (V_b) by the factors k_1 , k_2 and k_3 :

$$V_z = V_b \,.\, k_1 \,.\, k_2 \,.\, k_3 \qquad ...(2.5)$$

where V_b = the basic wind speed in m/s at 10 m height (Fig. 2.1 and Table 2.6)

k_1 = probability factor (or risk coefficient)

k_2 = terrain, height and structure size factor

k_3 = topography factor.

3.1 Basic wind speed : For basic wind speed, India has been divided into six zones. Fig. 2.1 gives basic wind speed map of India, as applicable to 10 m height above mean ground level. Basic wind speed for some important cities/towns is given in Table 2.6

TABLE 2.6. BASIC WIND SPEED IN SOME IMPORTANT CITIES/TOWNS

City/Town	*Basic Wind Speed (m/s)*	*City/town*	*Basic Wind Speed (m/s)*
Agra	47	Jhansi	47
Ahmadabad	39	Jodhpur	47
Ajmer	47	Kanpur	47
Almora	47	Kohima	44
Amritsar	47	Kurnool	39
Asansol	47	Lshadweep	39
Aurangabad	39	Lucknow	47
Bahraich	47	Ludhiana	47
Bangalore	33	Madras	50
Barauni	47	Madurai	39
Bareilly	47	Mandi	39
Bhatinda	47	Mangalore	39
Bhilai	39	Moradabad	47
Bhopal	39	Mysore	33
Bhubaneshwar	50	Nagpur	44
Bhuj	50	Nainital	47
Bikaner	47	Nasik	39
Bokaro	47	Nellore	50
Bombay	44	Panjim	39
Calcutta	50	Patiala	47
Calicut	39	Patna	47
Chandigarh	47	Pondicherry	50
Coimbatore	39	Port Blair	44
Cuttack	50	Pune	39
Darbhanga	55	Raipur	39
Darjeeling	47	Rajkot	39
Dehra Dun	47	Ranchi	39
Delhi	47	Roorkee	39
Durgapur	47	Rourkela	39
Gangtok	47	Simla	39
Gauhati	50	Srinagar	39
Gaya	39	Surat	44
Gorakhpur	47	Tiruchchirrappalli	47
Hyderabad	44	Trivandrum	39
Imphal	47	Udaipur	47
Jabalpur	47	Vadodara	44
Jaipur	47	Varanasi	47
Jamshedpur	47	Vijaywada	50
		Visakhapatnam	50

3.2 Probability factor (or risk coefficient) : k_1

Basic wind speeds given in Fig. 2.1 have been worked out for 50 years return period. The *design life* of structure is based on the functional aspect as well as the importance of the structure. The factor k_1 is based on statistical concepts which take account of the degree of reliability required and period of time in years during which there will be exposure to wind, *i.e.* life to the structure. If the suggested life of the structure is more than 50 years (such as nuclear power reactors, satellite communication towers etc.), factor k_1 is found by the relation

$$k_1 = \frac{X_{N,P}}{X_{50,\,0.63}} = \frac{A - B\left[\log_e\left\{-\frac{1}{N}\log_e(1 - P_N)\right\}\right]}{A + 4B} \qquad \text{...(2.6)}$$

where

N = mean probable design life of structure in years,

P_N = risk level in N years (nominal value = 0.63),

$X_{N,P}$ = extreme wind speed for given values of N and P_N and $X_{50,\,0.63}$ = extreme wind speed for $N = 50$ years and $P_N = 0.63$

A and B are coefficients having following values for different wind speed zones :

Zone	*A*	*B*
33 m/s	83.2	9.2
39 m/s	84.0	14.0
44 m/s	88.0	18.0
47 m/s	88.0	20.5
50 m/s	88.8	22.8
55 m/s	90.8	27.3

The value of k_1 are given in Table 2.7

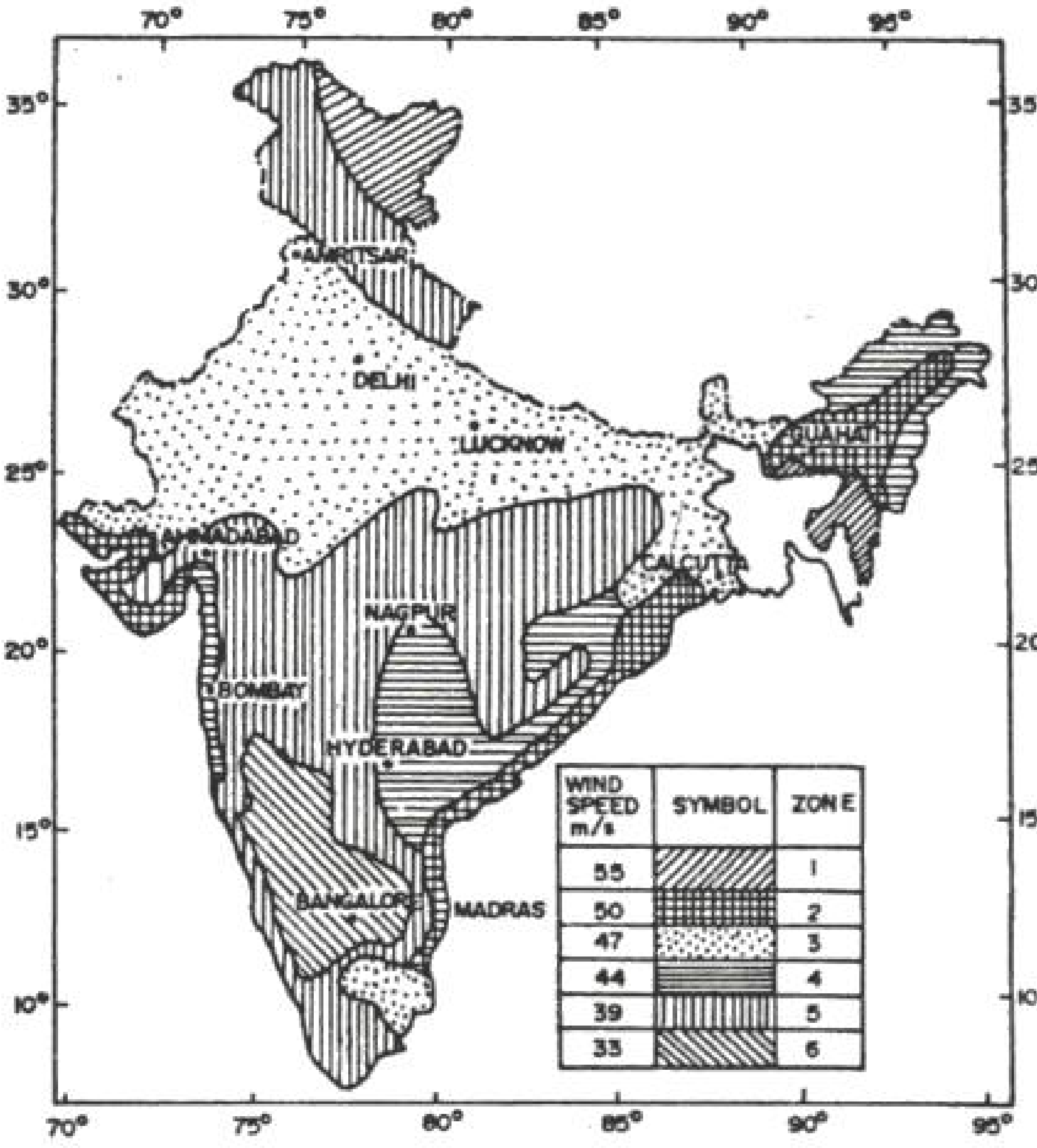


FIG. 2.1. BASIC WIND SPEED

TABLE 2.7. VALUES OF FACTOR k_1

Class of structure	*Mean probable design life of structure (years)*	*k_1 factor for basic design speed (m/s)*					
		33	*39*	*44*	*47*	*50*	*55*
1. All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
2. Temporary sheds and structures under construction	5	0.82	0.76	0.73	0.71	0.70	0.67
3. Buildings and structures presenting a low degree of hazard to life and property in event of failure	25	0.94	0.92	0.91	0.9	0.9	0.89
4. Important buildings and structures such as hospitals and communication buildings (towers, power plant structures etc.)	100	1.05	1.06	1.07	1.07	1.08	1.08

3.3 Terrain, height and structure size factor (k_2 factor)

This factor takes into account terrain roughness, height and size of structure.

Terrain categories : Selection of terrain categories is made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain on which a specific tructure stands is grouped under the *four* categories.

Category 1 : This represents exposed open terrain with few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5 m. This includes open sea coasts and flat treeless plains.

Category 2 : This represents open terrain with well scattered obstructions having height generally between 1.5 to 10 m. This includes air fields, open park lands and underveloped sparsely built-up outskirts of towns and suburbs. Open land adjacent to sea coast may also be included in this cateogry.

Category 3 : This represents terrain with numerous closely spaced obstructions having the size of building structures upto 10 m in height with or without a few isolated tall structures. This category includes well wooded areas, and shrubs, towns and industrial areas fully or partially developed.

Category 4 : This represents terrain with numerous large high closely spaced obstructions. This category includes large city centres, generally with obstructions above 25 m and well developed industrial areas fully or partially developed.

TABLE 2.8. VALUES OF FACTOR k_2

Height (m)	*Terrain Category 1*			*Terrain Category 2*			*Terrain Category 3*			*Terrain Category 4*		
	Class			*Class*			*Class*			*Class*		
	A	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
10	1.05	1.03	0.99	1.00	0.98	0.93	0.91	0.88	0.82	0.80	0.76	0.67
15	1.09	1.07	1.03	1.05	1.02	0.97	0.97	0.94	0.87	0.80	0.76	0.67
20	1.12	1.10	1.06	1.07	1.05	1.00	1.01	0.98	0.91	0.80	0.76	0.67
30	1.15	1.13	1.09	1.12	1.10	1.04	1.06	1.03	0.96	0.97	0.93	0.83
50	1.20	1.18	1.14	1.17	1.15	1.10	1.12	1.09	1.02	1.10	1.05	0.95
100	1.26	1.24	1.20	1.24	1.22	1.17	1.20	1.17	1.10	1.20	1.15	1.05
150	1.30	1.28	1.24	1.28	1.25	1.21	1.24	1.21	1.15	1.24	1.20	1.10
200	1.32	1.30	1.26	1.30	1.28	1.24	1.27	1.24	1.18	1.27	1.22	1.13
250	1.34	1.32	1.28	1.32	1.31	1.26	1.29	1.26	1.20	1.28	1.24	1.16
300	1.35	1.34	1.30	1.34	1.32	1.28	1.31	1.28	1.22	1.30	1.26	1.17
350	1.37	1.35	1.31	1.36	1.34	1.29	1.32	1.30	1.24	1.31	1.27	1.19
400	1.38	1.36	1.32	1.37	1.35	1.30	1.34	1.31	1.25	1.32	1.28	1.20
450	1.39	1.37	1.33	1.38	1.36	1.31	1.35	1.32	1.26	1.33	1.29	1.21
500	1.40	1.38	1.34	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.22

Note : Intermediate values may be obtained by linear interpolation. It is permissible to assume constant wind speed between two heights, for simplicity.

Structure size : Buildings or structures are classified into the following three different classes depending upon their size (*i.e.*, greater horizontal or vertical dismension).

Class A : Structures having maximum dimension less than 20 m

Class B : Structures having maximum dimension between than 20 to 50 m

Class C : Structures having maximum dimension greater than 50 m.

Height : The design wind speed is a function of height at which the design wind speed is being computed. Table 2.8 gives the values of k_2 factor, to obtain design wind speed variation with height, in different terrains and for different classes of building structures. It should be very clearly noted that for a given structure, out of the three factors k_1 , k_2 and k_3 , factors k_1 and k_3 are fixed depending upon zone, life of structure, terrain category and class of structure while factor k_2 varies with the *height of the element* (of the structure) at which the design wind speed is being computed.

3.4. Topography factor k_3 : The basic wind speed V_b given in Fig. 2.1 takes into account of the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliffs, steep excarpments or ridges.

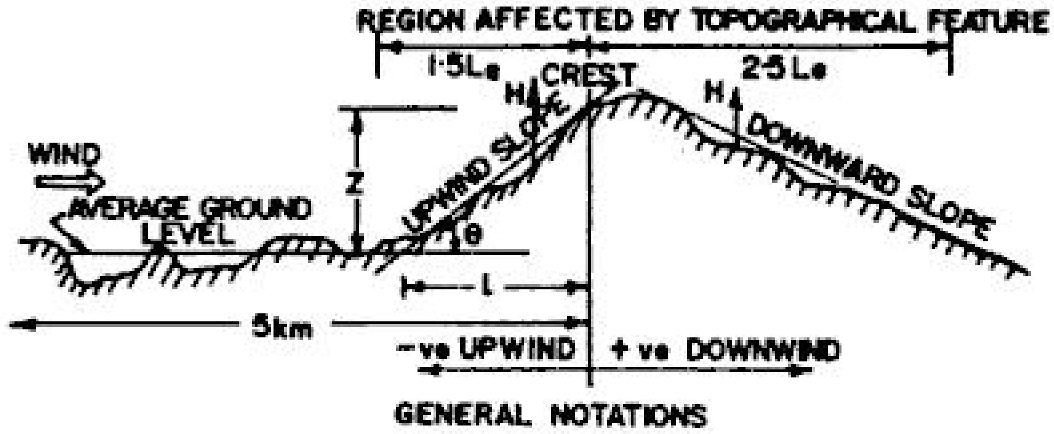


FIG. 2.2. GENERAL NOTATIONS

The effect of topography will be significant at a site when the upwind slope (θ) is greater than about 3°, and below that, the value of k_3 may be taken to be equal to 1.0. The value of k_3 (confined in the range of 1.0 to 1.36) for slopes greater than 3° is given by

$$k_3 = 1 + C \cdot s \qquad ...(2.7)$$

where C has the following values :

Slope	*Value of C*
$3° < \theta \leq 17°$	$1.2\left(\frac{Z}{L}\right)$
$> 17°$	0.36

where Z is the height of crest or hill and L is the projected length of upwind zone from average ground level to crest in wind direction (Fig. 2.2)

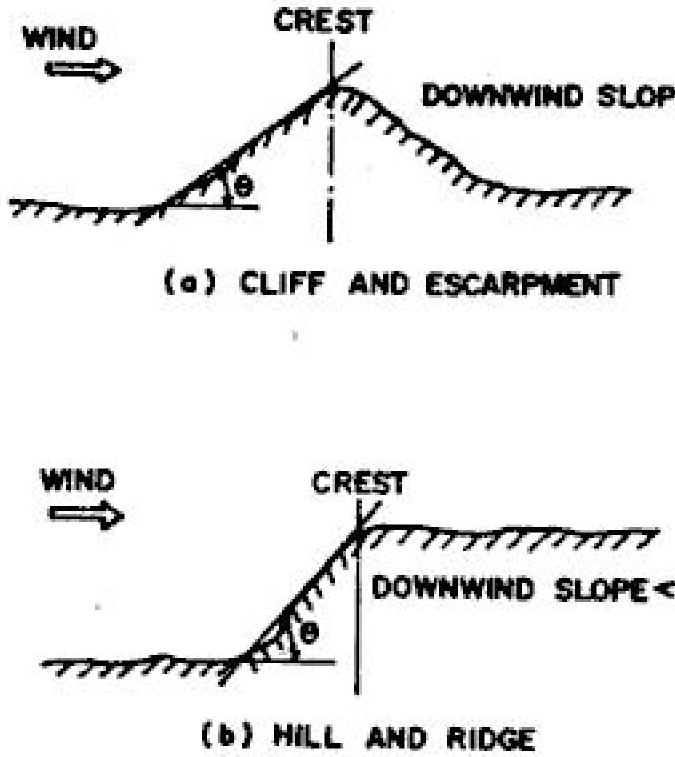


FIG. 2.3. TOPOGRAPHICAL FEATURES

Factor s is determined from Fig. 2.4 for cliffs and escarpments and from Fig. 2.5 for hills and ridges. In Figs. 2.4 and 2.5, H is the height of the crest above ground level and, x is the distance from the summit or crest relative to the effective length L_e. The effective horizontal length of the hill, depends upon the slope θ, and its value will be as under:

Slope	L_e
$3 < \theta < 17°$	L
$> 17°$	$\frac{Y}{0.3}$

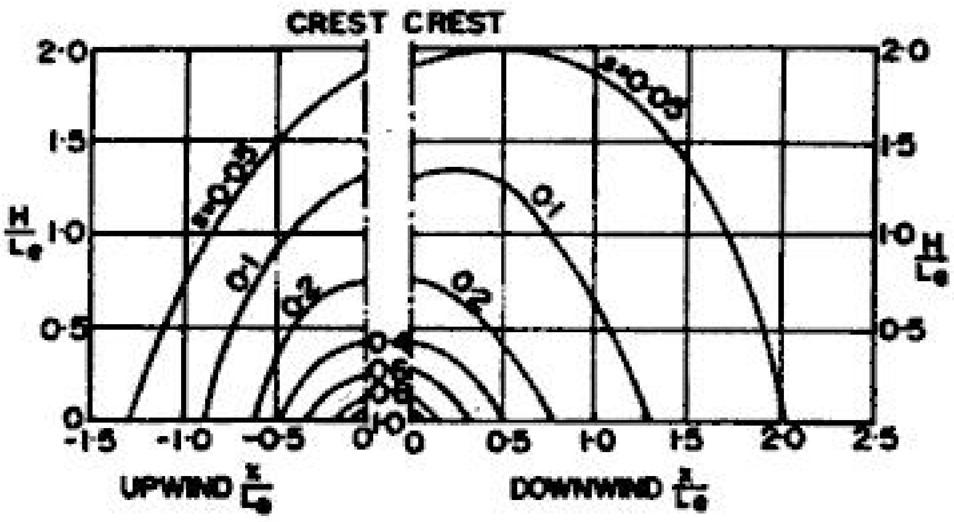


FIG. 2.4. FACTOR s FOR CLIFF AND ESCARPMENT

where Y is the effective height of the feature. The influence of topographic features is considered by modifying the effective length as follows :

Up wind effective length $= 1.5\ L_e$

Downwind effective length $= 2.5\ L_e$

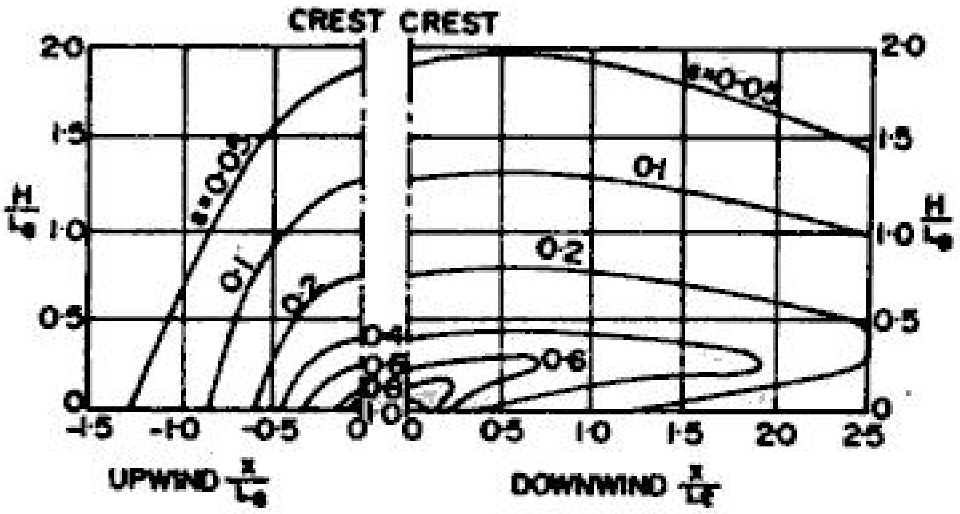


FIG. 2.5. FACTOR s FOR RIDGE AND HILL

4. Design wind pressure

The design wind pressure at any height above mean ground level is obtained by the following relationship :

$$p_z = 0.6\, V_z^2 \qquad ...(2.8)$$

where p_z = design wind pressure in N/m^2 at height z

V_z = design wind velocity in m/s at height z

5. Design wind loads

The wind load on a building shall be calculated for the following:

(*a*) The building as a whole

(*b*) Individual structural members

(*c*) Individual structural elements (such as roofs and walls)

(*d*) Individual structural units (such as glazing and fixing)

6. Wind load on building as a whole

The total wind load on a particular building or structure is given by

$$F = C_f \cdot A_e \cdot p_z \qquad ...(2.9)$$

where F = wind force acting in a direction specified

A_e = effective frontal area of the structure

$\bar{p}_z$ = design wind pressure

C_f = force coefficient for the building

The overall force coefficients for rectangular clad buildings of uniform section with flat roof in uniform flow shall be as given in Fig. 2.6, and for other clad buildings of uniform section shall be as given in Table 2.9. The force coefficient for solid shapes mounted on a surface are given in Table 2.10. The values of force coefficients differ for the wind acting on different faces of a building or structure. In order to determine the critical load, the total wind load should be calculated for each wind direction.

Building of circular shapes: For coefficients for buidings of circular cross-section shapes shall be as given in Table 2.9.

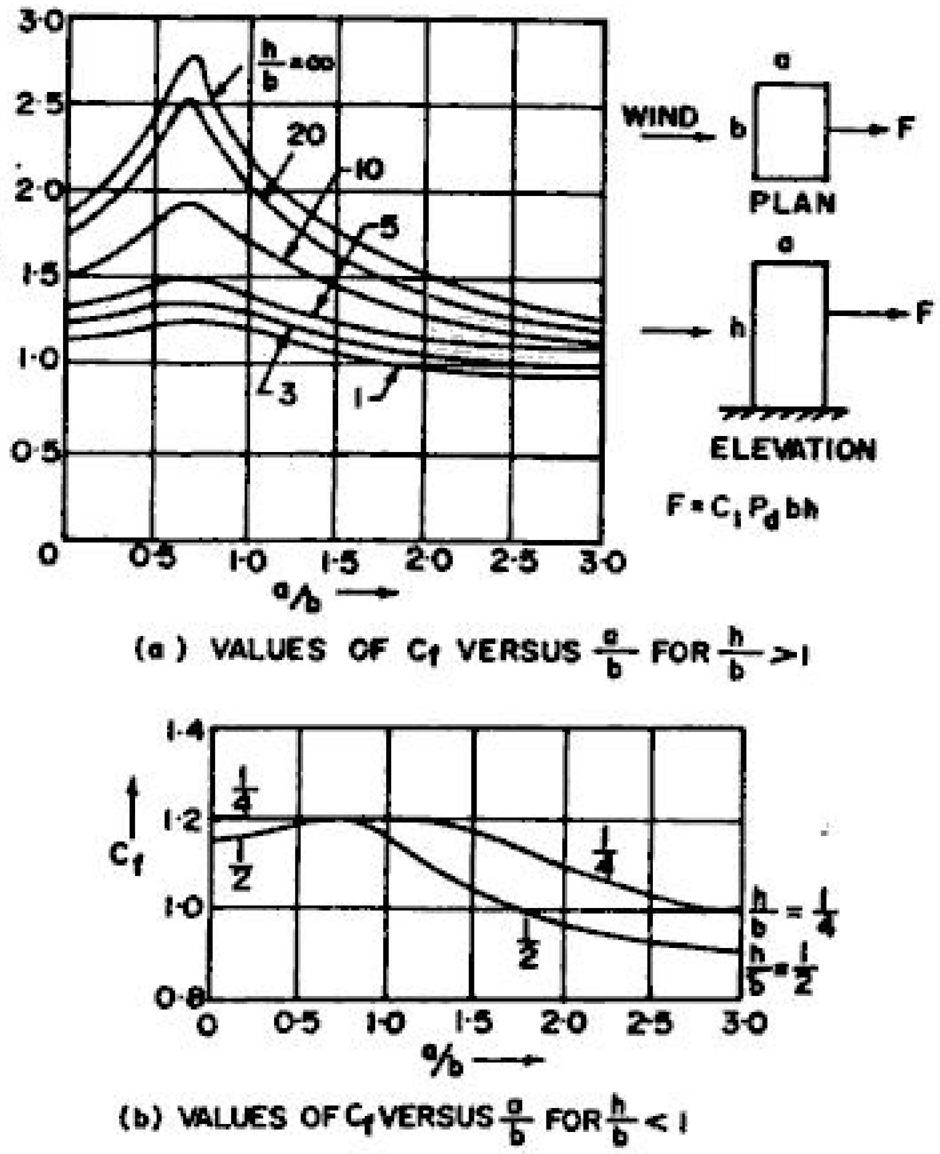


FIG. 2.6. FORCE CO-EFFICIENTS FOR RECTANGULAR CLAD BUILDINGS

TABLE 2.9 FORCE COFFICIENTS OF CLAD BUILDING OF UNIFORMA SECTION (ACTING IN THE DIRECTION OF WIND)

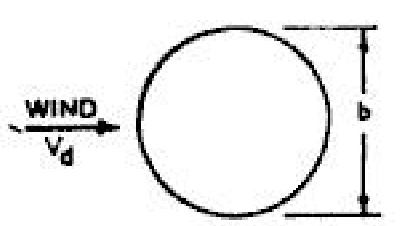
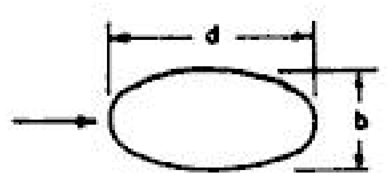
Plan Shape		$V_d.b$ m^2/s	C_f *for Height/Breadth Ratio*						
			Upto 1/2	1	2	5	10	20	∞
	All surfaces	<6	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	Rough or with projection	≥6							
	Smooth	>6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	Ellipse *b/d* = 1/2	<10	0.5	0.5	0.5	0.5	0.6	0.6	0.7
		≥10	0.2	0.2	0.2	0.2	0.2	0.2	0.2

TABLE 2.9 (Contd.)

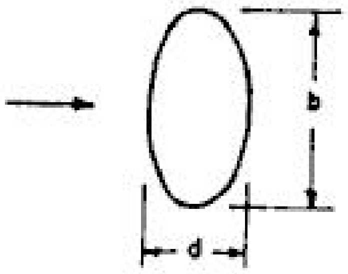
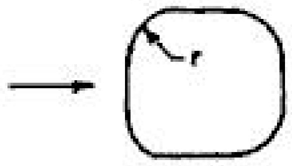
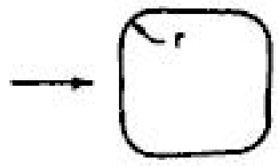
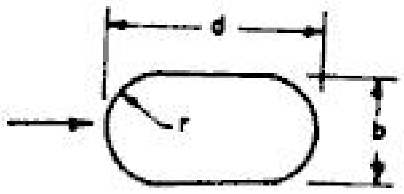
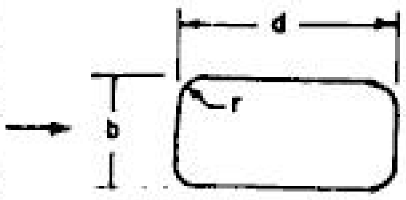
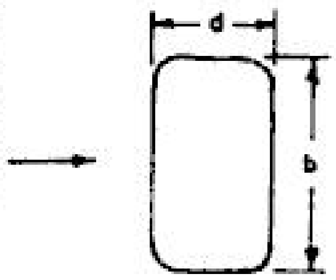
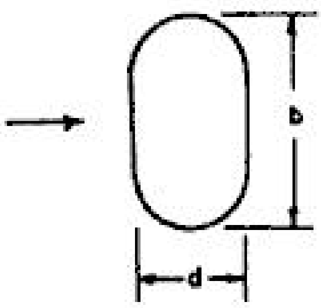
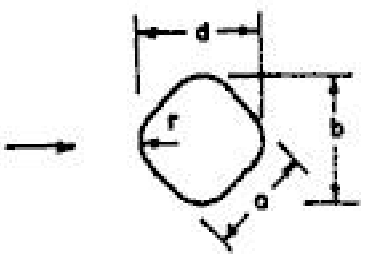
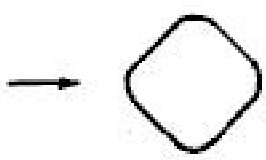
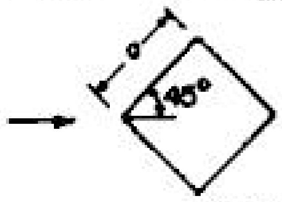
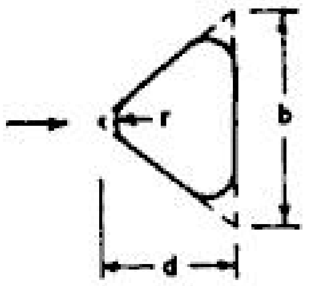
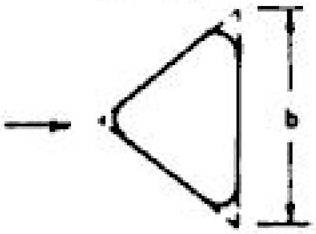
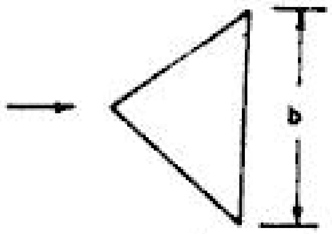
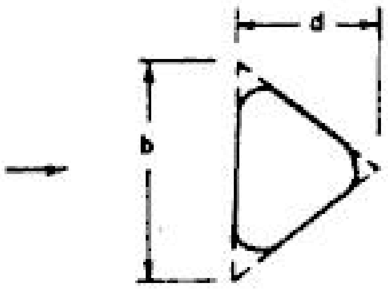
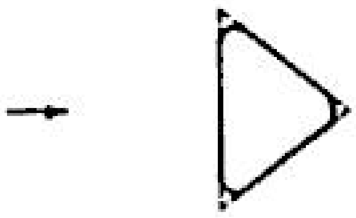
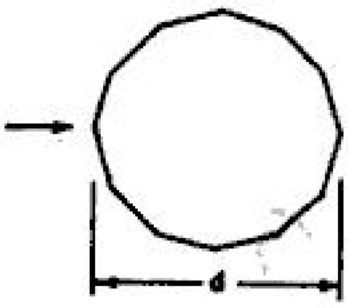
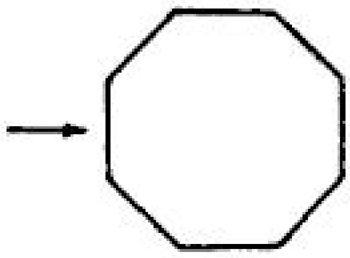
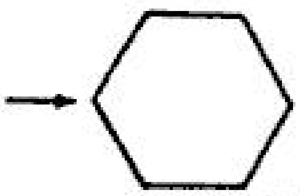
	Ellipse *b/d* =2	<8	0.8	0.8	0.9	1.0	1.1	1.3	1.7
		≥8	0.8	0.8	0.9	1.0	1.1	1.3	1.5
	*b/d* =1 *r/b* =1/3	<4	0.6	0.6	0.6	0.7	0.8	0.8	1.0
		≥4	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	*b/d* =1 *r/b* =1/6	<10	0.7	0.8	0.8	0.9	1.0	1.0	1.3
		≥10	0.5	0.5	0.5	0.5	0.6	0.6	0.6
	*b/d* =1/2 *r/b* =1/2	<3	0.3	0.3	0.3	0.3	0.3	0.3	0.4
		≥3	0.2	0.2	0.2	0.2	0.3	0.3	0.3
	*b/d* =1/2 *r/b* =1/6	All values	0.5	0.5	0.5	0.5	0.6	0.6	0.7
	*b/d* =2 *r/b* =1/12	All values	0.9	0.9	1.0	1.1	1.2	1.5	1.9
	*b/d*=2 *r/b*=1/4	<6	0.7	0.8	0.8	0.9	1.0	1.2	1.6
		≥6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	*r/a*=1/3	<10	0.8	0.8	0.9	1.0	1.1	1.3	1.5
		≥10	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	*r/a* =1/12	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6

TABLE 2.9 (Contd.)

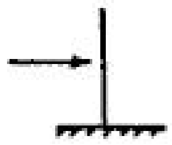
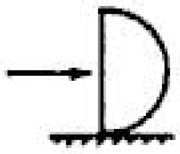
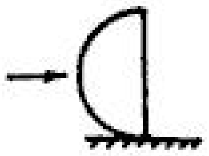
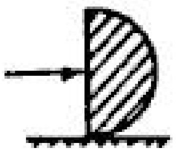
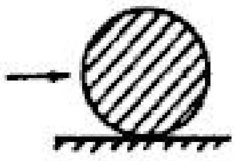
	*r*/*a* =1/48	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
	*r*/*b* =1/4	<11	0.7	0.7	0.7	0.8	0.9	1.0	1.2
		≥11	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	*r*/*a* =1/12	All values	0.8	0.8	0.8	1.0	1.1	1.2	1.4
	*r*/*b*=1/48	All values	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	*r*/*b* =1/4	<8	0.7	0.7	0.8	0.9	1.0	1.1	1.3
		≥8	0.4	0.4	0.4	0.4	0.5	0.5	0.5
	1/48<*r*/*b* <1/12	All values	1.2	1.2	1.2	1.4	1.6	1.7	2.1
	12-sided polygon	<12	0.7	0.7	0.8	0.9	1.0	1.1	1.3
		≥12	0.7	0.7	0.7	0.7	0.8	0.9	1.1
	Octagon	All values	1.0	1.0	1.1	1.2	1.2	1.3	1.4
	Hexagan	All values	1.0	1.1	1.2	1.3	1.4	1.4	1.5

Note : 1. Structures that, because of their size and design wind velocity, are in the supercritical flow regime may need further calculation to ensure that the greatest loads do not occur at some wind speed below the maximum when the flow will be subcritical.

2. The coefficients are for buildings without projections, except where otherwise shown.

3. In the above table $V_d.b$ is used as an indication of the airflow regime.

TABLE 2.10 FORCE COEFFICIENTS FOR SOLID SHAPES MOUNTED ON A SURFACE

Side Elevation	*Description of Shape*	C_f
	Circular Disc	1.2
	Hemispherical Bowl	1.4
	Hemispherical Bowl	0.4
	Hemispherical Solid	1.2
	Spherical Solid	0.5 For $V_d D < 7$ 0.2 For $V_d D \geq 7$

7. Wind load on structural members

Table 2.11 gives the force coefficients for individual structural members of infinite length. For members of finite length, the coefficients should be multiplied by a factor K that depends on the ratio l/b where l is the length of the member and b is the width across the direction of wind. The values of K are given in Table 2.12.

The force coefficients are given for two mutually perpendicular directions, normal and transverse, designated as C_{fn} and C_{ft} respectively.

Normal force, $F_n = C_{fn} \cdot p_d K l b$ ...(2.10 *a*)

Transverse force, $F_t = C_{ft} p_d K l b$ ...(2.10 *b*)

where p_d is the design wind pressure.

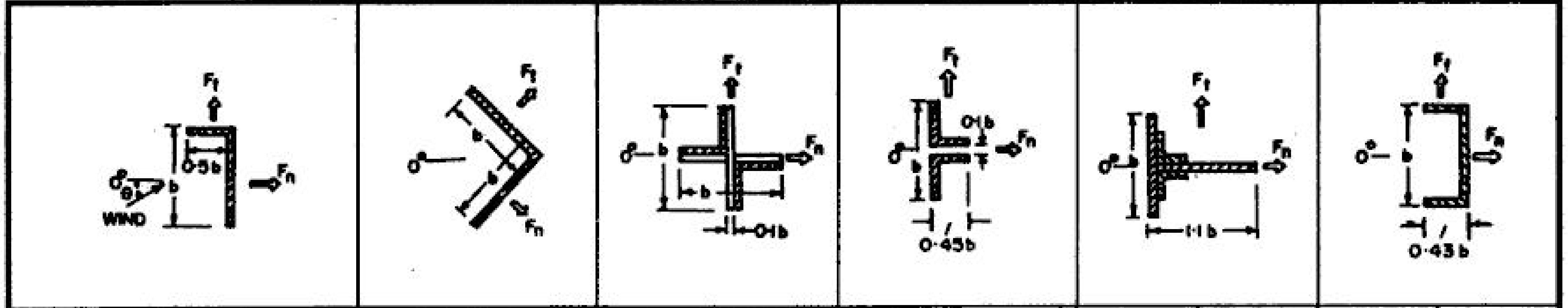
While estimating the value of K, following special cases should be noted.

(*i*) Where any member abuts on to a plate or wall in such a way that free flow of air around that end of the member is prevented, then the ratio of l/b shall be doubled for the purpose of determining K ; and

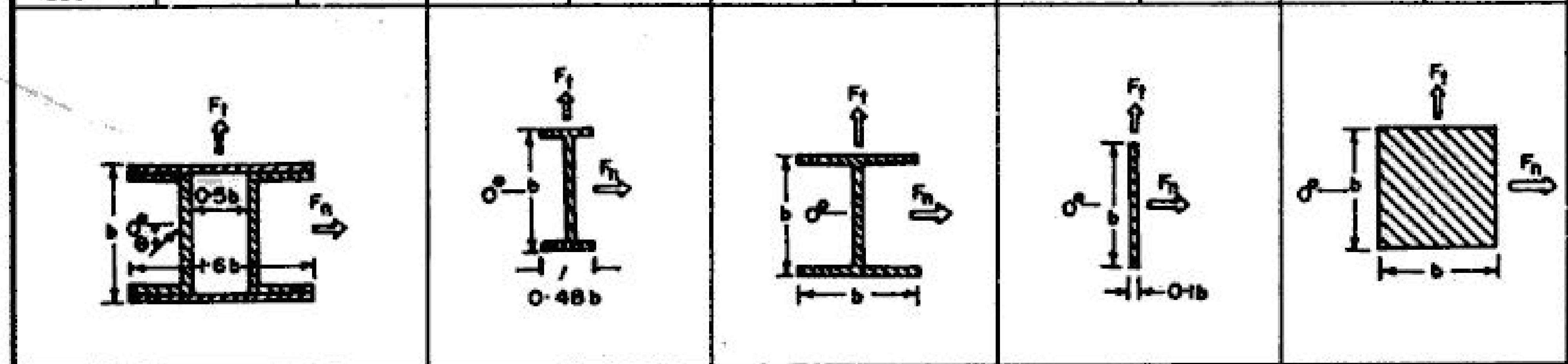
(*ii*) When both the ends of a member are so obstructed, the ratio l/b shall be taken as infinity for the purpose of determining K.

Wires and cables : Force coefficients for wires and cables are given in Table 2.13, depending upon the diameter (D) and the design wind speed (V_d) and the surface roughness.

TABLE 2.11. FORCE COEFFICIENTS (C_f) FOR INDIVIDUAL STRUCTURAL MEMBERS OF INFINITE LENGTH



θ	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees												
0	+ 1.9	+ 0.95	+ 1.8	+ 1.8	+ 1.75	+ 0.1	+ 1.6	0	+ 2.0	0	+ 2.05	0
45	+ 1.8	+ 0.8	+ 2.1	+ 1.8	+ 0.85	+ 0.85	+ 1.5	− 0.1	+ 1.2	+ 0.9	+ 1.85	+ 0.6
90	+ 2.0	+ 1.7	− 1.9	− 1.0	+ 0.1	+ 1.75	− 0.95	+ 0.7	− 1.6	+ 2.15	0	+ 0.6
135	− 1.8	− 0.1	− 2.0	+ 0.3	− 0.75	+ 0.75	− 0.5	+ 1.05	− 1.1	+ 2.4	− 1.6	+ 0.4
180	− 2.0	+ 0.1	− 1.4	− 1.4	− 1.75	− 0.1	− 1.5	0	− 1.7	± 2.1	− 1.8	0



θ	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
degrees										
0	+ 1.4	0	+ 2.05	0	+ 1.6	0	+ 2.0	0	+ 2.0	0
45	+ 1.2	+ 1.6	+ 1.95	+ 0.6	+ 1.5	+ 1.5	+ 1.8	+ 0.1	+ 1.55	+ 1.55
90	0	+ 2.2	+ 0.5	+ 0.9	0	+ 1.9	0	+ 0.1	0	+ 2.0

Note : In this table, the force coefficient C_f is given in relation to the dimensions b and not, as in other cases, in relation to effective frontal area A_e

TABLE 2.12. REDUCTION FACTOR *K* FOR INDIVIDUAL MEMBERS

l/b or *l/D*	2	5	10	20	40	50	100	∞
Circular cylinder, subcritical flow	0.58	0.62	0.68	0.74	0.82	0.87	0.98	1.00
Circular cylinder, supercritical flow ($DV_d \geq 6\ m^2/s$)	0.80	0.80	0.82	0.90	0.98	0.99	1.00	1.00
Flat plate perpendicular to wind ($DV_d \geq 6\ m^2/s$)	0.62	0.66	0.69	0.81	0.87	0.90	0.95	1.00

TABLE 2.13. FORCE COEFFICIENTS FOR CABLES AND WIRES

Flow regime	*Force coefficient*			
	Smooth surface	*Moderately smooth wire (galvanized or painted)*	*Fine stranded cables*	*Thick stranded cables*
$DV_d < 0.6\ m^2/s$	-	-	1.2	1.3
$DV_d \geq 0.6\ m^2/s$	-	-	0.9	1.1
$DV_d < 0.6\ m^2/s$	1.2	1.2	-	-
$DV_d \geq 0.6\ m^2/s$	0.5	0.7	-	-

8. Wind load on structural elements such as roofs and walls

When calculating the wind load on individual structural elements such as roofs and walls, and individual cladding units .and their fittings, it is essential to take account of the pressure difference between the opposite faces of such elements or units. For clad structures, it is, therefore, necessary to know the internal pressure as well as the external pressure. The wind load F, acting in a direction normal to the individual structural element or cladding unit is

$$F = (C_{pe} - C_{pi})\,A\,.\,p_z \qquad \text{...(2.11)}$$

where C_{pe} = external wind pressure coefficient

C_{pi} = internal pressure coefficient

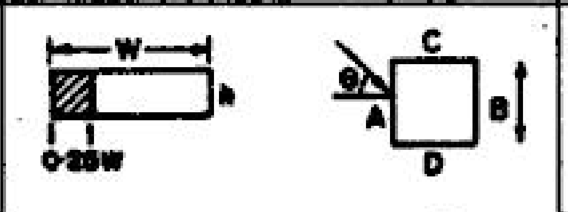
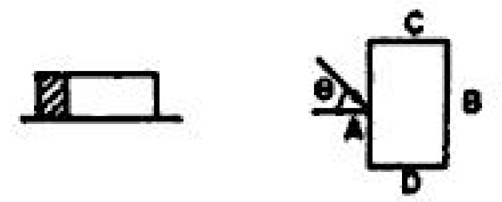
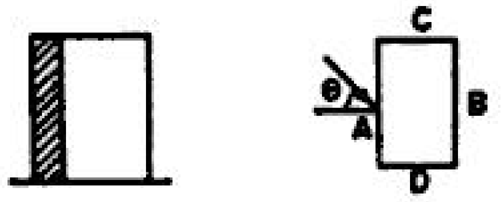
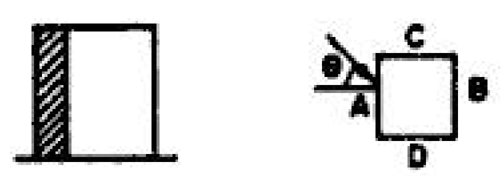
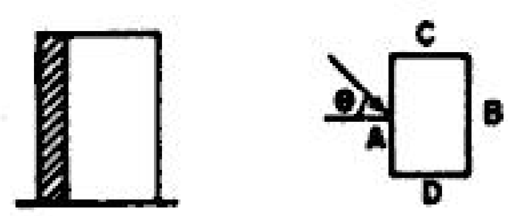
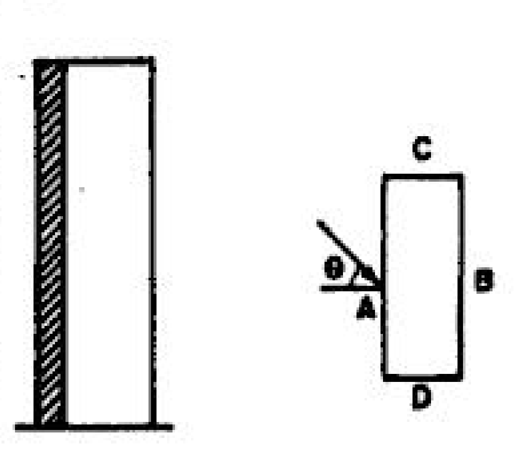
A = surface area of structural element or cladding 'unit, and

p_z = design wind pressure

If the surface design pressure varies with height, the surface areas of the structural element may be subdivided so that the specified pressures are taken over appropriate areas. Positive wind load indicates the force acting towards the structural element and negative away from it.

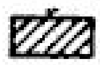
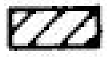
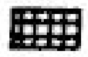
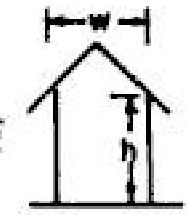
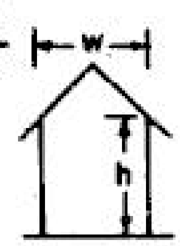
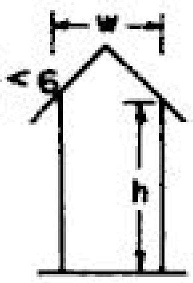
External air pressure coefficients (C_{pe}). Table 2.14 gives the average external pressure coefficient for the walls of clad buidlings of rectangular plan. In addition, local pressure concentration coefficenty are also given. Table 2.15 gives the average external pressure coefficients and pressure concentration coefficients for pitched roofs of rectangular clad buildings. Where no pressure concentration coefficients are given, the average coefficients shall apply.

TABEL 2.14. EXTERNAL PRESSURE COEFFICIENTS (C_{pe}) FOR WALLS OF RECTANGULAR CLAD BUILDINGS

Building height ratio	*Building plan ratio*	*Elevation*	*Plan*	*Wind angle θ degrees*	C_{pe} *for surface* A	B	C	D	*Local* C_{pe}
$\frac{h}{w} < \frac{1}{2}$	$1 < \frac{l}{w} < \frac{3}{2}$			0	+ 0.7	− 0.2	− 0.5	− 0.5	− 0.8
				90	− 0.5	− 0.5	+ 0.7	− 0.2	
	$\frac{3}{2} < \frac{l}{w} < 4$			0	+ 0.7	− 0.23	− 0.6	− 0.6	− 1.0
				90	− 0.5	− 0.5	+ 0.7	− 0.1	
$\frac{1}{2} < \frac{h}{w} < \frac{3}{2}$	$1 < \frac{l}{w} < \frac{3}{2}$			0	+ 0.7	− 0.25	− 0.6	− 0.6	− 1.1
				90	− 0.5	− 0.5	+ 0.7	− 0.25	
	$\frac{3}{2} < \frac{l}{w} < 4$			0	+ 0.7	− 0.3	− 0.7	− 0.7	− 1.1
				90	− 0.5	− 0.5	+ 0.7	− 0.1	
$\frac{3}{2} < \frac{h}{w} < 6$	$1 < \frac{l}{w} < \frac{3}{2}$			0	+ 0.8	− 0.25	− 0.8	− 0.8	− 1.2
				90	− 0.8	− 0.8	+ 0.8	− 0.25	
	$\frac{3}{2} < \frac{l}{w} < 4$			0	+ 0.7	− 0.4	− 0.7	− 0.7	− 1.2
				90	− 0.5	− 0.5	+ 0.8	− 0.1	
$\frac{h}{w} > \infty$	$\frac{l}{w} = \frac{3}{2}$			0	+ 0.95	− 1.85	− 0.9	− 0.9	− 1.25
				90	− 0.8	− 0.8	+ 0.9	− 0.85	
	$\frac{l}{w} = 1.0$			0	+ 0.95	− 1.25	− 0.7	− 0.7	− 1.25
				90	− 0.7	− 0.7	+ 0.95	− 1.25	
	$\frac{l}{w} = 2$			0	+ 0.85	− 0.75	− 0.75	− 0.75	− 1.25
				90	− 0.75	− 0.75	+ 0.85	− 0.75	

Note. *h* is height to eaves or parapet, *l* is greater horizontal dimension of building and *w* is the lesser horizontal dimension of a building.

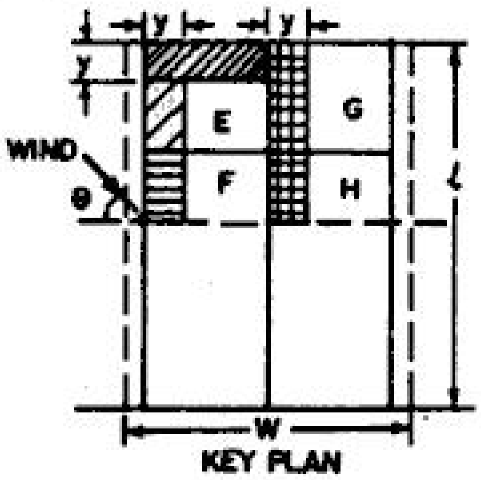
TABLE 2.15. EXTERNAL PRESSURE COEFFICIENT (C_{pe}) FOR PITCHED ROOFS OF RECTANGULAR CLAD BUILDING

Building height ratio	*Roof angle* α	*Wind angle* θ 0° *E F*	*G H*	*Wind angle* θ 0° *E F*	*F H*	*Local Coefficients* 			
	degress								
	0	− 0.8	− 0.4	− 0.8	− 0.4	− 2.0	− 2.0	− 2.0	−
	5	− 0.9	− 0.4	− 0.8	− 0.4	− 1.4	− 1.2	− 1.2	− 1.0
$\frac{h}{w} < \frac{1}{2}$ 	10	− 1.2	− 0.4	− 0.8	− 0.6	− 1.4	− 1.4		− 1.2
	20	− 0.4	− 0.4	− 0.7	− 0.6	− 1.0			− 1.2
	30	0	− 0.4	− 0.7	− 0.6	− 0.8			− 1.1
	45	+ 0.3	− 0.5	− 0.7	− 0.6				− 1.1
	60	+ 0.7	− 0.6	− 0.7	− 0.6				− 1.1
	0	− 0.8	− 0.6	− 1.0	− 0.6	− 2.0	− 2.0	− 2.0	−
$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$ 	5	− 0.9	− 0.6	− 0.9	− 0.6	− 2.0	− 2.0	− 1.5	− 1.0
	10	− 1.1	− 0.6	− 0.8	− 0.6	− 2.0	− 2.0	− 1.5	− 1.2
	20	− 0.7	− 0.5	− 0.8	− 0.6	− 1.5	− 1.5	− 1.5	− 1.0
	30	− 0.2	− 0.5	− 0.8	− 0.8	− 1.0			− 1.0
	45	+ 0.2	− 0.5	− 0.8	− 0.8				
	60	+ 0.6	− 0.5	− 0.8	− 0.8				
	0	− 0.7	− 0.6	− 0.9	− 0.7	− 2.0	− 2.0	− 2.0	−
	5	− 0.7	− 0.6	− 0.8	− 0.8	− 2.0	− 2.0	− 1.5	− 1.0
$\frac{3}{2} < \frac{h}{w} < 6$ 	10	− 0.7	− 0.6	− 0.8	− 0.8	− 2.0	− 2.0	− 1.5	− 1.2
	20	− 0.8	− 0.6	− 0.8	− 0.8	− 1.5	− 1.5	− 1.5	− 1.2
	30	− 1.0	− 0.5	− 0.8	− 0.7	− 1.5			
	40	− 0.2	− 0.5	− 0.8	− 0.7	− 1.0			
	50	+ 0.2	− 0.5	− 0.8	− 0.7				
	60	+ 0.5	− 0.5	− 0.8	− 0.7				

Note 1. h is the height to eaves or parapet and w is the lesser horizontal dimension of a building.

Note 2. Where no local coefficients are given, the overall coefficients apply.

Note 3. For hipped roofs the local coefficient for the hip ridge may be conservatively taken as the appropriate ridge value.



KEY PLAN

$y = h$ or $0.15\ w$, whichever is the lesser

Internal air pressure coefficient (C_{pi}) : Internal pressure in a building depends upon the degree of *permeability* of cladding to the flow of air. The internal air pressure may be positive or negative depending on the direction of flow of air in relation to openings in the buiding. The permeability of cladding may be (*i*) zero, (*ii*) normal, (*iii*) medium and (*iv*) large, as defined in Table 2.16, where the value of internal pressure coefficients (C_{pi}) are also given. A positive sign (+) indicates that the pressure is compression on the surface. The internal pressure coefficient is algebraically added to the external pressure coefficient and the analysis which indicates greater distress of the member shall be adopted. Evidently, for a critical condition, the the external pressure is combined with – ve internal pressure; similarly the – ve external pressure is combined with +ve internal pressure to obtain the maximum net pressure.

TABLE 2.16. INTERNAL AIR PRESSURE COEFFICIENTS

Permeability	*Opening in relation to wall area (%)*	*Internal pressure coefficient*
1. zero	0	±0
2. Normal	5	±0.2
3. Medium	5 – 20	±0.5
4. Large	>20	±0.7

Buildings with one open side or opening exceeding 20% of wall area may be assumed to be subjected to internal positive pressure or suction similar to those for buildings with large openings. A few examples of buildings with one sided openings are shown in Fig. 2.7, indicating values internal pressure coefficients with respect to the direction of wind.

9. Wind load on glazing and fixings

Areas such as ridges, eaves, cornices and corners of roofs are subjected to high local suction pressures. The coefficients for local suction are given in Tables 2.14, and 2.15. These coefficients should be used to calculate the forces on these local areas of roof sheeting, glass panels and their fixtures.

10. Wind load on lattice towers

Force coefficient for lattice towers of square or equilateral triangle section with flat sided members for wind blowing against any face shall be taken as given in Table 2.17. Here solidity ratio (φ) is equal to the effective area (*i.e.* projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

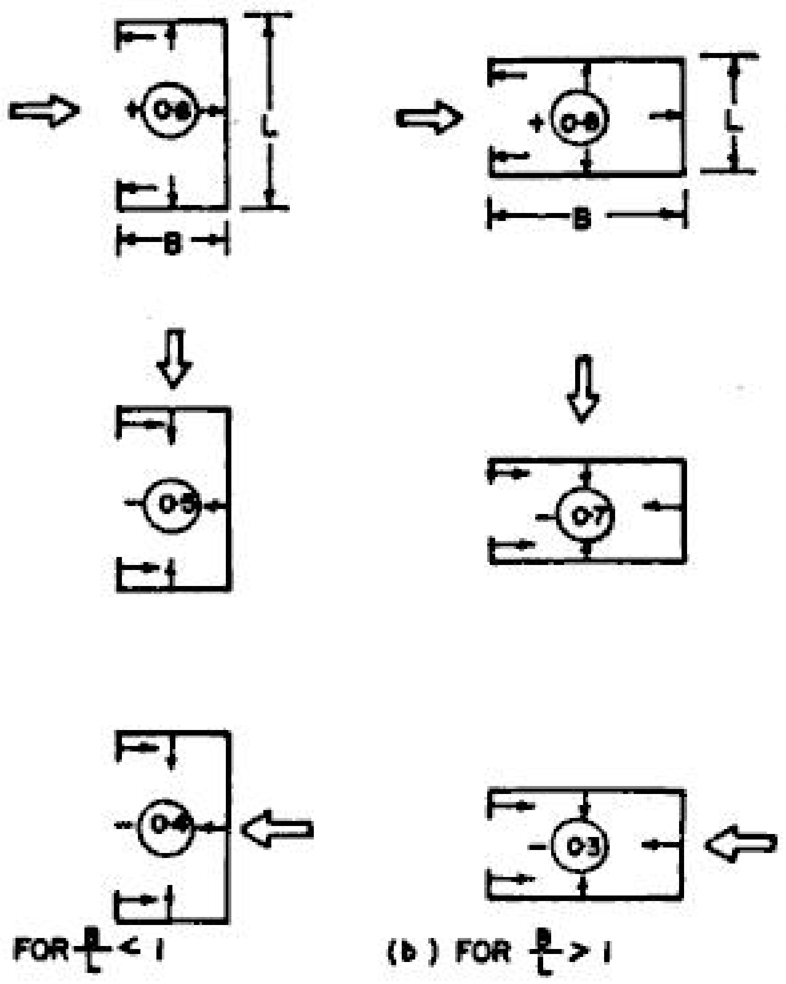


FIG. 2.7. VALUES OF C_{pi} FOR BUILDINGS WITH LARGE ONE SIDED OPENINGS (TOP CLOSED)

TABLE 2.17. OVERALL FORCE COEFFICIENTS FOR TOWERS COMPOSED OF FLAT SIDED MEMBERS

Solidity ratio φ	*Force coefficients for*	
	Square towers	*Equilateral triangular towers*
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

For square lattice towers with flat sided members, the maximum load which occurs when the wind blows into a corner shall be taken as 1.2 times the load for the wind blowing against a face.

11. Wind load on frames

(*a*) ***Single frames***

Force coefficients for a single frame having either (*a*) all flat sided members, or (*b*) all circular members in which all the members of the frame have either $D V_d < 6\ m^2/s$ or $D V_d > 6\ m^2/s$, shall be as given in Table 2.18 according to the type of the member, the diameter (D), the design wind speed (V_d) and the solidity-ratio (φ)

TABLE 2.18. FORCE COEFFICIENTS FOR SINGLE FRAMES

Solidity ratio φ	*Force coefficient C_f for*		
	Flat sided members	*Circular section*	
		Sub-critical flow ($D V_d < 6\ m^2/s$)	*Super critical flow* ($D V_d \geq 6 m^2/s$)
0.1	1.9	1.2	0.7
0.2	1.0	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8
0.75	1.6	1.5	1.4
1.0	2.0	2.0	2.0

Multiple frame buildings : Shielding effect

When the structure has two or more parallel frames, the wind ward frame will have *shielding effect* upon the frames to the leeward side. The wind load on the parts of the frames that are sheltered should be multiplied by a *shielding factor*(η) given in Table 2.19.

TABLE 2.19. SHIELDING FACTOR η FOR MULTIPLE FRAMES

Effective solidity ratio β	*Frame spacing ratio*				
	< 0.5	*1.0*	*2.0*	*4.0*	*>8.0*
0.0	1.0	1.0	1.0	1.0	1.0
0.1	0.9	1.0	1.0	1.0	1.0
0.2	0.8	0.9	1.0	1.0	1.0
0.3	0.7	0.8	1.0	1.0	1.0
0.4	0.6	0.7	1.0	1.0	1.0
0.5	0.5	0.6	0.9	1.0	1.0
0.7	0.3	0.6	0.8	0.9	1.0
1.0	0.3	0.6	0.6	0.8	1.0

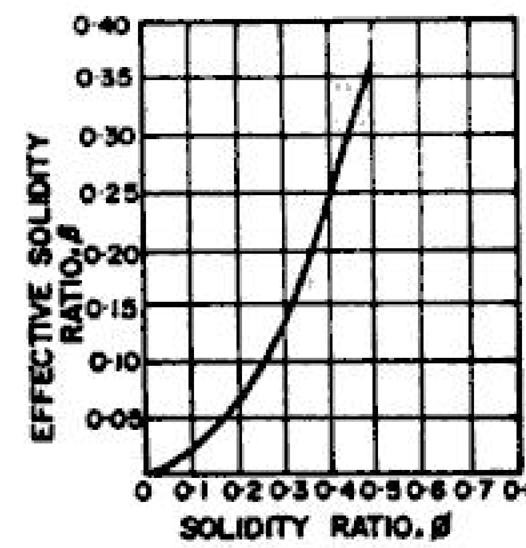


FIG. 2.8. EFFECTIVE SOLIDITY RATIO β FOR ROUND MEMBERS

In the above table, *frame spacing ratio* is equal to the distance centre to centre of the frams, beams or girders divided by the least overall dimension of the frame, beam or girder measured at right angles to the direction of the wind. The *effective solidity ratio* β is equal to φ for flat sided members. For members of circular section β is obtained from Fig. 2.8.

Where there are more than two frames of similar geometry and spacing, the wind load on the third and subsequent frames should be taken as equal to that on the second frame. The loads of the various frames should be added to obtain total load on the structure.

12. Frictional drag

In certain buidings of special shape, a force due to frictional drag shall be taken into account in addition to the wind force discussed above. For rectangular clad buildings, this addition is necessary only when d/h or d/b is greater than 4. The frictional drag force F', in the direction of wind, is given by the following formulae :

If $h \leq b$, $$F' = C_f' (d - 4h)\, b\, p_d + C_f' (d - 4h)\, 2h\, p_d \quad ...(2.12\ a)$$

If $h > b$, $$F' = C_f' (d - 4b)\, b\, p_d + C_f' (d - 4b)\, 2h\, p_d \quad ...(2.12\ b)$$

The first term in each case gives the drag on the roof and the second on the walls. The value of C_f' has the following values.

$C_f' = 0.01$ for smooth surfaces without corrugations or ribs across the
$C_f' = 0.02$ for surfaces with corrugations across the wind direction, and
$C_f' = 0.04$ for surfaces with ribs across the wind direction.

Here, d is the depth along the wind direction, b is transverse to the wind direction and h is the height on the building.

Example 2.1 Design wind speed and wind pressure

An industrial building is to be designed in Delhi. Compute the design wind speed and wind pressure at a height 20 m. The building has maximum dimension of 35 m.

Solution

For Delhi, the basic wind speed $V_b = 47$ m/s (Table 2.6)

An industrial building is to be designed for a 50 year life. Hence the risk coefficient $k_1 = 1.0$ (Table 2.7).

The terrain is in industrial area and hence it belongs to category 3. Also the structure has maximum dimension of 35 m and hence it belongs to class B. Hence from Table 2.8 for terrain category 3, structure class B and height equal to 20 m, $k_2 = 0.98$.

The ground is assumed to be plain. Hence $k_3 = 1$

$$\therefore \quad V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 = 47 \times 1 \times 0.98 \times 1 = 46.06 \text{ m/s}$$

Design wind pressure $p_z = 0.6\, V_z^2 = 0.6\, (46.06)^2 \approx 1273 \text{ N/m}^2 = \mathbf{1.273\ kN/m^2}$

Note : As per earlier code (IS 875-1964) the wind pressure would have been 1500 N/m^2

Example 2.2 ***Lateral pressure on a rectangular building :***

A tall building is to be erected at the outskirts of Madras. The terrain belongs to category 1 and the building is a class B structure. Determine the maximum lateral pressure (considering wind ward and leeward faces) at a height of 30 m if the building has medium permeability. The height to width ratio of the building is 6.5 and the length to width ratio is 1.5

Solution

For Madras, $V_b = 50$ m/s. Factor $k_1 = 1.0$; factor $k_3 = 1.0$ assuming plain ground.

For terrain category 1 and structure class B, we get $k_2 = 1.13$ for a height of 30 m (Table 2.8)

$$\therefore \quad V_z = 50 \times 1 \times 1.13 \times 1 = 56.5 \text{ m/s}$$

and $$p_z = 0.6\, V_z^2 = 0.6\, (56.5)^2 = 1915.4 \text{ N/m}^2$$

External pressure coefficient for rectangular clad buildings are (Table 2.14)

For windward face, $C_{pe} = +0.95$

For leeward face $C_{pe} = -1.85$

Since we are considering both the faces of the building (as a whole), the permeability effect will nullify.

$\therefore$ Lateral pressure $= (0.95 + 1.85)\ 1915.4 = 5363\ \text{N/m}^2 = \mathbf{5.363\ kN/m^2}$

(Note that the pressure direction on both the faces is along the wind.

Example 2.3. Design wind pressure on a slope

An industrial building of 15 m height is being built at Ajmer, near a hillok. The height of the hill is 160 m and the slope is 1 in 4. The building in proposed on the slope at a horizontal distance of 140 m from the base of the hill. Find the design wind pressure.

Solution

Basic wind pressure in Ajmer $= 47$ m/s. Risk coefficient $k_1 = 1.0$

Let us take the terrain of category 2 and building of class *B*.

For height of building 15 m above ground level, $k_2 = 1.02$

$$\theta = \tan^{-1}\frac{1}{4} = 14.04°$$

For $3° < \theta \le 17°$, $C = 1.2\dfrac{Z}{L}$

Here $Z = 160$ m and $L = 160 \times 4 = 640$ m (see Fig. 2.2)

$$\therefore \quad C = 1.2 \times \frac{160}{640} = 0.3$$

Again for $3° < \theta \le 17°$, $L_e = L = 640$ m

x= horizontal distance of the building from the crest, measured + ve towards the windward side.

$$= -(L - 100) = -(640 - 140) = -500 \text{ m}$$

$$\therefore \quad \frac{x}{L} = -\frac{500}{640} = 0.781$$

$$\frac{H}{L_e} = \frac{15}{640} = 0.023$$

Hence from Fig. 2.5, $s = 0.12$

$$\therefore \quad k_3 = 1 + C.s = 1 + 0.3 \times 0.12 = 1.036$$

$$\therefore \quad V_z = V_b . k_1 k_2 k_3 = 47 \times 1.0 \times 1.02 \times 1.036 = 49.67 \text{ m/s}$$

and $p_z = 0.6\,(V_z)^2 = 0.6\,(49.67)^2 = \mathbf{1480\ N/m^2}$

Example 2.4. Design wind pressure on building at hill top

A monumental building is being built on a hill top at Jodhpur. The size of the building is 20 m × 40 m and the height is 10 m. The height of the hill is 400 m, having a gradient of 1 in 5. The building is proposed at an average distance of 100 m from the crest, towards the downward slope. Compute the design wind pressure on the building.

Solution

Basic wind pressure for Jodhpur, $V_b = 47$ m/s

For a memorial building adopt $k_1 = 1.2$

Let us take the terrain of category 2, while the structure is of class *B*.

Hence for height of building equal to 10 m above ground, we get $k_2 = 0.98$. Let us now find topography factor k_3 given by

$$k_3 = 1 + C.s$$

Here $Z = 400 \quad ; \theta = \tan^{-1}\frac{1}{5} = 11.3°$

$$L = 400 \times 5 = 2000 \text{ m}$$

For $\theta < 17°\ ,\ L_e = L = 2000$ m.

and $C = 1.2\dfrac{Z}{L} = 1.2 \times \dfrac{400}{2000} = 0.24$

$$H = 10 \text{ m}$$

x = distance of building from crest = + 100 m

$\therefore$ $\dfrac{x}{L_e} = \dfrac{1600}{2000} = 0.05$;

$$\frac{H}{L_e} = \frac{10}{2000} = 0.005$$

Hence from Fig. 2.5, $s \approx 1$

$\therefore$ $k_3 = 1 + C.s = 1 + 0.24 \times 1 = 1.24$

Hence $V_z = V_b . k_1 k_2 k_3 = 47 \times 1.2 \times 0.98 \times 1.24 = 68.54$ m/s

and $p_z = 0.6\,(68.54)^2 = \mathbf{2818\ N/m^2}$

Example 2.5. Wind pressure on lattice tower

A tall lattice square tower with flat sided members is to be erected at Nagpur. Determine the maximum wind pressure acting on the effective area at a height of 80 m. The solidity ratio may be taken as 0.2 .

Solution

For Nagpur, $V_b = 44$ m/s. Adopt $k_1 = 1.0$ and $k_3 = 1.0$

Let the terrain be of category 4. The structure is of Class C. Hence from Table 2.8, $k_2 = 1.01$ (by linear interpolation)

$\therefore$ $V_z = 44 \times 1.0 \times 1.01 \times 1.0 = 44.44$ m/s

and $p_z = 0.6\,(44.44)^2 = 1185\ \text{N/m}^2$

Now from Table 2.17, $C_f = 3.3$

$\therefore$ Max. wind pressure normal to the face at 80 m height $= 3.3 \times 1185 = \mathbf{3910.5\ N/m^2}$

Also, max. wind pressure in diagonal direction of tower, at 80 m height $= 1.2 \times 3910.5 = \mathbf{4693\ N/m^2}$

Example 2.6 *An industrial building at the out skirts of Jaipur uses pitched steel trusses having slopes of 30°. The building has maximum dimension greater than 50 m. Determine the maximum wind pressure to be used for the design of roof, if the area of openings is about 15%. The height to width ratio of the building is 1.3, and the height of the building is less than 10 m.*

Solution

For Jaipur, $V_b = 47$ m/s. $k_1 = 1.0$ and $k_3 = 1.0$

The terrain is of category 2 and the building is of class *C*.

Hence $k_2 = 0.93$.

$\therefore \quad V_z = 47 \times 1 \times 0.93 \times 1 = 43.71$ m/s

and $\quad p_z = 0.6\,(43.71)^2 = 1146\ \text{N/m}^2 = 1.146\ \text{kN/m}^2$

For slope of 30°, $\frac{h}{w} = 1.3$ and from the gable end :

$$C_{pe} = -0.8 \text{ (on both sloping sides) (Table 2.15)}$$

Also $\quad C_{pi} = \pm 0.5$ for 15% opening (Table 2.16).

Hence maximum design pressure for roof

$$= (0.8 + 0.5)\,1.146 \approx \mathbf{1.49\ kN/m^2}$$

It is to be noted that the local effects (Table 2.15) are used primarily for the design of fixtures etc and not for the design of trusses or purlins.

2.5. EARTHQUAKE FORCES

Earthquakes cause random motion of ground which can be resolved in any three mutually perpendicular directions. This motion causes the structure to vibrate. The predominant direction of vibration is horizontal. The vibration intensity of ground expected at any location depends upon the magnitude of earthquake, the depth of focus, distance from epicentre and the strata on which the structure stands. The important structures should be designed for the maximum vibration intensity expected at the place.

The response of the structure to the ground vibration is a function of the nature of foundation soil, materials, form, size and mode of construction of the structure; and the duration and the intensity of ground motion. In the case of structures designed for horizontal seismic force only, it should be considered to act in any one direction at a time. Where both horizontal and vertical seismic forces are taken into account, horizontal force in any one direction at a time may be considered simultaneously with the vertical force. The vertical seismic force should be considered in case of structures in which stability is a criterion of design, or for overall stability analysis of structures.

Depending on the problem, one of the following two methods may be used for computing seismic force :

(*a*) Seismic coefficient method, and (*b*) Response spectrum method.

A brief discussion on the above two methods is given in Appendix B.

For detailed study, reference may be made to Indian Standard IS : 1893-1984 (Criteria for Earthquake Resistant Design of Structures).

2.6. SOIL AND HYDROSTATIC PRESSURES

In the design of structures, or parts of structures below ground- level, such as basement floors and walls, the pressures exterted by the soil or water or both should be duly accounted for on the basis of established theories. Due allowance should be made for possible surcharge from stationary or moving loads.

While determining the lateral soil pressures on slender structural members, such as pillars which rest in sloping soils, the width of the member should preferably be taken twice its actual width. The relieving pressure of soil in front of the structural member concerned may generally not be taken into account.

Safeguarding of structures and structural members against overturning and horizontal forward movement should be verified. Live loads having favourable effect should be disregarded for the purpose. Due consideration should be given to the possibility of soil being permanently or temporarily removed.

2.7. OTHER FORCES

Impact, vibrations, temperature effects, shrinkage, creep and such other phenomena produce effects on structures some of which may be similar to those caused by external loads. Adequate provision should be made for the effects of one or more of these phenomena separately or in appropriate combination with the specified external loads in accordance with the recommendations of the relevant design codes.

Erection Loads. : All loads required to be carried by the structure or any part of it due to placing or storage of construction materials and erection equipment including all loads due to operation of such equipment, should be considered as *erection loads*. Proper provision should be made to take care of all stresses due to such loads.

2.8. LOAD COMBINATIONS

A judicious combination of the working loads, specified in all the preceeding articles, keeping in view the probability of (*a*) their acting together and (*b*) their disposition in relation to other loads and the severity of stresses or deformations caused by the combination of the various loads, is necessary to ensure the required safety and economy in the design of a structure.

The following load combinations may be adopted

(*a*) Dead load alone

(*b*) Dead load + partial or full live load whichever causes the most critical condition in the structure.

(*c*) Dead load + wind or seismic loads.

(*d*) Dead load + such part of or whole of the specified live load whichever is most likely to occur in combination with specified wind or seismic loads + wind or seismic loads.

(*e*) Dead loads + such parts of the live load as would be imposed on the structure during the period of erection + wind or seismic loads + ecrection loads.

Note: For design purposes, wind load and seismic forces shall be assumed not to act simultaneously. Both forces shall, however, be investigated separatety and adequately provided for.

2.9. STRESS STRAIN RELATIONSHIP FOR MILD STEEL

Steel is the chief structural metal because it combines high strength (both in tension and compression), great stiffness (high modulus of elasticity), good ductility and ease of fabrication with relatively low price. Stress-strain curve is the single most important piece of information for determining the structural properties of a material. Fig. 2.9 (*a*) shows a typical stress strain diagram for structural carbon steel. The initial and structurally most important portion, upto a strain of about 0.02 (or 2%) is shown to larger scale in Fig. 2.9 (*b*).

From the stres-strain curve, we observe the following characteristic features:

1. Over a large initial range of stress, stress and strain are proportional, that is, the material behaves elastically upto the *proportional limit* σ_p.

2. The slope of the initial straight line, upto the proportional limit, is Young's modulus E, which for all structural steels, carbon or alloy, is very closely the same, and its value may be taken equal to 2×10^5 N/mm^2.

3. At stresses higher than σ_p, there follows a brief, slightly curved portion of the diagram until, at *upper yield point* σ_{yu}, the steel suddenly begins to yield.

4. Then, at a slightly lower stress σ_y (the *lower yield point*, or simply the yield point) the specimen keeps stretching (yielding) at practically constant stress upto a total elongation of about 1.5 percent of initial length. This amount is more than 10 times the maximum elastic elongation which was attained before yielding started and is known as the *plastic range.*

5. When the specimen is stretched further, it begins to harden, and an increasing force is required to produce further stretching. This phonomenon is called *strain hardening.*

6. Strain hardening continues until the specimen reaches *maximum* or *ultimate stress*, known as tensile strength.

7. Upon further stretching, some region near the middle suddenly narrows down (a process known as *necking*, which corresponds to the discending part of stress-strain diagram), and the specimen shortly thereafter breaks in two at the narrowest part of the neck.

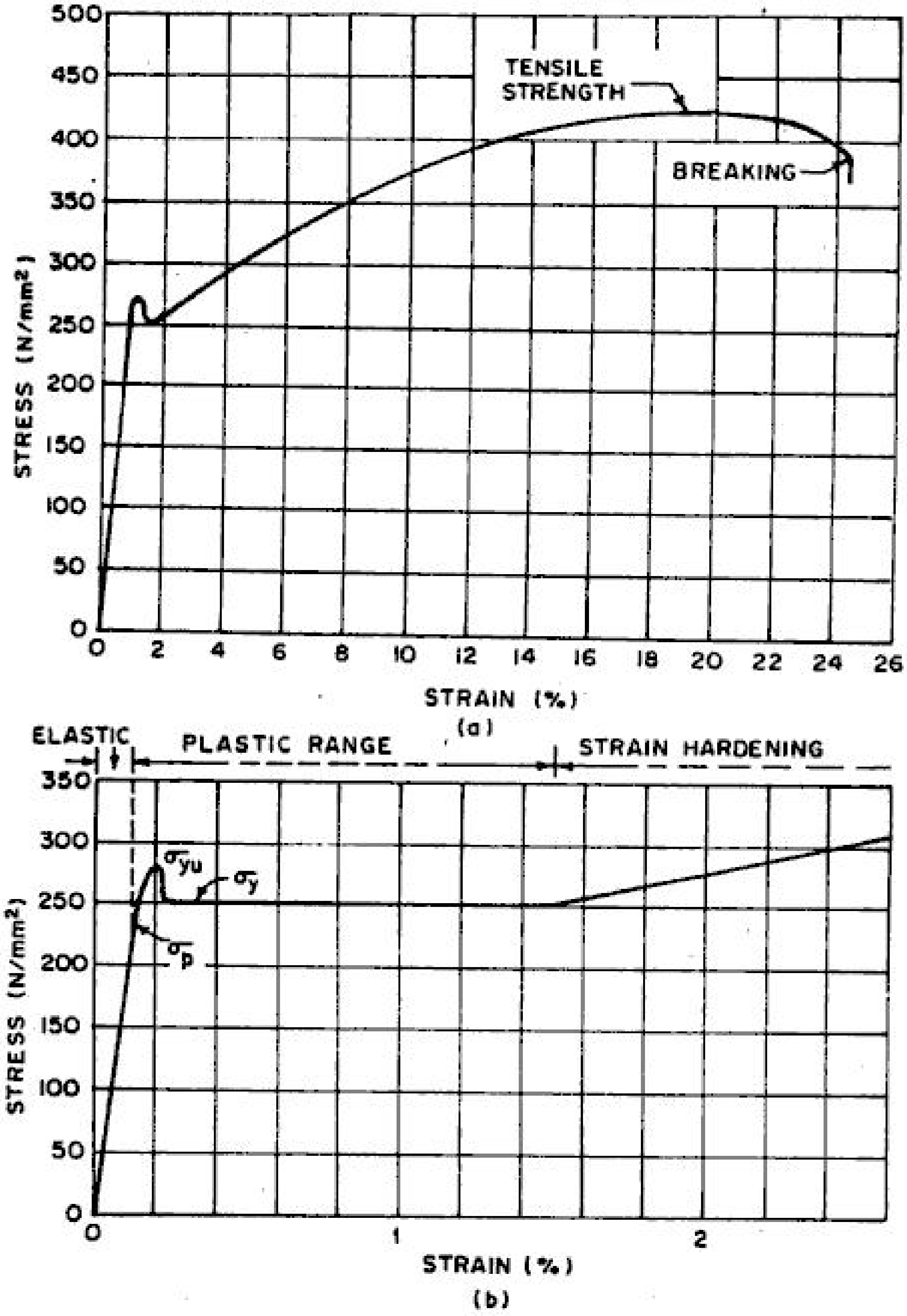


FIG. 2.9. TYPICAL STRESS STRAIN DIAGRAM FOR STRUCTURAL CARBON STEEL

8. The total elongation upto failure is seen to amount to more than 20% of the original length. This ability of structural steel to undergo sizable permanent (plastic) deformation before fracture is known as its *ductility* and is a most important property.

9. The most important structural properties of steel are : (*i*) modulus of elasticity which governs the amount of deflection under (elastic) service conditions, (*ii*) the yield point, that is, that stress below which permanent deformations are small and reversible, and (*iii*) the ductility. It is also seen that tensile strength is of secondary importance since in most cases, when the yield point is exceeded, a structure will distort into uselessness long before the tensile strength is reached.

2.10. FACTOR OF SAFETY

Fig. 2.9 (*b*) shows that should the applied stress reach the yield point, the member will suddenly elongate by a very sizable amount. Such yielding of the member will cause detrimental deformations of the structure of which it is part. It is necessary, therefore, at *design loads*, that is under service conditions, to keep the applied stress below the yield point by a certain margin. Hence *allowable stress* in a tension member is obtained by dividing the yield stress by a constant referred to as *safety factor* or *factor of safety* (F):

$$\text{Allowable stress } (\sigma_a) = \frac{f_y}{F} \qquad \text{...(2.7)}$$

In order to find the *minimum* value of factor of safety (F_{min}), let us define it in a rational method. The factor of safety (F) may be defined as the ratio of *computed strength* (S) of the structure or the structrual member to the respective *computed force P* carried by it :

$$F = \frac{S}{P} \qquad \text{...(a)}$$

The factors that influence the magnitude of *lowest* probable strength (S) are uncertainties in the mechanism failure, properties of material and workmanship. Similarly, the factors that influence the magnitude of the *highest* probable force P are the uncertainties in loading conditions and the structural behaviour. Because of these uncertainties, the *probable values* of S and P deviate from the *computed values*.

If ΔS is the maximum possible deviation of actual value from the computed value of strength, the *minimum probable value* of strength is given by

$$S_{min} = (S - \Delta S) \qquad \text{...(b)}$$

Similarly if ΔP is the maximum deviation of the actual value from the computed value of force, the *maximum probable* value of the force P is given by

$$P_{max} = (P + \Delta P) \qquad \text{...(c)}$$

Hence $$(S - \Delta S) \geq (P + \Delta P)$$

or $$S\left(1 - \frac{\Delta S}{S}\right) \geq P\left(1 + \frac{\Delta P}{P}\right)$$

From which minimum *factor of safety* ($F_{min.}$) is given by

$$F_{min.} = \frac{S}{P} = \left(\frac{1 + \Delta P/P}{1 - \Delta S/S}\right) \qquad \text{...(2.8)}$$

Assuming maximum deviations ΔS and ΔP to be 25% of the computed values of S and P, we have

$$F_{min.} = \frac{1 + 0.25}{1 - 0.25} = 1.67 \qquad \text{...(2.9)}$$

2.11. PERMISSIBLE STRESSES

The following are various types of stresses developed in steel structures :

(*i*) Direct tensile stress

(*ii*) Direct compressive stress

(*iii*) Bending stresses (tensile or compressive)

(*iv*) Shear stresses

(*v*) Bearing stresses

(*i*) ***Axial tensile stress*** : The *permissible stress* in axial tension (σ_{at}) on the net effective area of the sections shall not exceed

$$\sigma_{at} = 0.6 f_y \qquad ...(2.10)$$

where f_y = minimum yield stress of steel (MPa or N/mm^2)

(*ii*) ***Axial compressive stress*** : The direct stress in compression on the gross sectional area of axially loaded compression members shall not exceed 0.6 f_y nor the permissible stress σ_{ac} calculated using the following formula :

$$\sigma_{ac} = 0.6 \frac{f_{cc} \cdot f_y}{\left[(f_{cc})^n + (f_y)^n \right]^{1/n}} \qquad ...(2.11)$$

where σ_{ac} = Permissible stress in axial compression (MPa or N/mm^2)

f_{cc} = elastic critical stress in compression= $\dfrac{\pi^2 E}{\lambda^2}$

E = modulus of elasticity of steel; 2×10^6 N/mm^2

$\lambda\,(= l/r)$ = slenderness ratio of the member

n = a factor, assumed as 1.4

The valuess of σ_{ac} for some of the Indian Standard Structural Steels are given in Table 7.5.

(*iii*) ***Bending stresses*** : The maximum bending stress in tension or in compression in extreme fibre calculated on the effective section of a beam shall not exceed maximum permissible bending stress in tension (σ_{bt}) or in compression (σ_{bc}) obtained as follows nor the values specified by Eq. 2.13 given in the next para :

$$\sigma_{bt} \text{ or } \sigma_{bc} = 0.66 f_y \qquad ...(2.12)$$

Maximum permissible bending compressive stress in beams and plate girders

For beams and plate girders, bent about the axis of maximum strength (*x-x* axis), the maximum bending compressive stress on the extreme fibre, calculated on the effective section shall not exceed the maximum permissible bending compressive stress σ_{bc} obtained by the following formula :

$$\sigma_{bc} = 0.66 \frac{f_{cb} \cdot f_y}{\left((f_{cb})^n + (f_y)^n \right]^{1/n}} \qquad ...(2.13)$$

f_{cb} = elastic critical stress in bending

n = a factor assumed as 1.4.

The method of computing f_{cb} has been explained in detail, in Chapter 8 (Design of flexural members).

(*iv*) ***Shear stress:*** The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, shall not exceed the value τ_{vm} given by

$$\tau_{vm} = 0.45 f_y \qquad ...(2.14)$$

where τ_{vm}= max. permissible shear stress

(*v*) ***Bearing stress*** : The bearing stress in any part of a beam when calculated on the net area of contact shall not exceed the value of σ_p determined by the following formula :

$$\sigma_p = 0.75 f_y \qquad ...(2.15)$$

where σ_p = max. permissible bearing stress.

3

Steelwork Connections : I
Riveted Connections

3.1. TYPES OF 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, they 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 indeterminate, 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 steelwork:

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.

3.2. 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. Fig. 3.1 shows the essential steps in process of riveting, during which a *hot rivet* is driven in its plastic state, and a head is formed 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° F to 1950° 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.

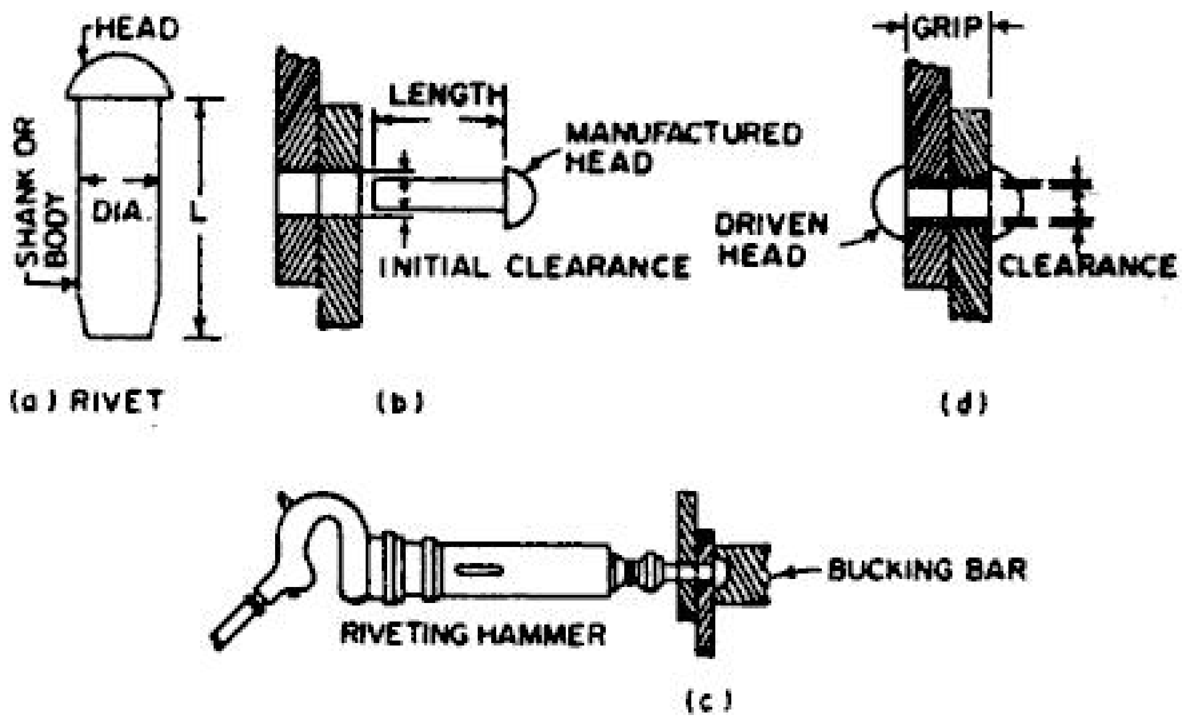


FIG. 3.1. ESSENTIAL STEPS IN RIVETING

3.3. RIVET SIZE, RIVET HOLE AND CONVENTIONAL SYMBOLS

The diameter of unheated rivet, before driving is known as the *nominal diameter*. Rivets are manufactured in nominal diameters of 12, 14, 16, 18, 20, 22, 24, 27, 30, 33, 36, 39, 42 and 48 mm. The diameter of *rivet hole* is made larger than the nominal diameter of the rivet by 1.5 mm for rivets less than or equal to 24 mm and by 2 mm for diameters exceeding 24 mm. The *grip* of the rivets is equal to the total thickness of plates to be joined by the rivet. The *length* of undriven rivet is the sum of (*i*) grip, (*ii*) length required for the head to be formed, and (*iii*) an additional length to fill up the space between the rivet and plate holes.

Several types of heads are used in structural design as shown in Fig. 3.2. The commonly used

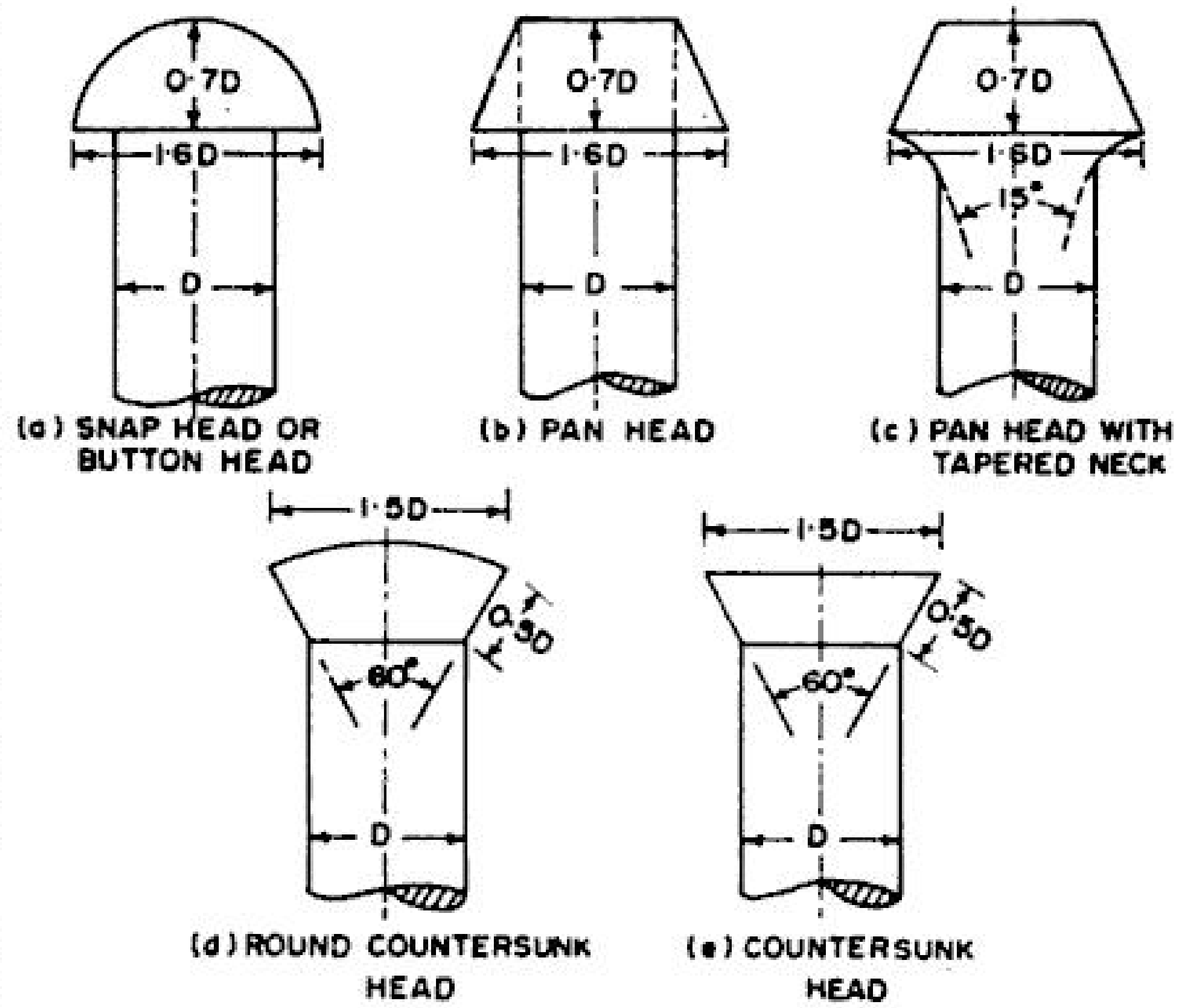


FIG. 3.2. DIFFERENT TYPES OF RIVET HEADS.

head is the *snap head* (Fig. 3.2 *a*) which is also known as *button head* or *round head. Counter-sunk* heads are used to provide a flush surface.

Rivet holes are made in the plates to be joined by (*i*) drilling or (*ii*) punching. Drilled holes are always preferred because the holes are perfect. Punched holes damage the plates to be joined.

Rivets are generally made of structural rivet steel conforming to IS 1929-1961 (for diameter between 12 to 48 mm) and IS 2155-1962 (for diameter below 12 mm). *High tensile steel rivets* if used are manufactured from steel conforming to IS : 1149-1982.

Rivets used for various types of structural connections are represented on the drawing through conventional symbols shown in Fig. 3.3.

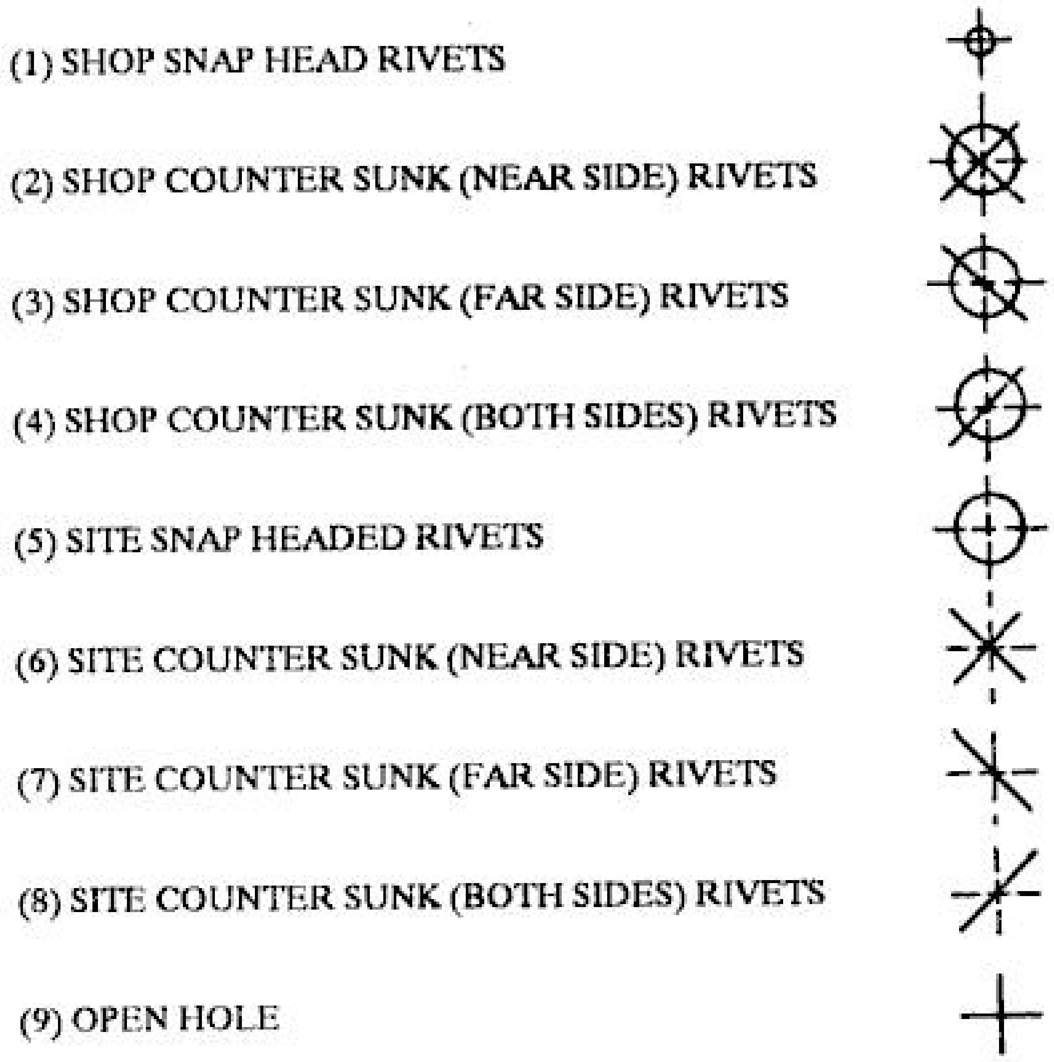


FIG. 3.3. CONVENTIONAL SYMBOLS FOR RIVETING

3.4. COMMON DEFINITIONS (Fig. 3.4)

*1. **Nominal diameter of Rivet***: It is the diameter of unheated rivet. It is the *stated diameter* of the rivet, available in the market.

*2. **Gross diameter of Rivet:*** It is the diameter of the rivet in the hole, measured after driving. It is taken equal to the diameter of the hole itself.

*3. **Gross area of rivet*** : It is the area calculated on the basis of gross diameter of the rivet.

*4. **Pitch of rivets** (p)* : It is the distance between centres of two adjacent rivets in a row.

*5. **Gauge line***: It is the line of rivets which is parallel to the direction of stress.

*6. **Gauge distance or 'gauge'** (g)*: It is the perpendicular distance between two adjacent gauge lines. This is also known as *back pitch*.

*7. **Edge distance*** : It is the distance of the edge of the member or the cover plates from the centre of extreme rivet hole.

*8. **Lap :*** It is the distance normal to the joint between edges of the overlapping plates in a lap joint or between the joint and the end of cover plates in a butt joint.

Note : There is a lot of confusion in the available literature about the nomenclatures 'pitch' and 'gauge'. To avoid confusion, *pitch* may be defined

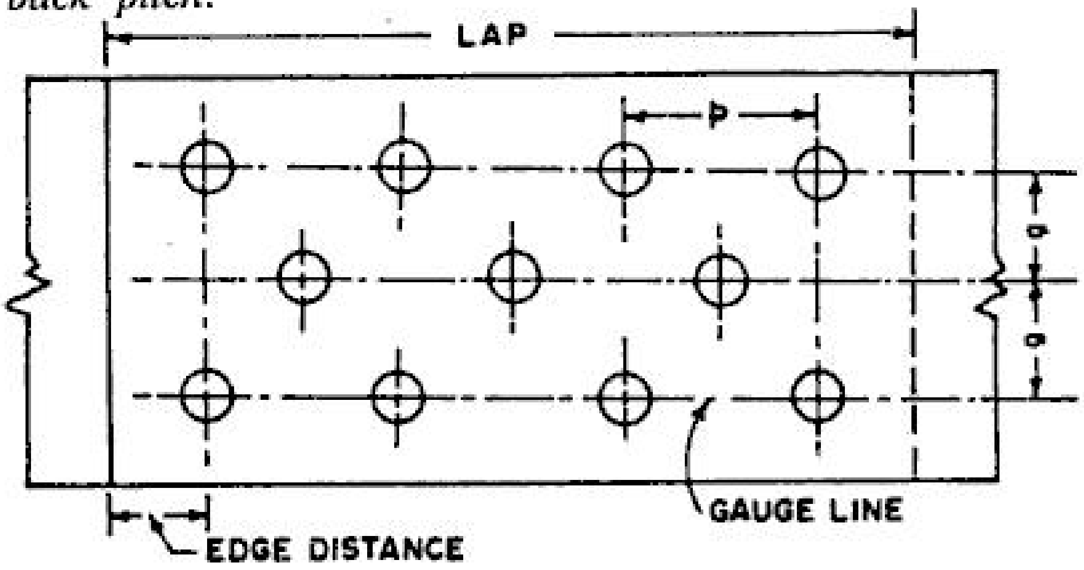


FIG. 3.4. DEFINITION SKETCH.

as the centre to centre distance between rivets *measured along the long directions* of the joint while gauge is the distance between rivet centres measured along the *short* direction.

3.5. WORKING STRESSES IN RIVETS

1. The working stresses (or maximum permissible stresses) in mild steel shop rivets as per IS: 800-1984 are given in Table 3.1.

2. The permissible stress in a high tensile steel rivet shall be those given in Table 3.1 *multiplied* by the ratio of tensile strength of the rivet material to the tensile strength as specified in IS: 1148-1973.

Note: For field rivets the permissible stresses shall be reduced by 10 percent.

TABLE 3.1. MAXIMUM PERMISSIBLE STRESSES IN RIVETS

Type of Rivet	*Axial Tension*, σ_{tf} N/mm²(MPa)	*Shear*, τ_{vf} N/mm² (MPa)	*Bearing*, σ_{pf} N/mm² (MPa)
1. Power driven rivets	100	100	300
2. Hand driven rivets	80	80	250

3. The calculated bearing stress of a rivet on the parts connected by it shall not exceed : (*a*) the value of f_y for hand driven rivets, and (*b*) the value 1.2 f_y for power driven rivets, where f_y is the yield stress of the connected parts.

Where the end distance of a rivet (that is, the edge distance in the direction in which it bears) is less than a limit of twice the effective diameter of the rivet, the permissible bearing stress of that rivet on the connected part shall be reduced in the ratio of the actual and the distance to that limit.

4. ***Combined shear and tension***: Rivets and bolts subjected to both shear and axial tension shall be so proportioned that the calculated shear and axial tension do not exceed the allowable stresses τ_{vf} and σ_{tf} and the expression $\left(\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{\sigma_{tf,cal}}{\sigma_{tf}}\right)$ does not exceed 1.4.

3.6. TYPES OF RIVETED JOINTS

A riveted joint may be classified according to (*a*) arrangement of rivets and plates (*b*) mode of load transmission, and (*c*) nature and location of load with respect of rivet group.

(*a*) **Arrangement of Rivets and Plates.**

According to the arrangement of rivets and plates, riveted joints may be of the following types:

(1) *Lap Joint*:

(*i*) Single riveted

(*ii*) double riveted.

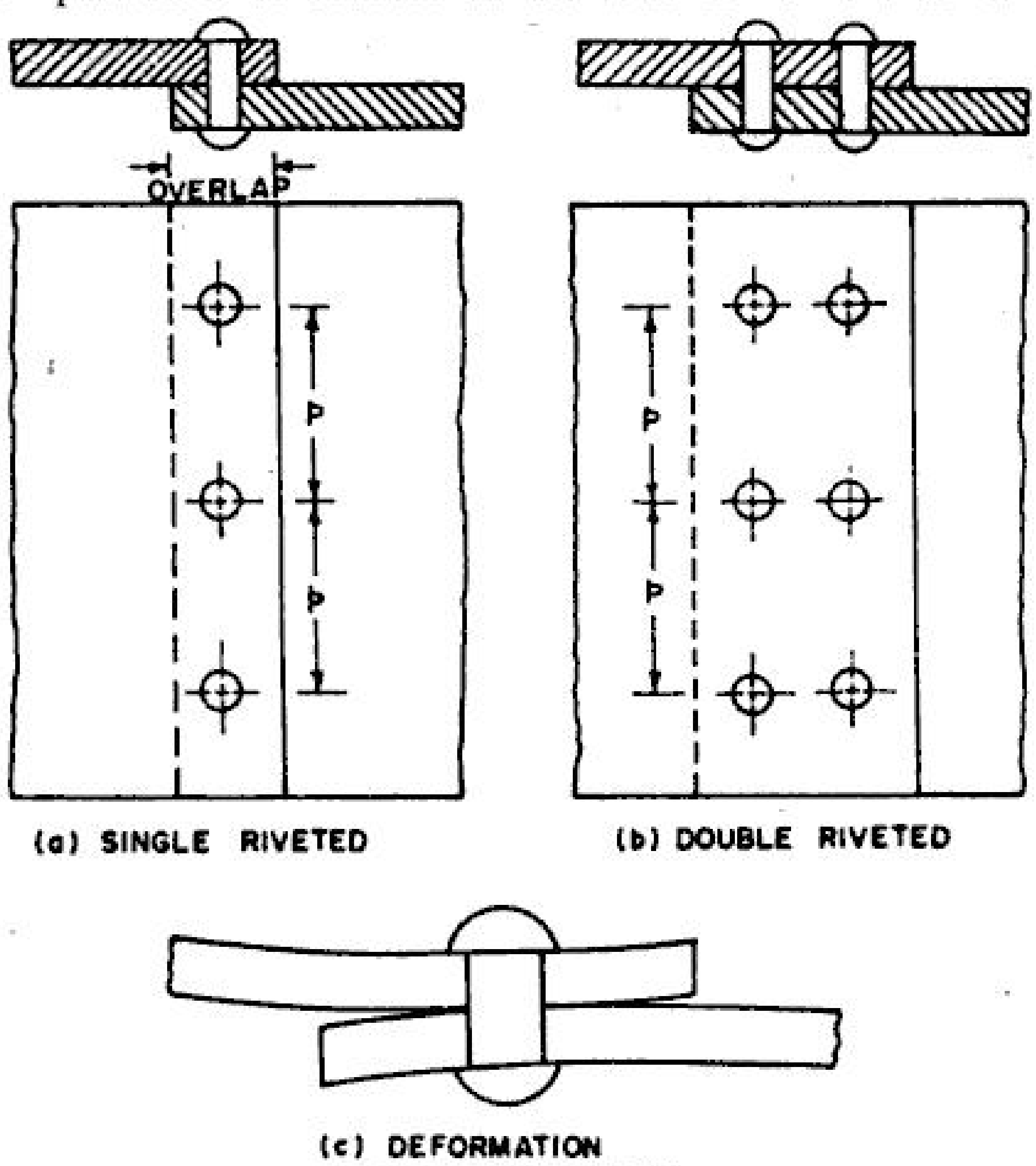


FIG. 3.5. LAP JOINTS

(2) *Butt joint*:

(*i*) Single riveted butt joint with single cover plate.

(*ii*) Single riveted butt joint with double cover plate

(*iii*) Doubled riveted butt joint with double cover plate.

Fig. 3.5 (*a*) shows single riveted lap joint, while 3.5 (*b*) shows double riveted lap joint. The two lines of pull in the joined plates are not in alignment, resulting in bending stresses tending to distort the joint, as shown in Fig. 3.5 (*c*).

Fig. 3.6 (*a*) and (*b*) show single riveted butt joint and double riveted butt joint respectively, in which the edges of the plates come flush and the *cover plates* are used to join them. In a single cover butt joint, bending stresses may develop, tending to distort the joint, as shown in Fig. 3.7. This possibility is completely eliminated by using a double cover butt joint.

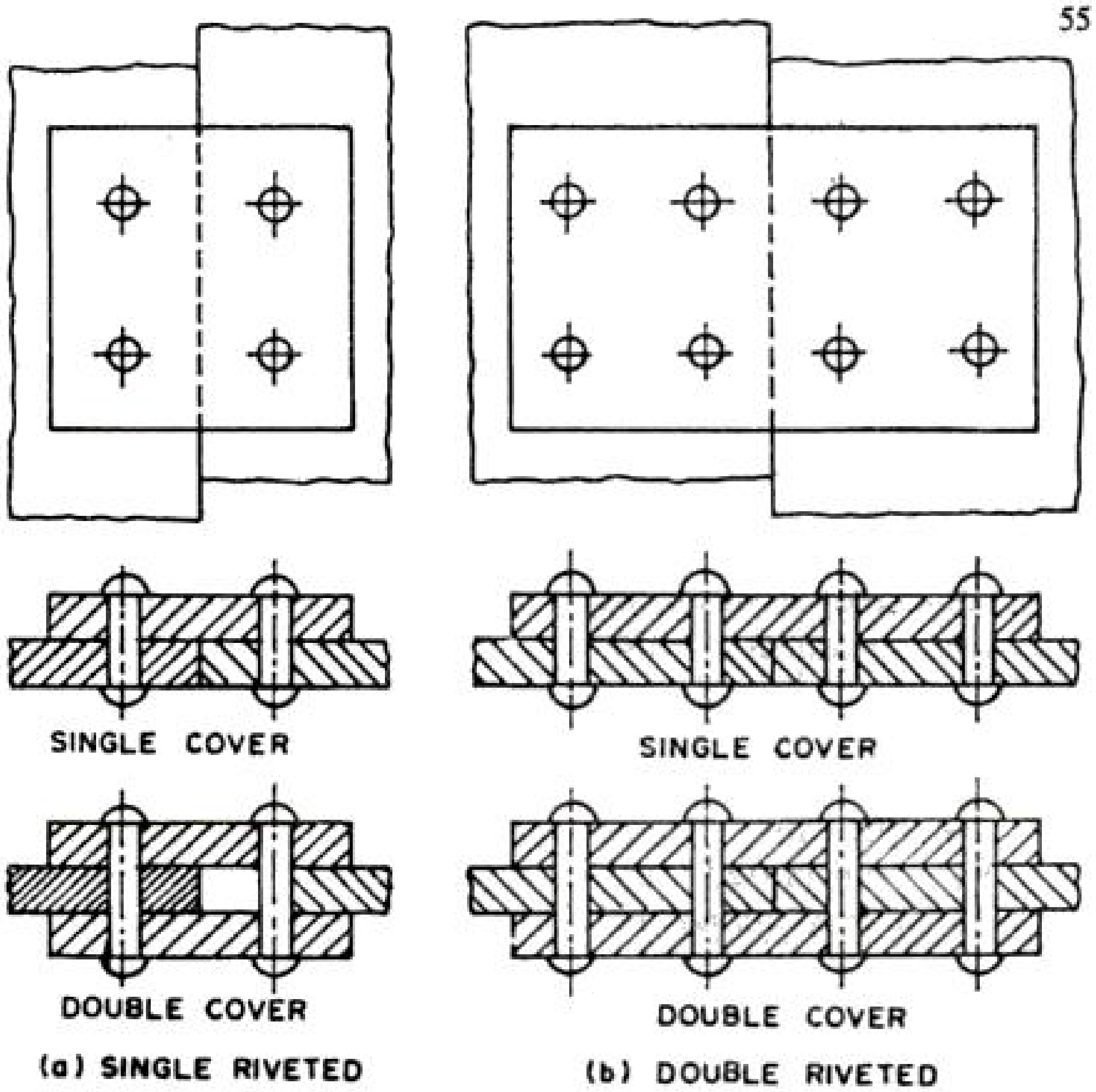


FIG. 3.6. BUTT JOINTS.

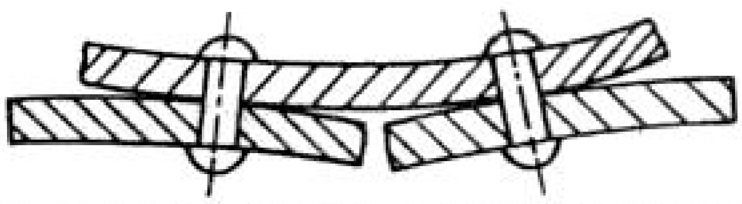


FIG. 3.7. DISTORTION OF BUTT JOINT WITH SINGLE COVER PLATE

(*b*) **Mode of load transmission**

Riveted connections can also be classified according to the *mode* of load transmission by the rivets. If the load is transmitted through *bearing* between the plate and the shank of the rivet producing shear in the rivet (Fig. 3.8), the rivet is said to be in *shear*. When the load is transmitted by shear in only *one section* of the rivet, the rivet is said to be in *single shear* (Fig. 3.8 *a*). When the loading of the rivet is such as to have the load transmitted in *two shear planes*, the rivet is said to be in *double shear* (Fig. 3.8 *b*). When the load is transmitted by shear in *more than two planes*, the rivet is said to be in *multiple shear* (Fig. 3.8 *c*).

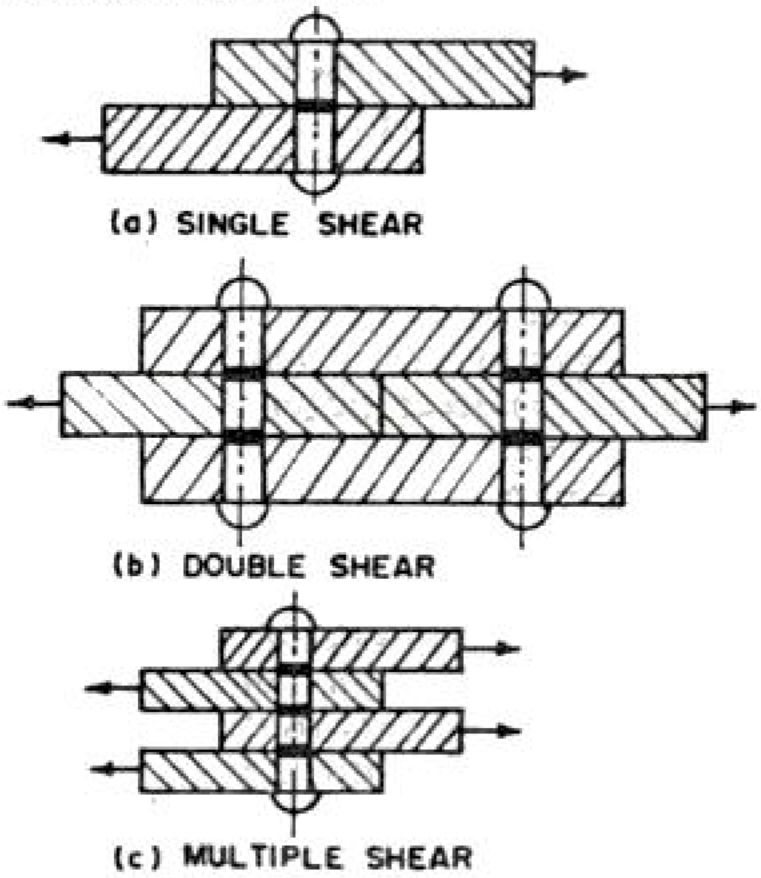


FIG. 3.8. RIVETS IN SHEAR.

If however, the load is transmitted through bearing between the plate and the head of the rivet, producing tension in the rivet, the rivet is said to be in tension, as shown in Fig. 3.9.

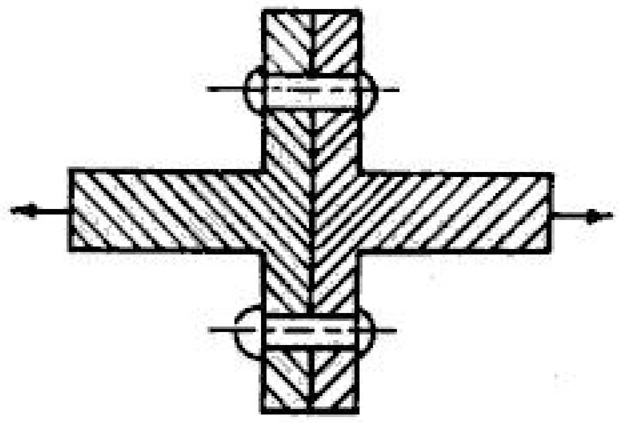
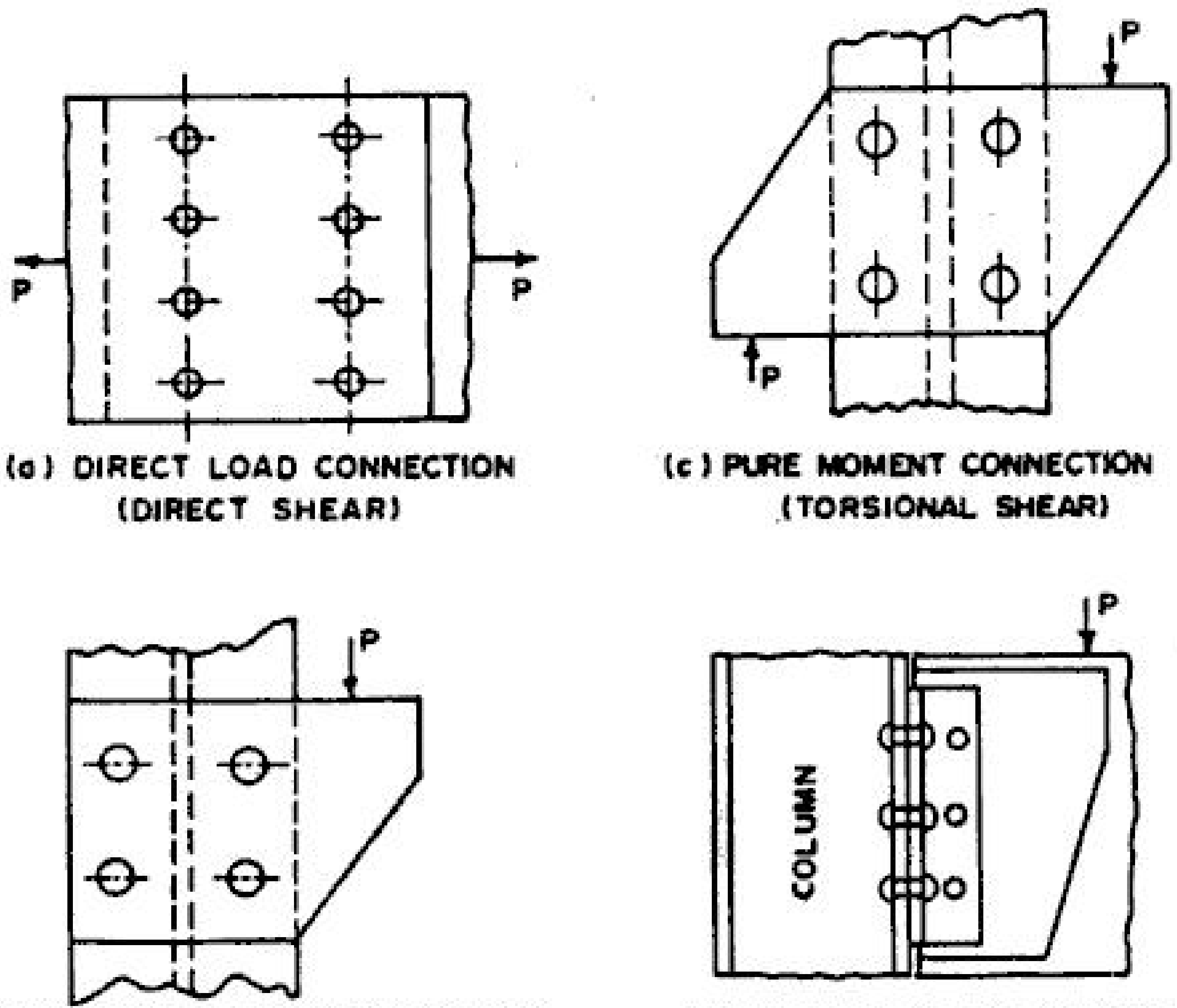


FIG. 3.9. RIVETS IN TENSION.

(c) Nature and location of load

A riveted connection can also be classified according to the nature and location of load with respect to the rivet group. When the load passes through the centroid of rivet cross-sectional area, the connection is said to be a *direct load connection* (Fig. 3.10 *a*) in which the rivets are subjected to direct shear stresses. When the load does not pass through the centroid of the rivet group, it is said of be an *eccentric load connection* (Fig. 3.10 *b*) or direct shear or torsional shear connection. When the load transmitted consists of a *pure torque or moment,* it is said to be *pure moment connection* (Fig. 3.10 *c*) in which the rivets are subjected to *torsional shear stresses*. If however, the load transmitted is such that the rivets are both in shear as well as in tension, it is said to be *moment-shear connection* or a *tension-shear connection* (Fig. 3.10 *d*). Beam to column moment connections or column-bracket connections fall under this category.

There may be two more types of *special connections*: (*a*) Eccentric tension connection and (*b*) pure couple connection. These are shown in Fig. 3.11 (*b*) and (*c*) respectively, for joined members shown in Fig. 3.11 (*a*). In these connections, the rivet is subjected to both the *direct tension* as well as *bending stress* (compression as well as tension).



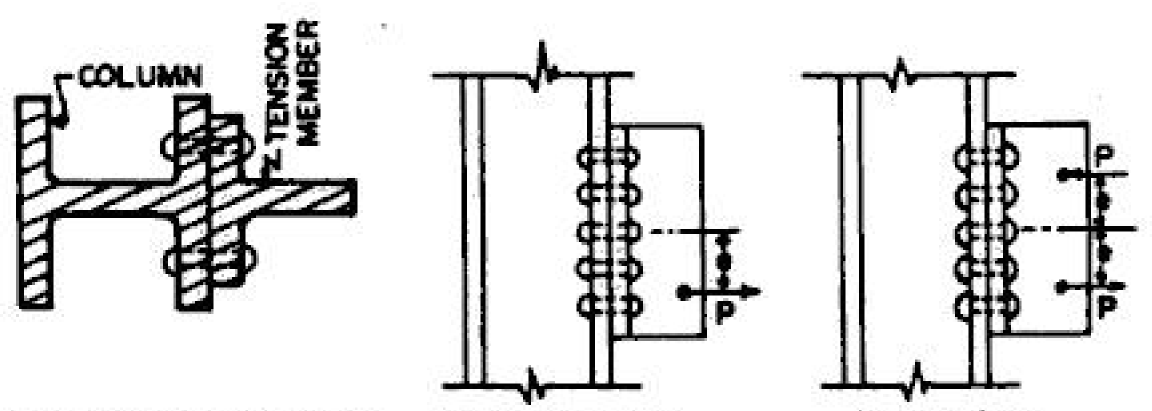
(a) DIRECT LOAD CONNECTION (DIRECT SHEAR)

(c) PURE MOMENT CONNECTION (TORSIONAL SHEAR)

(b) ECCENTRIC LOAD CONNECTION (DIRECT SHEAR AND TORSIONAL SHEAR)

(d) MOMENT SHEAR CONNECTION (TORSIONAL SHEAR AND DIRECT TENSION)

FIG. 3.10. TYPES OF RIVETED CONNECTIONS.



(a) PLAN OF CONNECTION (b) ECCENTRIC TENSION (c) COUPLE

FIG. 3.11. SPECIAL CONNECTIONS INDUCING DIRECT TENSION AS WELL AS BENDING STRESSES

3.7. MODES OF FAILURE OF A RIVETED JOINT

A riveted joint may fail in one of the following ways.

1. *Tension failure* in the plate (Fig. 3.12)
2. *Shearing failure* across one or more planes of the rivets (Fig. 3.13)
3. *Bearing failure* between the plates and the rivets (Fig. 3.14)
4. *Plate shear* or *shear out failure* in the plate (Fig. 3.15)

1. Tension failure in the plates

The plate can fail by *tearing off* across the pitch length (Fig. 3.12 *a*) due to lack of tensile strength of the plate on a section along the row of the rivets. The plate may also tear along the diagonal (Fig. 3.12 *b*). However, this type of failure is unlikely to happen if the back pitch is atleast 13 times the rivet diameter. We will investigate the possibility of failure represented in Fig. 3.12 (*a*) only, as it is assumed that sufficient *back pitch* has been provided.

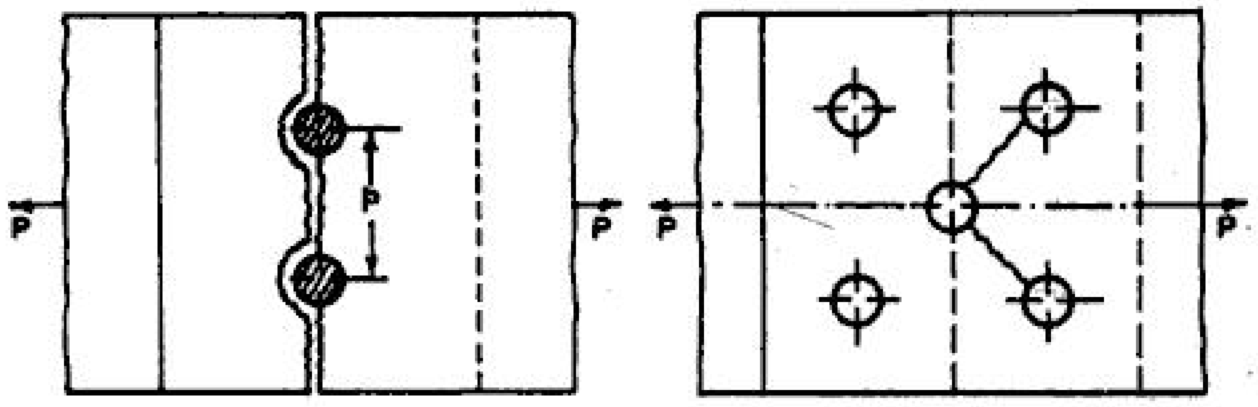


FIG. 3.12. TENSION FAILURE OF PLATES

Let P_{ut} = Pull applied per pitch length, for tension failure

f_t = Ultimate tensile strength of the plate material.

p = pitch of rivets, measured perpendicular to the direction of force.

t = thickness of the plate

d = gross diameter (or formed diameter) of the rivet.

$\therefore \quad P_{ut} = f_t \times$ resisting section

or
$$P_{ut} = f_t\,(p - d)\,t \qquad ...(3.1)$$

2. Shearing failue of the rivet

In a riveted joint, the rivets may themselves fail in shear. The tendency is to cut through the rivet across the section lying in the plane between the plates it connects (Fig. 3.13 *a, b*). In analysing this possible manner of failure, one must always note whether a rivet acts in *single shear* or *double shear*. In the latter case, two cross-sectional areas of the same rivet resist the applied force. The shearing stress is assumed to be *uniformly distributed* over the cross-section of the rivet.

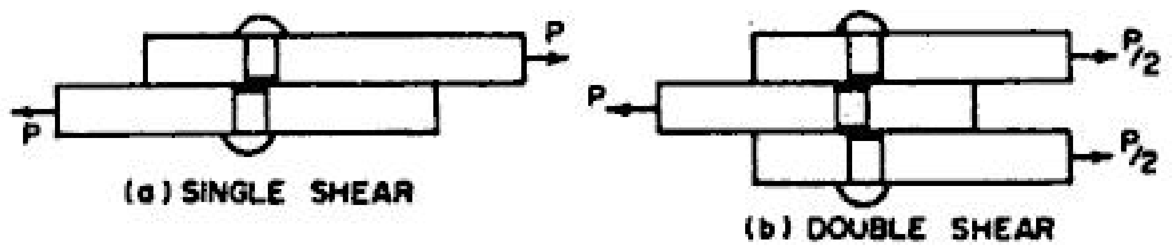


FIG. 3.13. SHEAR FAILURE OF RIVET

Let P_{us} = pull required, per pitch length, for shear failure

f_s = ultimate shear strength of the rivet material

d = gross diameter of rivet.

Resisting area of rivet section $= \frac{\pi}{4} d^2$ in single shear

and $= 2\frac{\pi}{4} d^2$ in double shear

$\therefore \qquad P_{us} = \frac{\pi}{4} d^2 f_s$, for single shear

and $\qquad P_{us} = 2 \times \frac{\pi}{4} d^2 f_s$, for double shear ...(3.2)

3. Bearing failure between plates and rivets.

A rivet joint may fail if a rivet itself is deformed by the plate acting on it (Fig. 3.14 *a*) or if the rivet crushes the material of the plate which it bears. The bearing failure of rivet is more common. In calculating the resistance to bearing, it is assumed that the bearing stress is uniform, as shown in Fig. 3.14 (*c*).

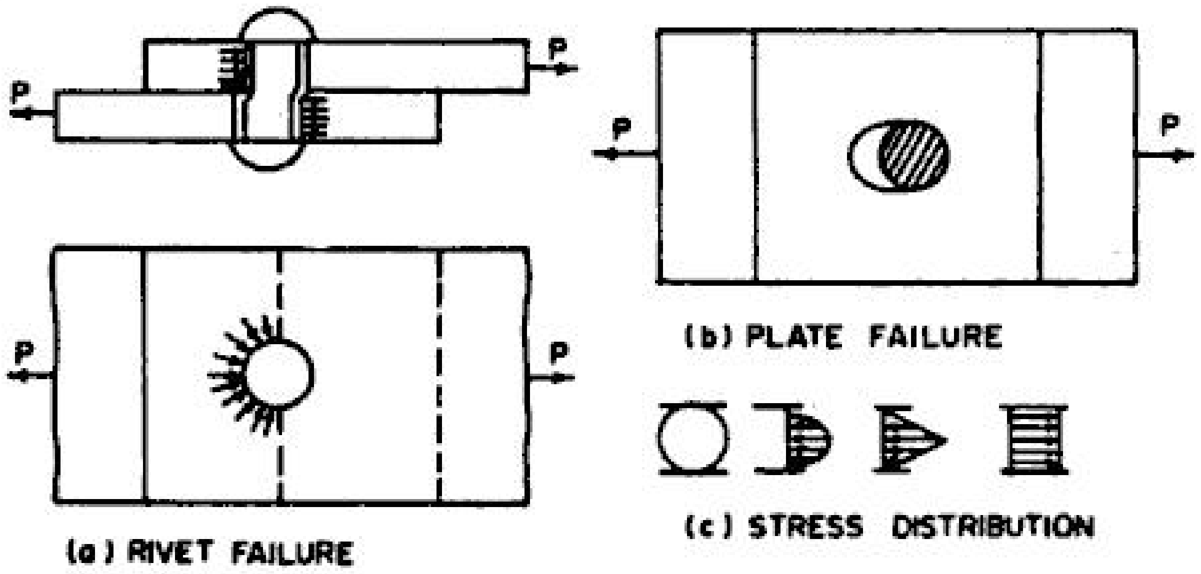


FIG. 3.14. BEARING FAILURE.

Let $\quad P_{ub}$ = pull required, per pitch length, for bearing failure of rivet.

f_b = ultimate crushing strength of rivet material.

d = gross diameter of rivet.

t = thickness of the plate.

P_{ub} = (intensity of radial pressure, at failure) × projected area.

$\therefore \qquad P_{ub} = f_b \,.\, d \,.\, t$ ...(3.3)

Equations 3.1, 3.2 and 3.3 are applicable for single riveted joint. If, however there are *n rows of rivets* per pitch length, we have

$P_{ut} = (p - d)\, t \,.\, f_t$ ...(3.1 *a*)

$P_{us} = n \,.\, \frac{\pi}{4} d^2 f_s$ for single shear ...(3.2 *a*)

or $\qquad = 2n \frac{\pi}{4} d^2 f_s$ for double shear

and $\qquad P_{ub} = n\, d\, t \,.\, f_b$ ...(3.3 *a*)

4. Shear-out failure of the plate

Fig. 3.15 (*a*) shows the 'shear-out' failure of plate, which can be prevented by providing a sufficient edge distance beyond the rivet.

Sometimes, *splitting failure* of plate may occur, which can also be prevented by providing a sufficient edge distance. Since the *edge distance* is governed by *code requirements* (see § 3.9), *'shear-out'* stresses are not calculated.

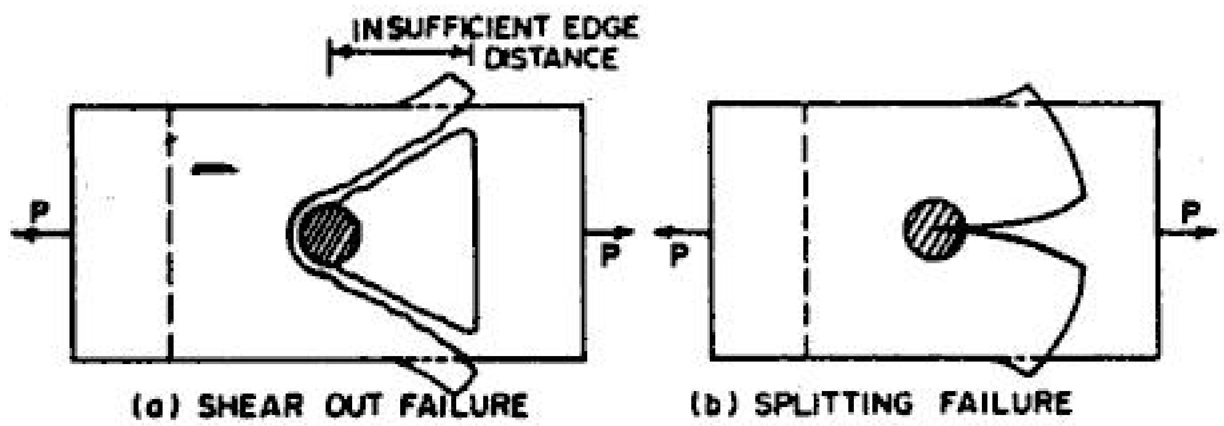


FIG. 3.15. SHEAR OUT FAILURE OF PLATE

3.8. STRENGTH OF RIVETED JOINT

In the previous section, we have computed the values of pull required for the failure of a riveted joint, using ultimate strength values for the material. However, same expressions can be used to compute the *strength* of a riveted joint, by using the *permissible values of the stresses in the materials*, as shown below :

1. Strength of plate

Let σ_{at} = permissible stress in plate in axial tension.

Strength of plate P_t, per pitch length is given by

$$P_t = \sigma_{at} (p - d) t \quad ...(3.4)$$

2. Strength of Rivet or Rivet Value

The strength of rivet, commonly known as *rivets value* (*R*) is the smaller of (*i*) shearing strength of rivet and (*ii*) bearing strength of rivet.

Let τ_{vf} = permissible shearing stress in rivet.

σ_{pf} = allowable bearing stress in rivet.

∴ Shearing strength of rivet, $P_s = \tau_{vf} \cdot \frac{\pi}{4} d^2$ in single shear ...(3.5)

or $= \tau_{vf} \cdot 2 \frac{\pi}{4} d^2$ in double shear

Bearing strength of rivet, $P_b = \sigma_{pf} \times d \times t$ ...(3.6)

Strength of riveted joint and efficiency of joint

The strength of a riveted joint will be lesser of values given by Eqs 3.4, 3.5 and 3.6.

Strength of solid plate, $P = \sigma_{at} . p . t$ (per pitch length) ...(3.7 *a*)

The efficiency η of joint is defined as the ratio of the strength of the joint to the strength of the plate.

Thus, $$\eta = \frac{\text{Least of } P_t, P_s \text{ and } P_b}{P} \quad ...(3.7\ b)$$

It should be noted that Eqs. 3.4, 3.5 and 3.6 give the strength of a single riveted joint. However, if there are *n* rows of rivets per pitch length, we have

$$P_t = \sigma_{at} (p - d) t \quad ...(3.4\ a)$$

$$P_s = n . \tau_{vf} \frac{\pi}{4} d^2 \text{ in the single shear}$$

or $$= n\, \tau_{vf} . 2 \frac{\pi}{4} d^2 \text{ in the double shear} \quad ...(3.5\ a)$$

and $$P_b = n\, \sigma_{pf} . d . t \quad ...(3.6\ a)$$

3.9. DESIGN OF RIVETED JOINT : AXIAL LOAD

When the line of action of the load coincides with the centre of gravity of rivet areas, the design of connection is based on the *nominal stresses*, which must not exceed the allowable stresses. The following assumptions, which are nearly correct for load approaching the ultimate, are made in computing the nominal stresses.

Assumptions

1. Initial tensile stresses in the rivet is neglected.
2. The frictional resistance to slip between the plates is neglected.

3. The plates are rigid.
4. The rivet fills the hole completely.
5. Deformation of the plates under the load is neglected.
6. Shearing deformation of the rivets is assumed proportional to the shearing stress.
7. Shearing stress in the rivets is assumed to be uniformly distributed over the rivet cross-section.
8. Unit shearing stress in all the rivets of a joint is uniform.
9. Tensile stress concentration due to rivet holes in the plates are neglected.
10. Bearing stress between rivets and plates is assumed to be uniformly distributed over the nominal contact surface between the rivets and plates.
11. Bending of rivets is neglected.

1. Pitch of Rivets

The pitch of rivets in a riveted joint should be such that under a pull P_t, the permissible stress in the rivet for shearing and bearing are not exceeded.

From Eq. 3.4, $P_t = \sigma_{at}(p-d)\,t$

Strength of rivet = Rivet value $R = P_s$ or P_b which is less.

$\therefore$ $P_t \le R$

or $$\sigma_{at}(p-d)\,t \le R \qquad \text{...(3.8 } a\text{)}$$

From Eq. 3.7, *pitch* (p) can be determined. If Eq. 3.8 a is fulfilled, the strength of joint will be P_t, and will be lesser than (or at the most equal to) P_s or P_b. The efficiency of the joint designed in this manner will be given by

$$\eta = \frac{P_t}{P} = \frac{(p-d)\,t\,\sigma_{at}}{p\,t\,\sigma_{at}} = \frac{p-d}{p} \qquad \text{...(3.8 } b\text{)}$$

IS 800-1984 lays down the following specifications for the pitch of rivets.

(*a*) ***Minimum Pitch*** : The distance between centres of rivets should be not less than 2.5 times the nominal diameter of rivet.

(*b*) ***Maximum Pitch***

(*i*) The distance between centres of any two adjacent rivets (including tacking rivets) shall not exceed $32t$ or 300 mm, whichever is less, where t is the thickness of the thinner outside plate.

(*ii*) The distance between centres of two adjacent rivets, in a line lying in the direction of stress, shall not exceed $16t$ or 200 mm, whichever is less in tension members, and $12t$ or 200 mm, which ever is less in compression members. In the case of compression members in which forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the rivets for a distance from the abutting faces equal to 1.5 times the width of the member.

(*iii*) The distance between centres of any two consecutive rivets in a line adjacent and parallel to an edge of an outside plate shall not exceed (100 mm + $4t$) or 200 mm, which ever is less in compression or tension members.

(*iv*) When rivets are staggered at equal intervals and the gauge does not exceed 75 mm, the distances specified in (*ii*) and (*iii*) between centres of rivets, may be increased by 50 percent.

2. Edge Distance

(*a*) The minimum distance from the centre of any hole to the edge of a plate shall not be less than that given in Table 3.2.

(*b*) Where two or more parts are connected together, a line of rivets shall be provided at a distance of not more than 37 mm + 4 *t* from the nearest edge, where *t* is the thickness in mm of the thinner outside plate. In the case of work not exposed to weather, this may be increased to 12 *t*.

TABLE 3.2. EDGE DISTANCE OF HOLES

Nominal dia. of rivet (mm) (1)	*Dia. of hole (mm)* (2)	*Distance to sheared or hand flame cut hole (mm)* (3)	*Distance to Rolled, machine flame cut, sawn or planed hole (mm)* (4)
12 or below	13.5 or below	19	17
14	15.5	25	22
16	17.5	29	25
18	19.5	32	29
20	21.5	32	29
22	23.5	38	32
24	25.5	44	38
27	29.0	51	44
30	32.0	57	51
33	35.0	57	51

3. Diameter of Rivet

The diameter of rivet for a given plate thickness is generally chosen from the *Unwin's formula* :

$$d = 6.04\sqrt{t} \qquad ...(3.9)$$

where t = thickness of plate in mm and d = dia. of rivet in mm.

The diameter of the rivet so found is rounded off to the available size of rivets which are manufactured in nominal diameter of 12, 14, 16, 18, 20, 22, 24, 27, 30, 33, 36, 39, 42 and 48 mm. Commonly used sizes for structural steel works are 16, 18, 20 and 22 mm.

For computation of strength etc, the *formed diameter* (equal of the diameter of hole) is used.

4. Tacking rivets

Tacking rivets are generally used when a structural member consists of two angles (or such other sections), so as the two act as one unit. These rivets are also known as *stitch rivets*. As per IS : 800-1984, when the maximum distance between centres of two adjacent rivets exceeds the specified maximum pitch, taking rivets not subjected to calculated stress shall be used.

The tacking rivets shall have a pitch in line not exceeding 32 times the thickness of outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the pitch in line shall not exceed 16 times the thickness of outside plate or 200 mm, whichever is less. In both cases, the lines of rivets shall not be apart at a distance greater than these pitches. All these requirements shall apply to compression members generally, subjected to the stipulation in the code affecting the design and construction of compression members.

In tension members composed of two flats, angles, channels or tees in contact back-to-back or separated back-to-back by a distance not exceeding the aggregate thickness of the connected parts, with solid distance pieces where the parts are separated, shall be provided at pitch in line not exceeding 1000 mm.

For compression members, the tacking rivets shall be at a pitch in line not exceeding 600 mm.

5 Rivets through packings

Number of rivets carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2.5 percent for each 2.0 mm thickness of packing except that, for packing having a thickness of 6 mm or less, no increase need be made. For double shear connections packed on both sides, the number of additional rivets required shall be determined from the thickness of the thicker packing. The additional rivets should preferably be placed in an extension of the packing.

6 Long grip rivets

Where the grip of rivets carrying calculated load exceeds 6 times diameter of the holes, the number of rivets required by normal calculation shall be increased by not less than one percent for each additional 1.5 mm of grip; but the grip shall not exceed 8 times the diameter of the holes.

7. Countersunk heads of rivets

For countersunk heads, one half of the depth of the counter sinking shall be neglected in calculating the length of rivet in bearing. For rivets in tension with countersunk heads, the tensile value shall be reduced by 33.3 percent. No reduction need be made in shear.

8. Members meeting at a point

(*i*) For triangulated frames designed on the assumption of pin jointed connections, members meeting at a joint shall, where practicable, have their centroidal axes meeting at a point ; and wherever practicable the centre of resistance of a connection shall be on the line of action of the load so as to avoid an eccentricity or moment on the connections.

(*ii*) Where eccentricity of members or of connection is present, the members and the connections shall provide adequate resistance to the induced bending moments.

(*iii*) Where the design is based on non-intersecting members at a joint, all stresses arising from the eccentricity of the members shall be calculated and stress kept within the limits specified.

3.10. RIVETED JOINT IN FRAMED STRUCTURES

In the case of a roof truss, bridge truss or other similar framed structure, the load (P) to be carried by the member is always known. The number of rivets (n) required to connect the member to the other member is given by

$$n = \frac{P}{\text{strength of one rivet}} = \frac{P}{R} \qquad \text{...(3.10)}$$

where P is the pull or push carried by the member.

In the case of tension member carrying pull, the arrangement of rivets found above on each side of joint is of utmost importance, since it will directly determine the width of flat. Rivets in a joint may be arranged in two forms:

(*a*) Chain Riveting (Fig. 3.16 *a*) and

(*b*) Diamond Riveting (Fig. 3.16 *b*)

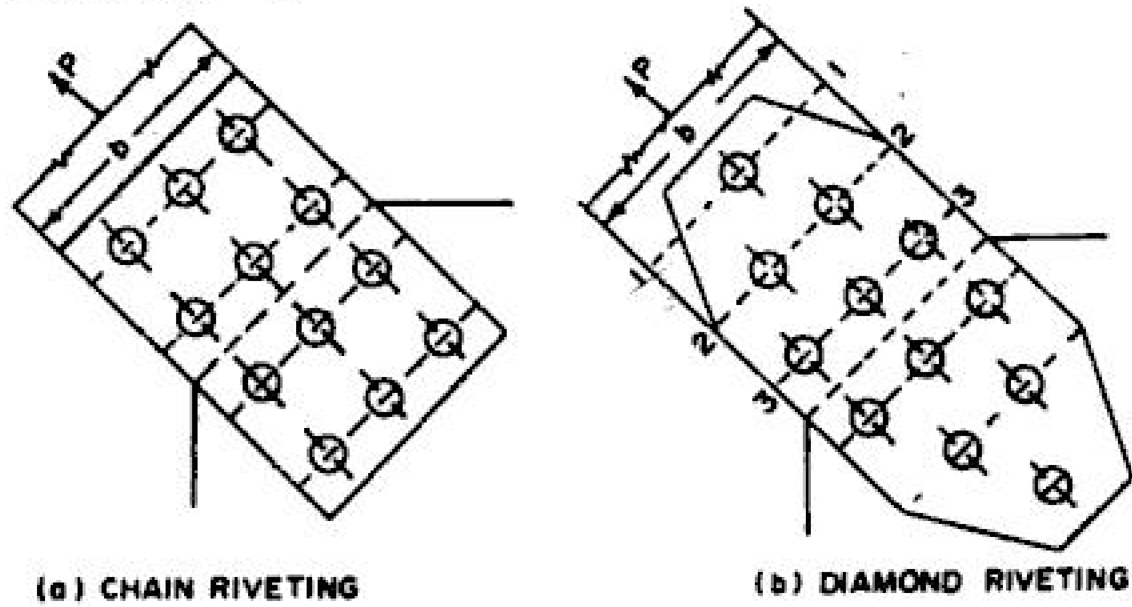


FIG. 3.16. ARRANGEMENT OF RIVETS.

In *chain riveting* shown in Fig. 3.16 (*a*) for a case when $n=6$, the flat is weakened by three rivet holes. The width b of the flat in this must be such that

$$P = \sigma_{at}(b - 3d)\,t \qquad \text{...(3.11 } a)$$

or

$$b = \frac{P}{t \cdot \sigma_{at}} + 3d \qquad \text{...(3.11)}$$

In the case of diamond riveting shown in Fig. 3.16 (*b*), the flat is weakened at the section 1-1 by one rivet hole only and the width of the flat is given by

$$P = \sigma_{at}(b - d)\,t$$

or

$$b = \frac{P}{t\,\sigma_{at}} + d \qquad \text{....(3.11 } b)$$

This width is *less* by $2d$ than the previous case. The saving in the width of flat by this amount is of utmost importance in the case of long bridge diagonals. The diamond riveting is, therefore used in preference to chain riveting. It can be shown that the joint get successively stronger at section 2-2, 3-3 etc.

The strength of joint at section 2-2 is

$$P_2 = \sigma_{at}(b - 2d)\,t + \text{(strength of one rivet in front)}$$

Similarly, the strength of the joint at section 3-3 is

$$P_3 = \sigma_{at}(b - 3d)\,t + \text{(strength of three rivets in front)}$$

Out of P, P_2 and P_3, usually P is the least. Therefore the efficiency of the joint is given by

$$\eta = \frac{(b-d)\,t\,\sigma_{at}}{b\,t\,\sigma_{at}} = \frac{b-d}{b} \qquad \text{...(3.12)}$$

3.11. RIVETED JOINTS IN CYLINDRICAL AND SPHERICAL SHELLS

In the case of *cylindrical shells*, hoop stress is $\frac{pD}{2t}$, where p is the internal pressure, D is the diameter of the shell and t is the thickness of the plate. If the shell is riveted longitudinally, the efficiency of the joint should also be taken into account while finding its thickness. Similarly, in the case of spherical shells, hoop stress is $\frac{pD}{4t}$. Hence

For cylindrical shells,

$$t = \frac{pD}{2\,\sigma_{at}\,\eta} \qquad \text{...(3.13)}$$

For spherical shells,

$$t = \frac{pD}{4\,\sigma_{at}\,\eta} \qquad \text{...(3.14)}$$

Example 3.1. *Determine the rivet value of 20 mm diameter rivets connecting 12 mm thick plates, if it is in (a) single shear (b) double shear. The permissible stress for rivet in shear and bearing are 80 N/mm² and 250 N/mm² and for plate in bearing is 250 N/mm².*

Solution:

Gross diameter of rivet, $d = 20 + 1.5 = 21.5$ mm

Gross Area of rivet $= \frac{\pi}{4}d^2 = \frac{\pi}{4}(21.5)^2 = 363.05\ \text{mm}^2$

Bearing strength of rivet, $P_b = \sigma_{pf} \times d \times t = 250 \times 21.5 \times 12 = 64500$ N ...(*i*)

Strength of rivet in single shear, $P_{s_1} = \tau_{vf} . \frac{\pi}{4} d^2 = 80 \times 363.05 = 29044$ N ...(*ii*)

Strength of rivet in double shear, $P_{s_2} = \tau_{vf} . 2 \times \frac{\pi}{4} d^2 = 2 \times 29044 = 58088$ N ...(*iii*)

Hence Rivet value in single shear = smaller of P_b and P_{s1}= **29044 N.**

Rivet value in double shear = smaller of P_b and P_{s2}= **58088 N**

Example 3.2. *A double riveted double cover butt joint in plates 16 mm thick is made with 20 mm rivets at 80 mm pitch. Calculate the pull per pitch length at which the joint will fail and also its efficiency. Take $f_t = 480\ N/mm^2$, $f_b = 760\ N/mm^2$ and $f_s = 380\ N/mm^2$.*

Solution

Gross diameter or formed diameter of rivet= 20 + 1.5 = 21.5 mm. For the tension failure in the plates,

$$P_{ut} = f_t\,(p-d)\,t = 480(80-21.5)16 = 449280 \text{ N} = 449.28 \text{ kN}$$

Since there are two rivets in one pitch length, and each rivet is in double shear, we have

$$P_{us} = 2\left(2\times\frac{\pi}{4}d^2\right) f_s = \pi(21.5)^2 \times 380 = 551836 \text{ N} \approx 551.84 \text{ kN}$$

Also, $$P_{ub} = 2d.t.f_b = 2\times21.5\times16\times760 = 522880 \text{ N} = 522.83 \text{ kN}$$

The joint will thus fail at pull of 449.28 kN, the plates giving way by tearing off.

∴ Strength of the joint = 449.28 kN.

Strength of plate $= P_u = p\,.\,t f_t = 80 \times 16 \times 480 = 614400$ N = 614.4 kN

$$\therefore \quad \eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} = \frac{449.28}{614.4} \times 100 = \mathbf{73.12\ \%}$$

Example 3.3. *A single riveted lap joint is used to connect 12 mm thick plates, by providing 20 mm diameter rivets at 50 mm pitch. Determine the strength of the joint and joint efficiency. Take working stress in shear in rivets = 80 N/mm², working stress in bearing in rivets= 250 N/mm² and working stress in axial tension in plates $= 0.6 f_y$ where $f_y = 260\ N/mm^2$.*

Solution: Formed diameter of rivets = 20 + 1.5 = 21.5 mm

$$\sigma_{at} = 0.6 f_y = 0.6 \times 260 = 156 \text{ N/mm}^2$$

Strength of plate in tension, per pitch length;

$$P_t = \sigma_{at}\,(p - d)\,t = 156(50 - 21.5) \times 12 = 53352 \text{ N} \quad ...(1)$$

Strength of rivet in single shear:

$$P_s = \tau_{vf} \times \frac{\pi}{4} d^2 = 80 \times \frac{\pi}{4}(21.5)^2 = 29044 \text{ N} \quad ...(2)$$

Strength rivet in bearing:

$$P_b = \sigma_{pf} \times d \times t = 250 \times 21.5 \times 12 = 64500 \text{ N} \quad ...(3)$$

∴ Strength of joint = minimum of the above three values = **29044N**

Strength of solid plate, $P = \sigma_{at} pt = 156 \times 50 \times 12 = 93600$ N ...(4)

$$\therefore \quad \text{Joint efficiency} = \frac{\text{Least of } P_t, P_s \text{ and } P_b}{P} = \frac{29044}{93600} \times 100 = \mathbf{31.03\%}$$

Example 3.4. *Determine the load which can be transmitted per pitch length of a double cover butt joint connected by 24 mm diameter shop rivets at 100 mm pitch. The thickness of main plates and cover plates are 16 mm and 12 mm respectively. Take allowable tensile strength of plates equal to 150 N/mm², allowable shear stress in rivets equal to 100 N/mm² and allowable stress in bearing for rivets equal to 300 N/mm². Also, determine the efficiency of the joint.*

Solution Given : $\sigma_{pf} = 300\ \text{N/mm}^2$, $\tau_{vf} = 100\ \text{N/mm}^2$ and $\sigma_{at} = 150\ \text{N/mm}^2$

Gross diameter of rivets = 24 + 1.5 = 25.5 mm

(*i*) strength of rivet in double shear :

$$P_s = \tau_{vf} \cdot 2\frac{\pi}{4}d^2 = 100 \times 2 \times \frac{\pi}{4}(25.5)^2 = 102141\ \text{N} \quad ...(i)$$

(*ii*) Strength of rivet in bearing : As the total thickness of cover plates is more than the thickness of the main plate, the strength of the rivet will be found for bearing on main plates.

$$P_b = \sigma_{pf} \times d \times t = 300 \times 25.5 \times 16 = 122400\ \text{N} \quad ...(ii)$$

(*iii*) Strength of plate, per pitch length:

$$P_t = \sigma_{at}(p - d)\,t = 150(100 - 25.5)\,16 = 178800 \quad ...(iii)$$

∴ Strength of joint per pitch length = minimum of (*i*), (*ii*) and (*iii*) = 102141 N

∴ Load which can be transmitted = **102141 N**

Strength of solid plate $= \sigma_{at} \cdot p \cdot t = 150 \times 100 \times 16 = 240000$ N

$$\therefore \quad \eta \text{ of joint} = \frac{102141}{240000} \times 100 = \mathbf{42.56\%}$$

Example 3.5. *Fig. 3.17 shows the joint of a boiler shell made of 12 mm thick plates using 20 mm dia. rivets at a pitch of 90 mm. The two cover plates of the butt joint are 8 mm thick, but are of unequal lengths. Determine the strength of the joint per pitch length and its efficiency. Take the following values of permissible stresses:* (*i*) $\sigma_{at} = 150\ N/mm^2$ (*ii*) $\sigma_{pf} = 250\ N/mm^2$ *and* (*iii*) $\tau_{vf} = 80\ N/mm^2$.

Solution:

Here we observe that rivets in row 1 are in single shear, and they bear against 8 mm plate. However, rivets in row 2 are in double shear, and they bear against 12 mm main plate because the total thickness of the two coverplates is more than that of the main plate.

Formed diameter of rivet

= 20 + 1.5 = 21.5 mm

(*a*) ***Rivet value in section 1-1***

Strength of rivet in single shear

$$= \tau_{vf} \cdot \frac{\pi}{4}d^2$$

$$= 80 \times \frac{\pi}{4}(21.5)^2 = 29044\ \text{N}$$

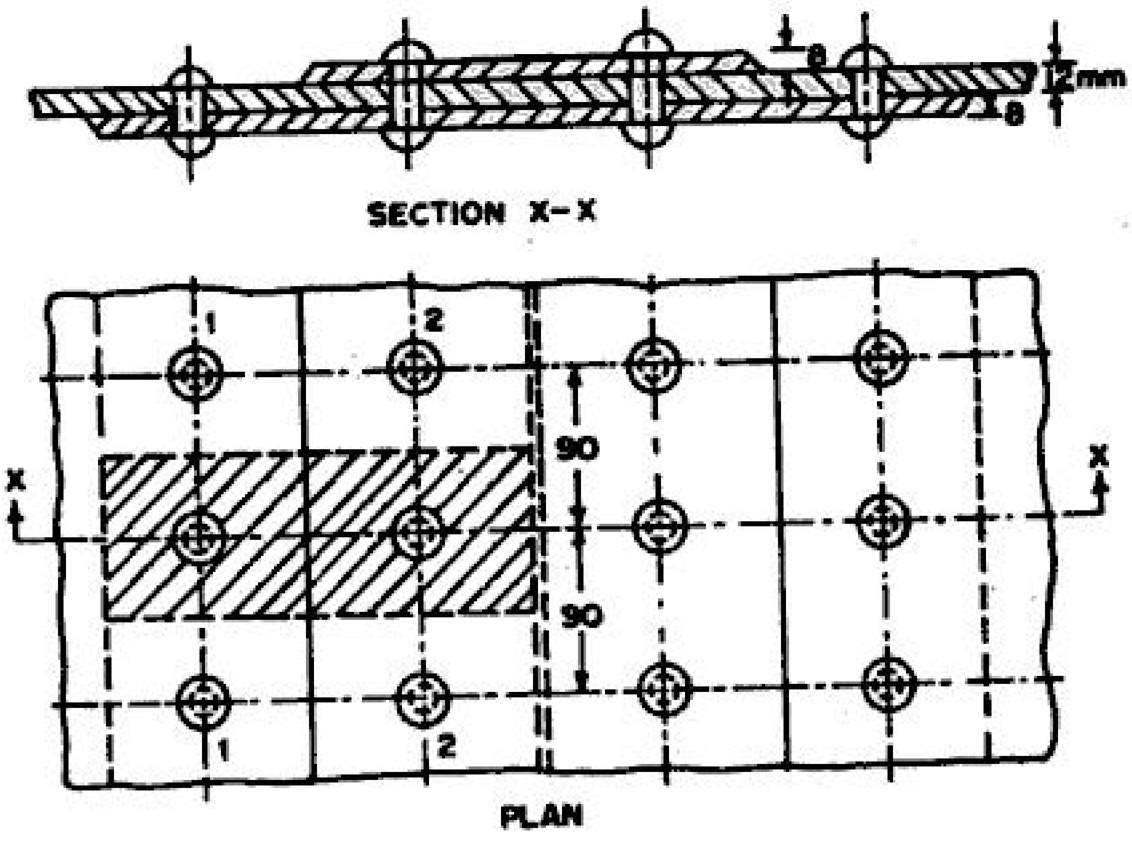


FIG. 3.17.

Strength of rivet in bearing on 8 mm plate $= \sigma_{pf} \times d \times t$

$= 250 \times 21.5 \times 8 = 43000$ N

$\therefore$ Rivet value in section 1-1 $= R_1 = 29044$ N

(b) Rivet value in section 2-2

Strength of rivet in double shear $= \tau_{vf} \times 2\frac{\pi}{4}d^2 = 2\times29044 = 58088$ N

Strength of rivet in bearing on 12 mm plate $= \sigma_{pf} \times d \times t$

$= 250 \times 21.5 \times 12 = 64500$

$\therefore$ Rivet value in section 2-2 $= R_2 = 58088$ N.

(c) Strength of joint

Consider one pitch length of the joint, as shown hatched in Fig. 3.17. We will consider three possible chances of failure : (*i*) failure of rivets at section 1-1 and 2-2 (*ii*) tearing of main plate at section 1-1 and (*iii*) tearing of cover plates at section 2-2.

(*i*) *Strength of joint on the basis of failure of rivets at section 1-1 and 2-2*

$= R_1 + R_2 = 29044 + 58088 = 87132$ N ...(*i*)

(*ii*) *Strength of main plate, at section 1-1*

Strength of the joint on the basis of strength of main plate

$= \sigma_{at}(p - d)\,t = 150(90 - 21.5) \times 12 = 123300$ N ...(*ii*)

(*iii*) *Strength of cover plates at section 2-2*

Strength of the joint on the basis of strength of cover plates at section 2-2

$= \sigma_{at}\,(p - d)\,t = 150(90 - 21.5)2 \times 8 = 164400$ ...(*iii*)

(*iv*) *Strength of main plate at section 2-2*

Strength of joint at 2-2 = Strength of main plate at 2-2 + strength of rivet at 1-1

$= \sigma_{at}\,(p - d)\,t + R_1 = 150(90 - 21.5)12 + 29044 = 152344$ N ...(*iv*)

$\therefore$ Strength of joint per pitch length = least of (*i*), (*ii*) (*iii*) and (*iv*) = 87132N

Strength of solid plate $= \sigma_{at}\,.\,p\,.\,t = 150 \times 90 \times 12 = 162000$ N

$$\therefore \quad \eta \text{ of joint} = \frac{\text{strength of joint}}{\text{strength of solid plate}} = \frac{87132}{162000} \times 100 = \mathbf{53.79\,\%}$$

Example 3.6. *Two plates 80 mm wide and 10 mm thick are joined with a triple riveted butt joint as shown in Fig. 3.18. The rivets at 1-1, 2-2 and 3-3 are of 18 mm, 20 mm and 22 mm diameter, while the two cover plates, each 6 mm thick, are of unequal length.*

Determine the strength of the joint and its efficiency, if the allowable values of stresses are: $\sigma_{at} = 150\,N/mm^2$, $\sigma_{pf} = 250\,N/mm^2$ and $\tau_{vf} = 80\,N/mm^2$.

Solution

Formed diameter of 18 mm rivet $= 18 + 1.5 = 19.5$ mm

Formed diameter of 20 mm rivet $= 20 + 1.5 = 21.5$ mm

Formed diameter of 22 mm rivet $= 22 + 1.5 = 23.5$ mm.

1. Rivet value of 18 mm rivet: The rivet is in single shear.

Strength of rivet in single shear $= \tau_{vf} \times \frac{\pi}{4}d^2 = 80 \times \frac{\pi}{4}(19.5)^2 = 23892$ N

Strength of rivet in bearing on 6 mm plate $= \sigma_{pf} \cdot d \cdot t$

$= 250 \times 19.5 \times 6 = 29250$ N

$\therefore$ Rivet value of 18 mm rivet $= R_1 = 23892$ N

2. Rivet value of 20 mm rivet: The rivet is in single shear.

Strength of rivet in single shear

$= \tau_{vf} \times \frac{\pi}{4} d^2 = 80 \times \frac{\pi}{4}(21.5)^2$

$= 29044$ N

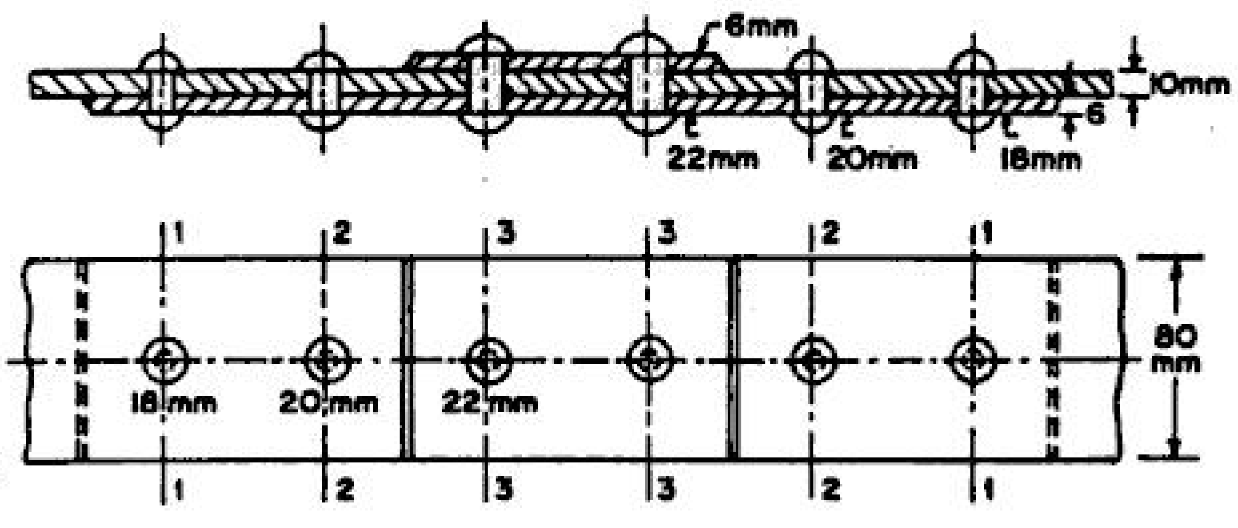


FIG. 3.18.

Strength of rivet in bearing on 6 mm plate $= \sigma_{pf} \cdot d \cdot t$

$= 250 \times 21.5 \times 6 = 32250$ N

$\therefore$ Rivet value of 20 mm rivet $= R_2 = 29044$ N

3. Rivet value of 22 mm rivet: The rivet is in double shear. Also, since the total thickness of cover plates is more than the main plate, the bearing strength of the rivet will be on the basis of 10 mm plate.

$\therefore$ Strength of rivet in double shear $= \tau_{vf} \times 2 \times \frac{\pi}{4} d^2$

$= 80 \times 2 \times \frac{\pi}{4}(23.5)^2 \approx 69398$ N

Strength of rivet in bearing on 10 mm plate $= \sigma_{pf} \cdot d \cdot t.$

$= 250 \times 23.5 \times 10 = 58750$ N

$\therefore$ Rivet value of 22 mm rivet $= R_3 = 58750$ N

(*a*) strength of joint on the basis of rivet values

$= R_1 + R_2 + R_3 = 23892 + 29044 + 58750 = 111686$ N ...(*i*)

(*b*) Strength of joint on the basis of failure of main plate at section 1-1

$= \sigma_{at}(p - d)\,t = 150(80 - 19.5) \times 10 = 90750$ N ...(*ii*)

(*c*) Strength of joint on the basis of failure of main plate at section 2-2

= tearing of main plate at section 2-2 + rivet value of rivet at 1-1

$= \sigma_{at}(p - d)\,t + R_1 = 150\,(80 - 21.5)10 + 23892 = 111642$ N ...(*iii*)

(*d*) Strength of joint on the basis of failure of main plate at section 3-3.

= tearing of main plate at section 3-3 + Rivet value of rivets at 2-2 + rivet value of rivets at 1-1 $= \sigma_{at}(p - d)t + R_2 + R_1$

$= 150(80 - 23.5)10 + 29044 + 23892 = 137686$ ...(*iv*)

(*e*) Strength of joint on the basis of failure of cover plates at 3-3

$= \sigma_{at}(p - d)t = 150(80 - 23.5)\,(6 + 6) = 101700$ N ...(*v*)

$\therefore$ *Strength of joint* = Least of (*i*), and (*ii*), (*iii*), (*iv*) and (*v*) = 90750 N

Strength of solid plate $= \sigma_{at} \cdot p \cdot t = 150 \times 80 \times 10 = 120000$ N

$\therefore$ η of joint $= \frac{90750}{120000} \times 100 =$ **75.6 %**

Example 3.7. *A double cover butt joint is used to connect plates of 12 mm thick. Design the riveted joint and determine its efficiency. Use power driven rivets and take permissible stresses as per IS : 800-1984. Take permissible axial tension in plate = $0.6 f_y$ where $f_y = 250\ N/mm^2$.*

Solution

The diameter of the rivet is found on the basis of Unwins formula:

$$d = 6.04\sqrt{t} = 6.04\sqrt{12} = 20.9 \text{ mm}$$

Since Unwin's formula gives slightly higher values, adopt nominal diameter of rivet = 20 mm.

∴ Gross diameter of rivet = 20 + 1.5 = 21.5 mm

The permissible stresses for shop rivets as per IS 800-1984 are as follows :

$$\tau_{vf} = 100 \text{ N/mm}^2;\ \sigma_{pf} = 300 \text{ N/mm}^2.$$

$$\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150 \text{ N/mm}^2$$

Let p be the pitch of the rivets.

Strength of rivet in double shear

$$P_s = \tau_{vf} \cdot 2\frac{\pi}{4}d^2 = 100 \times 2 \times \frac{\pi}{4}(21.5)^2 = 72610 \text{ N}$$

Strength of rivet in bearing on main plate

$$P_b = \sigma_{pf} \cdot d\, t = 300 \times 21.5 \times 12 = 77400 \text{ N}$$

∴ *Rivet value* = 72610 N ...(*i*)

Strength of plate per pitch length = $\sigma_{at}(p - d)t$

$= 150(p - 21.5)12 = 1800(p - 21.5)$ N ...(*ii*)

Equating this to the rivet value, we get

$$1800(p - 21.5) = 72610$$

From which $p = 61.8$ mm

Adopt $p = 60$ mm. Adopt thickness of each cover plates

$$= \frac{5}{8} \times \text{ thickness of main plate}$$

$$= \frac{5}{8} \times 12 = 7.5 \text{ m}$$

Keep thickness = 8 mm

$$\text{Efficiency of joint} = \frac{p - d}{p} \times 100 = \frac{60 - 21.5}{60} \times 100 = \mathbf{64.2\ \%}$$

Example 3.8. *Two plates 12 mm and 10 mm thick are joined by a triple riveted lap joint, in which the pitch of the central row of rivets is half the pitch of rivets in the outer rows. Design the joint and find its efficiency. Take $\sigma_{at} = 150\ N/mm^2$, $\tau_{vf} = 80\ N/mm^2$ and $\sigma_{pf} = 250\ N/mm^2$.*

Solution : Since the Unwin's formula always gives slightly higher value of diameter of the rivet, we will use smaller of the thickness of the two plates.

$$\therefore \quad d = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.1 \text{ mm.}$$

Use 20 mm rivets.

Gross diameter of rivets = 20 + 1.5 = 21.5 mm.

(*i*) *Rivet Value*

Strength of rivets in single shear

$= \tau_{vf} . \frac{\pi}{4} d^2$

$= 80 \times \frac{\pi}{4} (21.5)^2 = 29044$ N

Strength of rivets in bearing on 10 mm plate $= \sigma_{pf} . d . t$

$= 250 \times 21.5 \times 10 = 53750$ N

$\therefore$ Rivet value $R = 29044$ N ...(*i*)

(*ii*) Strength of thinner plate per pitch length, along section 1-1.

$= \sigma_{at} (p - d)t = 150(p - 21.5)10$

$= 1500(p - 21.5)$

$= 1500p - 32250$ ...(*ii*)

(*iii*) Strength of thinner plate per pitch length, along section 2-2

$= \sigma_{at} (p - 2d)t + R$

$= 150(p-2\times21.5)10+29044$

$=1500(p-43)+29044$ ...(*iii*)

$= 1500p - 35456$

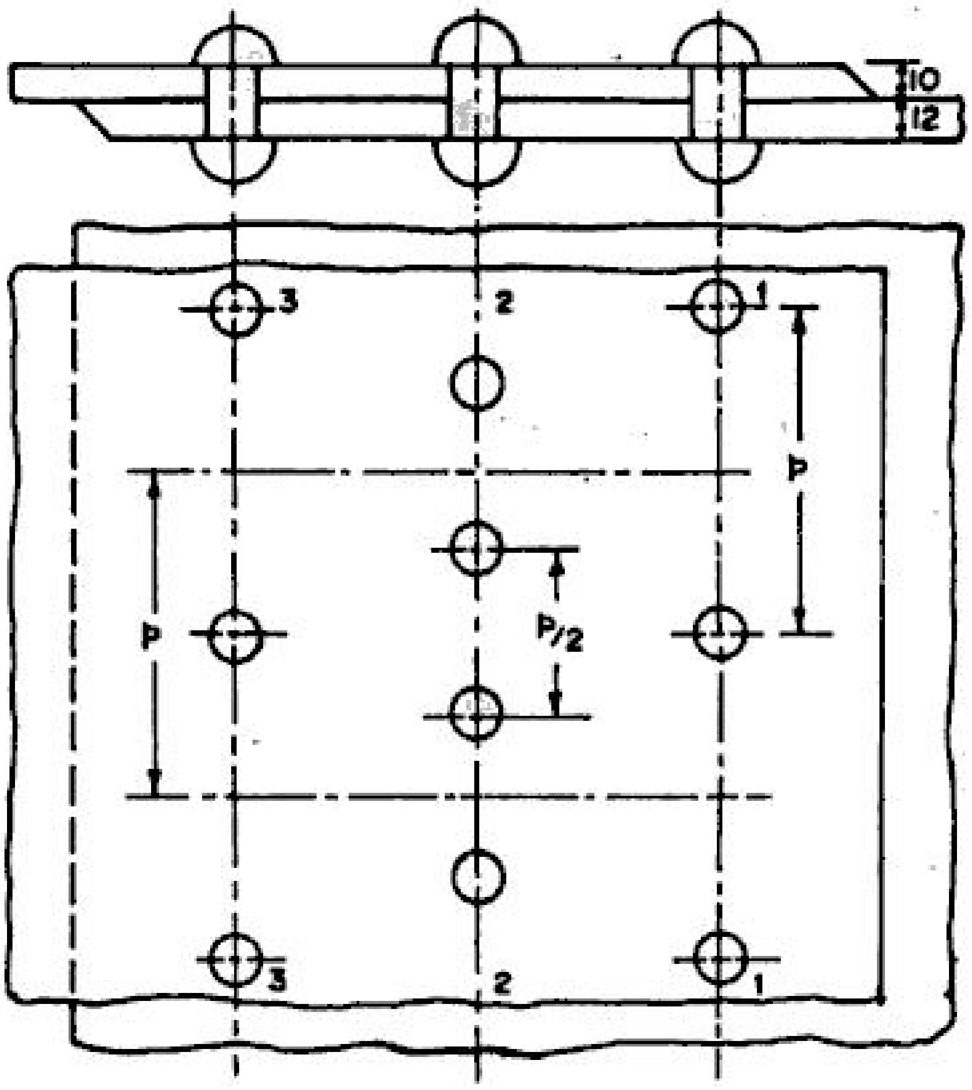


FIG. 3.19.

Hence section 2-2 is weaker, along which the strength of plate is $(1500p - 35456)$. For maximum joint efficiency, the strength of plate per pitch length should be equal to strength of rivets per pitch length.

$\therefore \quad 1500p - 35456 = 4R = 4 \times 29044$

From which, $p \approx 101$ mm

Minimum permissible pitch $= 2.5 \times 21.5 = 53.8$ mm

Maximum permissible pitch $= 32t = 32 \times 10 = 320$ mm

Keep pitch equal to 54 mm for the inner row and $2 \times 54 = 108$ mm for the outer rows.

$\therefore$ Strength of joint $= 1500p - 35456 = 1500\times108-35456= 126544$ N

or equal to $4R(= 4 \times 29044 = 116176$ N) whichever is less

$= 116176.$

Strength of solid plate $= \sigma_{at} p\, t = 150 \times 108 \times 10 = 162000$ N

$\therefore$ Efficiency of joint $= \frac{116176}{162000} \times 100 = $ **71.7%**

Example 3.9. *Plates 25 mm thick are connected by a treble riveted butt joint with two cover straps. The pitch of rivets in the outer row is twice the pitch of those in other rows and the diameter of rivets is 24 mm. Taking the resistance of rivets in double shear equal to 1.75 times their resistance in single shear, determine p for equal tearing and shearing resistance. Also determine the efficiency of the joint. Assume $\sigma_{at} = 90\ N/mm^2$ and $\tau_{vf} = 60\ N/mm^2$.* (U.P.S.C. 1967)

Solution

Formed diameter of rivets $= d = 24 + 1.5 = 25.5$ mm

Consider strip width of joint equal to pitch p.

Let us assume tearing along section 1-1.

Permissible load per pitch length

$= \sigma_{at}(p-d)\,t = 90(p-25.5)25$

$= 2250p - 57375$ N ...(1)

Again, assuming tearing along section 2-2 and shearing of rivets in section 1-1,

Permissible load per pitch length

$= \sigma_{at}(p-2d)t + \tau_{vf} \times 1.75 \times \frac{\pi}{4}d^2$

$= 90(p - 2 \times 25.5) \times 25 + 60 \times 1.75 \times \frac{\pi}{4}(25.5)^2$

$= 2250p - 61126$ ...(2)

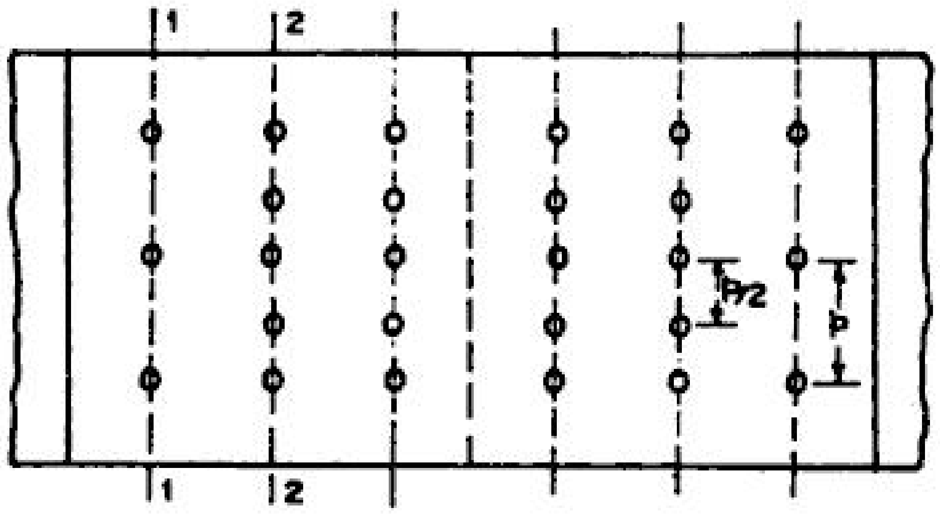


FIG. 3.20.

Equations (1) and (2) give two modes of failure due to tearing of plates , out of which Eq. (2) gives the smaller load. Hence permissible load, per pitch length, in tearing

$$= 2250p - 61126 \quad ...(a)$$

Let us now consider the mode of failure due to shearing of rivets. In the strip of joint equal to pitch p, there are $2 + 2 + 1 = 5$ rivets. Each rivet is in double shear.

Hence permissible load, per pitch length,

$$= 5 \times \tau_{vf} \times 1.75 \times \frac{\pi}{4}d^2 = 5 \times 60 \times 1.75 \times \frac{\pi}{4}(25.5)^2 = 268120 \text{ N} \quad ...(b)$$

Equating (*a*) and (*b*), we get

$2250p - 61126 = 268120$, from which, $p = \mathbf{146.3}$ mm

Permissible load which can be carried by solid plate

$$= \sigma_{at} . p . t = 90 \times 146.3 \times 25 = 329175 \text{ N}$$

$\therefore \quad \eta$ of the joint $= \frac{268120}{329175} \times 100 = \mathbf{81.5\%}$

Example 3.10. *A butt joint is used to connect two 20 mm thick plates as shown in Fig. 3.21. The diameter of rivets is 18 mm. Determine the efficiency of the joint. Take $f_t = 470\ N/mm^2$, $f_s = 375\ N/mm^2$ and $f_b = 750\ N/mm^2$.*

Solution

Formed diameter of the rivets $= 18 + 1.5 = 19.5$ mm

Each rivet is in double shear. Strength of each rivet in double shear

$= P_{us} = 2 \times \frac{\pi}{4}d^2 \times f_s$

$= 2 \times \frac{\pi}{4}(19.5)^2 \times 375$

$= 223986$ N

Strength of each rivet in bearing $= P_{ub} = f_b \times d \times t$

$= 750 \times 19.5 \times 20$

$= 292500$ N

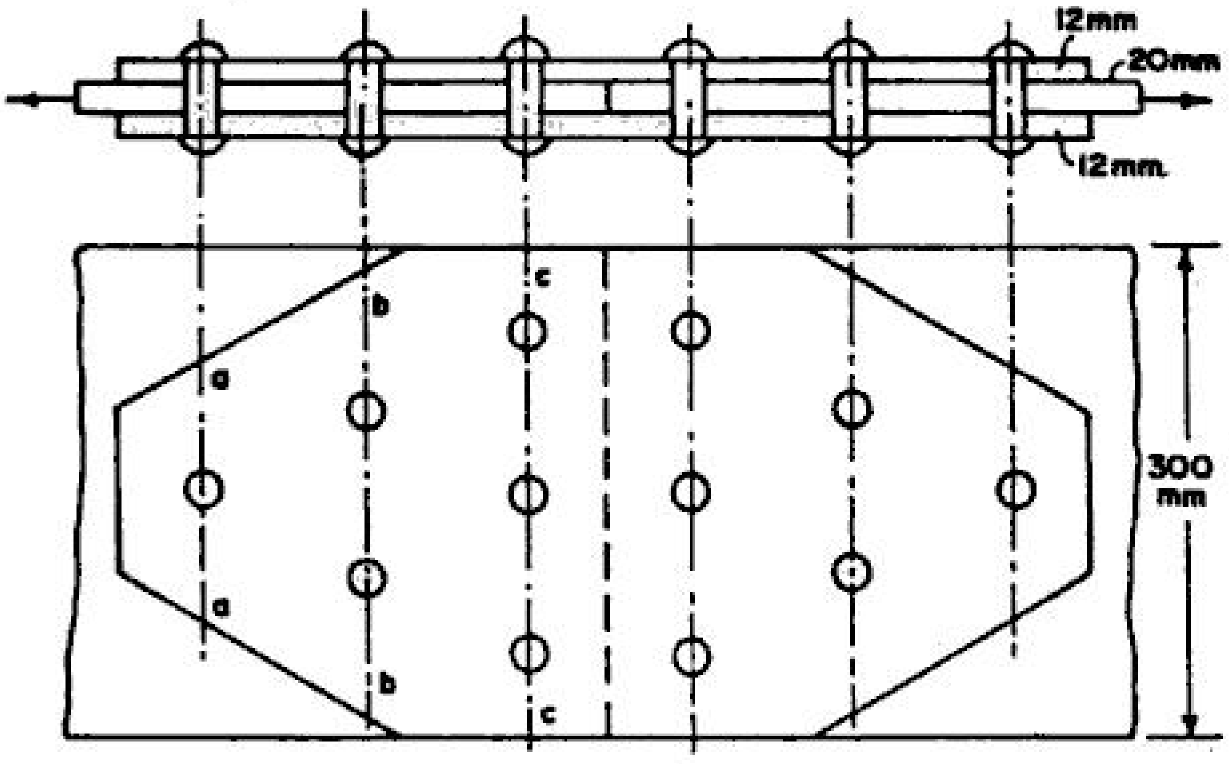


FIG. 3.21

∴ Rivet value $R = 223986$ N

Let us now investigate various modes of failure.

(*i*) Considering the tearing of main plate at *a-a*,

$$P_{u1} = (b - d)\,t.f_t = (300-19.5)20\times470 = 2.636\times10^6 \text{ N} \quad ...(i)$$

(*ii*) considering tearing of main plate at *b-b* and shearing of rivet at *a-a*,

$$P_{u2} = (b - 2d)\,t.f_t + P_{us}$$
$$= (300-2\times19.5)20\times470+223986 = 2.677\times10^6 \text{ N} \quad ...(ii)$$

(*iii*) considering tearing of main plate at *c-c* and shearing of rivets at *a* and *b*,

$$P_{u3} = (b - 3d)\,t f_t + 3P_{us}$$
$$= (300-3\times19.5)20\times470+3\times223986 = 2.942\times10^6 \text{ N} \quad ...(iii)$$

(*iv*) Considering failure by shearing of all the rivets,

$$P_{u4} = 6P_{us} = 6\times223986 = 1.344\times10^6 \text{ N} \quad ...(iv)$$

(*v*) Considering failure of bearing surfaces,

$$P_{u5} = 6\,P_{ub} = 6\times292500 = 1.755\times10^6 \text{ N} \quad ...(v)$$

(*vi*) Considering tearing of cover plates at *c-c*,

$$P_{u6} = (b - 3d)2t'.f_t = (300 - 3\times19.5)2\times12\times470$$
$$= 2.724\times10^6 \text{ N} \quad ...(vi)$$

The most likely mode of failure is (*iv*) in which the joint fails by shearing of the rivets, the strength of joint being $= 1.344\times10^6$ N.

Strength of solid plate $= b.t.f_t = 300\times20\times470 = 2.82\times10^6$ N

$$\therefore \quad \eta \text{ of joint} = \frac{1.344\times10^6}{2.82\times10^6}\times100 = \mathbf{47.7\%}$$

Example 3.11. *The diagonal of a bridge truss is made of 16 mm thick flat and has to transmit a pull of 600 kN. The diagonal is to be connected to 16 mm thick gusset plate by a double cover butt joint with 20 mm rivets.*

Calculate the number of rivets and width of flat required. Take permissible stresses as follows: $\sigma_{at} = 150\ N/mm^2$; $\tau_{vf} = 100\ N/mm^2$ *and* $\sigma_{pf} = 300\ N/mm^2$. *Sketch the joint and calculate the efficiency of the joint. Also determine (i) the actual stresses induced in the flat and the rivets; and (ii) thickness of cover plates.*

Solution

Formed diameter of rivet $= 20 + 1.5 = 21.5$ mm.

The rivets are in double shear. Strength of rivet in double shear is:

$$P_s = 2\times100\times\frac{\pi}{4}(21.5)^2 = 72610 \text{ N}$$

Strength of rivet in bearing against main plate,

$$P_b = 21.5\times16\times300 = 103200 \text{ N}$$

∴ Strength of rivet $= R = 72610$ N

$$\therefore \quad \text{Number of rivets required} = \frac{600\times1000}{72610} = 8.26$$

Hence provide 9 rivets to each side, and arrange them in diamond riveting pattern, shown Fig. 3.22.

Let the width of the flat $= b$. Assuming the section to be weakened by one rivet hole only,

$$P_t = (b - d)\, t\, \sigma_{at}$$
$$= (b - 21.5)\, 16 \times 150$$

Equating this to external load, we get

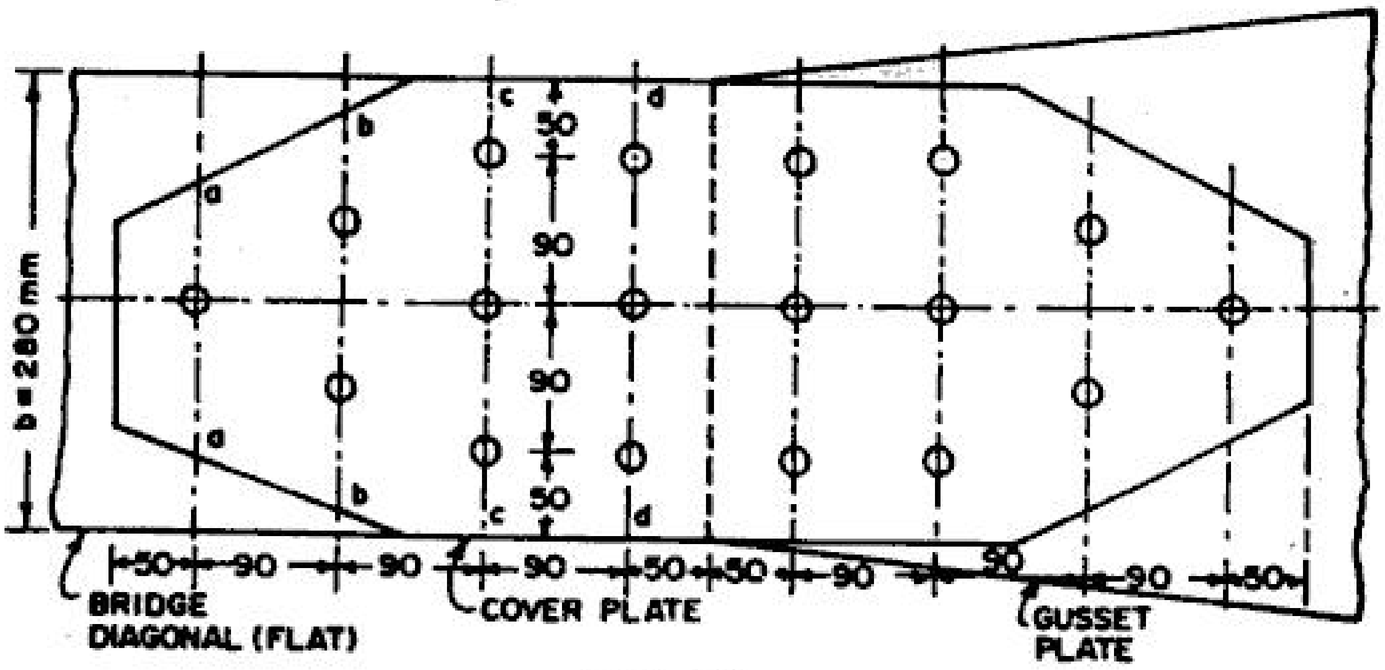


FIG. 3.22.

$$(b - 21.5)16 \times 150 = 600 \times 10^3$$

From which, $b = 271.5$ mm

Keep $b = 280$ mm

$$\therefore \quad \eta = \frac{b-d}{b} = \frac{280 - 21.5}{280} \times 100 = \mathbf{92.3\%}$$

Provide pitch and back pitch $= 90$ mm. Provide edge distance $= 50$ mm

Actual stress in rivets

$$f_s = \frac{600 \times 1000}{2 \times \frac{\pi}{4}(21.5)^2 \times 9} = 91.81 \text{ N/mm}^2$$

$$f_b = \frac{600 \times 1000}{21.5 \times 16 \times 9} = 193.8 \text{ N/mm}^2$$

Actual stresses in flat

At section $a - a$, $\quad P = f_1\,(b - d)\, t$

$$\therefore \quad f_1 = \frac{600000}{(280 - 21.5)\, 16} = 145 \text{ N/mm}^2$$

At section $b - b$, $\quad P = f_2(b - 2d)\, t + \frac{P}{n}$

$$\therefore \quad 600000 = f_2(280 - 2 \times 21.5) \times 16 + \frac{600000}{9}$$

from which $\quad f_2 = 140.6 \text{ N/mm}^2.$

At section $c-c$, $\quad P = f_3(b - 3d)t + \frac{3P}{n}$

$$\therefore \quad 600000 = f_3\,(280 - 3 \times 21.5)\, 16 + \frac{3 \times 600000}{9}$$

$$\therefore \quad f_3 = 116 \text{ N/mm}^2$$

At section d-d, $\quad P = f_4\,(b - 3d)\, t + \frac{6P}{n}$

$$\therefore \quad 600000 = f_4(280 - 3 \times 21.5)16 + \frac{6 \times 600000}{9}$$

$$\therefore \quad f_4 = 58 \text{ N/mm}^2$$

Thickness of cover plates

Let the thickness of each cover plate be t'.

$\therefore$ Strength of cover plates at section $d-d$, against tearing

$$= (b-3d)2t'\,\sigma_{at} = (280-3\times21.5)\times2\,t'\times150 = 64650\,t'$$

$\therefore$ $64650\,t' = 600\times10^3$

Form which $t' = 9.28$ mm ;

Also, $t' = \frac{5}{8}t = \frac{5}{8}\times16 = 10$ mm

Hence keep $t' = \mathbf{10}$ **mm.**

Actual stress in cover plate, at $d-d = \dfrac{600\times1000}{(280-3\times21.5)\times2\times10} = \mathbf{139.2\ N/mm^2}$

Example 3.12. *A lower chord of truss has a vertical member AB and a diagonal member AC meeting at a point A in it, as shown in Fig. 3.23 along with the axial forces. Design the joint, using hand driven rivets, taking permissible tensile stress in the angles as $0.6f_y$ where $f_y = 250\ N/mm^2$.*

Solution Assume 12 mm thick gusset plate. Minimum thickness of angles = 10 mm. The diameter of the rivet is found on the basis of Unwin's formula:

$$D = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.1 \text{ mm}$$

Use 20 mm rivets. Gross dia. of rivets

$$= 20 + 1.5 = 21.5 \text{ mm}$$

Minimum pitch $= 2.5D = 2.5\times21.5 = 53.75$ mm.

Provide pitch $= 60$ mm.

For hand driven rivets, we have

$$\tau_{vf} = 80\ N/mm^2\ ;\ \sigma_{pf} = 250\ N/mm^2$$

$$\sigma_{at} = 0.6f_y = 0.6\times250 = 150\ N/mm^2$$

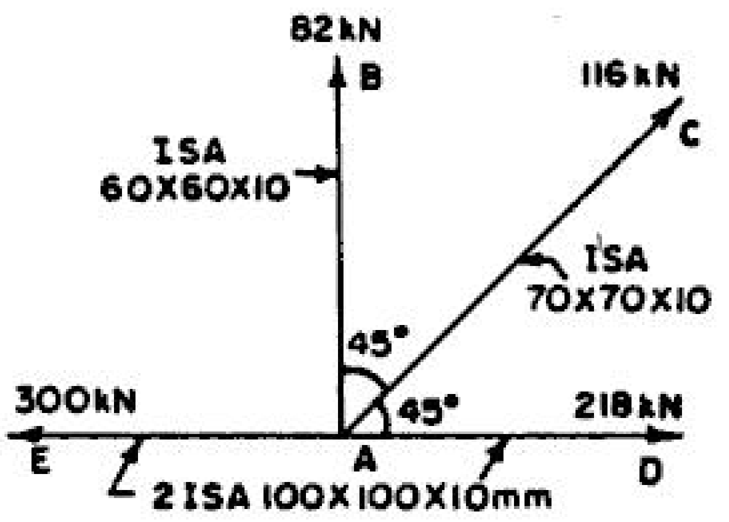


FIG. 3.23.

(a) Member AB

(*i*) Strength of rivet in single shear $= P_{us} = \frac{\pi}{4}d^2\,.\,\tau_{vf}$

$$= \frac{\pi}{4}(21.5)^2\,80 = 29044 \text{ N}$$

(*ii*) Strength of rivets in bearing on 10 mm thick angle

$$= P_{ub} = \sigma_{pf}\,.\,d\,.\,t = 250\times21.5\times10 = 53750 \text{ N}$$

(*iii*) Strength of angle per pitch length $= P_{ut}$

$$= \sigma_{at}(p-d)t = 150(60-21.5)10 = 57750 \text{ N}$$

$\therefore$ Rivet value $R = 29044$ N

No. of rivets required $= \dfrac{\text{Force in member } AB}{\text{Rivet Value}} = \dfrac{82\times10^3}{29044} = 2.83$

Provide 3 rivets.

(b) Member AC

(*i*) Strength of rivet in single shear $= P_{us} = \frac{\pi}{4}d^2\,\tau_{vf}$

$= \frac{\pi}{4}(21.5)^2 80 = 29044$ N

(*ii*) Strength of rivet in bearing on 10 mm thick angle$= P_{ub}$

$= \sigma_{pf} d \,.\, t = 250 \times 21.5 \times 10 = 53750$ N

(*iii*) Strength of thinner angle per pitch length

$= P_{ut} = \sigma_{at}(p - d)t = 150(60 - 21.5)10 = 57750$ N

Rivet value $R = 29044$ N

No. of rivets required $= \frac{\text{Force in member } AC}{\text{Rivet Value}} = \frac{116 \times 10^3}{29044} = 3.99$

Provide 4 rivets.

(c) Member ED

Net force in member $ED = 300 - 218 = 82$ kN

(*i*) Strength of rivet in double shear$= P_{us}$

$= 2 \times \frac{\pi}{4} d^2 \tau_{vf}$

$= 2 \times \frac{\pi}{4}(21.5)^2 80 = 38088$ N

(*ii*) Strength of rivet in bearing on 12 mm thick gusset plate$= P_{ub}$

$= \sigma_{pf} d.t = 250 \times 21.5 \times 12 = 64500$ N

(*iii*) Strength of angle per pitch length $= P_{ut}$

$= \sigma_{at}(p - d)\, t$

$= 150\,(60 - 21.5)10 = 57750$ N.

Rivet value $= R = 38088$ N.

No. of rivets required$= \frac{82 \times 10^3}{38088} = 2.15$

Provide 3 rivets.

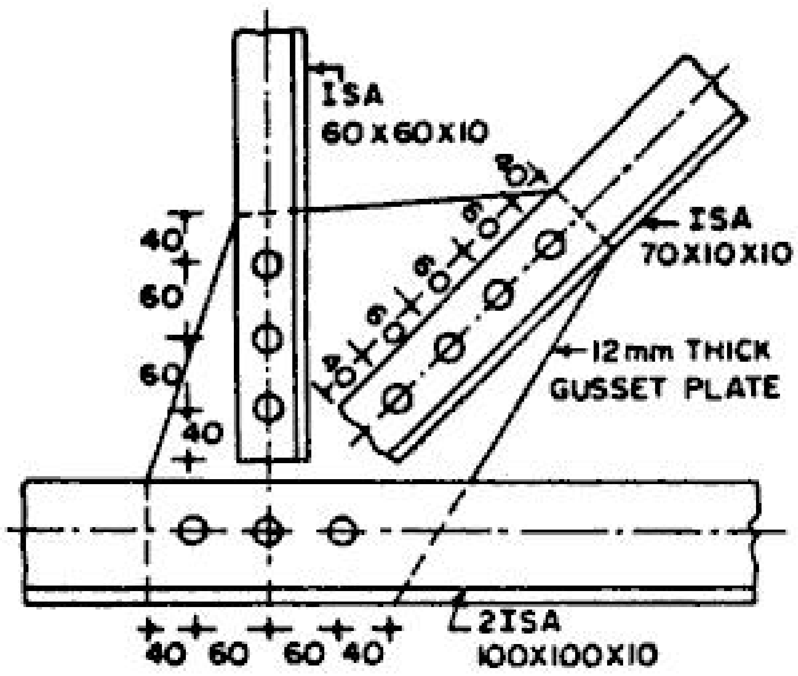


FIG. 3.24.

3.12. RIVETED JOINT SUBJECTED TO MOMENT ACTING IN THE PLANE OF THE JOINT

Sometimes, a riveted joint may be subjected to a load which may not pass through the C.G. of the rivet group. Such a connection is known as *eccentric load connection*. There may be two types of such connections :

1. When the line of action of the load is in the plane of the group of rivets, but is away from the C.G. of rivets (Fig. 3.25 *a*), or when the joint is subjected to a *pure moment* acting in the plane of the joint, and

2. When the line of action of the load does not lie in the plane of the group of rivets (Fig. 3.28).

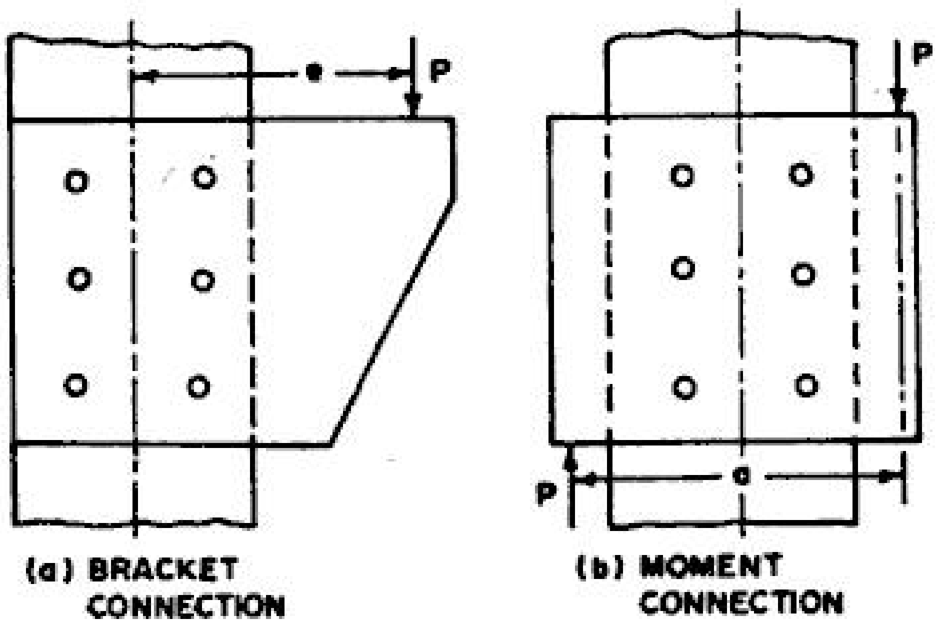


FIG. 3.25. ECCENTRIC LOAD AND MOMENT CONNECTION

Fig. 3.25 (*a*) shows a bracket connection in which the load P acts at an eccentricity

e from the C.G. of the rivet group, both being in the same plane. Fig. 3.25 (b) shows a riveted joint subjected to a pure moment M in the plane of the joint.

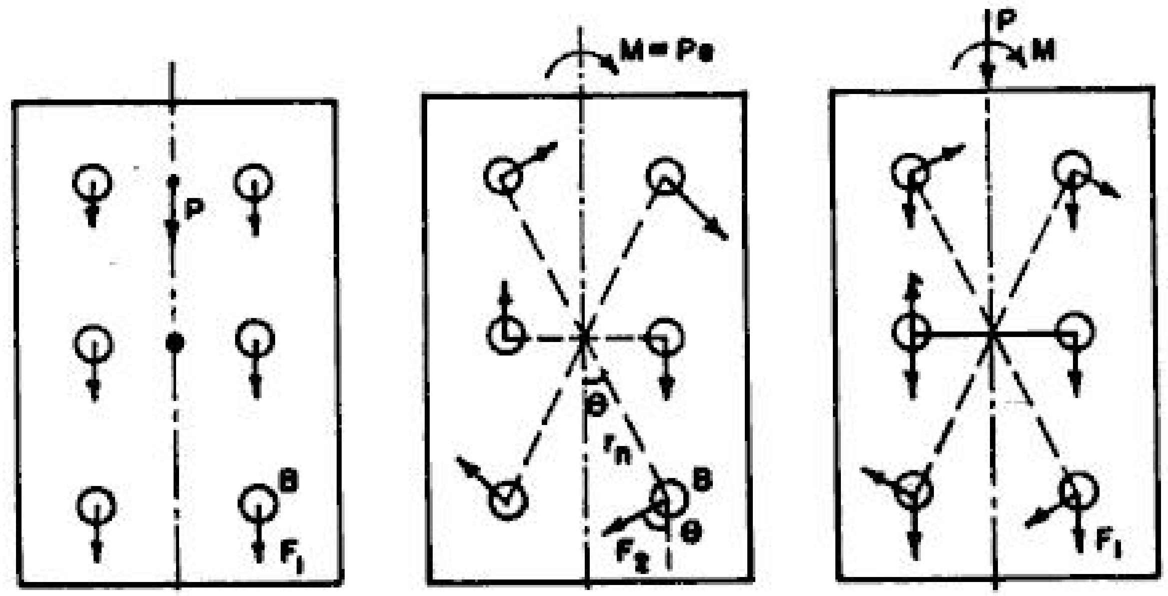
In the connection of Fig. 3.25 (a), the rivets group is subjected to (i) a direct load P passing through the centroid of the group and (ii) a moment $M = P.e$. The two effects are shown separately in Fig. 3.26.

Due to direct load P, a shear stress f_1 will be induced in such a way that

$$f_1 = \frac{P}{\Sigma A}$$

where ΣA is the sum of the rivet area. If A is the area of cross-section of any particular rivet, the force in the rivet due to direct load is given by

$$F_1 = f_1 . A = \frac{P.A}{\Sigma A} \quad ...(3.15\ a)$$



(a) DIRECT FORCE (b) MOMENT (c) COMBINED

FIG. 3.26

If all the rivets are of equal area, the above equation reduces to :

$$F_1 = \frac{P.A}{nA} = \frac{P}{n} \quad ...(3.15)$$

where n = total number of the rivets.

The direction of this force will be vertically downwards, acting through the centre of each rivet, as shown in Fig. 3.26 (a).

Due to moment $M (= P.e)$, each rivet will be subjected to torsional shear stress f_2 (Fig. 3.26 b). In order to find stress f_2 due to the moment (or couple), it is assumed that (i) rivets are perfectly elastic, and (ii) the unit stress on each rivet is proportional to its radius vector and acts in a direction perpendicular to the radius vector.

Thus, $f_2 \propto r$ where r is the distance of any rivet from the C.G. of the group.

$$\therefore \quad f_2 = k.r \quad ...(1)$$

If F_2 is the force in the rivet, we have

$$F_2 = Af_2 = A\,k\,r \quad ...(2)$$

The moment of resistance of the rivet is $= F_2 . r = A\,k\,r^2$

Total moment of resistance of the rivet group $= \Sigma A\,k\,r^2 = k\,\Sigma A\,r^2$

Equating this to the external moment,

$$k\,\Sigma A\,r^2 = P.e$$

or

$$k = \frac{P.e}{\Sigma A.r^2} \quad ...(3)$$

Substituting the value of k in (1) and (2), we get

$$f_2 = \frac{P.e\,r}{\Sigma A\,r^2}$$

and

$$F_2 = \frac{P.e.A.r}{\Sigma A.r^2} = \frac{M.A.r}{\Sigma A\,r^2} \quad ...3.16\ (a)$$

If all the rivets are of equal area,

$$F_2 = \frac{P.e.r}{\Sigma r^2} = \frac{M.r}{\Sigma r^2} \quad ...(3.16)$$

In order to find the resultant force R on a rivet, it must be noted that F_1 acts in the direction of P while F_2 acts in a direction perpendicular to the radius vector. The maximum value of R in a rivet group will be for the rivet in which F_1 and the resolved part of F_2 in the direction of F_1 are additive. If F_2 makes angle θ with the direction of F_1, we have

$$R = \sqrt{(F_2 \cos\theta + F_1)^2 + (F_2 \sin\theta)^2}$$

or

$$R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta} \quad ...(3.17)$$

The resultant force R in the heavily loaded rivet or any other rivet can also be found graphically by parallelogram of forces.

If, in the place of eccentric load, a pure moment M acts on the rivet group, the load carried by any rivet will be given by

$$R = \frac{M.r.A}{\Sigma A r^2} \text{ in general} \quad ...(3.18\ a)$$

or

$$R = \frac{M.r}{\Sigma r^2} \quad ...(3.18)$$

for the particular case when all the rivets have the same area. The direction of R in this case will be perpendicular to the radius vector.

Design of the bracket connection

In the analysis problem, if the number of rivets in a bracket connection (Fig. 3.25 *a*) are known, then the load P, acting at a given eccentricity, can be easily found by using Eqs. 3.15, 3.16 and 3.17. However, in a design problem, the number of the rivets has to be *assumed*, to start with, for a given load P acting at a given eccentricity, or for a given moment. This can be done as follows, for a connection having large number of rivets.

Let there be l number of rivet lines, with n number of rivets in each line, having pitch p, as shown in Fig. 3.27 (*a*). Fig. 3.27 (*b*) shows the stress diagram for one rivet line, wherein the stress in each rivet is proportional to its distance from the neutral axis.

Let f' be the stress in the outermost rivet, and R is the rivet value. We have

$$f' = \frac{R}{p} \quad ...(1)$$

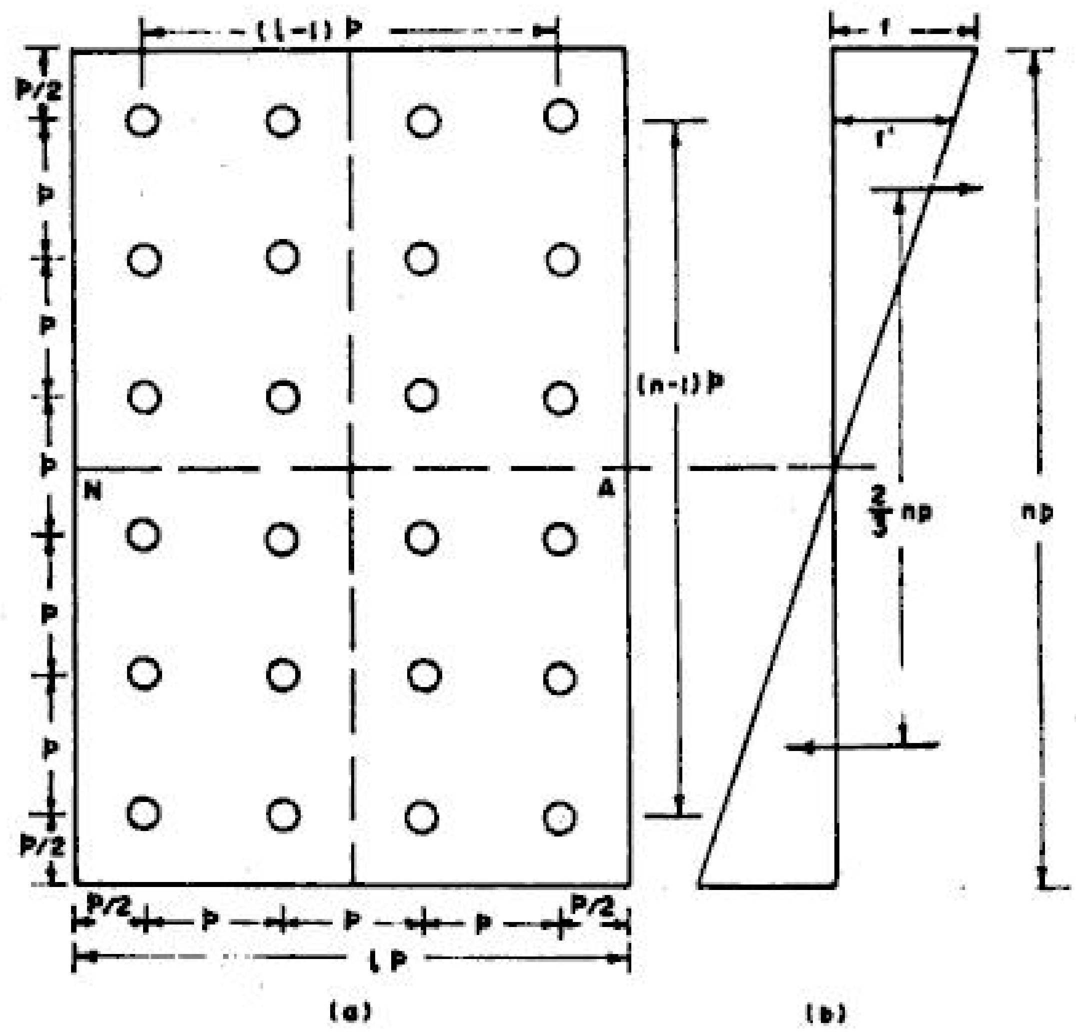


FIG. 3.27.

Hence the maximum stress f of an equivalent rectangular beam is

$$f = f' \cdot \frac{np}{(n-1)p} = \frac{R}{p} \cdot \frac{n}{n-1} \quad \text{...(2)}$$

Let M = total moment to be resisted $= P.e$

Moment shared by one rivet line $= \frac{M}{l}$

Section modulus $= \frac{1}{6}(np)^2$

$\therefore$ Fibre stress

$$f = \frac{M/l}{\frac{1}{6}(np)^2} \quad \text{...(3)}$$

Equating (2) and (3), we get

$$\frac{R}{p} \cdot \frac{n}{n-1} = \frac{M/l}{\frac{1}{6}(np)^2}$$

or

$$R = \frac{6M.p}{l\,(np)^2}\left(\frac{n-1}{n}\right) \quad \text{...(3.19)}$$

or

$$n = \sqrt{\frac{6M}{l\,pR}\left(\frac{n-1}{n}\right)} \quad \text{...3.20 (a)}$$

If the value of n is large (say more than 6), the factor $\left(\frac{n-1}{n}\right)$ can be taken approximately equal to unity. Then

$$n = \sqrt{\frac{6M}{lpR}} \quad \text{...(3.20)}$$

3.13. RIVETED JOINT SUBJECTED TO MOMENT ACTING PERPENDICULAR TO THE PLANE OF JOINT

Fig. 3.28 (*a*) shows a bracket connection in which the moment (= *eP*) is acting in a plane perpendicular to the plane of joint. In such a case, each rivet is subjected to *tension* in addition of *direct shear*. The external load *P* tries to *rotate* the connection about a *neutral axis*, the location of which depends upon the initial tension, if any, in the rivets.

Case 1 : Initial tension in rivets : If hot driven rivets are used, they will have initial tension when they cool and compress the plates together.

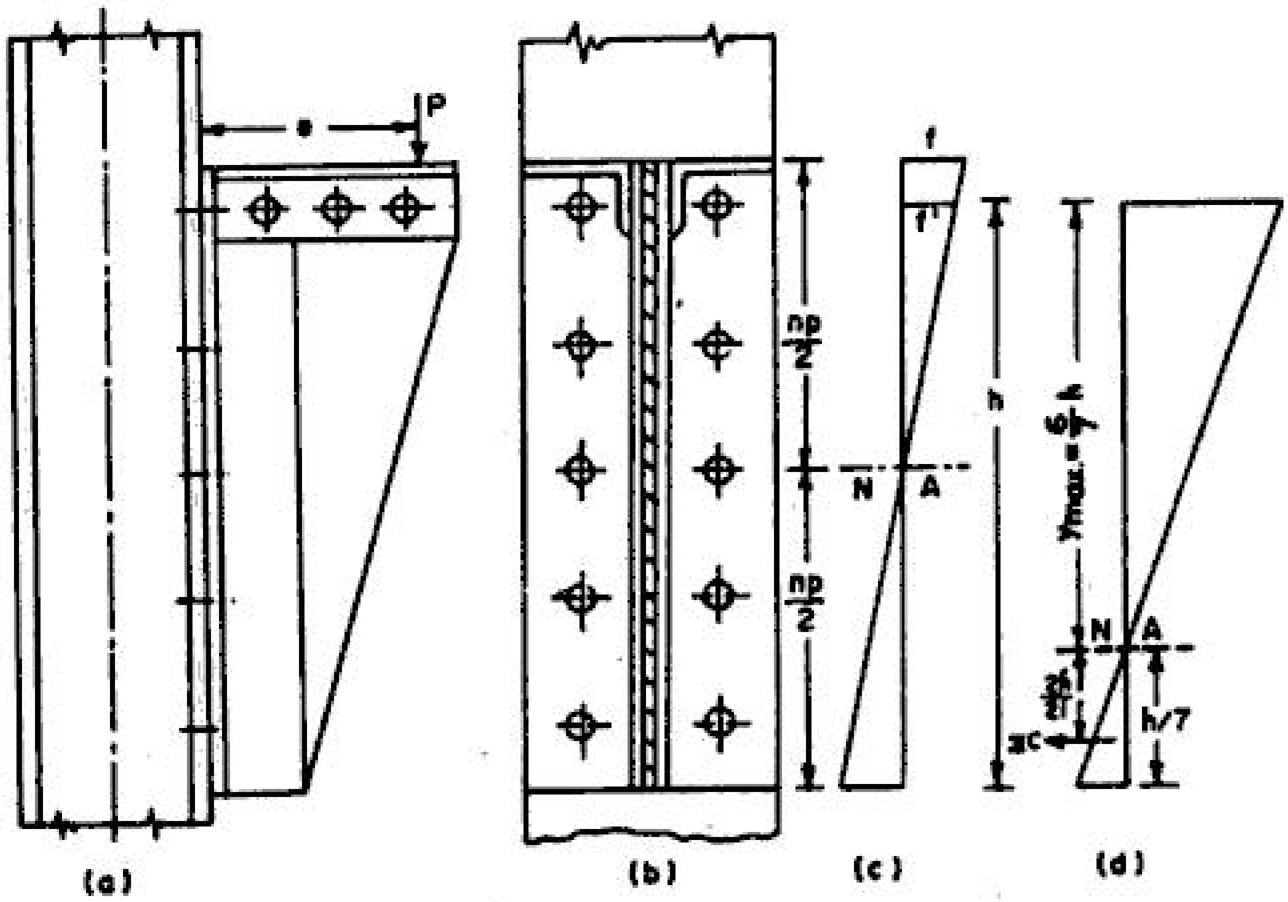


FIG. 3.28.

In that case, the N.A will lie at the mid-height h of the strip, as shown in Fig. 3.28 (*c*), where $h = np$ for a connection having n number of rivets in each line, provided at a pitch p. Let there be l number of rivets lines.

$$\text{Stress} \quad f = \frac{M}{I} y = \frac{M}{\frac{1}{12}(l \,.\, b)\,(np)^3} \times \frac{np}{2}$$

or
$$f = \frac{6M}{l\,b\,(np)^2} \qquad ....(1)$$

Hence effective tensile force T in the extreme rivet is

$$T = f \,.\, b \,.\, p = \frac{6M \,.\, bp}{l \,.\, b\,(np)^2}$$

or
$$T = \frac{6M}{l\,p\,n^2} \qquad ...(2) \qquad ...(3.21)$$

The maximum value of T should be equal to rivet value R. Hence from Eq. 3.21, putting $T = R$, we get

$$n = \sqrt{\frac{6M}{l\,p\,R}} \qquad ...(3.22)$$

From the above equation, number of rivets can be computed. Again, calculated tensile stress in the extreme rivet is given by

$$\sigma_{tf,\,cal} = \frac{T}{A} = \frac{6M}{l\,p\,n^2 A} \qquad ...(3.23)$$

where A is the effective area of cross section of the extreme rivet. It may be noted that Eq. 3.22 is similar to Eq. 3.20.

Case 2 : No initial tension in rivets: If there is no initial tension in the rivets, as is generally the case in cold driven rivets, the N.A. does not pass through the C.G. of rivet groups. But instead, it passes through a point which lies much *lower* than the C.G of rivet groups. According to British practice, the N.A passes through the centre of bottom most rivet. In the American practice, however, the line of rotation (*i.e.* the N.A.) is assumed at a distance of $\frac{1}{7}$th the *effective depth* from the bottom of the bracket. The *effective depth (h) is the depth from the centre line of the topmost rivet to the bottom of the bracket*, as shown in Fig.. 3.28 (*d*). We will follow this commonly accepted practice.

The rivets which lie above line of rotation will be in *tension*, in addition to *direct shear*, while those which lie below the line of rotation will be in compression and direct shear.

The tensile force T in any rivet above the line of rotation will be proportional to its distance y from the line of rotation.

$$\therefore \qquad T \propto y \quad \text{or} \quad T = ky \qquad ...(i)$$

The moment of resistance due to this tensile force is given by

$$\delta M = T.y = k\,.\,y^2 \qquad ...(ii)$$

Hence the total moment of resistance provided by the rivets in tension is

$$M_t = \Sigma\, k\,y^2 = k\,\Sigma\, y^2$$

or
$$M_t = \frac{T}{y}\Sigma\, y^2 \qquad ...(iii)$$

Hence
$$T = \frac{M_t y}{\Sigma y^2} \qquad ...(3.24)$$

$$\text{Total tensile force } \Sigma T = \frac{M_t \Sigma y}{\Sigma y^2}$$

For equilibrium, total tensile force must be equal to total compressive force ΣC

$$\therefore \qquad \Sigma C = \frac{M_t \Sigma y}{\Sigma y^2}$$

This compressive force acts at a distance of $\frac{2}{3}\left(\frac{h}{7}\right) = \frac{2h}{21}$ from the N.A. Hence taking moment about the N.A., we get

External moment = Moment resisted by rivets in tension + moment resisted by rivets and T section in compression

$$\therefore \qquad M = M_t + \Sigma C \cdot \frac{2h}{21}$$

or
$$M = M_t + \frac{M_t \Sigma y}{\Sigma y^2} \cdot \frac{2h}{21}$$

Hence
$$M_t = \frac{M}{1 + \frac{2h}{21} \cdot \frac{\Sigma y}{\Sigma y^2}} \qquad ...(3.25)$$

Thus, the moment resisted by rivets in tension is known. From this, the tensile force T_{max} in the extreme rivet can be found from Eq. 3.24 :

$$T_{max} = \frac{M_t \cdot y_{max}}{\Sigma y^2} \qquad ...(3.24\ a)$$

where $y_{max} = \frac{6}{7} h$.

Hence the tensile stress in the extreme rivet is given by $\sigma_{tf,cal} = \frac{T_{max}}{A} = \frac{M_t \cdot y_{max}}{A \Sigma y^2}$ ...(3.26)

Direct shear stress

In both the cases, the direct S.F. in any rivet is given by $F = \frac{P}{n}$

Hence direct shear stress in extreme rivet is

$$\tau_{vf,cal} = \frac{F}{A} = \frac{P}{nA} \qquad ...(3.27)$$

Interaction Equation

Thus the extreme rivet, in both the cases, is subjected to axial tensile stress $\sigma_{tf,cal}$ and direct shear stress $\tau_{vf,cal}$. Tests have shown that the *strength* of bearing type fasteners subjected to both shear and tension can be approximated by an equation in the form of ellipse. The general form of *interaction equation* can be written as

$$\left(\frac{\sigma_{tf,cal}}{\sigma_{tf}}\right)^2 + \left(\frac{\tau_{vf,cal}}{\tau_{vf}}\right)^2 = 1 \qquad ...(3.28)$$

According to IS : 800-1984, the rivets subjected to shear and externally applied tensile force should be so proportioned that

$$\left(\frac{\sigma_{tf,cal}}{\sigma_{tf}} + \frac{\tau_{vf,cal}}{\tau_{vf}}\right) \leq 1.4 \qquad ...(3.29)$$

Design of bracket connection

If the rivets have initial tension, as in the case of hot driven rivets, the number of rivets can be found using Eq. 3.22. If, however, the rivets do not have initial tension, slightly less number of rivets will be required; in such a case, n may be found by modifying Eq. 3.22 as under

$$n = 0.8\sqrt{\frac{6M}{lpR}} \qquad ...(3.30)$$

Example 3.13. *A load of 100 kN is carried by a bracket riveted to the flange plate of a stanchion, as shown in Fig. 3.29. Each rivet is of 24 mm diameter. Calculate the maximum shear stress in any rivet.*

Solution

Formed diameter of rivets

$$= 24 + 1.5 = 25.5 \text{ mm}$$

$$M = P.e = 100 \times 10^3 \times 200 = 20 \times 10^6 \text{ N-mm}$$

Since the area of section of all the rivets is the same,

$$F_1 = \frac{P}{n} = \frac{100000}{8} = 12500 \text{ N}$$

$$\Sigma r^2 = 4(40^2 + 120^2) + 4(40^2 + 40^2) = 76800 \text{ mm}^2$$

Rivet B will be the most heavily loaded rivet, for which

$$r = \sqrt{40^2 + 120^2} = 126.5 \text{ mm}$$

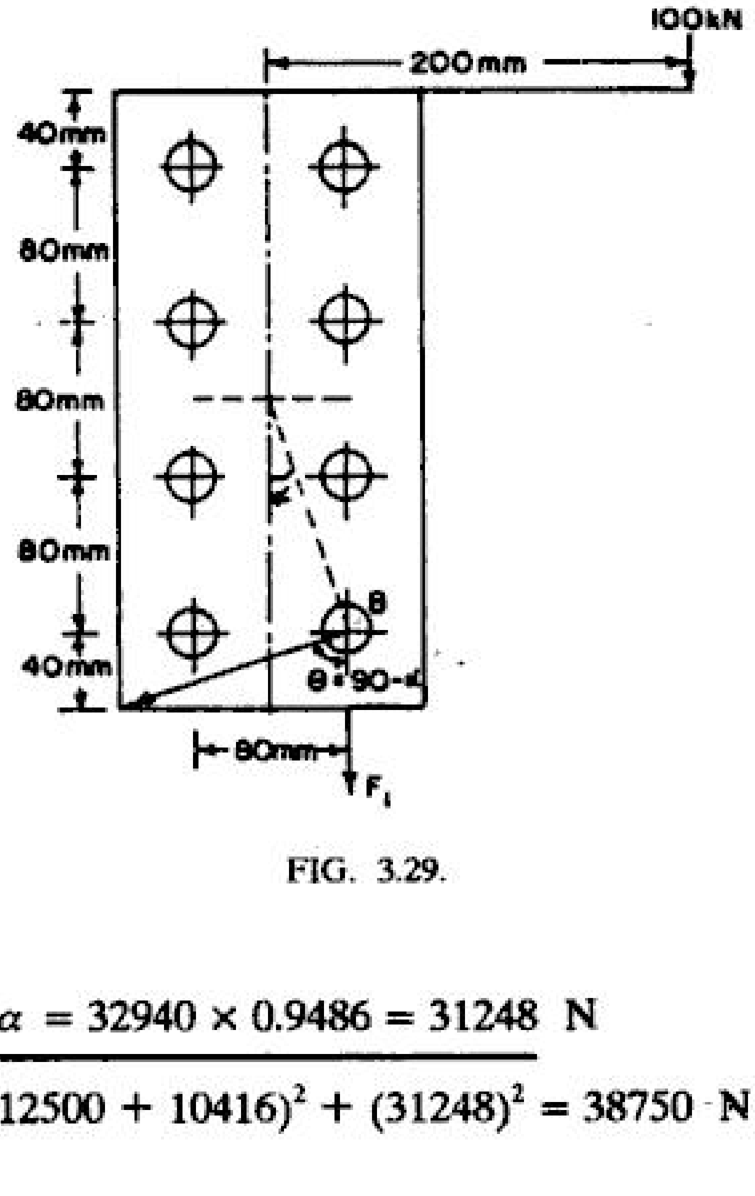


FIG. 3.29.

$$\therefore \quad F_2 = \frac{M.r}{\Sigma r^2} = \frac{20 \times 10^6 \times 126.5}{76800} = 32940 \text{ N}$$

$$\cos\alpha = \frac{120}{126.5} = 0.9486\,; \quad \sin\alpha = \frac{40}{126.5} = 0.3162$$

$$F_2 \sin\alpha = 32940 \times 0.3162 = 10416 \text{ N}\,; \qquad F_2\cos\alpha = 32940 \times 0.9486 = 31248 \text{ N}$$

$$R = \sqrt{(F_1 + F_2\sin\alpha)^2 + (F_2\cos\alpha)^2} = \sqrt{(12500 + 10416)^2 + (31248)^2} = 38750 \text{ N}$$

If the rivets are in single shear,

$$f_s = \frac{38750}{\frac{\pi}{4}(25.5)^2} = \mathbf{75.9 \text{ N/mm}^2}$$

Example 3.14. *Calculate the shearing stress in the rivets B and C for the connection shown in Fig. 3.30. Rivets A and B have 14 mm diameter while rivet C has a diameter of 22 mm.*

Solution

Formed diameter of rivets A and $B = 14 + 1.5 = 15.5$ mm

Area of cross-section of rivets A and $B = \frac{\pi}{4}(15.5)^2 = 188.7 \text{ mm}^2$

Formed diameter of rivet $C = 22 + 1.5 = 23.5$ mm

Area of cross-section of rivet $C = \frac{\pi}{4}(23.5)^2 = 433.7 \text{ mm}^2$

To find the position of C.G. of rivet group, take moments of rivet areas about vertical line passing through C.

Thus $\bar{x} = \dfrac{2 \times 188.7 \times 90}{(2 \times 188.7) + 433.7} = 41.9$ mm

$$\Sigma A = (2 \times 188.7) + 433.7 = 811.1 \text{ mm}^2$$

$$M = P.e. = 20000(120 + 41.9)$$
$$= 3238000 \text{ N-mm}$$

In the rivet B,

$$F_{B1} = \frac{PA}{\Sigma A} = \frac{20000 \times 188.7}{811.1} \doteq 4653 \text{ N}$$

Similarly, in the rivet C

$$F_{C1} = \frac{PA}{\Sigma A} = \frac{20000 \times 433.7}{811.1} = 10690 \text{ N}$$

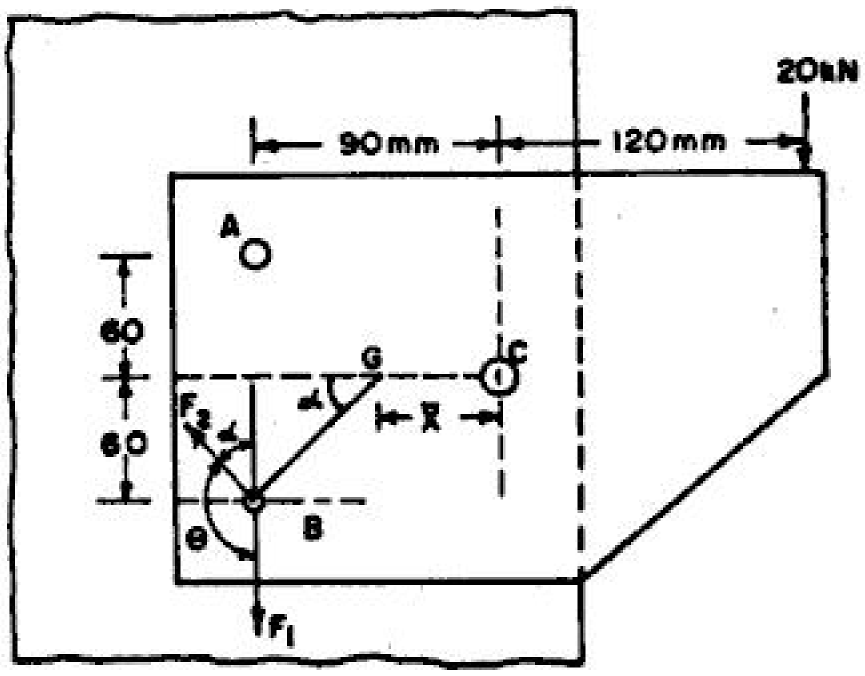


FIG. 3.30.

$$\Sigma A r^2 = 2 \times 188.7\,(48.1^2 + 60^2) + 433.7\,(41.9)^2 = 2993200$$

$$F_{B2} = \frac{M.r.A}{\Sigma A r^2} = \frac{3238000 \times 188.7\sqrt{(48.1)^2 + 60^2}}{2993200} = 15700 \text{ N}$$

and $$F_{C2} = \frac{M.rA}{\Sigma A r^2} = \frac{3238000 \times 41.9 \times 433.7}{2993200} = 19660 \text{ N}$$

$$R_C = 10690 + 19660 = 30350 \text{ N}$$

$$\alpha = \tan^{-1}\frac{60}{90 - 41.9} = 51.28°,$$

∴ $$\theta = 180° - 51.28° = 128.72°$$

$$R_B = \sqrt{F_{B1}^2 + F_{B2}^2 + 2F_{B1}.F_{B2}\cos\theta}$$

$$R_B = \sqrt{(4653)^2 + (15700)^2 + 2 \times 4653 \times 15700 \cos 128.72°} = 13295 \text{ N}$$

$$f_B = \frac{R_B}{188.7} = \frac{13295}{188.7} = 70.45 \text{ N/mm}^2$$

$$f_C = \frac{R_C}{433.7} = \frac{30350}{433.7} = 69.98 \text{ N/mm}^2$$

Example 3.15. *A load of 100 kN is carried by a bracket riveted to the flange plate of a stanchion, as shown in Fig. 3.29. Using 8 rivets, calculate the diameter of the rivets. Each rivet is in single shear. Take $\tau_{vf} = 100$ N/mm^2.*

Solution. (Fig. 3.29, Example 3.13). As found in Example 3.13,

$$M = 20 \times 10^6 \;;\; F_1 = 12500 \text{ N}$$
$$F_2 = 32940 \text{ N} \;;\; R = 38750 \text{ N}$$

$$\text{Shear stress} = \frac{38750}{A}$$

But this should be equal to permissible shear stress (τ_{vf}).

∴ $$\frac{38750}{A} = \tau_{vf} = 100.$$

From which $$A = \frac{38750}{100} = 387.50 \text{ mm}^2$$

$\therefore$ Dia. $d = \sqrt{\frac{4}{\pi}A} = \sqrt{\frac{4}{\pi} \times 387.50} = 22.2$ mm

This is the formed diameter of the rivet.

Actual dia. of rivet $= 22.2 - 1.5 = 20.7$

Hence adopt d = **22 mm**

Example 3.16. *Solve example 3.13 if the load of 100 kN acts at an inclination of 60° to the horizontal.*

Solution

Apply equal and opposite forces P, in the direction of given load $P = 100$ kN. The given system is thus equivalent to

(*i*) A direct load P acting through the C.G. of the rivet group and in the direction of applied load, and

(*ii*) A moment $M = P.e$

F_1 due to direct load $= \frac{P}{n} = \frac{100000}{8} = 12500$ N, in the direction of P.

F_2 due to moment $= \frac{M.r}{\Sigma r^2}$

From Fig. 3.31, $CE = 200 \tan 60° = 346.4$ mm

$GE = 364.4 - (40 + 80 + 40) = 186.4$ mm

Draw GA perpendicular to DE.

$GA = e = GE \cos 60° = 186.4 \times \frac{1}{2} = 93.2$ mm

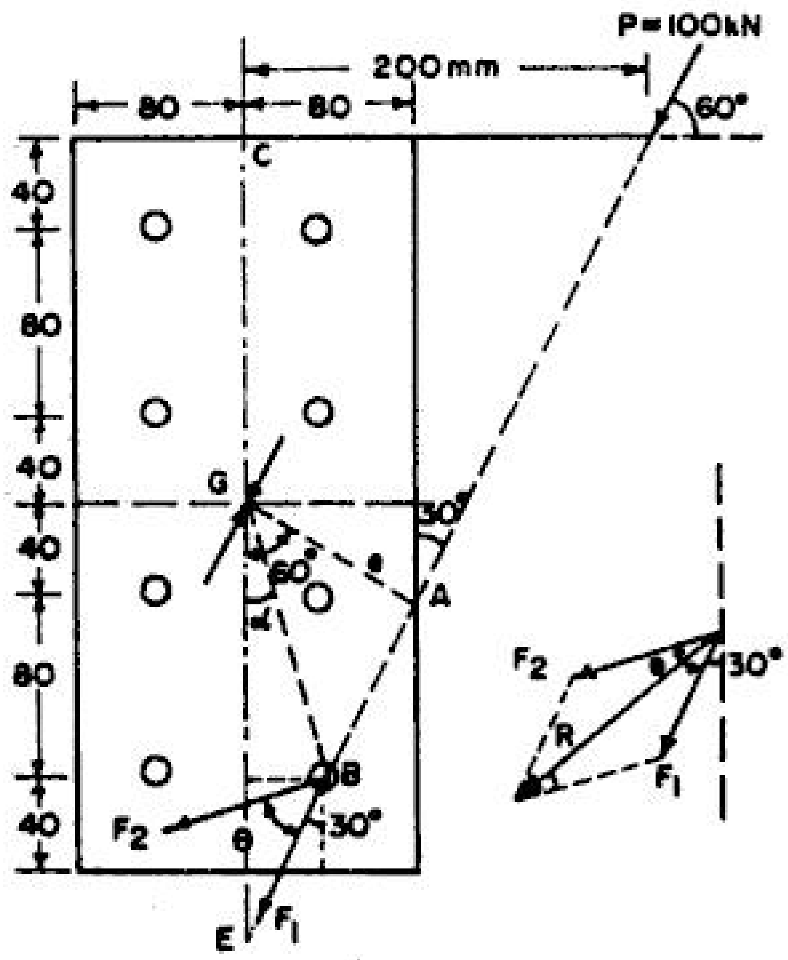


FIG. 3.31.

$M = P.e = 100000 \times 93.2 = 93.2 \times 10^5$ N-mm

$\Sigma r^2 = 76800$ mm^2 as found in Example 3.13 and $r = 126.5$ mm

$$\therefore \quad F_2 = \frac{M.r}{\Sigma r^2} = \frac{93.2 \times 10^5 \times 126.5}{76800} = 15350 \text{ N}$$

The maximum F_2 will occur in rivet marked B. The resultant load $R = 26000$ N (found graphically)

Alternatively, $\alpha = \sin^{-1} \frac{40}{126.5} = 18.43°$

$\therefore$ Inclination of F_2 with vertical $= 90° - \alpha = 90° - 18.43° = 71.57°$

Inclination of F_1 with vertical $= 90° - 60° = 30°$

$\therefore$ Angle θ between F_1 and $F_2 = 71.57° - 30° = 41.57°$

$$\therefore \quad R = \sqrt{F_1^2 + F_2^2 + 2F_1 F_2 \cos\theta}$$

$$= \sqrt{(12500)^2 + (15350)^2 + 2\,(12500 \times 15350) \times \cos 41.57°}$$

$= 26057$ N

$$\therefore \quad \text{Maximum shear stress intensity} = \frac{26057}{\frac{\pi}{4}(25.5)^2} = \mathbf{51.02\ N/mm^2}$$

Example 3.17. *A load P kN is to be carried by a bracket riveted to the flange plate of a stanchion as shown in Fig. 3.32. Each rivet is of 22 mm dia. Calculate the maximum value of P which can be allowed so that the maximum stress in any rivet does not exceed the one as prescribed by IS : 800-1984. Use power driven rivets.*

Solution

Apply equal and opposite forces at c.g. of the rivet group, in the direction of the load P. The given system is equivalent to

(*i*) a direct load P acting through the c.g. of the rivet group, in the direction of applied load and

(*ii*) a moment $M = P.e$.

F_1 due to direct load $= \frac{P}{n} = \frac{P}{6}$ in the direction of P.

F_2 due to moment $= \frac{Mr}{\Sigma r^2} = \frac{P.e.r}{\Sigma r^2}$

$$AC = 125 + 150 = 275 \text{ mm};$$

$$AO = 150 + 55 = 205$$

$\therefore$ $OC = \sqrt{AC^2 + AO^2} = \sqrt{(275)^2 + (205)^2} = 343$ mm

$$\beta = \tan^{-1}\frac{AO}{AC} = \tan^{-1}\frac{205}{275} = 36.7°$$

$$\alpha = 60° - \beta = 60° - 36.7° = 23.3°$$

$$e = OC \sin\alpha = 343 \sin 23.3° = 135.66 \text{ mm}$$

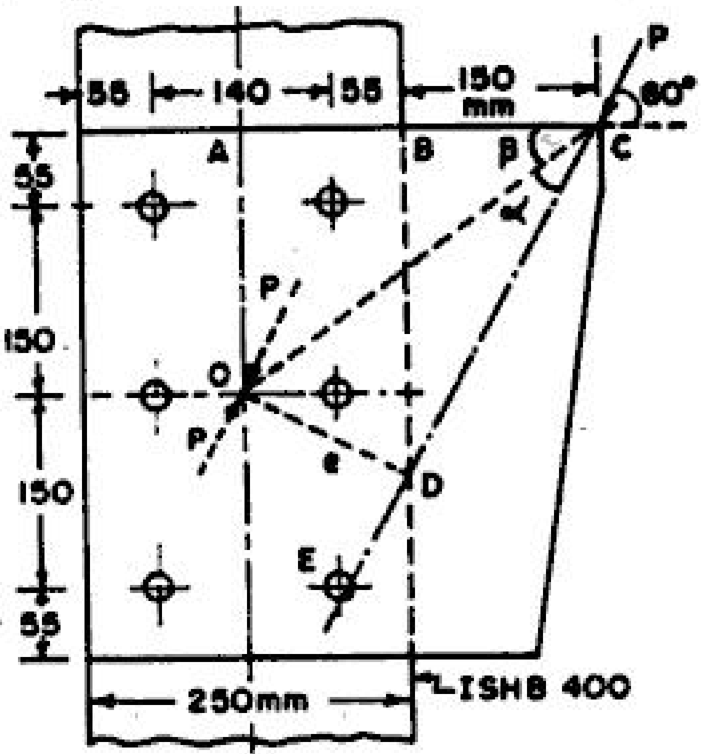


FIG. 3.32.

From Fig. 3.33, it is clear by inspection that rivet E is the most heavily loaded.

$$r \text{ of rivet } E = OE = \sqrt{150^2 + 70^2} = 165.53 \text{ mm}$$

$$\Sigma r^2 = 4(165.53)^2 + 2(70)^2 = 119400 \text{ mm}^2$$

$\therefore$ $F_{E2} = \frac{P.e.r}{\Sigma r^2} = \frac{P \times 135.66 \times 165.53}{119400} = 0.1881P$

$$F_{E1} = \frac{P}{6} = 0.1667P$$

Now $\gamma = \tan^{-1}\frac{70}{150} = 25.02°$

$\therefore$ $\theta = 60° - \gamma = 60° - 25.02° = 34.98°$

$\therefore$ $R_E = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

$$= \sqrt{(0.1667P)^2 + (0.1881P)^2 + 2 \times 0.1667P \times 0.1881P \times \cos 34.98°}$$

$\therefore$ $R_E = 0.3385P$ kN (where P is in kN)

Max. shear stress in rivet $E = \frac{R}{A} = \frac{0.3385P \times 1000}{\frac{\pi}{4}(23.5)^2}$

$$= 0.7804P \text{ N/mm}^2$$

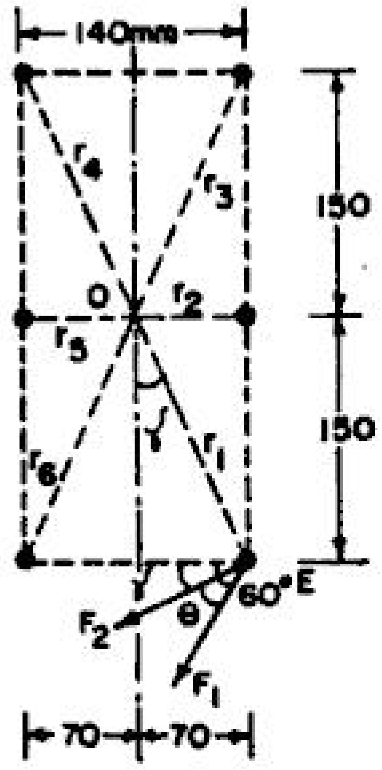


FIG. 3.33

Equating this to permissible stress $\tau_{vf} = 100$ N/mm², we get

$$P = \frac{100}{0.7804} = \textbf{128.13 kN}$$

Example 3.18. *A double plate bracket is provided using 12 mm thick plates connected to flanges of a steel column having flange thickness of 12.7 mm and transmit a load of 600 kN at an eccentricity of 200 mm. Design the bracket using 22 mm dia. power driven rivets.*

Solution

Axial load on one plate $= \frac{1}{2} \times 600 = 300$ kN.

Formed diameter of rivet $= 22 + 1.5 = 23.5$ mm

Taking $\tau_{vf} = 100$ N/mm^2 for power driven rivets,

Strength of rivet in single shear $= \frac{\pi}{4}(23.5)^2 \times 100 = 43374 \text{ N} = 43.374$ kN ...(*i*)

Thickness of flange of the section = 12.7 mm

Thickness of gusset plate = 12 mm. $\therefore$ $t = 12$ mm

Strength of rivet in bearings $= d \,.\, t \,.\, \sigma_{pf} = 23.5 \times 12 \times 300 = 84600$ N ..(*ii*)

Hence rivet value = 43.374 kN

Let us provide a pitch p = 60 mm

$$M = P.e = 300 \times 10^3 \times 200 = 60 \times 10^6 \text{ N-mm}$$

Now $n = \sqrt{\dfrac{6M}{lpR}}$. Here l = number of rivet lines = 2 (say)

n = number of rivets in each line.

$$\therefore \quad n = \sqrt{\frac{6 \times 60 \times 10^6}{2 \times 60 \times 43374}} = 8.3$$

Hence provide $n = 9$.

Total rivets

$= 2n = 2 \times 9 = 18.$

Arrange these as shown in Fig. 3.34 (*a*).

Force in rivet due to axial load of 300 kN:

$$F_1 = \frac{P}{2n} = \frac{300}{2 \times 9} = 16.67 \text{ kN}$$

Force in rivet due to moment:

$$F_2 = \frac{M.r}{\Sigma r^2}$$

$r_1^2 = (70)^2 = 4900$

$r_2^2 = (60)^2 + (70)^2 = 8500$

$r_3^2 = (120)^2 + (70)^2 = 19300$

$r_4^2 = (180)^2 + (70)^2 = 37300$

$r_5^2 = (240)^2 + (70)^2 = 62500$

$\Sigma r^2 = 2r_1^2 + 4(r_2^2 + r_3^2 + r_4^2 + r_5^2)$

$= 520200.$

Also for rivet E,

$r = r_5 = \sqrt{62500} = 250$ mm

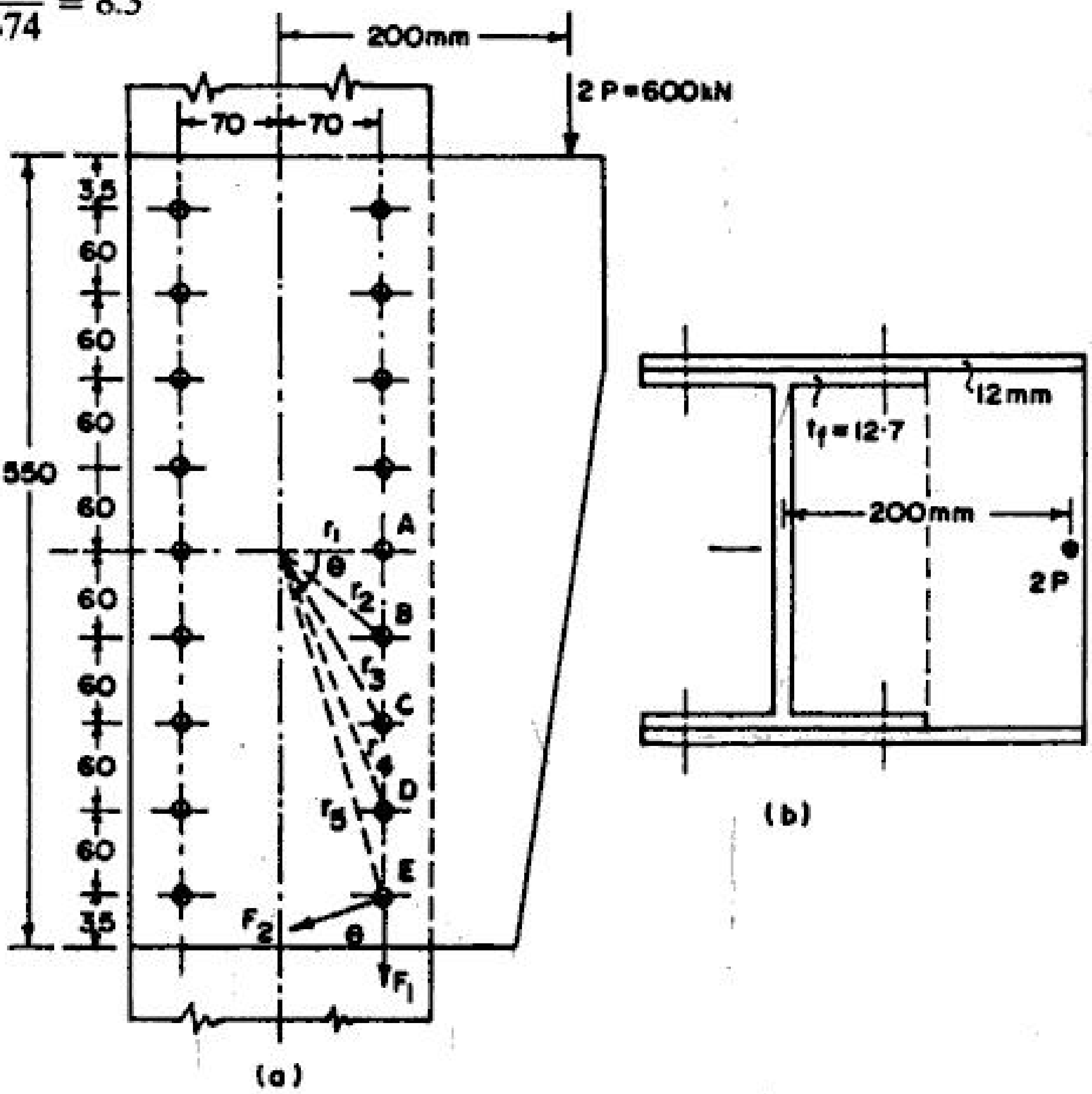


FIG. 3.34.

$$\therefore \quad F_{2E} = \frac{M.r}{\Sigma r^2} = \frac{60 \times 10^6 \times 250}{520200} = 28835 \text{ N} = 28.835 \text{ kN}$$

$$\text{Angle} \quad \theta = \tan^{-1}\frac{240}{70} = 73.74°; \ \cos\theta = 0.28$$

$$\therefore \quad R = \sqrt{F_1^2 + F_2^2 + 2F_1 F_2 \cos\theta}$$

$$= \sqrt{(16.67)^2 + (28.835)^2 + 2 \times 16.67 \times 28.835 \times 0.28} = 37.13 \text{ kN}$$

This is less than the rivet value of 43.374 kN. Hence the design is safe.

Example 3.19. *A gantry girder is attached to a steel stanchion, through a bracket connection, as shown in Fig. 3.35. Determine the maximum allowable load, if all the rivets are 16 mm dia and are power driven.*

Solution : For power driven rivets, $\tau_{vf} = 100 \text{ N/mm}^2$.

Formed dia. of rivet $= 16 + 1.5 = 17.5$ mm

Strength of rivets in single shear

$$= \frac{\pi}{4}(17.5)^2 \times 100 = 24053 \text{ N}$$

Strength of rivets in double shear $= 48106$ N

(a) Connection of girder to angles : Rivets A

These rivets connect the web of the girder to two angles, one angle to each side. The rivets are in single shear. There are in all $3 \times 2 = 6$ rivets.

$$\therefore \quad \text{Permissible } P = 6 \times 24053 = 144318 \text{ N} \quad ...(1)$$

(b) Connection of angles to bracket : rivets B

These rivets connect the two angles to the bracket plate. There are 3 rivets and each rivet is in double shear. However, the load P acts at an eccentricity of 40 mm. Hence the maximum force in the outermost rivet of rivet line B

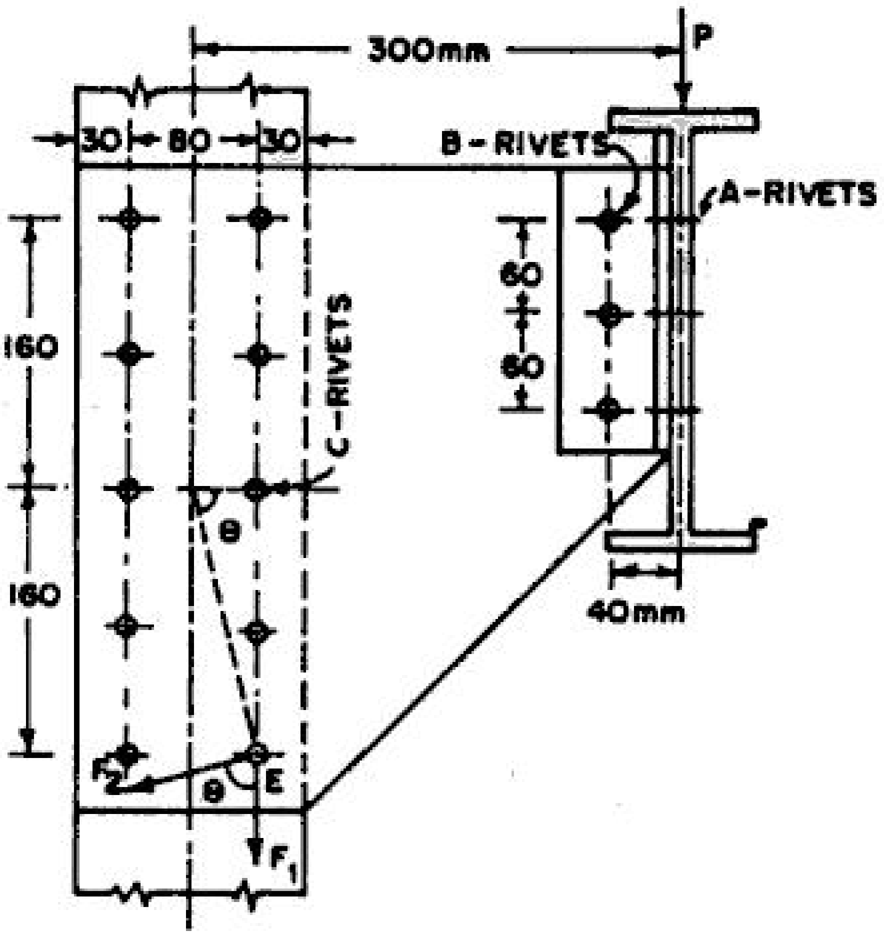


FIG. 3.35.

is

$$= \sqrt{\left(\frac{P}{3}\right)^2 + \left(\frac{P \times 40 \times 60}{2\,(60)^2}\right)^2} = 0.4714\,P$$

Equating this to the strength of rivet in double shear, we get

$$0.4714\,P = 48106$$

$$\text{or} \quad P = 102048 \text{ N} \quad ...(ii)$$

(c) Connection of bracket to stanchion: Rivets C

The load acts at an eccentricy $e = 300$ mm.

$$M = P.e = 300\,P \text{ N-mm.}$$

The maximum force will be induced in the outermost rivet E for which

$$\theta = \tan^{-1}\frac{160}{40} = 75.96° \ ; \ \cos\theta = 0.2426$$

$$\text{and} \quad r = \sqrt{(40)^2 + (160)^2} = 164.9 \text{ mm}$$

$$\Sigma r^2 = 4\,(164.9)^2 + 4\,[\,(80)^2 + (40)^2\,] + 2\,[\,(40)^2\,] = 144000 \text{ mm}^2$$

$$\text{Now} \quad F_{E1} = \frac{P}{10} = 0.1\,P$$

$$F_{E2} = \frac{P.e.r}{\Sigma r^2} = \frac{P \times 300 \times 164.9}{144000} = 0.3435\,P$$

Resultant $R = \sqrt{F_{E1}^2 + F_{E2}^2 + 2\,F_{E1}\,.\,F_{E2}\cos\theta}$

$$= \sqrt{(0.1\,P)^2 + (0.3435\,P)^2 + 2\,(0.1\,P)\,(0.3435\,P)\,(0.2426)}$$

$$= 0.3803\,P.$$

The rivet is in single shear.

$\therefore$ $0.3803P = 24053$

or $P = \dfrac{24053}{0.3803} = 63247$ N ...(*iii*)

The maximum permissible value of P will be the least of (*i*), (*ii*) and (*iii*).

$\therefore$ Permissible P **=63247 N**

Example 3.20. *Determine the maximum axial tension in the rivets of the connection shown Fig. 3.36. Taking permissible stresses as per IS : 800 for power driven rivets, determine the diameter of the rivet.*

Solution : For power driven rivets,

$\tau_{vf} = 100$ N/mm^2 ; $\sigma_{tf} = 100$ N/mm^2

Let us assume that the rivets do not have initial tension, and that the N.A. or line of rotation will lie at $h/7$ above the bottom edge of bracket.

$$h = (6 \times 70) + 35 = 455 \text{ mm}$$

$$\therefore\ \frac{h}{7} = \frac{455}{7} = 65 \text{ mm}$$

For both the lines of rivets,

$$\Sigma y = 2\,[40 + 110 + 180 + 250 + 320 + 390]$$
$$= 2580 \text{ mm}$$

$$\Sigma y^2 = 2\left[40^2 + 110^2 + 180^2 + 250^2 + 320^2 + 390^2\right]$$
$$= 726200 \text{ mm}^2$$

Moment resisted by rivets which are subjected to tension is

$$M_t = \frac{M}{1 + \dfrac{2h}{21} \cdot \dfrac{\Sigma y}{\Sigma y^2}} \qquad ...(3.25)$$

$$= \frac{150 \times 10^3 \times 200}{1 + \dfrac{2 \times 455}{21} \times \dfrac{2580}{726200}}$$

$$\approx 26 \times 10^6 \text{ N-mm}$$

The maximum tensile force in the top most rivet is

$$T_{max} = \frac{M_t\,.\,y_{max}}{\Sigma y^2} = \frac{26 \times 10^6 \times 390}{726200}$$

$$= \mathbf{13962\ N} \qquad ...(1)$$

Direct shear stress

$$F_s = \frac{P}{2n} = \frac{150 \times 10^3}{2 \times 7} = 10714 \text{ N} \qquad ...(ii)$$

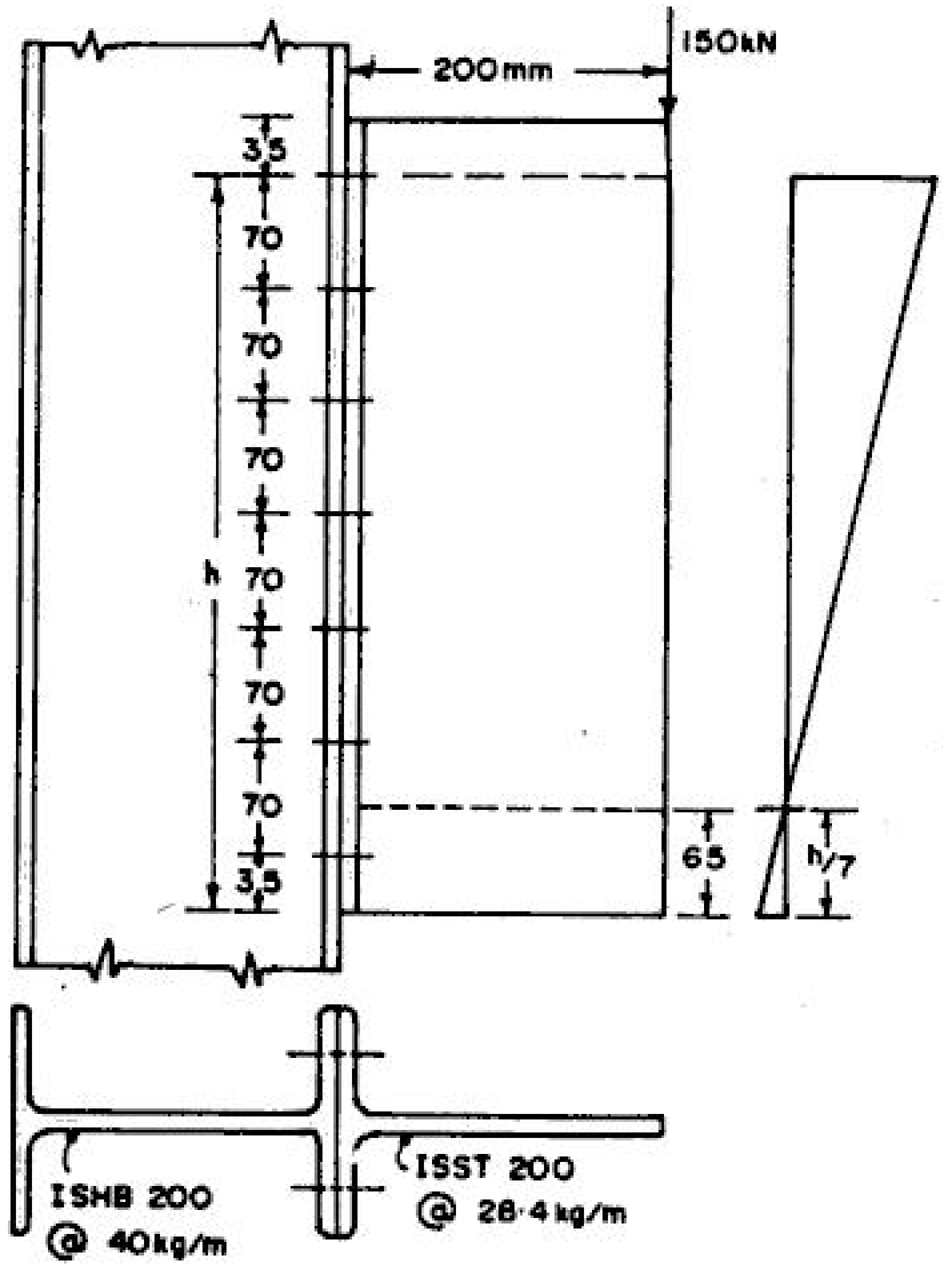


FIG. 3.36.

Let d be the formed diameter of rivet. Hence for the topmost rivets,

$$\sigma_{tf,cal} = \frac{T_{max}}{A} = \frac{13962}{\frac{\pi}{4}d^2} = \frac{17777}{d^2} \quad ...(a)$$

and

$$\tau_{vf,cal} = \frac{F_s}{A} = \frac{10714}{\frac{\pi}{4}d^2} = \frac{13641}{d^2} \quad ...(b)$$

The rivets are subjected to both shear as well as axial tension. Hence the following relationship should be satisfied.

$$\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{\sigma_{tf,cal}}{\sigma_{tf}} \le 1.4$$

or

$$\frac{13641/d^2}{100} + \frac{17777/d^2}{100} = 1.4$$

from which $d^2 = 224.4$ or $d = 15$ mm

Hence actual dia. of rivet $= 15 - 1.5 = 13.5$ mm. $\therefore$ Provide. 14 mm dia. rivets.

Example 3.21. *Design a riveted connection joining the bracket angles 2-ISA 100 × 100 × 8 mm using (a) power driven (hot) shop rivets, (b) power driven (cold) shop rivets, as shown in Fig. 3.37.*

Solution

(a) Power driven (hot) shop rivets.

Due to hot rivets, there will be *initial tension* in the rivets, due to which N.A. will lie at the mid-height of the bracket.

Let us use two rows of 20 mm diameter rivets at pitch of 60 mm. The number of rivets per line is given by Eq. 3.22

$$n = \sqrt{\frac{6M}{lpR}}$$

where $M = 200 \times 250 \times 10^3 = 50 \times 10^6$ N-mm

$l = 2$

$p = 60$ mm

$R = \frac{\pi}{4}(21.5)^2 \times 100 = 36305$ N.

$$\therefore \quad n = \sqrt{\frac{6 \times 50 \times 10^6}{2 \times 60 \times 36305}} = 8.3$$

Provide 9 rivets in each row. Area of each rivet $= \frac{\pi}{4}(21.5)^2 = 363.1$ mm².

The calculated tensile stress in extreme rivet is given by Eq. 3.23 :

$$\sigma_{tf,cal} = \frac{6M}{lpn^2A} = \frac{6 \times 50 \times 10^6}{2 \times 60\,(9)^2 \times 363.1} = 85 \text{ N/mm}^2$$

Direct shear load in each rivet

$$= F = \frac{P}{2n} = \frac{200 \times 10^3}{2 \times 9} = 11111.1\text{N. Hence,}$$

$$\tau_{vf,cal} = \frac{11111.1}{363.1} = 30.6 \text{ N/mm}^2$$

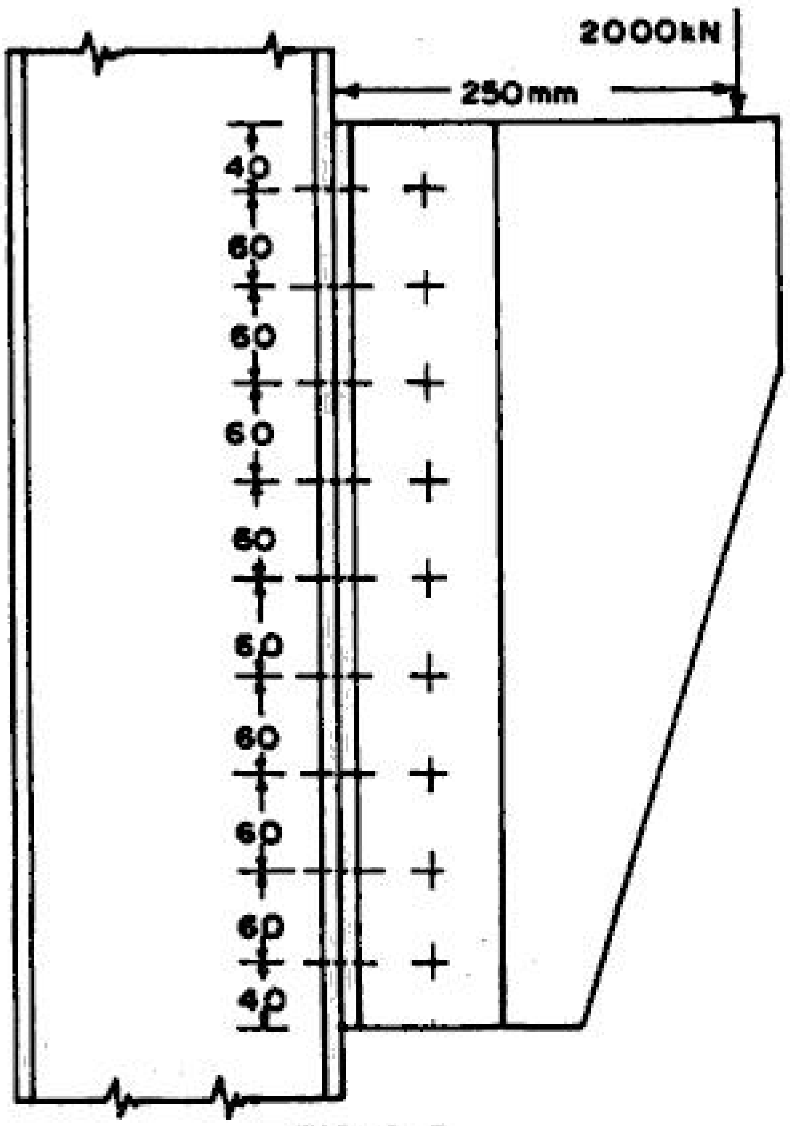


FIG. 3.37.

Now as per Code requirements,

$$\frac{\tau_{vf,\,cal}}{\tau_{vf}} + \frac{\sigma_{tf,\,cal}}{\sigma_{tf}} \le 1.4$$

$\therefore$ $$\frac{30.6}{100} + \frac{85}{100} \le 1.4$$

or $1.156 \le 1.4$. Hence the design is satisfactory.

(b) Power driven (cold) shop rivets

For these rivets, there will be no initial tension. The number of rivets for such a case is given by Eq. 3.30

$$n = 0.8\sqrt{\frac{6M}{lpR}}$$

Providing l = number of rows = 2, p = 60 mm and R = 36305 N as before, we get

$$n = 0.8\sqrt{\frac{6 \times 50 \times 10^6}{2 \times 60 \times 36305}} = 6.64$$

Hence provide 7 rivets, and arrange these as shown in Fig. 3.38. Keep the edge distance = 40 mm. Height $h = (60 \times 6) + 40 = 400$ mm

The N.A. will lie at $\frac{h}{7}\left(= \frac{400}{7} \approx 57 \text{ mm}\right)$ above the bottom edge of the bracket.

For both the lines of rivets,

$$\Sigma y = 2\,[43 + 103 + 163 + 223 + 283 + 343] = 2316 \text{ mm}$$
$$\Sigma y^2 = 2\,[43^2 + 103^2 + 163^2 + 223^2 + 283^2 + 343^2] = 572988$$

$$M_t = \frac{M}{1 + \frac{2h}{21}\frac{\Sigma y}{\Sigma y^2}}$$

$$= \frac{200 \times 10^3 \times 250}{1 + \frac{2 \times 400}{21} \times \frac{2316}{572988}}$$

$= 43.328 \times 10^6$ N-mm

The tensile force in the topmost rivet is

$$T_{max} = \frac{M_t \cdot y_{max}}{y^2}$$

$$= \frac{43.328 \times 10^6 \times 343}{572988}$$

= 25937 N. Hence,

$$\sigma_{tf,\,cal} = \frac{T_{max}}{A} = \frac{25937}{363.1}$$

$= 71.43 \text{ N/mm}^2$ ...(1)

Direct shear load in each rivet $= F = \frac{P}{2n} = \frac{200 \times 10^3}{2 \times 7}$

= 14286 N

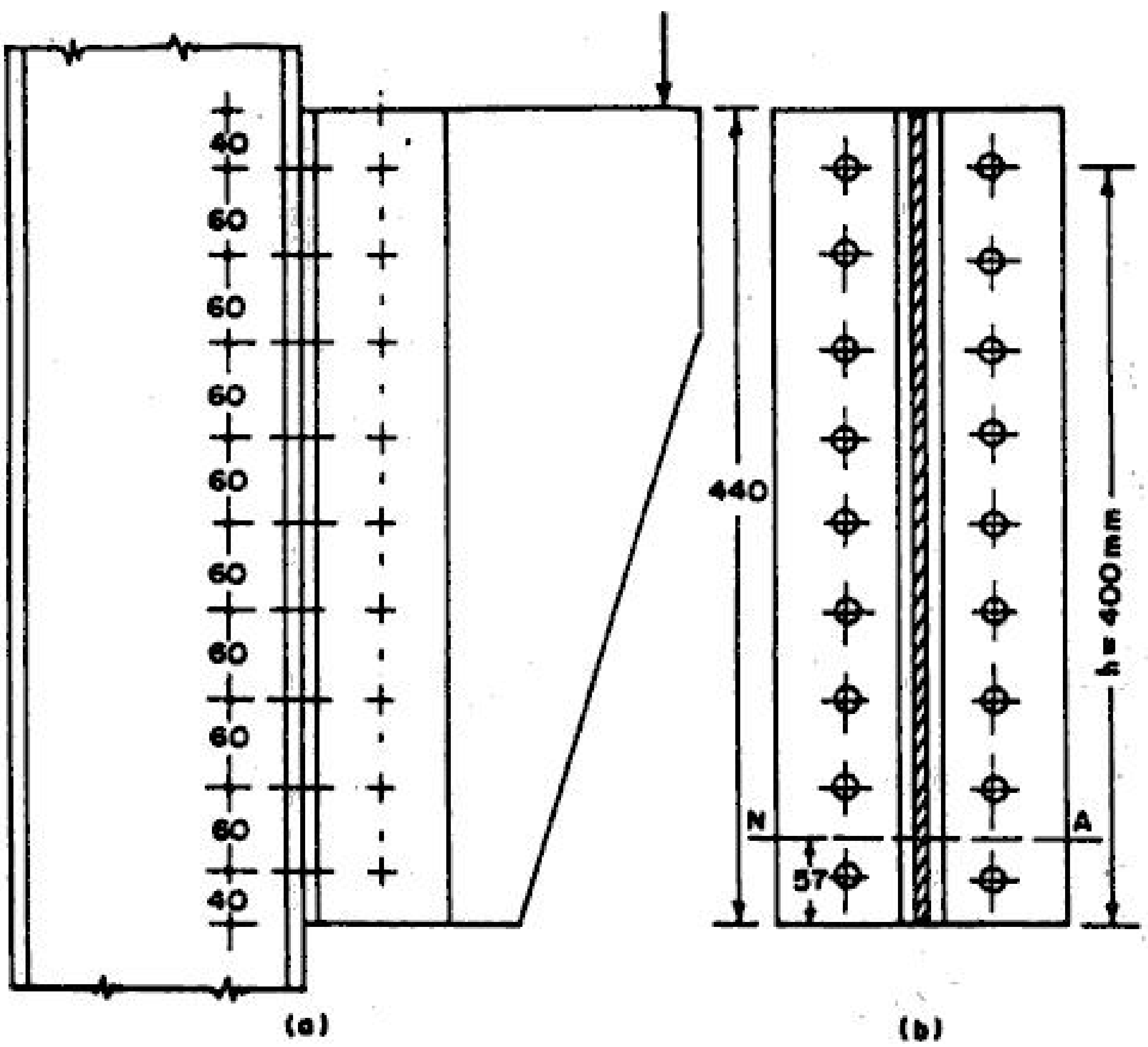


FIG. 3.38.

$$\therefore \quad \tau_{vf,cal} = \frac{F}{A} = \frac{14286}{363.1} = 39.34 \text{ N/mm}^2 \quad ...(2)$$

As per code requirements,

$$\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{\sigma_{tf,cal}}{\sigma_{tf}} \le 1.4$$

or
$$\frac{39.34}{100} + \frac{71.43}{100} \le 1.4$$

or $1.11 \le 1.4$. Hence the design is satisfactory.

PROBLEMS

1. A double riveted lap joint in plates 10 mm thick is made with 16 mm rivets at 60 mm pitch. Estimate how the joint will fail and calculate its efficiency if the tearing strength of the plates is 475 N/mm^2 and shearing and bearing strength of the rivets are 380 N/mm^2 and 750 N/mm^2 respectively. **Ans.** [The joint will fail in shear at a pull of 182700 N; $\eta = 59.2\%$]
2. A double riveted double cover butt joint is used to connect plates 12 mm thick. Determine the diameter of the rivet, rivet value, pitch and efficiency of the joint. Adopt the following working stresses : $\tau_{vf} = 102.5 \text{ N/mm}^2$; $\sigma_{pf} = 236 \text{ N/mm}^2$ and $\sigma_{at} = 150 \text{ N/mm}^2$.

   Ans. [22 mm; 66600 N ; 95 mm ; 75.2 %]
3. A bridge truss diagonal carries an axial pull of 500 kN. It is to be connected to a gusset plate 20 mm thick by a double cover butt joint with 22 mm rivets. If the width of the flat tie bar is 250 mm, determine the thickness of the flat. Design an economical joint and determine the efficiency of the joint. Use power driven rivets and take permissible stresses as per IS Code. The permissible stress in plate for axial tension may be taken as $0.6f_y$ where $f_y = 250 \text{ N/mm}^2$.
4. A double riveted butt joint, in which the pitch of rivet in the outer rows is twice that in the inner rows, connects two 16 mm thick plates with two cover plates each 12 mm thick. The diameter of rivets is 22 mm. Determine the pitches of the rivets in the two rows if the working stresses are not to exceed the following limits :

   Tensile stress in plates : 100 N/mm^2 ; Shear stress in rivets : 75 N/mm^2

   Bearing stress in rivets and plates : 150 N/mm^2 ; Make a fully dimensioned sketch of the joint. (Based on U.P.S.C. 1954) **Ans.** [128 mm; 64 mm]
5. A tie bar is attached to a gusset plate by four rivets arranged at the corners as shown in Fig. 3.39, the pitch of the rivets being p. The pull is applied symmetrically. If the rivet D is now removed, the pull being maintained at its former value, calculate by what percentage the load on each of the rivets is increased ? **Ans.** [A : 17.6 % ; B : 21.2% ; C : 67.5 %]
6. Explain the assumptions usually made in estimating the load carried by each rivet in a riveted joint which is subjected to both shear and bending in the plane of the joint. In the arrangement shown in Fig. 3.40, five rivets are symmetrically

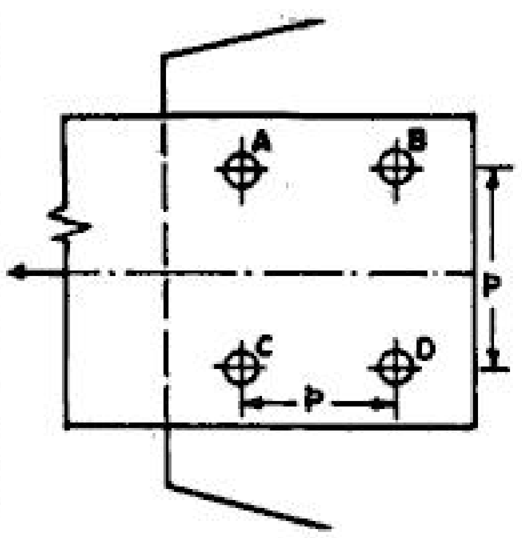


FIG. 3.39.

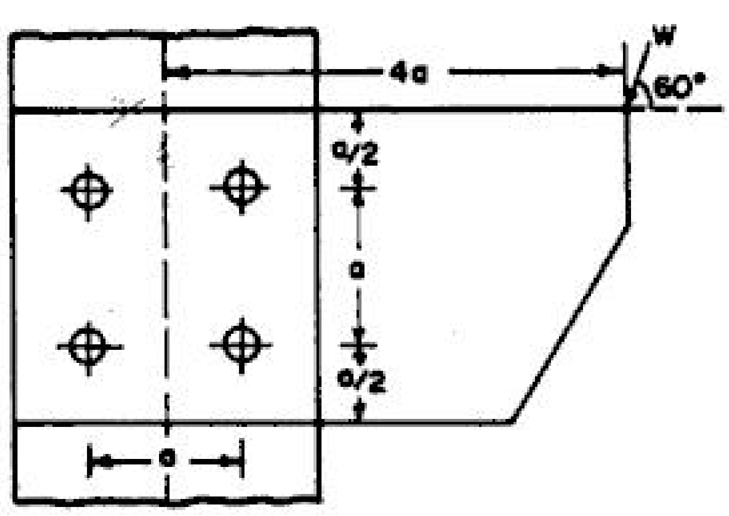


FIG. 3.40.

arranged at the corners and the centre of square of side a. Find in terms of W, the load transmitted by the most heavily loaded rivet. **Ans.**[1.24 W]

7. A riveted steel bracket connection has 22 mm diameter rivets 12 in number arranged as shown in Fig. 3.41. Determine the load P so that allowable stress in the extremely loaded rivet is just reached. Take safe permissible stress in bearing in rivet = 236 N/mm^2, and safe permissible stress in shearing in rivet = 102.5 N/mm^2.
(J.U. 1965) **Ans.** [186 kN]

8. If the worst rivet in the system shown in Fig. 3.42 may be stressed to 100 N/mm^2, calculate the safe value for the eccentric load P. The rivets are 24 mm in diameter and are in single shear.
Ans. [140 kN]

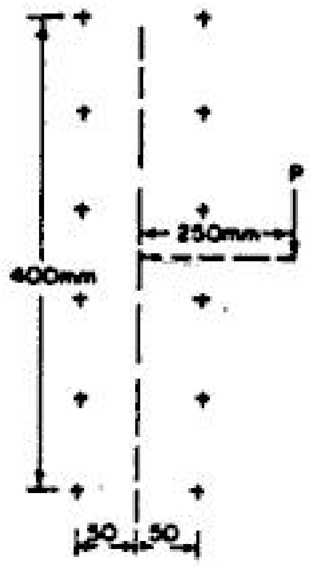


FIG. 3.41.

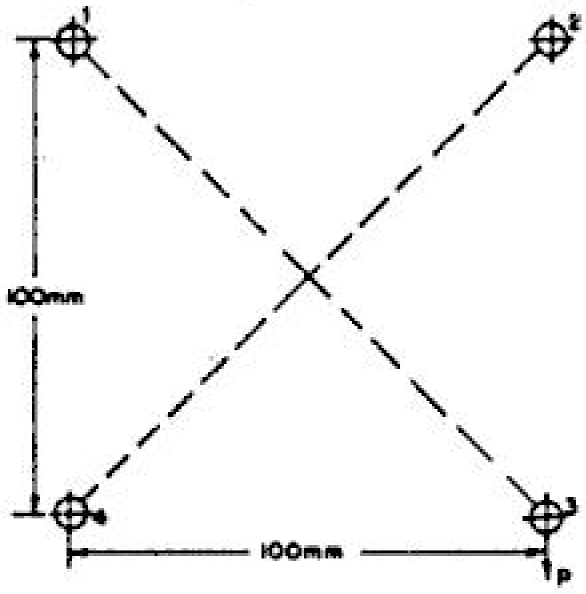


FIG. 3.42.

9. Calculate the maximum load carried by any rivet shown in Fig. 3.43. Rivets A and B are 200 mm^2 cross-sectional area and rivet C of 400 mm^2 area.

10. Design a bracket connection shown in Fig. 3.44, if it carries a load of 120 kN at an eccentricity of 350 mm from the centre line. Use power driven rivets. The thickness of bracket is 16 mm and the thickness of flange of the stanchion is 12 mm.

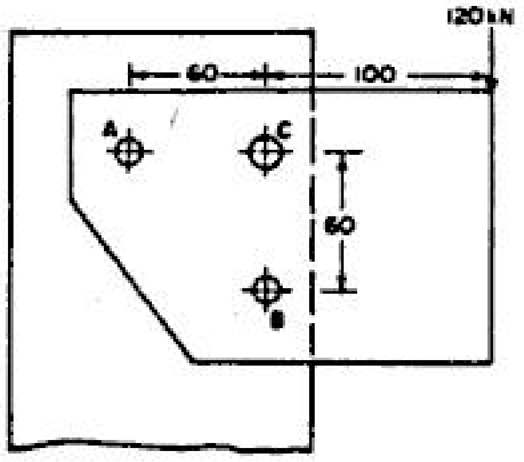


FIG. 3.43.

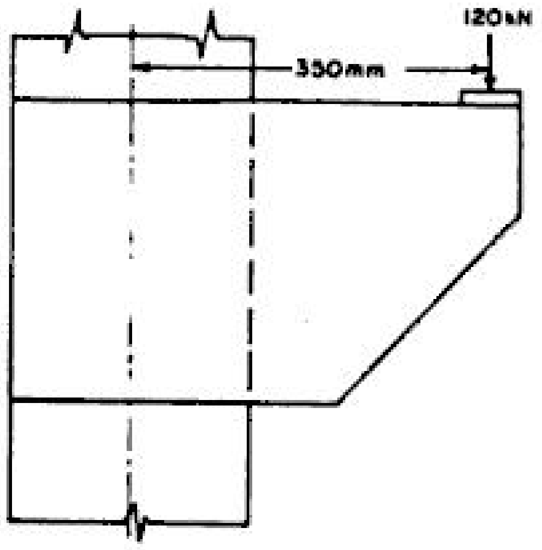


FIG. 3.44.

11. The flange of a tee section 200 mm × 200 mm is riveted to the flange of a rolled steel column of I-section to form a bracket which carries a vertical load of 250 kN at a distance of 200 mm from the face of the stanchion, as shown in Fig. 3.45. Design the riveted connection. Use shop driven rivets, each 20 mm dia.

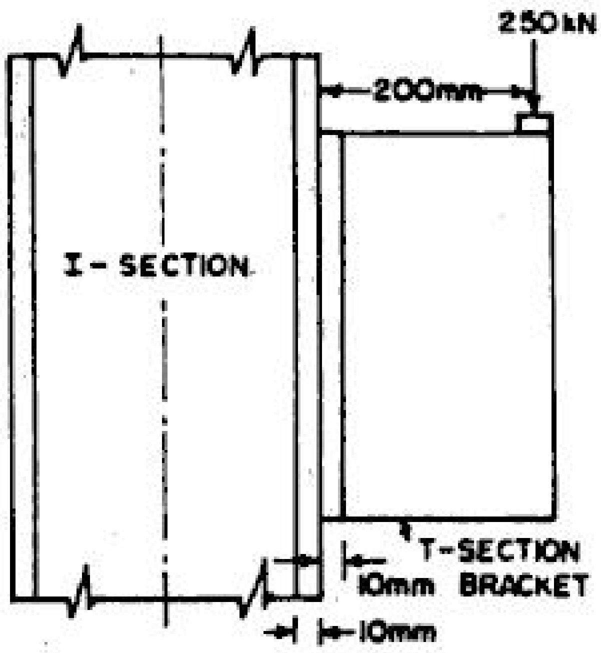


FIG. 3.45.

4

Steelwork Connections : II Bolted & Pinned Connections

4.1. INTRODUCTION

In chapter 1, we have seen that there are three fundamental structural members—tension members, compression members, and bending members. These structural members are often composed of structural sections which are jointed together. The most common types of structural steel connections are riveted connections, bolted connections and welded connections, though riveted connections are fast becoming absolete because of their low strength, high installation cost and other disadvantages. In *bolted connections*, bolts and nuts are used. There are several types of bolts that can be used for connecting structural steel members. The three types of bolts used in structural applications are (*i*) unfinished or black bolts, (*ii*) turned and fitted bolts and (*iii*) high-strength bolts. In *pinned connections*, pins are used for jointing the members.

Advantages of bolted connections

1. The bolting operation is very silent, in contrast to the hammering noise in riveting.
2. Bolting is a cold process, and hence there is no risk of fire.
3. Bolting operation is far more quicker than riveting.
4. There is no risk involved in the bolting, in contrast to the risk of flying rivets in riveting work.
5. Less man-power is required in making the connections.

Disadvantages of bolted connections

1. The bolted connections, if subjected to vibratory loads, result in reduction in strength if they get loosened.
2. Bolted connections for a given diameter of bolt, have lesser strength in axial tension since the net area at the root of the threads is less.
3. Unfinished bolts have lesser strength because of non-uniform diameter.
4. In the case of black bolts, the diameter of hole is kept 1.5 mm more than the diameter of the bolt, and this extra clearance does not get filled up, in contrast to the riveted joints.

4.2. BOLT TYPES

A bolt is a metal pin with a head formed at one end and the shank threaded at the other end in order to receive a nut. Structural bolts are classified as under :

(a) ***According to type of shank***
  (i) Unfinished or black bolts
  (ii) Turned bolts.

(b) ***According to material and strength***
  (i) Ordinary structural bolt
  (ii) High strength steel bolt

(c) ***According to Shape of head and nut***
  (i) Square bolt
  (ii) Hexagonal bolt

(d) ***According to Pitch and fit of thread***
  (i) Standard pitch bolt
  (ii) Coarse pitch bolt
  (iii) Fine pitch bolt

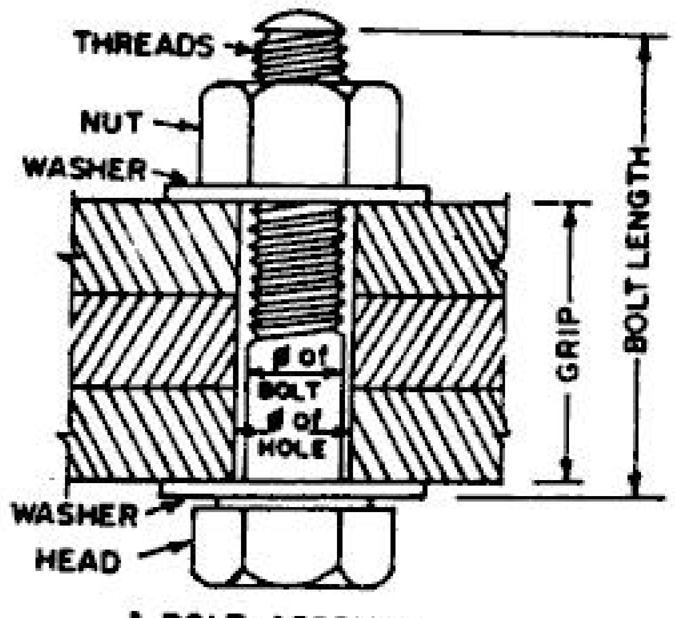


FIG. 4.1. BOLT ASSEMBLY

In common steel structural work, however, the following three bolt types are recognised:

1. Ordinary unfinished or black bolts
2. Turned and fitted bolts
3. High strength bolts.

4.3. ORDINARY UNFINISHED OR BLACK BOLTS

These are manufactured from black round bars of low carbon steel, and the surface of the shank is left unfinished, that is rough as rolled. The head is formed by forging. The diameter under the thread is usually 1.5 to 3 mm less than the shank. They remain loose in the holes which are usually made 1.5 mm larger in diameter than the nominal diameter of the bolt. Since the bearing of such bolts on the walls of the holes remains imperfect, the allowable stresses in these bolts are kept lower than the other types of bolts. They are therefore used only for ordinary field work and light loads—specially during erection operations.

Table 4.1 summarises the dimensions, range of lengths and other information about black bolts, as recommended by IS : 1364-1983.

TABLE 4.1. GENERAL DIMENSIONS OF HEXAGONAL HEAD BLACK BOLT IS : 1364 (Part I)–1983

Diameter (d) mm	6	8	10	12	16	20	24	30
Head dia. (e), mm	11.05	14.38	17.77	20.03	26.75	33.53	39.98	50.85
Head thickness, mm	4	5.3	6.4	7.5	10	12.5	15	18.7
*Thread length * (b), mm*	18	22	26	30	38	46	54	66
Pitch of thread, mm	1.0	1.25	1.5	1.75	2.0	2.5	3.0	3.5
Washer, out. dia. mm	18	25	30	40	50	60	IS : 5370–1969	
Washer, inner dia. mm	6.4	8.4	10.5	13.0	17.0	21.0		
Washer thickness, mm	1.5	2.0	2.5	3.0	3.0	4.0		

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

Permissible stresses

Table 4.3 gives permissible stresses in bolts in clearance holes (*i.e.* unturned bolts), along with those for turned bolts.

4.4. TURNED AND FITTED BOLTS

These are specially made from black round bars of mild steel containing low carbon content, but are turned down to exact diameter. The diameter of the shank is finished by turning to a diameter which is larger than the nominal diameter of the bolt by 1.2 mm for bolts M8 to M16 and by 1.3 mm for larger sizes. Flat face of the nut and head on the inner side are usually machined. Washers should also be machined on both the faces. These bolts will fit the bolt holes, which are larger by 1.5 mm. The holes for such bolts should either be reamed or drilled. These bolts provide much better bearing contact between the bolts and the holes. Strength of such bolts approximately equals that of rivets, and thus is greater than that of black bolts.

Table 4.2 summarises the dimensions, range of length and other information about fit bolts, as recommended by IS 3640-1982.

TABLE 4.2. GENERAL DIMENSIONS OF HEXAGONAL HEAD FIT BOLTS (IS : 3640–1982)

Nominal dia (d), mm	8	10	12	16	20	24	30
Bearing dia. d_3 *, mm*	9.2	11.2	13.2	17.2	21.3	25.3	32.3
Head thickness, mm	5.5	7.0	8.0	10.0	13.0	15.0	19.0
Head diameter, e, mm	–	17.6	19.86	26.17	32.95	39.55	50.85
Thread Length * *(b), mm*	16.5	19.5	22.5	27.0	30.5	36.5	43.0

*For l above 50 upto 150 mm.

For l upto 50 mm, b is less by 2 mm, and

For l >150 mm, b is 5 mm longer.

Permissible stresses

Table 4.3 gives the maximum permissible stresses in close tolerance and turned bolts, along with the bolts in clearance holes. In calculating the axial tensile stress in a bolt (or screwed tension rod), the net area shall be used.

TABLE 4.3. MAX. PERMISSIBLE STRESSES IN BOLTS

Description of bolt		*Axial tension* σ_{tf} *MPa (or N/mm²)*	*Shear* τ_{vf} *MPa (or N/mm²)*	*Bearing* σ_{pf} *MPa (or N/mm²)*
1.	Bolts in clear-ance holes	120	80	250
2.	Close tolerance and turned bolts	120	100	300

Edge distance of holes

The requirements for pitch and edge distance for bolts are the same as for rivets. Hence for taking the minimum edge distance, Table 3.2 may be referred.

Behaviour of unturned and turned bolts under load

Fig. 4.2 (*a*) shows a black bolt (unturned bolt), connecting two plates *A* and *B*. These bolts fit loosely and there is no clamping action on the plates. When load *P* is applied on the two plates (Fig. 4.2 *b*) there is shearing action on the plates, resulting in the slipping of the plates until the unturned shank of the bolt comes in contact with the edges of the hole. The load is thus transmitted by bearing on the bolt and shear in its shank. The shank of bolt bears against the edge of the plate, resulting in high local stresses and consequent localised yielding of material. Hence such a bolt is not desirable if the value of *P* is high. In contrast to this, Fig. 4.2 (*c*) shows the behaviour of turned and fitted bolt, when the plates are subjected to a load *P*. Since the bolt fits tightly in the reamed or drilled hole of plates, no slip would occur. The load, for such a case, is directly transferred by bearing and shear in the bolt. No localised bearing stresses are produced. Due to this, higher value of *P* can be permitted.

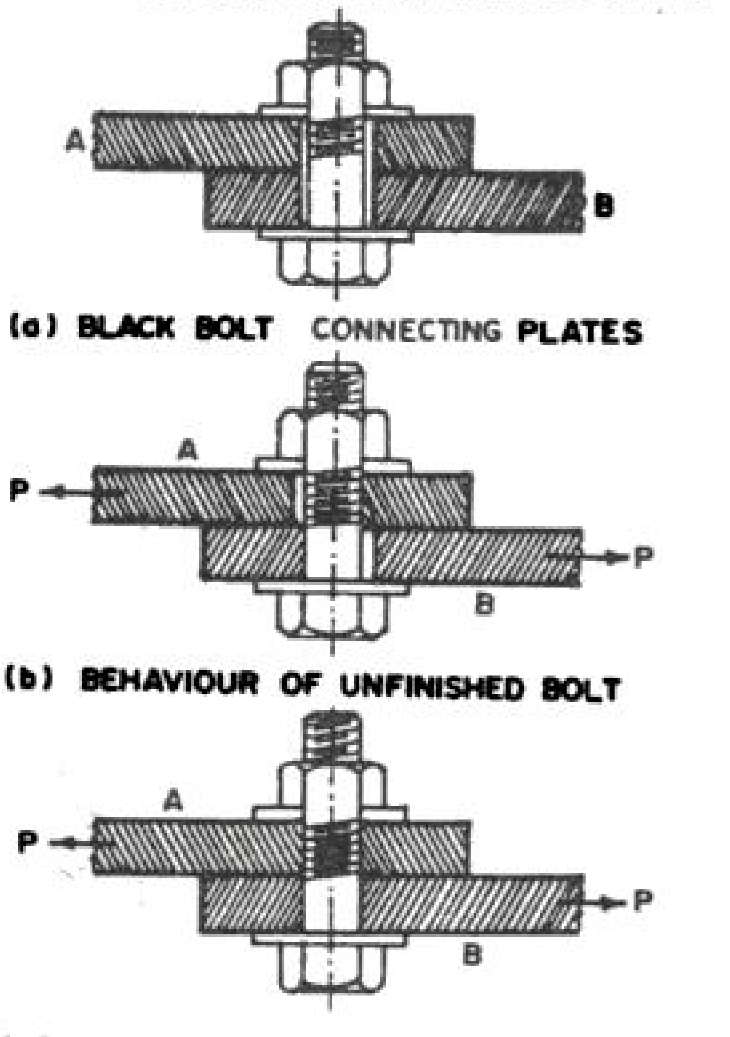


FIG. 4.2. BEHAVIOUR OF BOLTS UNDER LOADS

4.5. HIGH STRENGTH FRICTION GRIP BOLTS

High strength friction grip bolts are comparatively a recent development. They are made of high strength steel and their surface is kept unfinished, *i.e.* as rolled and rough. Due to this, they remain loose fit in the holes, similar to the unturned block bolts. However, high initial tension is developed in such bolts in the initial stage of tightening, and this tension clamps the joining plates between the bolt head and the nut. The tightening of the bolt to a very high tension, reaching their proof load, is done through calibrated torque wrenches. *This high pre-compression causes clamping action due to which the load is transmitted from one plate to the other by friction, with negligible slip.* The bearing of the bolt on the hole surface does not come to play at all. The joint so produced is a rigid one, which remain fully tight even under dynamic load, free from fatigue.

Fig. 4.3 shows the load transmission by a friction grip bolt. In an ordinary bolted joint, the force from one side is transferred to the other side through the interlocking and bearing of the bolts. In a friction grip joint, however, the force is transmitted by virtue of friction between the interfaces. To develop this friction a normal load is applied to the joint by using high strength bolts tightened to proof load. By usual law of friction

$$P = \mu T \qquad \text{...(4.1)}$$

where

T = clamping force induced by the bolt

μ = coefficient of friction between the interfaces, and

P = load carrying capacity of the joint in shear

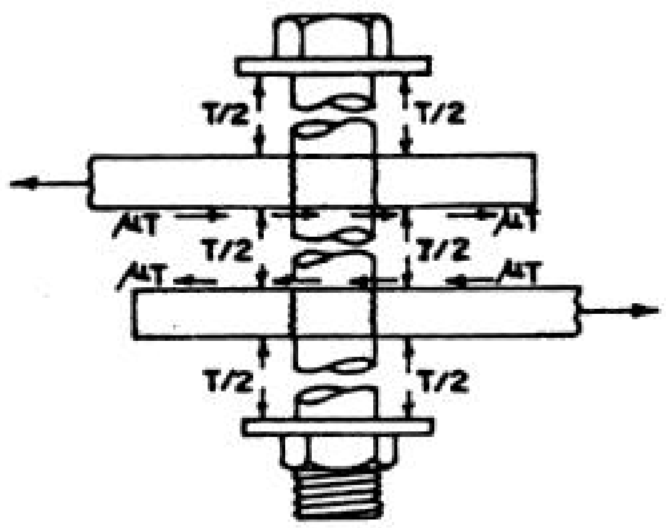


FIG. 4.3. LOAD TRANSMISSION BY FRICTION IN HIGH STRENGTH FRICTION GRIP BOLT

Hence if the actual applied load is equal to P or less, the joint will withstand it, and transfer it without any slip. When the actual load exceeds this value, there occurs a major slip, and, as load is further increased, gradual slipping brings the bolt in contact with the edges of the plate. The coefficient of friction is termed as *slip factor*. The slip factor is defined as the ratio between the force causing the large displacement between the two interfaces of the plates connected together and the force normal to the interfaces due to the tension in the bolts. A slip factor of 0.45 is stipulated by IS : 4000-1967 for surfaces which are free of paint, dirt, loose rust and mill scale. The high tensile friction grip bolts are commonly abbreviated as HTFG bolts.

Advantages of high strength bolts

1. It gives rigid joint as there is no slip between plates at working loads.
2. It gives high static strength due to high frictional resistance.
3. Smaller load is transmitted at net section of plates.
4. There are no shearing or bearing stresses in the bolts.
5. It has high fatigue strength.
6. As the bolts are in tension upto proof load they do not permit loosening of the nut and the washer.

Disadvantages of high strength bolts

1. The material cost of these bolts is much higher — it is about 50% greater than that of ordinary bolts and about 3 times that of rivets.

2. Special attention is required for workmanship in installing and tightening these bolts, specially in regard to giving them right amount of tension.

Structural uses : Field connections

High tensile friction grip bolts may be used as alternate to rivets or welds in the following types of connections:

(*i*) Column splices in all tier structures 60 m or more in height.

(*ii*) Column splices in tier structures 30 to 60 m in height, if the least horizontal dimension is less than 40 percent of the height.

(*iii*) Column splices in tier structures less than 30 m in height, if the least horizontal dimension is less than 25 percent of the height.

(*iv*) Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependant, in structure over 40 m height.

(*v*) Roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces and crane supports, in all structures carrying cranes of over 50 kN capacity.

(*vi*) Connections of supports of running machinery, or of other live loads which produce impact or reversal of stress, and

(*vii*) Any other connections stipulated on the design plans.

In all other cases, it is enough if field connections are made by normal type of bolts.

Minimum edge distance : The minimum distance from the centre of bolt hole to any edge, shall be the same as prescribed for rivets (IS : 800-1984). Hence Table 3.2 may be referred.

Dimensions for high tensile friction grip bolts and nuts

Table 4.4 shows typical dimensional relations of high tensile friction grip bolts with hexagonal head for sizes ranging from 12 to 39 mm and Table 4.5 gives preferred length-diameter combinations. Table 4.6 gives dimensions for high-tensile friction grip nuts. Similarly, Table 4.7 gives the dimensions for plain washers. The various dimensions stated in these Tables are marked in Fig. 4.4. Sometimes, square taper washers may also be used, the details about which may be obtained from IS : 3757-1966.

TABLE 4.4 DIMENSIONS FOR HIGH TENSILE FRICTION GRIP BOLTS (IS : 3757–1972)

Size		*M12*	*M16*	*M20*	*M22*	*M24*	*M27*	*M30*	*M33*	*M36*	*M39*
d		12	16	20	22	24	27	30	33	36	39
d_a		15.2	19.2	24.4	26.4	28.4	32.4	35.4	38.4	42.4	45.4
d_1	*Min*	20	25	30	34	39	44	48	53	58	63
b	*	30	38	46	50	54	60	66	72	78	84
	†	–	44	52	56	60	66	72	78	84	90
c	*Nom.*	0.4	0.6	0.8	0.8	0.8	0.8	1.0	1.0	1.0	1.0
e	*Min.*	23.91	29.56	35.03	39.55	45.20	50.85	55.37	60.79	66.44	72.09
k (*js16*)	*Nom.*	8	10	13	14	15	17	19	21	23	25
	Max.	8.45	10.45	13.55	14.55	15.55	17.55	19.65	21.65	23.65	25.65
	Min.	7.55	9.55	12.45	13.45	14.45	16.45	18.35	20.35	22.35	24.35
r		0.6	0.6	0.8	0.8	0.8	1.0	1.0	1.0	1.0	1.0
s (*h15*)	*Nom.*	22	27	32	36	41	46	50	55	60	65
	Max.	22.00	27.00	32.00	36.00	41.00	46.00	50.00	55.00	60.00	65.00
	Min.	21.16	26.16	31.00	35.00	40.00	45.00	49.00	53.80	58.80	63.80

Note 1. — For the dimension l see Table 4.5

Note 2. — The dimension d_1 shall not exceed the actual width across the flat.

Note 3. — Sizes shown in brackets are of second preference.

* For lengths up to 130 mm.

† For lengths over 130 mm up to 200 mm.

Designation of bolts, nuts and washers

High tensile friction grip bolts are available in two symbols: 10 K symbol and 8G symbol depending upon the mechanical properties of the material used. The bolt is designated by the *size, length* and *symbol* representing the mechanical properties and also the IS *number*. For example, a friction grip bolt of size M16 and length 100 mm, conforming to the mechanical properties of 10K are designated as: Friction grip bolt M16 × 100 IS : 3757-10K

Similarly, the nuts and washers are designated by size, symbol representing the mechanical properties (*i.e.* 10 K or 8 G) and the IS number. For example, a friction grip nut of size M20, conforming to the mechanical properties of 10 K is designated as : Friction grip nut M20 IS : 3757-10K

Similarly, a plain washer of size 21, conforming to mechanical properties of 10K is designated as:

Plain washer 21 IS : 3757-10K

The high tensile friction grip bolts are commonly abbreviated as HTFG bolts.

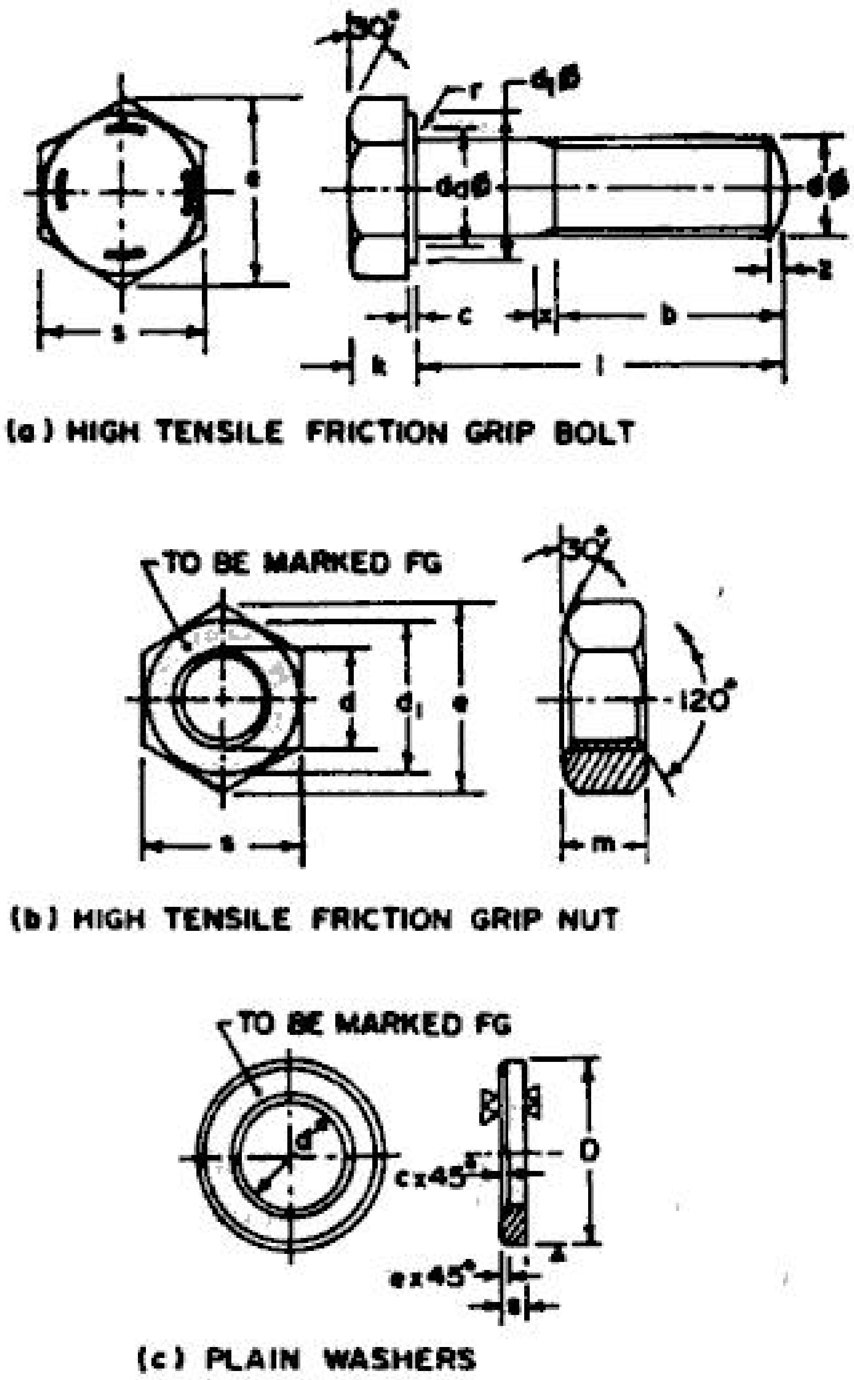


FIG. 4.4. HTFG BOLTS, NUTS AND WASHERS.

TABLE 4.5. PREFERRED LENGTH-DIAMETER COMBINATIONS FOR HIGH TENSILE FRICTION GRIP BOLTS (IS : 3757-1972)

l (mm)	*M 12*	*M 16*	*M 20*	*M 22*	*M 24*	*M 27*	*M 30*	*M 33*	*M 36*	*M 39*
30	✓									
35	✓									
40	✓	✓								
45	✓	✓								
50	✓	✓								
55	✓	✓	✓							
60	✓	✓	✓	✓						
65	✓	✓	✓	✓	✓					
70	✓	✓	✓	✓	✓	✓				
75		✓	✓	✓	✓	✓				
80		✓	✓	✓	✓	✓	✓			
90		✓	✓	✓	✓	✓	✓			
100		✓	✓	✓	✓	✓	✓	✓	✓	✓
110			✓	✓	✓	✓	✓	✓	✓	✓
120			✓	✓	✓	✓	✓	✓	✓	✓
130			✓	✓	✓	✓	✓	✓	✓	✓
140					✓	✓	✓	✓	✓	✓
150					✓	✓	✓	✓	✓	✓
160						✓	✓	✓	✓	✓
170							✓	✓	✓	✓
180							✓	✓	✓	✓
190								✓	✓	✓
200								✓	✓	✓

Note :- Preferred lengths are given between the bold lines :

TABLE 4.6 DIMENSIONS FOR HIGH-TENSILE FRICTION GRIP NUTS

d		*M12*	*M16*	*M20*	*M22*	*M24*	*M27*	*M30*	*M33*	*M36*	*M39*
d_1	*Min*	20	25	30	34	39	44	48	53	58	63
s (h15)	*Nom.*	22	27	32	36	41	46	50	55	60	65
	Max.	22.00	27.00	32.00	36.00	41.00	46.00	50.00	55.00	60.00	65.00
	Min.	21.16	26.16	31.00	35.00	40.00	45.00	49.00	53.80	58.80	63.80
c	*Max.*	25.4	31.2	36.9	41.6	47.3	53.1	57.7	63.5	69.3	75.0
m (j16)	*Nom.*	10	13	16	18	19	22	24	26	29	31
	Max.	10.45	13.55	16.55	18.55	19.65	22.65	24.65	26.65	29.65	31.80
	Min.	9.55	12.45	15.45	17.45	18.35	21.35	23.35	25.35	28.35	30.20

Note — The dimension d_1 shall not exceed the actual width across the flat.

TABLE 4.7 DIMENSIONS FOR PLAIN WASHERS.

d Nom.		13	17	21	23	25	28	31	34	37	40
For bolt size		M12	M16	M20	M22	M24	M27	M30	M33	M36	M39
D	*Nom.*	24	30	36	40	44	50	55	60	65	71
c	*Max.*	1.9	1.9	1.9	2.5	2.5	3.0	3.0	3.4	3.4	3.4
	Min.	1.6	1.6	1.6	2	2	2.5	2.5	2.8	2.8	2.8
s	*Nom.*	3	4	4	4	4	5	5	5	5	5
e	*Nom.*	0.5	1	1	1	1	1	1	1	1	1

4.6. PROOF LOADS OF HTFG BOLTS

Table 4.8 gives the proof loads for 10K and 8G bolts of various diameters. The values originally given in kg have been converted into kN.

TABLE 4.8 PROOF LOADS OF HTFG BOLTS (IS : 3757-1966)

Bolt dia (mm)	*Proof Load*			
	10 K Bolts		8 G Bolts	
	kg (f)	kN	kg (f)	kN
12	5900	57.9	5058	49.6
14	8050	78.9	7500	73.5
16	10790	105.8	9120	89.4
18	13440	131.8	11520	113.0
20	17150	168.2	14700	144.1
22	21210	208.0	18180	178.3
24	23710	232.5	21180	207.7
27	32130	315.1	27450	269.2
30	39270	385.1	33660	330.1
33	48580	476.4	41640	408.3
36	57190	560.8	49020	480.7
39	68320	669.9	58560	574.2

Note : The proof load of 10 K bolt is based on 70 kg/mm^2 and that of 8G bolt is based on 60 kg/mm^2 on the stress area of the bolt.

4.7. TYPES OF BOLTED CONNECTIONS

Connections serve primarily to transmit load from or to intersecting members. The simplest form of bolted connection is the ordinary *lap joint* (Fig. 4.5 *a*). Some joints in structures are of this general type, but it is not a commonly used detail due to the tendency of the connected members to deform. A more common type of connection is the *butt joint* shown in Fig. 4.5(*b*). Other commonly used bolted connections are shown in Fig. 4.6. Fig. 4.7 shows some of the common types of bolted building connections.

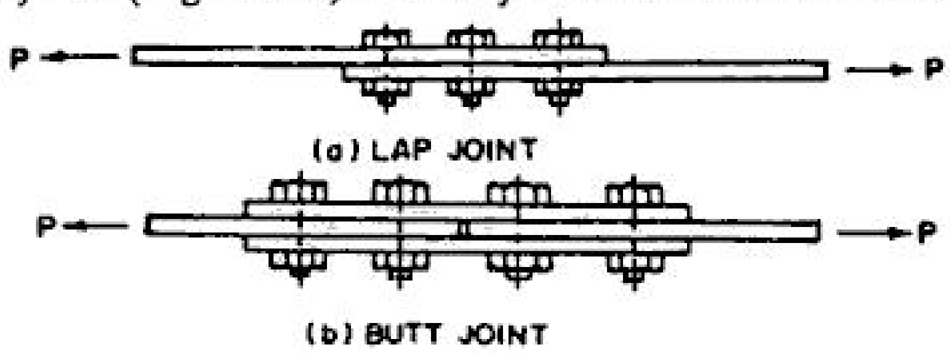


FIG. 4.5. TYPES OF JOINTS.

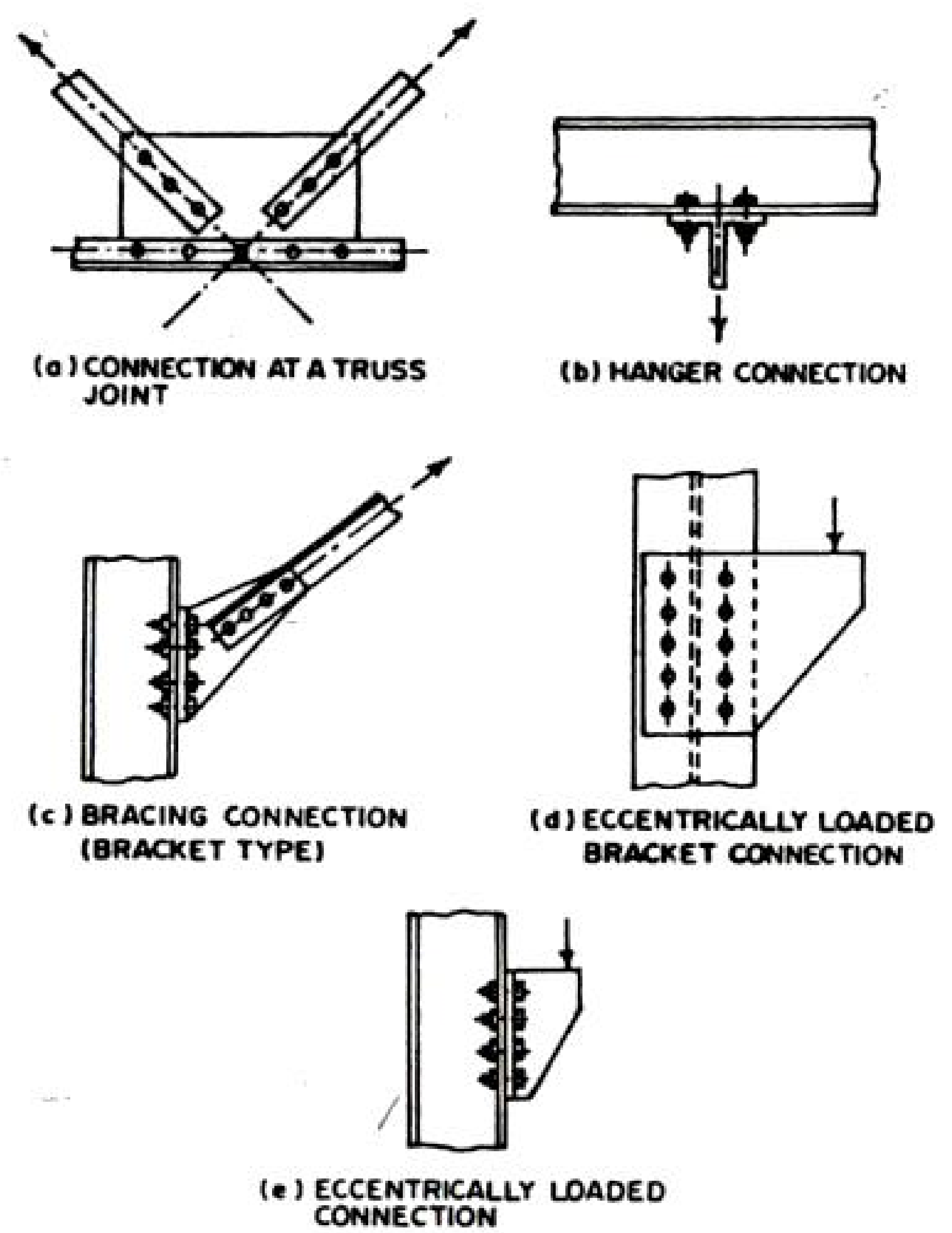


FIG. 4.6. COMMON TYPES OF BOLTED CONNECTIONS.

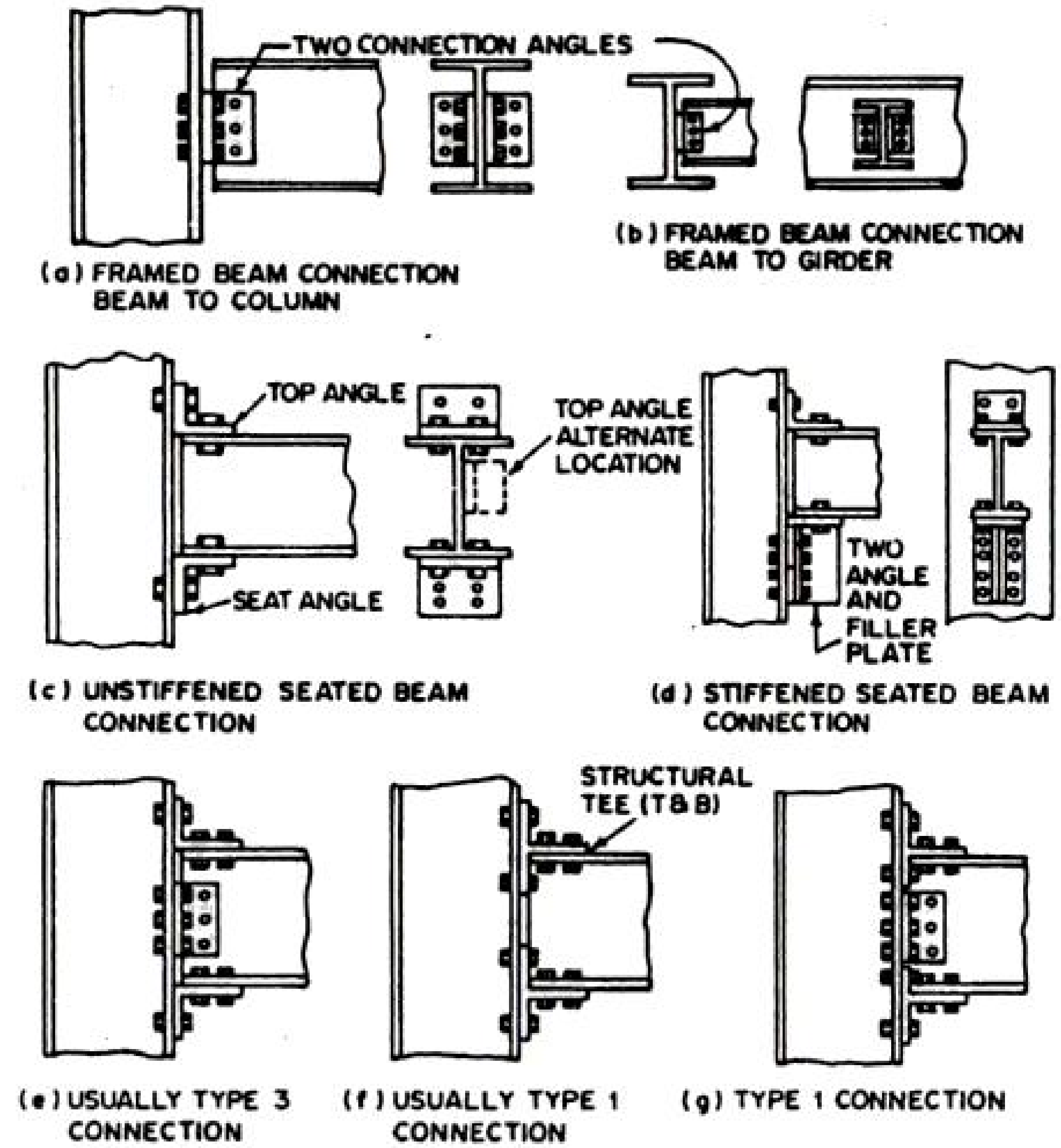


FIG. 4.7. COMMON TYPES OF BOLTED BUILDING CONNECTIONS.

4.8. DESIGN OF BOLTED SHEAR CONNECTION

From Eq. 4.1, we have

$$P = \mu T \qquad ...(i)$$

If there are n interfaces, we have

$$P = n \mu T \qquad ...(ii)$$

Dividing this by a factor of safety (F), we have

$$\text{Shear per bolt} = \frac{\mu}{F} \times n \times T \qquad ...(4.2)$$

$$\therefore \quad \text{Shear per bolt} = \frac{\text{Slip factor}}{\text{Factor of safety}} \times \text{ No. of interfaces} \times \text{proof load.}$$

where

shear per bolt = Contribution of one bolt to the total shear strength of the joint
Slip factor = Coefficient of friction = 0.45
T = Proof load = Maximum permissible load in the bolt, as given in Table 4.8

The total number of effective interfaces is determined by common contact surfaces between adjacent load transmitting structural members with forces in opposite directions; this is, excluding packing pieces through which the bolt passes.

The factor of safety (F) is taken as 1.4 for all loads except wind load in which case in may be reduced to 1.2. The factor of safety allows for the stress relaxation in the bolts which may be of the order of 10 percent. Research has proved that stress relaxation in the bolt occurs mostly during first few days after the bolt is tightened.

From Eq. 4.2, it is clear that greater the number of interfaces, smaller is the value of the required proof load corresponding to a given shear. This could mean that smaller diameter bolt will be required if the number of interfaces are more, and vice-versa.

The various types of shear connections considered in this chapter are

(*i*) Lap joint (*ii*) Butt joint with double cover plates (*iii*) Moment connection
(*iv*) Flange plate connection (*v*) Bracket connection

Example 4.1. ***Bolted Lap Joint***

Design a doubly bolted lap joint for plates 16 mm thick to carry its full load. Take permissible axial tension in plate 0.6 f_y where f_y= 250 N/mm².

Solution

Load carried by the plate per pitch length

$$= \sigma_{at} \times p \times t$$
$$= (0.6 \times 250)\, p \times 16 = 2400\, p \text{ N}$$

Fig. 4.8 shows the lap joint. There is only *one interface* to transmit shear. Since there are two bolts per pitch length, the load carried by each bolt $= \frac{1}{2} \times 2400\, p\, n = 1200\, p$ N

Hence from Eq. 4.2, taking $n = 1$,

$$1200\, p = \frac{\mu}{F} \times n \times T$$

$$\therefore \quad T = \text{proof load}$$
$$= \frac{(1200\,p)\,F}{\mu\, n} = \frac{1200\,p\,(1.4)}{0.45 \times 1}$$
$$= 3733.33\, p$$

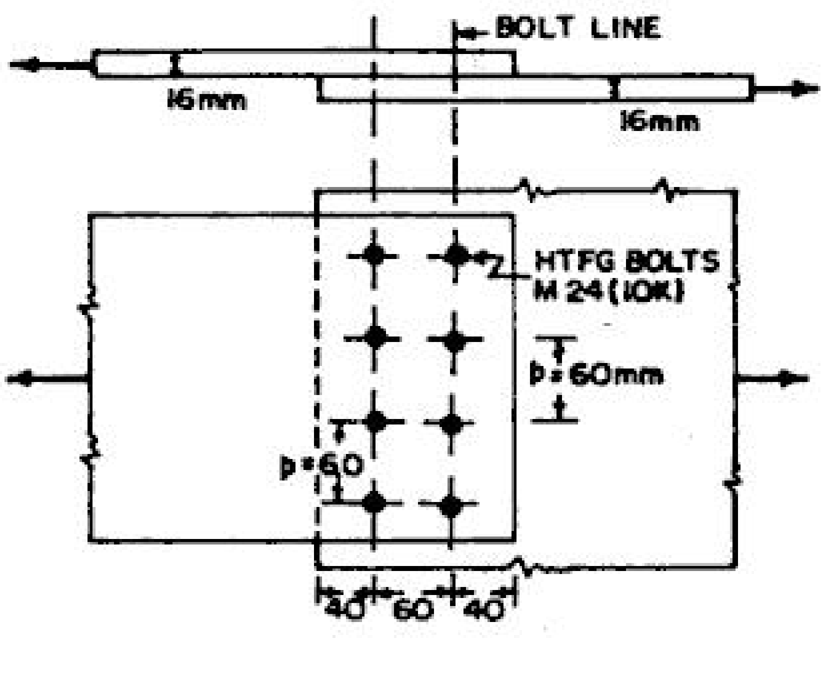


FIG. 4.8.

Providing bolts at 60 mm pitch,

$$\text{Proof load, } T = 3733.33 \times 60 = 224000 \text{ N } = 224 \text{ kN.}$$

From Table 4.8, we get $T = 232.5$ kN for 24 mm dia. 10K bolt. Hence provide HTFG bolts M24 (10K) with a pitch of 60 mm and edge distance of 40 mm.

Example 4.2. ***Bolted butt joint with double cover plates***

Redesign the joint of Example 4.1, using butt joint with double cover plates.

Solution

In the case of butt joint with double cover plates, shown in Fig. 4.9, there will be *two interfaces* (*i.e* $n = 2$) for each bolt.

Hence we have

$$1200\,p = \frac{\mu}{F} \times n \times T = \frac{0.45}{1.4} \times 2 \times T$$

From which

$$T = \frac{(1200\,p) \times 1.4}{0.45 \times 2} = 1866.67\,p$$

Thus we find that the proof load T has been reduced to half, because of two interfaces per bolt. This will require lesser value of bolt dia. and, inturn, less value of pitch p.

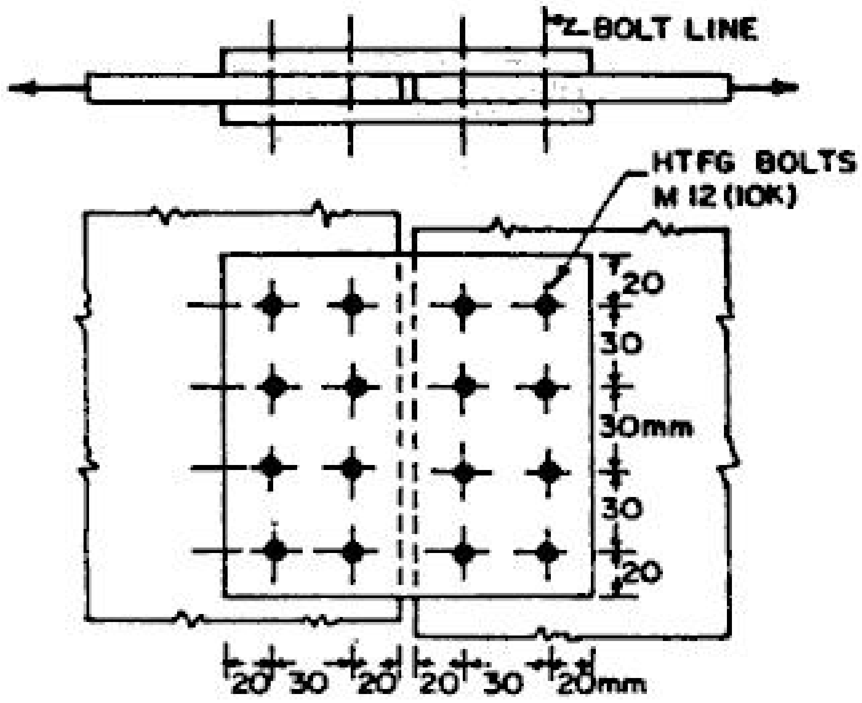


FIG. 4.9.

Keeping $p = 30$ mm, we get

$$T = 1866.67 \times 30 = 56000 \text{ N } = 56 \text{ kN}$$

Hence provide HTFG bolts M12 (10K) at a pitch of 30 mm and a side distance of 20 mm. The joint is shown in Fig. 4.9.

Example 4.3. ***Bolted Moment Connection***

A top column of section ISHB 400 @ 77.4 kg/m transmits a moment of 5 kN-m to the bottom column of section ISHB 450 @ 87.2 kg/m. Design the moment connection.

Solution

It is assumed that the direct load is transmitted from the upper column to the lower column by *web connection* (shear connection). The moment of 5 kN-m will be transmitted through *flange connection*, providing bolts in the flanges.

$$\text{Shear on the bolts at each flange} = \frac{M}{h} = \frac{5 \times 10^3}{400} = 12.5 \text{ kN}$$

The bolts are in single shear, *i.e.* there will be one interface only at each flange,

$$\therefore \qquad \frac{\mu}{F} \times T = P$$

$$\text{or} \qquad \frac{0.45}{1.4} \times \text{Proof load} = 12.5$$

$$\therefore \qquad \text{Proof load} = \frac{12.5 \times 1.4}{0.45} = 38.89 \text{ kN}$$

Referring to Table 4.8, select HTFG bolts M12 (8G) 4 Nos. on each flange as shown in Fig. 4.10.

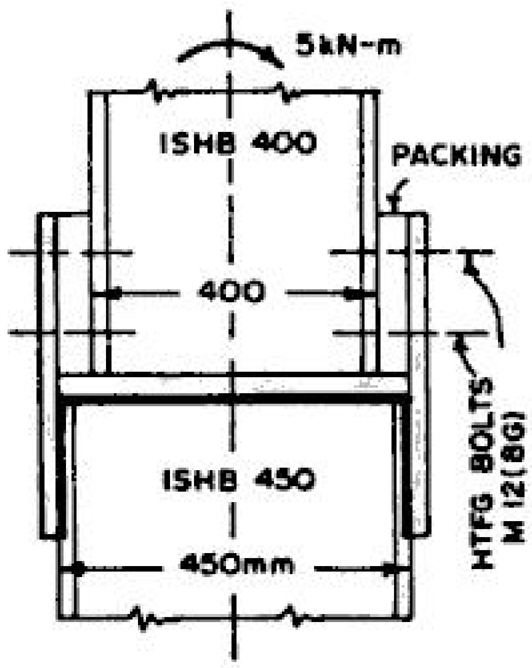


FIG. 4.10.

Example 4.4. *Flange bolts of a plate girder*

The section of a plate girder is shown in Fig. 4.11. The girder is subjected to a total shear of 3000 kN. Design the flange bolts for the plate girder.

Solution : Moment of inertia of the plate girder

$$= \frac{1}{12} \times 8\,(2000)^3 + 2\left[500 \times 40\,(1020)^2\right] = 4.7 \times 10^{10}\ \text{mm}^4$$

The longitudinal shear (q) per unit mm of the girder is given by

$$q = \frac{F}{I}(A\bar{y}) = \frac{3000}{4.7\times10^{10}}(500\times20\times1030)$$

$$= 0.657\ \text{kN/mm} \qquad ...(i)$$

Try HTFG bolt M16 (8G), for which proof load= 89.4 kN.

∴ Shear carrying capacity of a pair of bolts

$$= 2 \times \frac{0.45}{1.4} \times 89.4 = 57.47\ \text{kN} \qquad ...(ii)$$

$$\therefore \quad \text{Pitch} = \frac{57.47}{0.657} = 87.5\ \text{mm}.$$

Hence provide HTFG bolts M16 (8G) at 85 mm pitch.

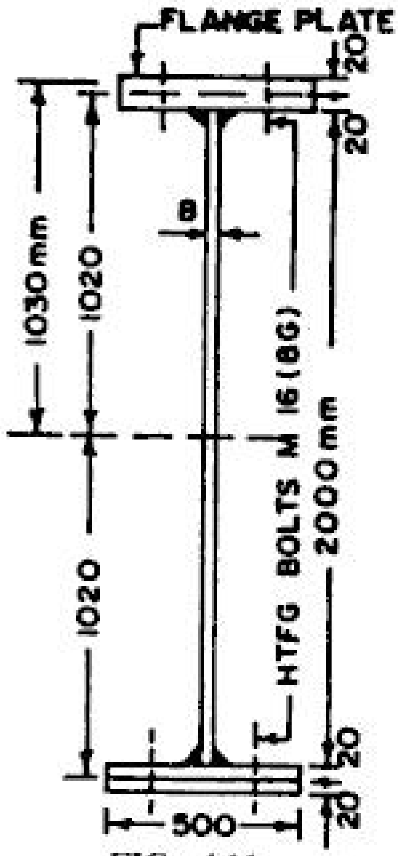


FIG. 4.11.

Example 4.5. *Bolted Bracket Connection : Moment Connection*

Fig. 4.12 shows a bracket, attached to a column, supporting a gantry girder. Design the bolted bracket connection, if the bracket carries a load of 300 kN at a lever arm of 400 mm.

Solution.

This is a typical case of shear and moment acting in the plane of the joint. The bolts will be subjected to (i) direct shear acting vertically downwards, and (ii) shear due to B.M., acting perpendicular to the radius vector. Assume the bolt pattern as shown in Fig. 4.12 (b) and (c), having 7 bolts in each of the two rows, at a pitch of 60 mm and edge distance of 40 mm. Load on each bracket $= \frac{1}{2} \times 300 = 150$ kN.

Direct shear $P_1 = \dfrac{150}{14} = 10.71$ kN

B.M. $M = P.e = 150 \times 400$

$= 60000$ kN-mm

From Eq. 3.16, we find that shear due to moment M is

$$P_2 = \frac{M \cdot r}{\Sigma r^2}$$

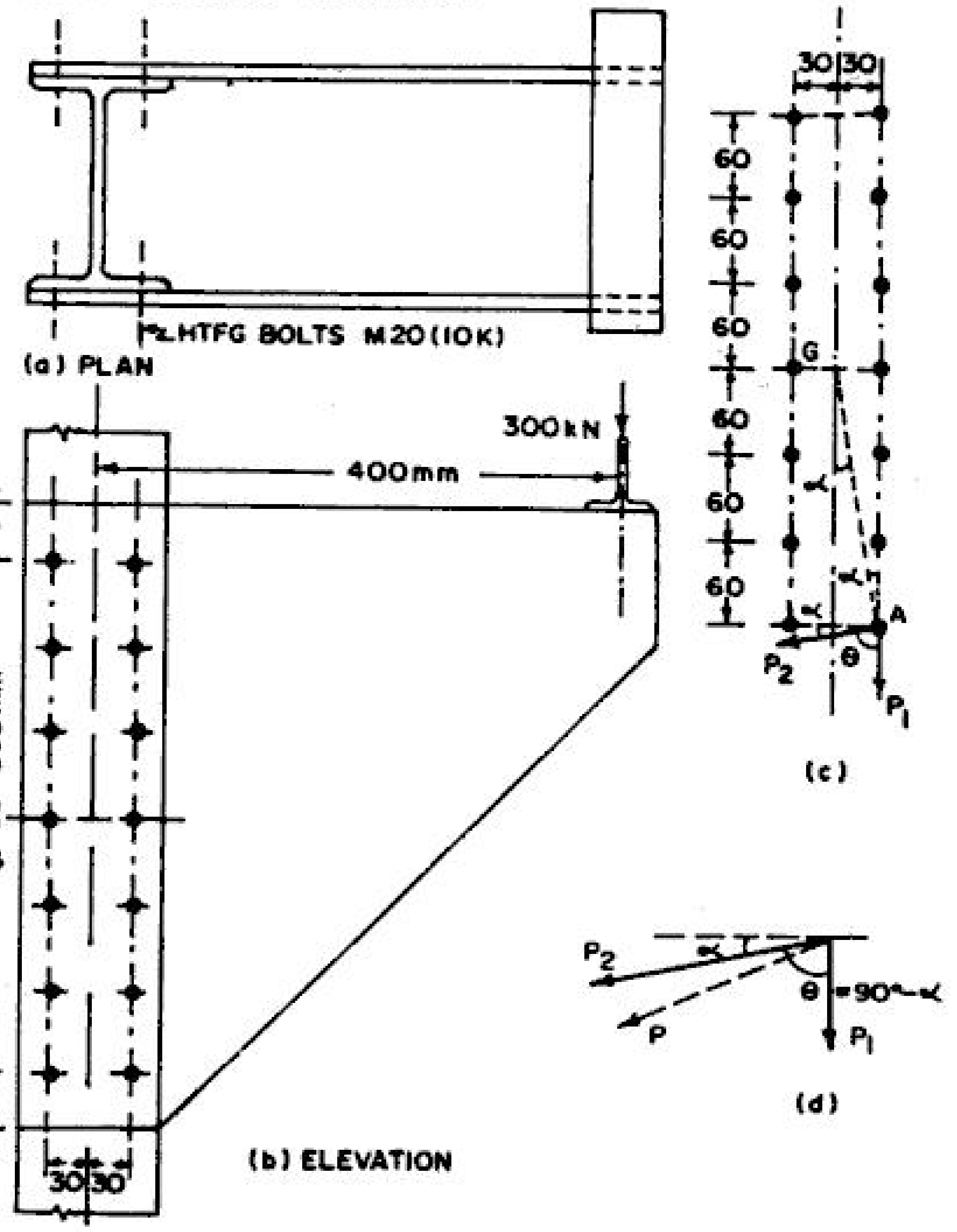


FIG. 4.12. BOLTED BRACKET CONNECTION.

where $\Sigma r^2 = \Sigma (x^2+y^2) = 4\left[(30)^2+(60)^2+(30)^2+(120)^2+(30)^2+(180)^2\right]+2\left[(30)^2\right] = 214200 \text{ mm}^2$

$$r \text{ for bolt } A = \sqrt{(30)^2 + (180)^2} = 182.48 \text{ mm}$$

$$\therefore \quad P_2 = \frac{60000 \times 182.48}{214200} = 51.11 \text{ kN}$$

$$\cos\alpha = \frac{180}{182.48} = 0.9864; \quad \sin\alpha = \frac{30}{182.48} = 0.1644$$

$$\therefore \quad P_{2V} = P_2 \sin\alpha = 51.11 \times 0.1644 = 8.403 \text{ kN}$$

$$P_{2H} = P_2 \cos\alpha = 51.11 \times 0.9864 = 50.415 \text{ kN}$$

$$\therefore \quad \text{Resultant shear } P = \sqrt{(P_1 + P_2 \sin\alpha)^2 + (P_2 \cos\alpha)^2} = \sqrt{(10.71 + 8.403)^2 + (50.415)^2}$$

$$= 53.92 \text{ kN}$$

Alternatively, $P = \sqrt{P_1^2+P_2^2 + 2 P_1 P_2 \cos\theta}$, where $\cos\theta = \sin\alpha = 0.1644$

$$= \sqrt{(10.71)^2+(51.11)^2+2\times10.71\times51.11\times0.1644} = 53.92 \text{ kN}$$

But from Eq. 4.2, $P = \frac{\mu n}{F} \times \text{Proof load}$ or $53.92 = \frac{0.45 \times 1}{1.4} \times \text{Proof load}$

From which, Proof load $= \frac{53.92 \times 1.4}{0.45} = 167.74$ kN

Choose HTFG bolts M20 (10 K) having proof load of 168.2 kN.

4.9. BOLTS SUBJECTED TO EXTERNAL TENSION

We have seen that strength of a joint with HTFG bolts is developed as a result of the clamping force induced in the joint. This clamping force is caused by the tension in the shank of the bolt tightened to its proof load. In some joints, such as the joint at the dead end of a pressure vessel or the upper part of the joint of a column bracket, the external load acting parallel to the shank of the bolts increases tension in the bolt, thus reducing the clamping action.

Fig. 4.13 (*a*) shows the end connection of a pressure vessel. When there is no internal pressure, the clamping force between the flange and the flange plate is *fully effective*. As the internal pressure increases, the flanges plate is pushed away from the flange introducing an additional tension on the bolt shank. This (additional) external tension reduces the clamping force. As the external tension increases, the clamping force decreases, till a stage comes when the clamping force may become zero. On further increase in the external ten-

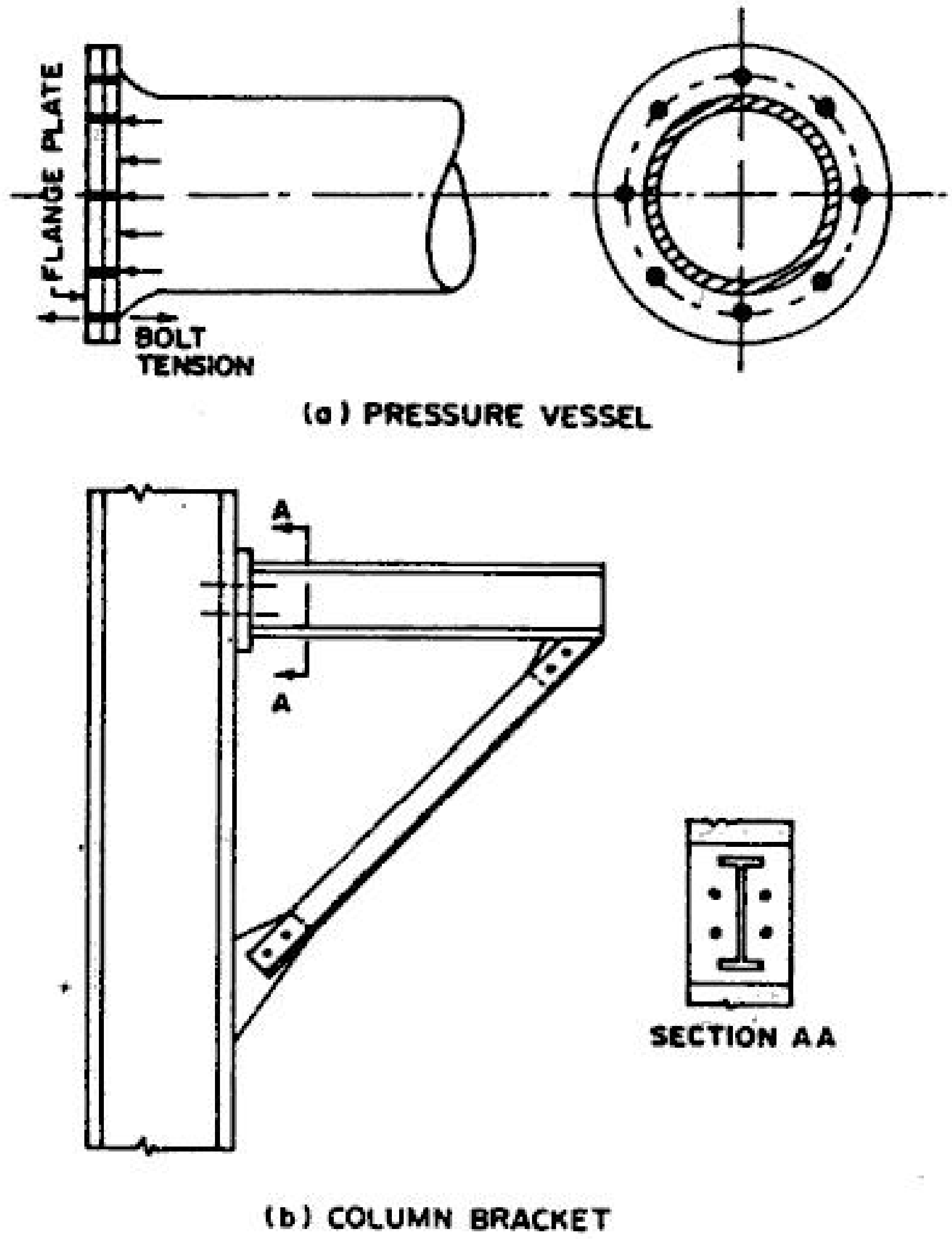


FIG. 4.13. EXTERNAL PRESSURE ON HTFG BOLTS

sion, *separation* of interfaces will take place. It is, therefore, very essential to take care to avoid separation of interfaces.

To ensure that the clamping force is not fully neutralized by the external tension to avoid separation of interface, the code (IS : 4000-1967) limits the external tension to 0.6 times the proof load and to 0.5 of the proof load when the joint is subjected to fatigue.

Example 4.6. *Bracket Connection under tensile load*

Fig. 4.14 (a) shows a bracket connection, carrying 144 kN load at a lever arm of 1200 mm. Design the bolted connection of the horizontal member with the flange of the column.

Solution

Since the inclined member of the bracket has inclination of 45°, the horizontal member (ISMB 200) will be subjected to a tensile force of 144 kN. Due to this, the bolts (Fig. 4.14 *b*) of the joint are subjected to *pure tension.*

Let us provide 4 HTFG bolts

$$\text{Tension per bolt} = \frac{144}{4} = 36 \text{ kN}$$

The bolts carry a non-repetitive tensile load.

$\therefore$ 0.6 Proof load = external tension = 36 kN

$$\therefore \quad \text{Proof load} = \frac{36}{0.6} = 60 \text{ kN}$$

Hence provide 4 Nos. HTFG bolts M14 (8G) bolts, each having a proof load of 73.5 kN.

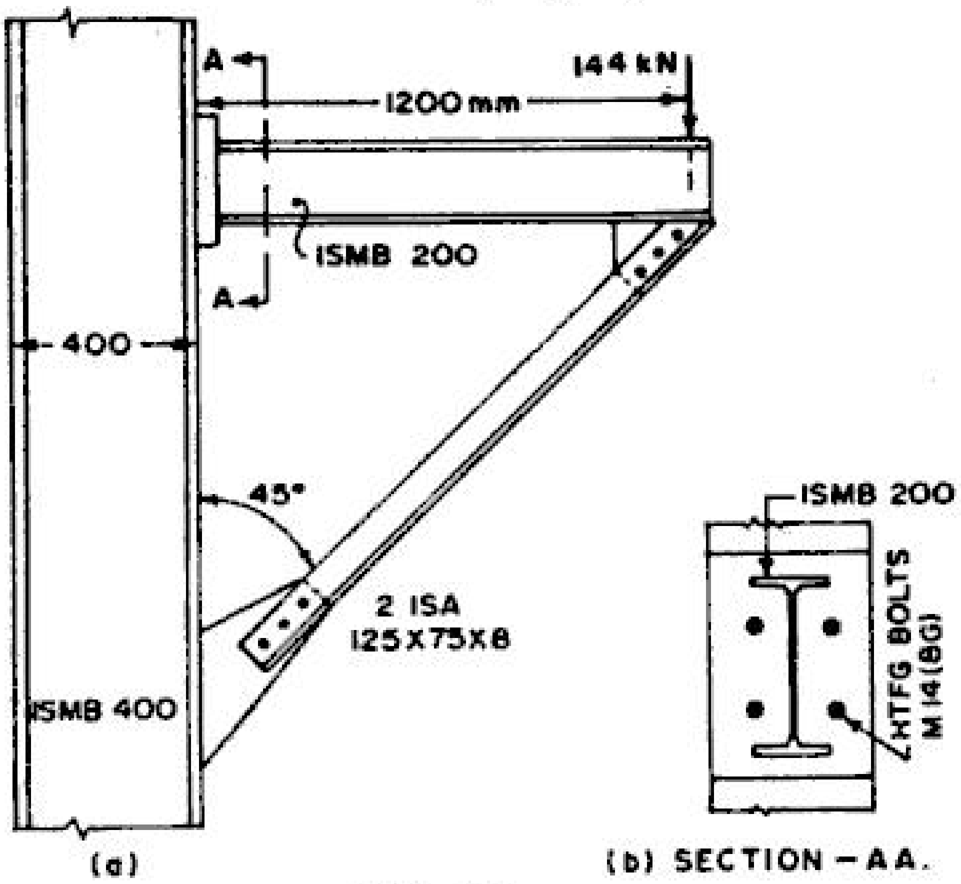


FIG. 4.14.

Example 4.7. *Bolted Flange Connection of a Gas Cylinder*

Design a bolted flange connection of a gas cylinder of internal diameter 900 mm, in which the internal pressure varies from 0.4 N/mm² to 0.6 N/mm²

Solution : Total maximum pressure of gas on the flange

$$= \text{Area of flange} \times \text{max. pressure} = \frac{\pi}{4}(900)^2 \times 0.6 = 381704 \text{ N}$$

The bolts are subjected to repetitive tension, since the pressure fluctuates from 0.4 to 0.6 N/mm². Hence the maximum external tension is limited to 0.5 times the Proof load.

$\therefore$ 0.5 × total proof load = 381704

From which, total proof load

$$= \frac{381704}{0.5} = 763408 \text{ N}$$

$\approx$ 763.41 kN

Providing HTFG bolts M16 (10 K) with a proof load of 105.8 kN,

$$\text{No. of bolts} = \frac{763.41}{105.8} = 7.22$$

Hence provide 8 bolts.

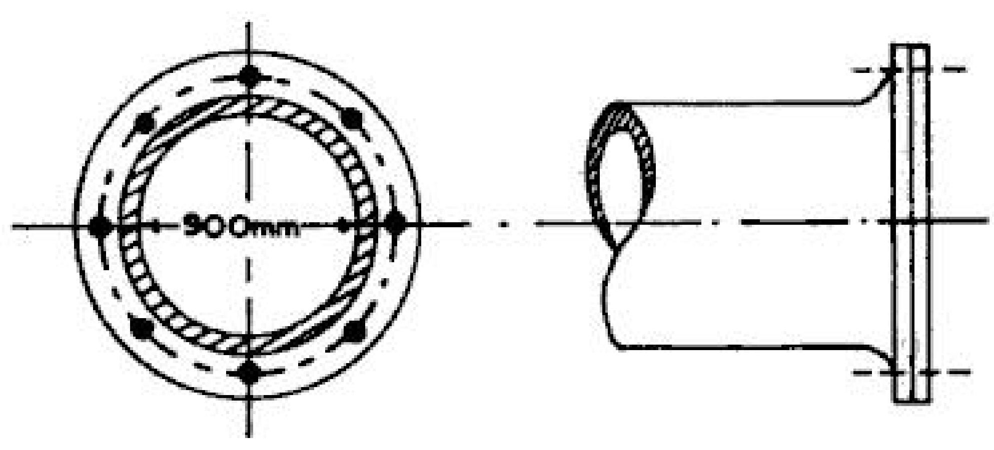


FIG. 4.15. FLANGE CONNECTION

4.10. COMBINED SHEAR AND TENSION

Connections subjected to combined shear and tension are commonly found in joints connecting columns and beams, brackets, stringer beams to floor beams etc. Experiments conducted for various combinations of tension and shear indicate that the curve for the relationship between shear and tension at failure is elliptical of the nature :

$$\frac{x^2}{k^2} + y^2 = 1 \qquad \text{...(4.3)}$$

where x = ratio of the calculated shear on the bolt to the shear at failure
y = ratio of the calculated tension on the bolt to the tension at failure
k = a constant dependent on the strength of bolt

Value of k is found to be 0.83 if the bolt fails in shank and 0.64 if the bolt fails in threads.

The ultimate strength of the bolt given by Eq. 4.3 increases with the grip of the bolt as the effect of shear loading on the shank bending becomes predominant. The above equation indicates the behaviour of bolts at failure, and, therefore, is not easily applicable to the normal design purposes. Clause 4.5 of IS : 4000-1967 gives the following simple formula, in the original version of the code:

$$\frac{\text{Calculated shear}}{\text{slip factor} \times \text{No. of interfaces}} + \text{calculated tension} \times F = \frac{\text{Proof Load}}{\text{Factor of safety}}$$

or

$$\frac{P}{\mu\, n} + P_T \,.\, F = \frac{\text{Proof Load}}{1.4} \qquad \text{...(4.4)}$$

where F = 1.2 when the tension is non-repetitive, or
= 1.43 when the tension is repetitive.
P_T = Calculated tension.

However, an amendment to the clause 4.5 of IS : 4000 was issued in October 1975. The amended clause is reproduced here:

Clause 4.5 (Amended) : Connections subject to External Tension in addition to Shear

An externally applied tension in the direction of bolt axes reduces the effective clamping action of a bolt which has been tightened to induce shank tension. To allow for this effect, the bolts shall be proportioned to satisfy the expression :

$$\frac{\text{Calculated shear}}{\text{Slip factor} \times n} \le \frac{\text{Proof load} - \text{Calculated tension} \times F}{\text{Factor of safety}}$$

or

$$\frac{P}{\mu \,.\, n} \le \frac{\text{Proof load} - P_T \,.\, F}{1.4} \qquad \text{...(4.5)}$$

The value of F shall be taken as 2.0 if the external tension is repetitive and 1.7 if it is non-repetitive. It will be seen that Eq. 4.4 and 4.5 are identical, if proper value of F is selected with each formula.

Example 4.8. ***Bolted bracket connection to flange of column***

Fig. 4.16 shows a bracket, bolted to the flange of a column section. Design the connection.

Solution

As a first trial, choose No. of bolts given by Eq. 3.22 :

$$n = \sqrt{\frac{6M}{l\,p\,R}} \qquad \text{...(4.6)}$$

where

R = Shear per bolt

$= \frac{\mu}{F} \times$ proof load.

l = No. of rows = 2

p = Pitch

$M = 200 \times 250$

$= 50000$ kN-mm

n = No. of bolts per line

Here, there are 3 unknowns : R, p and n.

Taking M 16 (10K) bolts, proof load = 105.8 kN, and providing these at p = 40 mm, we get,

R = Shear per bolt

$= \frac{\mu}{F} \times$ Proof load

$= \frac{0.45 \times 105.8}{1.4}$

$= 34.007$ kN

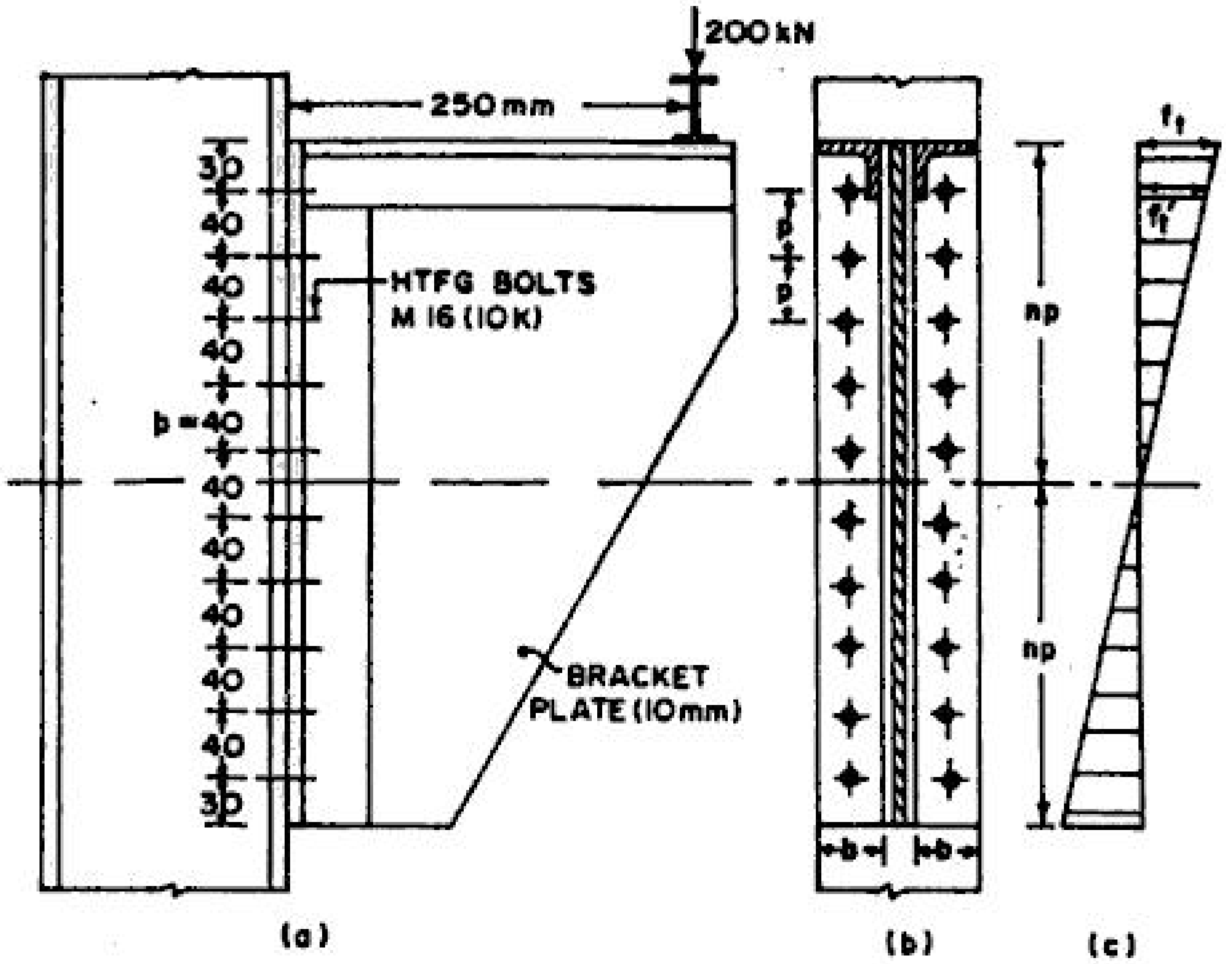


FIG. 4.16.

$$\therefore \qquad n = \sqrt{\frac{6 \times 50000}{2 \times 40 \times 34.007}} = 10.5$$

Provide 10 bolts in each row, at a pitch of 40 mm and edge distance of 30 mm, and arrange these as shown in Fig. 4.16.

Bending stress (tensile or compressive), in extreme fibre is

$$f = \frac{M}{I} y = \frac{M}{\frac{1}{12}(lb)(np)^3} \times \frac{np}{2}$$

or
$$f = \frac{6M}{l\,b(n\,p)^2} \qquad ...(4.7)$$

Effective tensile force P_T in the extreme rivet is

$$P_T = f \,.\, b\,p = \frac{6M}{l\,p\,n^2} \qquad ...(4.8)$$

This is an approximate expression for the tension in the extreme bolt which is assumed to be situated at $p/2$ (= 20 mm) from top fibre, though it is actually located at 30 mm from the top fibre.

$$\therefore \qquad P_T = \frac{6 \times 50000}{2 \times 40\,(10)^2} = 37.5 \text{ kN} \qquad ...(1)$$

$$\text{Direct shear } P = \frac{200}{2 \times 10} = 10 \text{ kN} \qquad ...(2)$$

From Eq. 4.5,

$$\frac{P}{\mu N} \leq \frac{\text{Proof load} - P_T F}{1.4} \qquad ...(4.5\ a)$$

Taking $N = 1$, and $F = 1.7$ (for non-repetitive load), we get

$$\text{Proof load} = \frac{1.4\,P}{\mu} + P_T F = \frac{1.4 \times 10}{0.45} + 37.5 \times 1.7 = 31.11 + 63.75 = 94.86 \text{ kN}$$

This is within the actual proof load (=105.8 kN) for M 16 (10K) bolts. Hence OK.

4.11. INSTALLATION OF HTFG BOLTS

The performance of a high strength friction grip (HTFG) bolt depends on the proper installation of the bolt. As indicated earlier, the characteristic of the friction grip joint depends on the proper clamping force induced by the high tensile bolts. This should correspond to the proof load according to the grade and the stress area of the bolt and should be within the permissible limits of variation. The permissible limits have not been specified in IS : 4000-1967 as the measurement of tension in all the bolts by direct method is not feasible at site.

HTFG bolts may be tightened by any of the following methods:

(*a*) Indirect methods :

(*i*) Torque control method (*ii*) Part turn method

(*b*) Direct methods : Direct tension indicator methods

(*i*) Load indicating bolt (*ii*) Load indicating washer.

The indirect methods achieve the desired effect with a variation of ± 15 to 30%. The direct method give results within ± 6 percent.

1. Torque control method : The method is based on the linear relationship between the torque necessary to turn the bolt and induced tension in the bolt upto the proof load. After the proof load, there is very little increase in the tension with the applied torque.

The relationship between the torque and the tension may be expressed as :

$$M = K\,T\,D \qquad \text{...(4.9)}$$

where M = applied torque

K = a non-dimensional torque coefficient

T = proof load

and D = nominal diameter of the bolt.

The constant K is a function of the inclination of the threads, coefficient of friction at the threads and also at the nut and the washer. The value of K may be taken as 0.15 if the bolt is slightly lubricated and as 0.25 if it is rusty.

Spanners called *torque limiting spanners* or the hand wrenches and impact wrenches working on the air pressure are used to apply the predetermined torque to the bolt. They are so designed that they are unable to apply any more torque than the preset value.

This limiting value is set by the use of the formula and then calibrated to give a more precise setting. This is done by tightening a sample bolt against a load cell or similar apparatus. The load cell is designed to show the direct tension induced in the bolt. The torque spanner is adjusted to produce a tension 10 percent higher than the proof load of the bolt . The bolt used for calibration is not to be used elsewhere as a friction grip bolt. Such calibration is made once or more per shift depending on the work load. It is also necessary when the condition of the bolt, size, length, grade, etc are changed. It may, therefore, be necessary to keep differently calibrated equipment ready for speedy work.

2. Part turn method : What is essentially required in a friction grip joint is preload in the bolt and the accompanying elongation of the bolt. This may be achieved by turning the nut after the bolt is made just tight. The method based on this principle is called *part turn method.* In this method the interfaces are cleaned of all dirt, burrs, loose milliscale etc. by using suitable tools. The members forming the joint are then carefully brought together without using any force or making undue deformation. The interfaces are again inspected to see whether any loose particles like that of sand, metal etc are lodged within or not. The bolts are then introduced and subjected to preliminary tightening so that they are just tight and it becomes impossible to turn them any further by fingers. If power operated wrenches are used, they are set to 5% of the total torque as required by the torque control method. This 5 percent is taken as a *just tight bolt condition* before giving the final turn as specified.

After this mark is made on the nut and the protruding thread portion of the bolt to record the initial position, the nut is tightened so that the relative turn corresponds to that specified in Table 4.9.

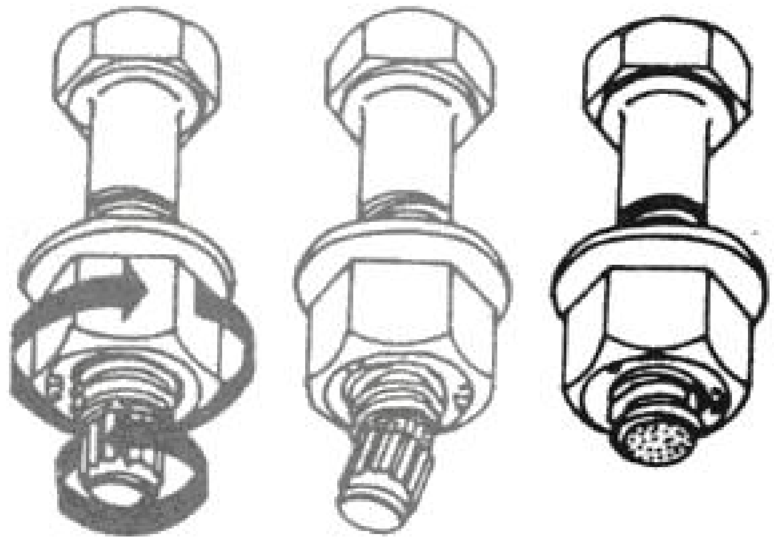
TABLE 4.9 FINAL TIGHTENING OF HTFG BOLTS (All dimension in mm)

Norminal diameter of bolt	*Grip of bolt for rotation of the nut (relative to the bolt shank)*	
	Not less than 1/2 turn	*Not less than 3/4 turn*
M 16	upto 114	—
M 20	upto 114	over 114 to 216
M 22	upto 114	over 114 to 280
M 24	upto 165	over 165 to 360
M 27	upto 165	over 165 to 360
M 33	upto 165	over 165 to 360

As will be seen, this method does not require any sophisticated tools to achieve a preload as ordinary spanners may be used for tighening. The method gives freedom to enginners on site to choose any lubricant or the condition of the bolt without affecting the final result. The inspection is also made easy as this needs to check the final position of the marks previously made on the nuts and the bolts. However, the efficiency of this method very much depends on the initial condition of the joint before giving the final turn.

3. Load indicating bolt : This is developed by GKN Bolts and Nuts Ltd. U.K. and also by Bethlehem Steel Corporation. In the GKN bolts, the head is so shaped that before tightening, it makes contact with the steel with its four corners only. This leaves a gap between the steel and the underside of the bolt head. As the bolt is tightened, the part of the bolt head yields and causes a gradual closure of this gap. The experiments show that to ensure a correct bolt load, it would only be necessary in practice to make sure that the widths of the gaps under the head were reduced to less than 0.8 mm. This may be done visually or by using a simple *feeler gauge.*

The load indicator bolt by Bethlehem Steel Corporation has a special feature in the form of adding a spline to the end of the bolt. A special wrench, by virtue of its construction, grasps both the nut and the spline, applying a clockwise turning force to the nut and a counter clockwise force to the spline (Fig. 4.17 *a*). When the fastener assembly reaches a proper torque, the tension is in excess of the specified minimum tension and the wrench will twist off the spline end. (Fig. 4.17 *b*). Inspection is quick; if the spline has been twisted off (Fig. 4.17 *c*), the tension is adequate.



(a) (b) (c)

FIG. 4.17. LOAD INDICATOR BOLT (BETHLEHEM STEEL CORPORATION)

4. Load indicating washer : This special washer is developed by Cooper and Turner Ltd U.K., and also by Bethlehem Steel Corporation. Fig. 4.18 shows the load indicator washer by Bethlehem Steel Corporation. The round hardened washer has a series of protusions on one face. The washer is usually inserted between the bolt head and the gripped plates with protusions bearing against the underside of the bolt head, leaving a gap. Upon tightening, the protusions are partially flatenned and the gap is reduced. Bolt tension is evaluated by measurement of the gap closure. When the gap is reduced to a prescribed dimension (say, 0.4 mm), the bolt has been properly tightened. The gap can be measured by a feeler gauge. If the gap is inadvertantly closed to less than that, it is to be allowed to remain so and on no account adjusted by slackening the nut.

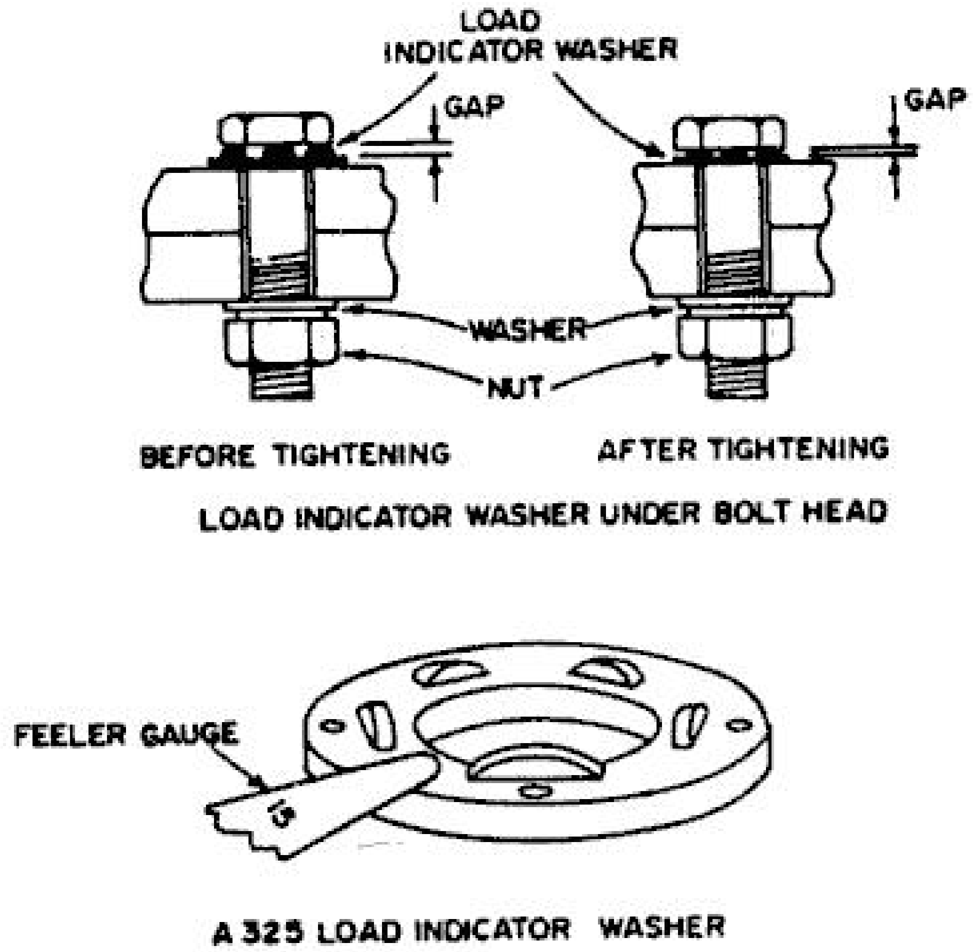


FIG. 4.18 LOAD INDICATOR WASHER (BETHLEHEM STEEL CORPORATION)

4.12. PINNED CONNECTIONS

Pinned connection is used to connect the members which are required to rotate relative to each other. Pins for structural purposes are cylindrical in shape, and are made of structural carbon steel, forged and machined to accurate dimensions. The size of pins may range from 9 mm dia. used for connecting strap iron bars to railway bridge pins 300 mm or more in diameter. The end of the one bar is forged in the form of a fork, with a hole drilled in it, while the end of the other bar is forged and an eye is made with a hole drilled in it. The eye bars can be inserted within the jaws of the fork, and a pin is inserted through the holes in them, thus producing the joint, as shown in Fig. 4.19 (*a*). Another form of pinned connection is shown in Fig. 4.19 (*b*.), where eye bars are formed at the ends of the both members to be jointed. A pin has also got the same function to perform as rivets or bolts, but since only one pin can do so, the diameter of the pin is always much larger than that of bolts or rivets. The pin is assumed to turn freely in the connection; therefore *clamping action due to initial tension is undesirable.*

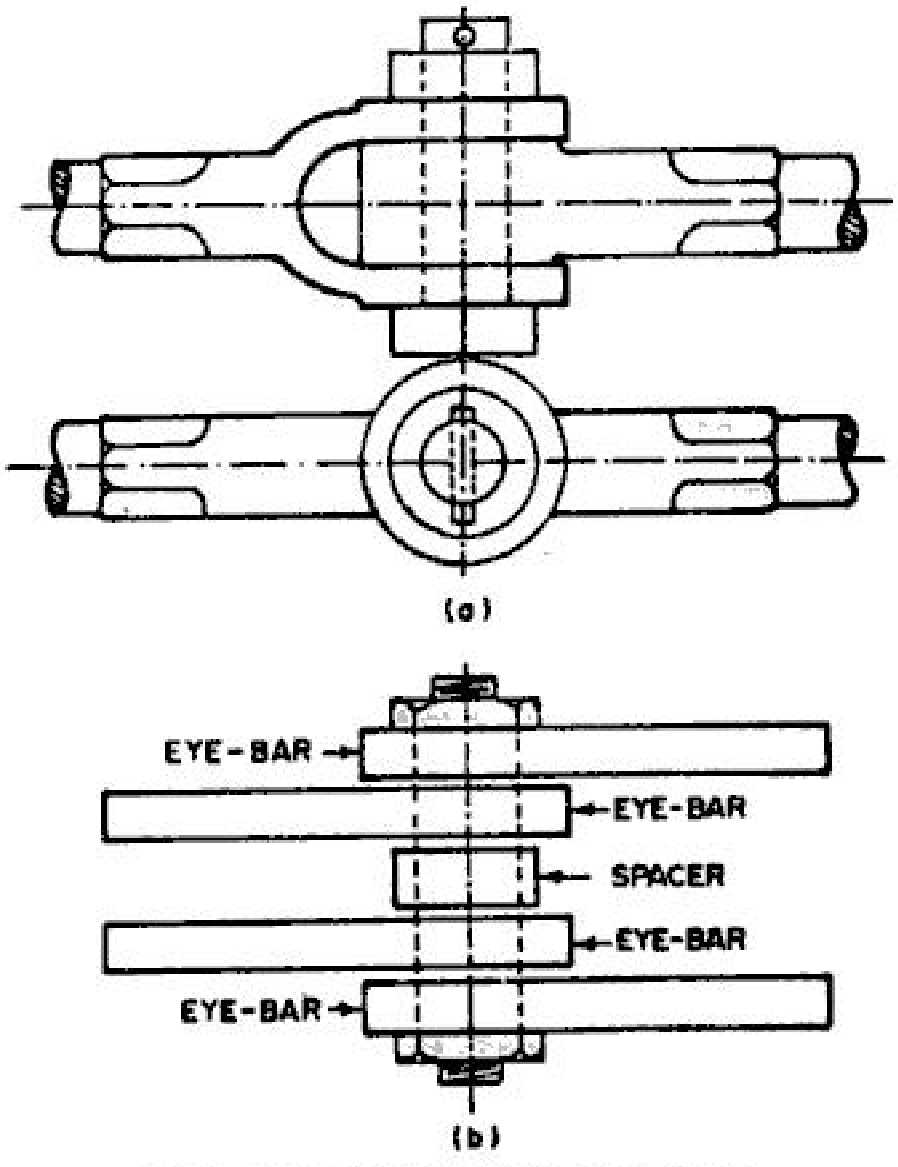


FIG. 4.19. PINNED CONNECTIONS

Types of pins : Fig. 4.20 shows most common types of pins. For smaller pins carrying light loads, a head is forged at one end and a *cotter pin* inserted at the other, as shown in Fig. 4.20 (*a*); alternatively two cotters may be used, one at each end. Fig. 4.20 (*b*) shows the so called *undrilled pin* having threaded ends and two recessed nuts screwed on the ends to hold the pin in place. Fig. 4.20 (*c*) shows a larger diameter pin (>250 mm), having long bolt passing through the recessed cap and the pin. Such a pin is known as *drilled pin*, in which the bolt holds the nuts and the pin together. Such an arrangement eliminates the use of large locknuts.

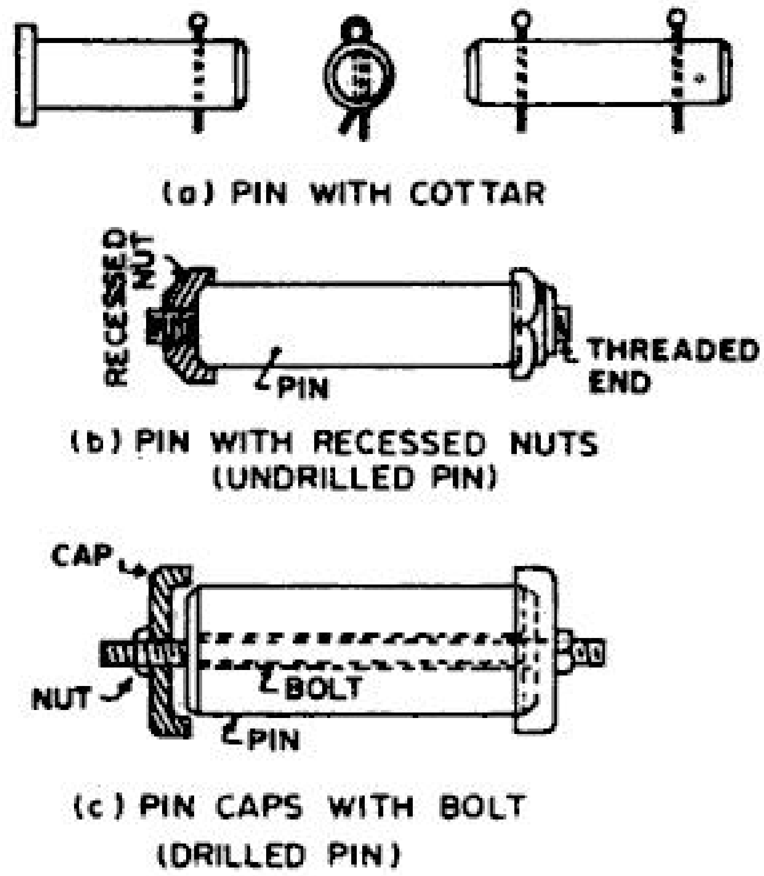


FIG. 4.20. TYPES OF PINS

Structural Uses of Pins : Pins may be divided into two categrories: (*i*) those on which the connecting members turn through large angles, such as in machine parts, crane booms etc. and (*ii*) those on which the connected members turn only through small angles, primarily due to the elastic deflections/deformations of the members. Pins are used in the following types of structures :

(*i*) Bascule bridges
(*ii*) Crane booms
(*iii*) Arch hinges
(*iv*) Diagonal bracings
(*v*) Columns supporting bridge girders of large lengths
(*vi*) Towers of suspension bridges
(*viii*) Hinge plates in cantilever bridges
(*viii*) Expansion joints
(*ix*) Rocker supports

Advantages and Disadvantages of pin connections

The analysis of a structure with pinned connection is very simple since the moment at pin-connection is zero. Members are free to rotate as per the requirements of elastic deflections/deformations. Also, they allow for thermal expansion/contraction. Secondary stresses to some extent are eliminated in a pinned joint. However, such a joint lacks rigidity and such joints make a loose noisy bridge under traffic vibrations. Hence they should be used only for very long spans and heavy loads. Pins cannot resist longitudinal tension. The longitudinal tension produces friction which in turn prevents free turning of pins. Pin holes at the ends of jointed members require expansive machine work.

Stresses in Pins : Pins are subjected to the following types of stresses

(*i*) Bending stresses
(*ii*) Shearing stresses
(*iii*) Bearing stresses

*1. **Bending Stresses***. In order to facilitate proper lubrication and free rotation, connected members are assembled on the pin with ample clearance between them, thus requiring longer pins, as shown in Fig. 4.21 (*a*). Fig. 4.21(*b*) and (*c*) show B.M. diagrams for pins, with concentrated loads and assumed uniform loads respectively.

The maximum fibre stress the pin, in terms of max. B.M. (M) is given by

$$f_b = \frac{M\,(d/2)}{\pi\, d^4/64} = \frac{32\,M}{\pi\, d^3} = \frac{10.2\,M}{d^3} \qquad \text{...(4.10)}$$

The B.M. (M), in terms of maximum bending stress σ_b is given by

$$M = \frac{\pi}{32}\frac{d^3}{M}\sigma_b = \frac{\sigma_b\, d^3}{10.2} \qquad \text{...(4.10 a)}$$

2. ***Shearing stresses***

If V is the max. shear force, the shear stress τ_v in the pin is given by

$$\tau_{v,\,max} = \frac{V}{I\,.\,b}A\,\bar{y} = \frac{V(\pi\, d^2/8)\,(4\,d/6\,\pi)}{(\pi\, d^4/64)\,d} = \frac{16\,V}{3\,\pi\, d^2} = \frac{4}{3}\frac{V}{A} \qquad \text{...(4.11)}$$

or $$\tau_{v,\,max} = \frac{4}{3}\tau_{v,\,av}$$

where d is the diameter of the pin.

In the case of pins, the ratio of span to depth is very small. Infact, a pin is a deep beam and hence the shear stresses computed by the simple beam theory (Eq. 4.11) give erroneous results. Due to this reason, *nominal shear stress*, based on uniform (rectangular) stress distribution is commonly used in the design of pins, given by the following expression :

$$\tau_v = \frac{V}{A} \qquad \text{...(4.12)}$$

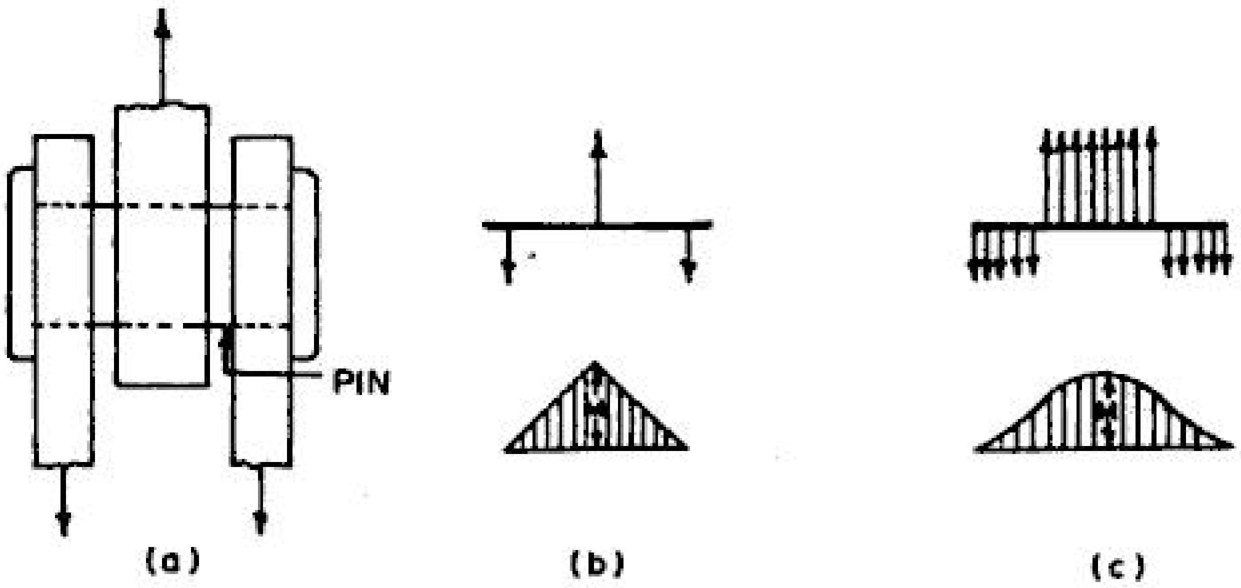


FIG. 4.21. BENDING MOMENT IN A PIN

3. ***Bearing Stresses*** : The bearing stresses depend upon the accuracy of fit between the plates and the pin. Assuming uniform distribution of bearing stress for proper fit, the nominal bearing stress (σ_{pa}) is given by

$$\sigma_{pa} = \frac{P}{t\,d} \qquad \text{...(4.13)}$$

where P = load carried by the joint
d = diameter of the pin
t = total thickness of plates bearing on the pin.

Allowable stresses in pins. The permissible shear and bearing stresses for a pin are taken the same as for a rivet. Similarly, the permissible bending stress is taken equal to 0.66 f_y. No longitudinal tensile stress is permitted. The permissible values are given in Table 4.10.

TABLE 4.10. PERMISSIBLE STRESSES IN PINS

	Type of stress	*Permissible value* N/mm^2 (MPa)
1.	Shear	100
2.	Bearing	300
3.	Bending	0.66 f_y

Example 4.9. *Design a pin to connect two pairs of parallel eye bars of size 180 × 50 mm. The space between the inner pairs is 90 mm apart. Take $f_y = 250\ N/mm^2$*

Solution

1. Pull transmitted by each eye bar

$= 180 \times 50\ (0.6 \times 250) \times 10^{-3} = 1350\ \text{kN}$

2. Max. B.M. at B is given by

$M = P \times 53 = 1350 \times 53 = 71550\ \text{kN-mm}$

$= 71.55 \times 10^6\ \text{N-mm}$

3. Max. shear force at B is

$V = P = 1350\ \text{kN}$

4. Required $Z = \dfrac{M}{\sigma_{bc}} = \dfrac{71.55 \times 10^6}{0.66 \times 250}$

$= 433636\ \text{mm}^3$

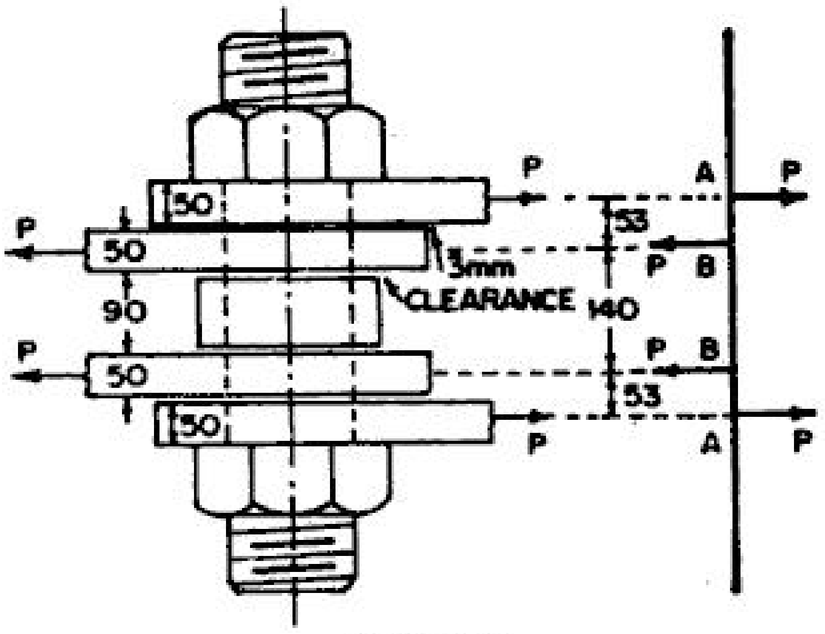


FIG. 4.22.

$$\therefore \qquad \frac{\pi}{32} d^3 = Z = 433636$$

From which $\qquad d = 164.1\ \text{mm}$

Provide 170 mm dia. pin.

5. *Check for shear* :

$$\tau_v = \frac{V}{A} = \frac{1350 \times 10^3}{\frac{\pi}{4}(170)^2} = 59.5\ \text{N/mm}^2$$

$< 100\ \text{N/mm}^2$. Hence safe.

6. *Check for bearing*

Bearing stress in pin, $\qquad \sigma_{pa} = \dfrac{P}{t\,d} = \dfrac{1350 \times 10^3}{50 \times 170}$

$= 158.8\ \text{N/mm}^2 < 300\ \text{N/mm}^2$.

Hence safe.

PROBLEMS

1. Write a detailed note on high tension friction grip bolted connection. What are the advantages of such a connection ?
2. Design a double bolted lap joint for a plate of 20 mm thickness to carry its full load.
3. Design a doubly riveted butt joint with double cover plates, for the data of problem 2.
4. Design a bolted bracket connection for the data of Example 3.13. Compare the two designs.
5. Design the double plate bracket with the data of Example 3.18.
6. Design the bracket connection with the data of Problem 3.11.

5

Steelwork Connections : III
Welded Connections

5.1. INTRODUCTION

Welding is a process of connecting pieces of metal by application of heat (fusion) with or without pressure. A metallic bond is established between the two pieces. This bond has the same mechanical and physical properties as the parent metal. A number of methods are used for the process of fusion. The oxyacetylene or gas welding and electric arc welding are the most important of these methods. The metal at the joint is melted by the heat generated from either an electric arc or an oxyacetylene flame and fuses with metal from a welding rod. After cooling, the parent metal (base metal) and the weld metal form a continuous and homogeneous joint. The welded connections have become so reliable that they are replacing riveted joints, both in structural as well as machine design.

There are numerous welding processes, shown in Table 5.1, but the one most commonly used in Civil Engineering structures is *electric-arc-welding*. In this process, heat is generated by an electric arc formed between a steel electrode and steel parts to be welded. The arc heat melts the base metal and the electrode simultaneously, and the electromagnetic field carries the molten metal of the welding rod (electrode) towards the base metal. Fusion takes place by the flow of material from the welding rod across the arc. No pressure is applied.

Arc welding process may be of three types: (*i*) shielded (*ii*) unshielded and (*iii*) submerged.

TABLE 5.1 WELDING PROCESS.

(*a*)		***Fusion welding***
	1.	Metal-arc welding : shielded, unshielded, submerged.
	2.	Carbon-arc welding : shielded, unshielded.
	3.	Inert-gas arc welding
	4.	Atomic-hydrogen arc welding
	5.	Gas welding (air or oxyacetelene)
	6.	Thermit welding
(*b*)		***Pressure Welding***
	1.	Forge welding
	2.	Pressure Thermit welding
	3.	Resistance welding (A-C)
	4.	Resistance welding (D-C)—seam and spot welding.

In the *shielded-metal arc welding* or the SMAW process (Fig. 5.1), the electrode is coated with certain mineral compounds which produce a gaseous 'shield' that helps to exclude oxygen and stabilize the arc. A part of the coating melts to form a fluid slag layer which rises to the top, retards the rate of cooling of weld metal, and also protects it from undesirable exposure to atmospheric gases.

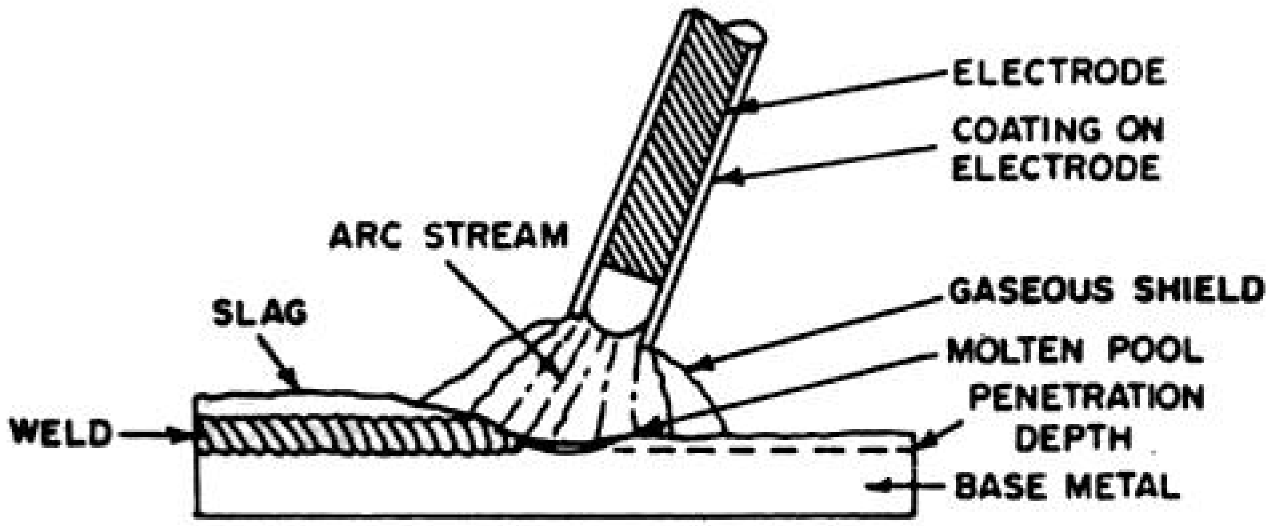


FIG. 5.1. SHIELDED-METAL ARC WELDING PROCESS

In the *non-shielded* metal arc welding, the electrode is uncoated, as shown in Fig. 5.2. Use of coated electrodes results in better quality welds than can be obtained with bare electrodes. Hence modern arc welding is done only with coated electrodes.

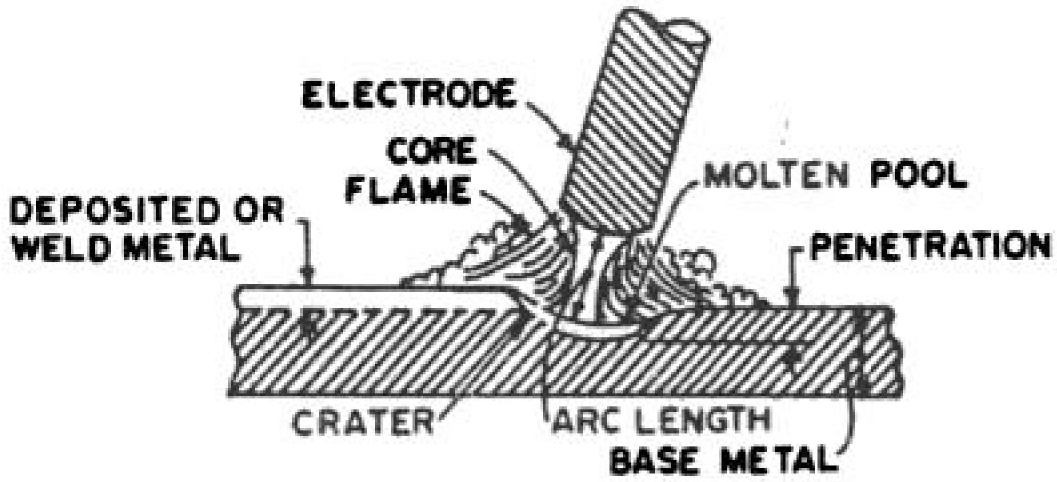


FIG. 5.2. UNSHIELDED METAL ARC WELDING PROCESS

In the *submerged-arc-welding process* or SAW process, the arc is 'submerged' or covered by a mound of fusible powdered flux, and the base electrode wire is fed mechanically from a reel. The arc, at all times, is covered by the flux, as shown in Fig. 5.3. The heat of the arc melts the parent metal, the electrode and part of the flux which forms a slag covering which can be removed later by brushing.

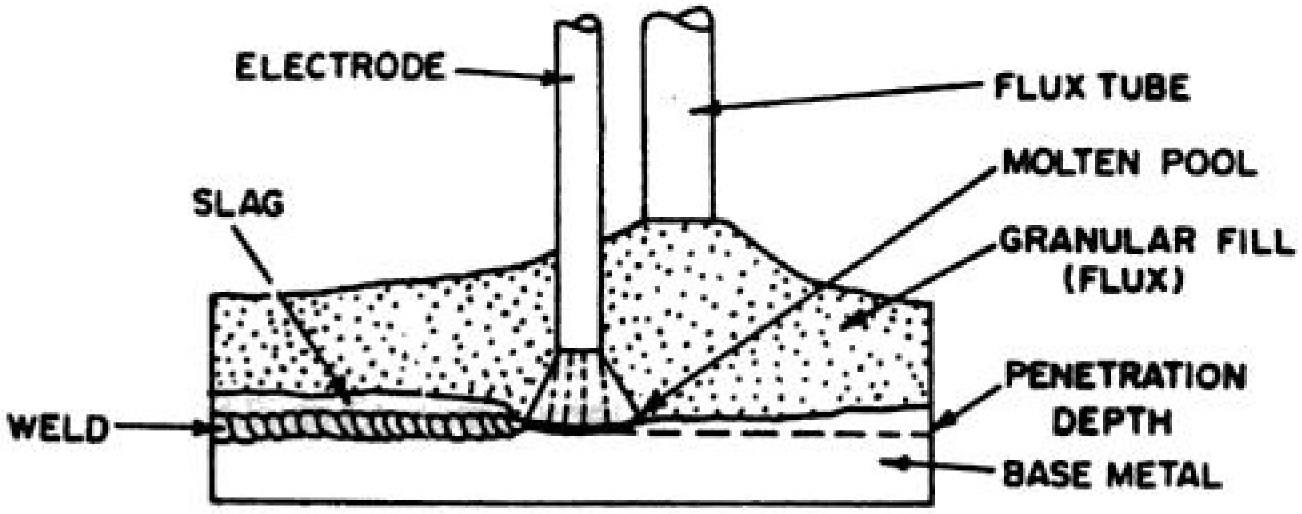


FIG. 5.3. SUBMERGED ARC WELDING PROCESS.

Gas welding is another popular welding process where heat is obtained from combustion of a gas fuel—commonly a mixture of oxygen and acetylene. The process is therefore also known as *oxy-acetelene welding*. The welding rod used for the process may be either shielded or unshielded.

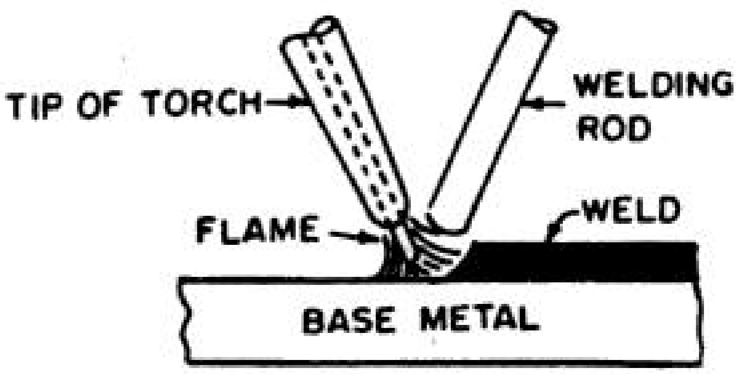


FIG. 5.4. GAS WELDING.

5.2. ADVANTAGES OF WELDING

Welded joints have the following advantages.

1. Welded joints are *economical* from the points of view of cost of labour and materials, both. The filler plates, gusset plates, connecting angles etc. are eliminated in welded joints. The smaller size of members, compared to those which may be used in riveted connections from the practical point of view, may be used here.

2. The *efficiency* of the welded joints is 100% as compared to an efficiency of 75% to 90% in case of riveted joints.

3. The *fabrication* of a complicated structure is *easier* by welded connection as in case of circular steel pipe. The alterations or additions in existing structures are facilitated by it.

4. The welding provides very *rigid joints*. This is in keeping with the modern trend of providing rigid frames.

5. The *noise* associated with the riveting work is a source of a great nuisance. There is *silence* in welding operation.

6. When riveting is done in populated localities, *safety* precautions to protect the public from flying rivets has to be taken. No such precautions are necessary in case of welding operation.

7. The welded structures look more *pleasing* in comparison to riveted one.

8. The welding work is done more *quicker* than the riveting work.

5.3. DISADVANTAGES OF WELDING

Notwithstanding the advantages narrated above, the welded connections have a number of disadvantages in comparison to the riveted connections. The same are narrated below :

1. No provision for expansion and contraction is kept in welded connection and therefore, there is possibility of *cracks* developing in such structures.

2. Due to uneven heating and cooling of the members during welding, the members may *distort* resulting in additional stress.

3. The *inspection* of welding work is more difficult and costlier than the riveting work.

4. The welding work requires a *skilled person* while semi- skilled person can do the riveting work.

5. On account of extreme heat, *fatigue* may take place.

6. There is a greater possibility of *brittle fracture* in welding than in riveting.

5.4. TYPES OF WELDS AND WELDED JOINTS

Welds are classified as follows.

(*a*) ***According to their position***: as (*i*) flat, (*ii*) horizontal, (*iii*) vertical and (*iv*) overhead (Fig. 5.5)

(*b*) ***According to type of weld*** : as (*i*) Butt weld (or groove weld), (*ii*) fillet weld (*iii*) slot weld, and (*iv*) plug weld (Fig. 5.5 and 5.6) and (*c*) ***According to type of joint*** : as (*i*) butt joint (*ii*) lap joint, (*iii*) tee joint, (*iv*) edge joint and (*v*) corner joint (Fig. 5.7)

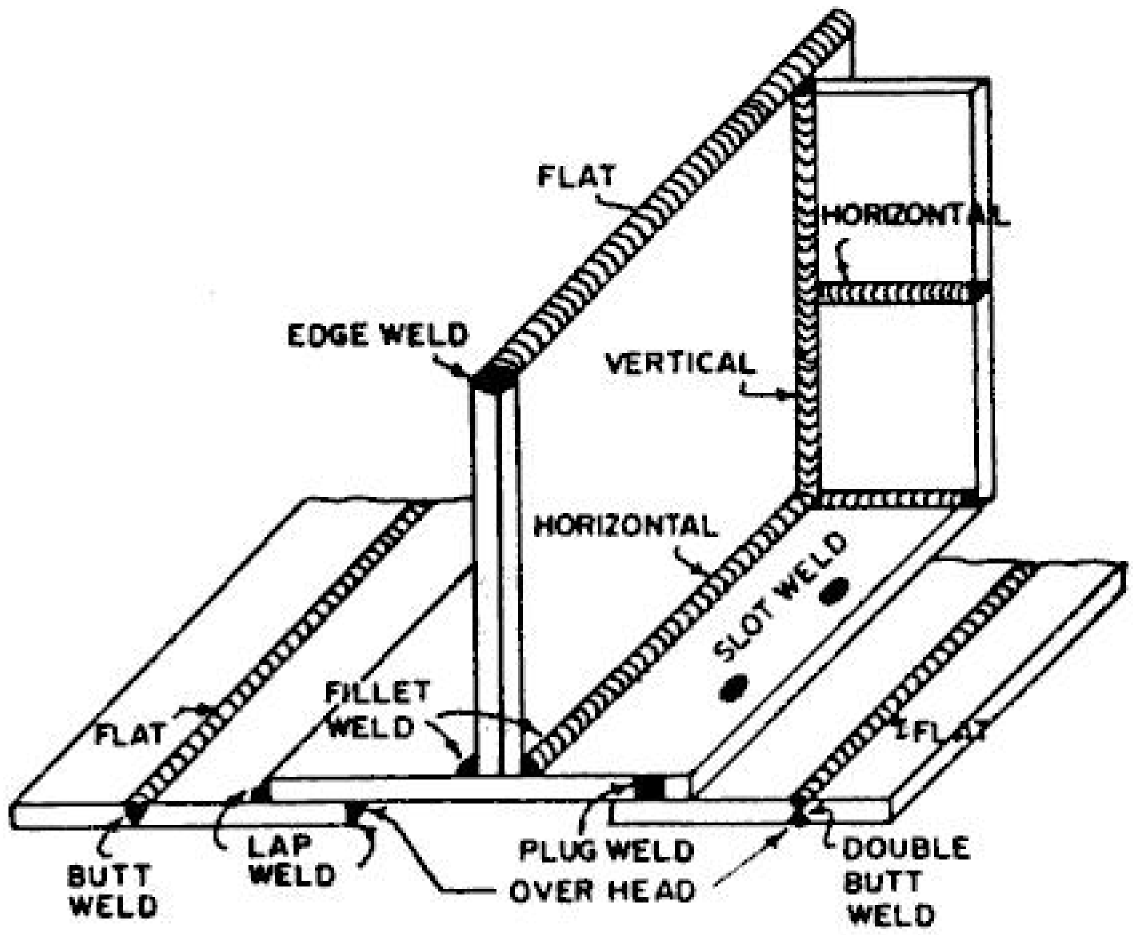


FIG. 5.5. PICTORIAL VIEW SHOWING VARIOUS TYPES OF WELDS.

Welding is commonly done in four *positions* : flat, horizontal, vertical and overhead, as shown in Fig. 5.5. Vertical and overhead welds are possible because molten metal is carried from the rod to the connected joint by the electromagnetic field of the arc and not by gravity. In the *flat* welding the direction of electromagnetic field is in the direction of gravity.

Butt weld or *grooved weld* is used when the plates or members to be connected are in the same plane, or when a T-joint is required, as shown in Fig. 5.6 (*a*). *Fillet welds* are used for lap joint (Fig. 5.7 *b*), T-joint (Fig. 5.7 *c*) or corner joint (Fig. 5.7 *e*). *Plug weld* (Fig. 5.6 *c*) and *slot weld* (Fig. 5.6 *d*) are used wherever sufficient space is not available for providing required length of fillet weld. The slot welds and plug welds are also used for equalizing stress in plates and to prevent buckling in case of wide plates.

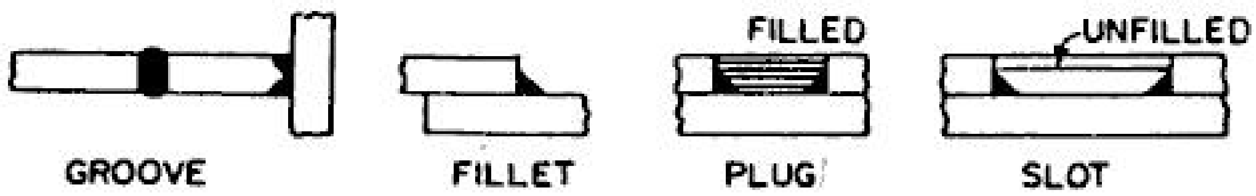


FIG. 5.6. TYPES OF WELDS

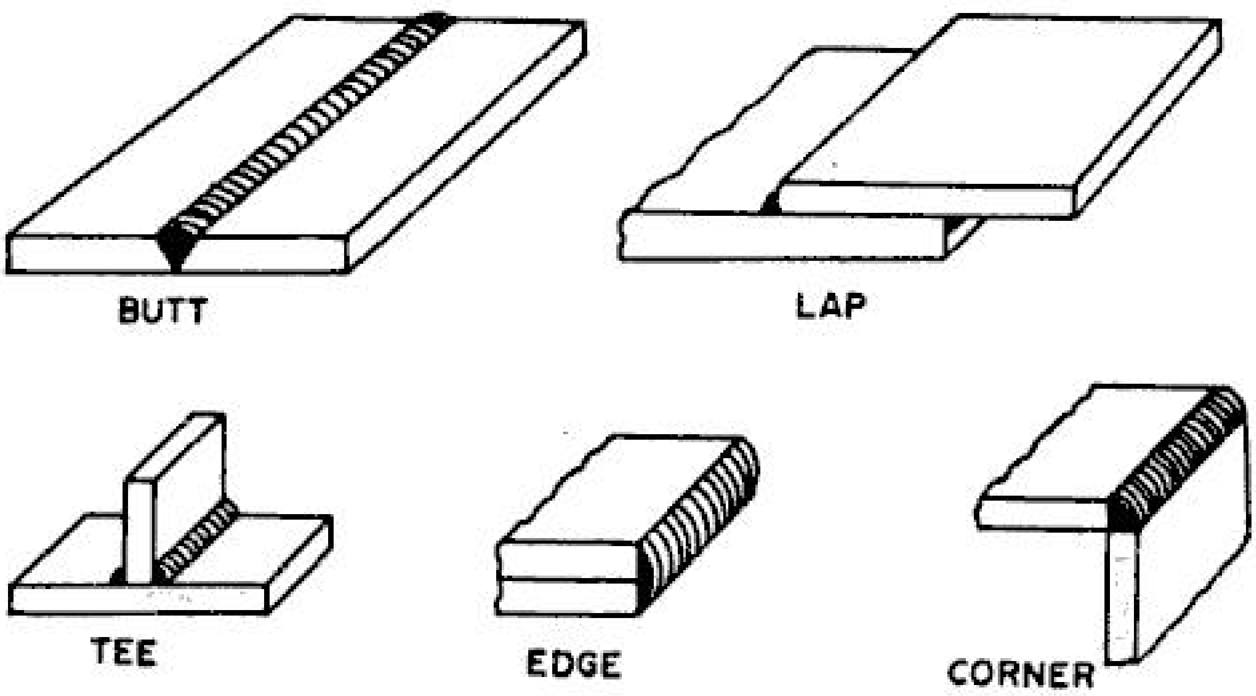


FIG. 5.7. TYPES OF WELDS

5.5. BUTT WELD OR GROOVE WELD

Butt weld or groove weld is used when the plates to be jointed are in the same plane, or when a T-joint is desired, as shown in Fig. 5.6 (*a*). A butt weld is designated according to the shape of groove made during the preparation of ends of the pieces to be jointed. The common types of butt welds are shown in Fig. 5.8.

A butt weld is specified by the size of the weld. The size of the butt weld is defined by the *effective throat thickness*. Reinforcement is the extra metal deposited proud of the surfaces of the pieces jointed, as shown in Fig. 5.9. The reinforcement may vary between 1 mm to 3 mm and is not included in the throat thickness.

The square butt joints shown in Fig. 5.8 (*a*) and (*b*) are used for thickness less than 8 mm. The *effective thickness* of the weld, called *throat thickness* (Fig. 5.9), is less than the thickness T of the plates jointed. It is taken as $\frac{3}{4}T$. In the single V-built joint (Fig. 5.8 *c*), the throat thickness is taken as $\frac{3}{4}T$. In double V-butt joint, the weld is fully effective and hence the throat thickness is taken equal to T. As a rule, in single U, single V and single J butt welds, where welding is done from one side, full penetration is not possible and hence effective throat thickness is taken equal to $\frac{3}{4}T$. In double-V, double U and double J butt welds, full penetration is possible and the effective thickness of throat is taken equal to the thickness of plates jointed. Whenever two plates of different thickness are jointed, the thickness of thinner plate must be taken into account.

Depending upon the *position*, butt welds may be (*i*) flat, (*ii*) vertical (*iii*) overhead and (*iv*) horizontal, as shown in Fig. 5.10.

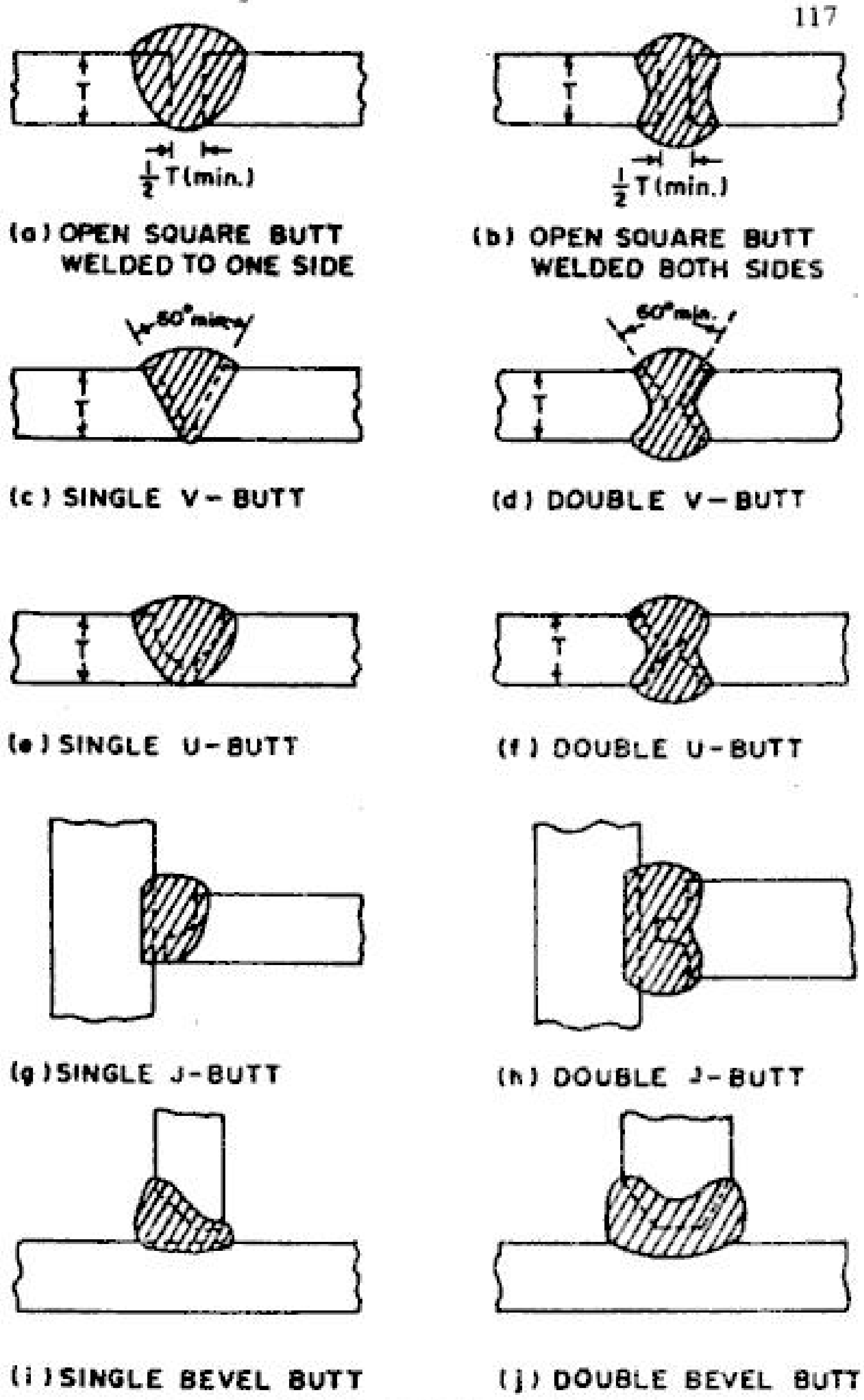


FIG. 5.8. TYPES OF BUTT WELDS.

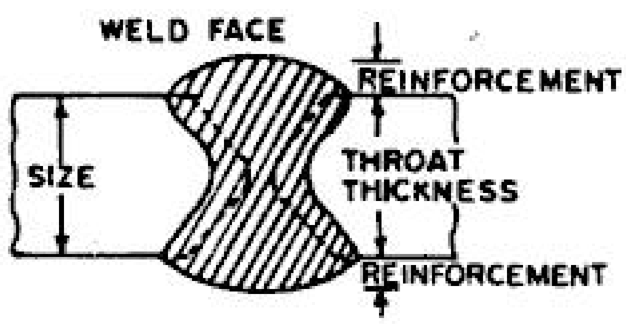


FIG. 5.9. BUTT WELD.

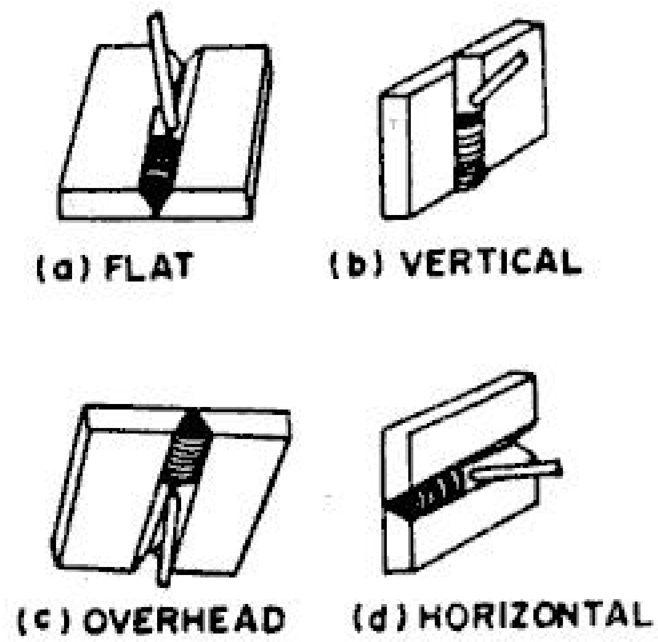


FIG 5.10. POSITIONS FOR WELDING : BUTT WELDS

5.6. FILLET WELDS

When the lapped plates are to be jointed, fillet welds are used. These are generally of right angled triangle shape. The outer surface is generally made *convex*, as shown in Fig. 5.11. A fillet weld is specified by the following:

(*i*) Size of wied

(*ii*) Throat thickness.

(*iii*) Length of weld.

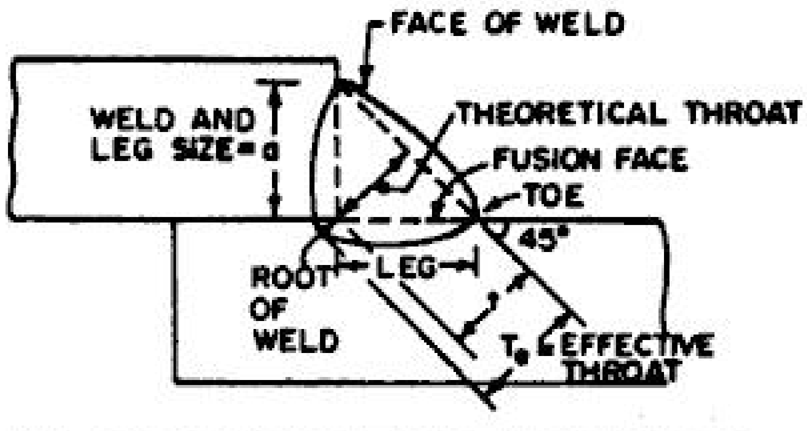


FIG. 5.11. FILLET WELD TERMINOLOGY

(*i*) **Size of weld**: The sides containing the right angle of the fillet are called *legs*. The *size* of the weld is specified by *minimum leg length*. The length of the leg is the distance from the root of the weld to the toe of the weld, measured along the fusion face. Table 5.1 gives the minimum size of single run fillet weld, as specified by IS : 816–1969.

TABLE 5.2. MINIMUM SIZE OF SINGLE RUN FILLET WELD.

Thickness of thicker part	*Min. size*
upto 10 mm	3 mm
10 to 20 mm	5 mm
20 to 32 mm	6 mm
32 to 50 mm	8 mm (first run) ; 10 mm (min.)

Note : When the minimum size of the weld is greater than the thickness of thinner part, the minimum size of the weld should be equal to the thickness of thinner part.

(*ii*) **Throat thickness:** The theoretical throat is the perpendicular distance between the root of the weld, and the hopotenuse joining the two ends of the legs. Reinforcement is neglected. The *effective throat thickness* is taken equal to the theoretical throat thickness, and when the angle between the fusion faces is 90°(as is generally the case), we have :

$$\text{Effective throat thickness } t = \frac{1}{\sqrt{2}} \times \text{ size of weld}$$

or

$$t \approx 0.7 \times \text{ size of weld.} \qquad ...(5.1)$$

where size of weld = minimum leg length

For angles other than 90° between the fusion faces,

Effective throat thickness = $k \times$ minimum leg length. Table 5.3 gives the values of k for different angles between the fusion faces, as per IS : 816-1969

TABLE 5.3 VALUES OF k

Angle	60° to 90°	91° to 100°	101° to 106°	107° to 113°	114° to 120°
k	0.7	0.65	0.60	0.55	0.50

It may be noted that a fillet weld is not used for jointing parts if the angle between the fusion faces is less than 60° or greater than 120°.

The maximum size of fillet weld at the square edge of a plate (Fig. 5.12 *a*) is 1.5 mm less than the plate thickness and in case of a weld at the rounded edges of flanges or the toe of an angle is kept three fourths the thickness of the edge (Fig. 5.12 *b*).

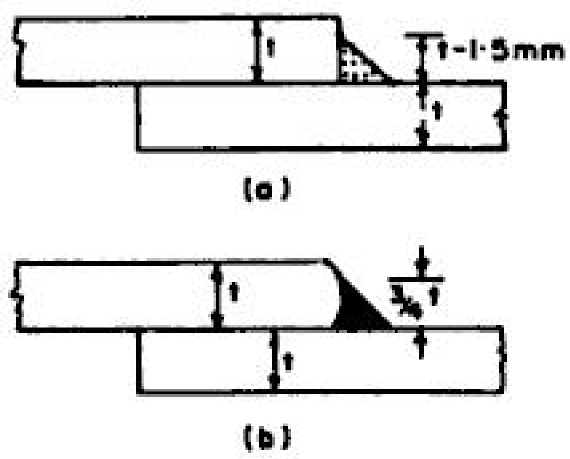


FIG. 5.12. MAXIMUM SIZE OF WELD.

When the fillet weld is placed parallel to the direction of the forces on both the sides of the member, it is called *side fillet weld*. When the weld is placed at the end of the member, such that it is perpendicular to the direction of the force, it is called *end fillet weld*. If the axis of the weld is inclined to the direction of force, it is known as *diagonal fillet weld*.

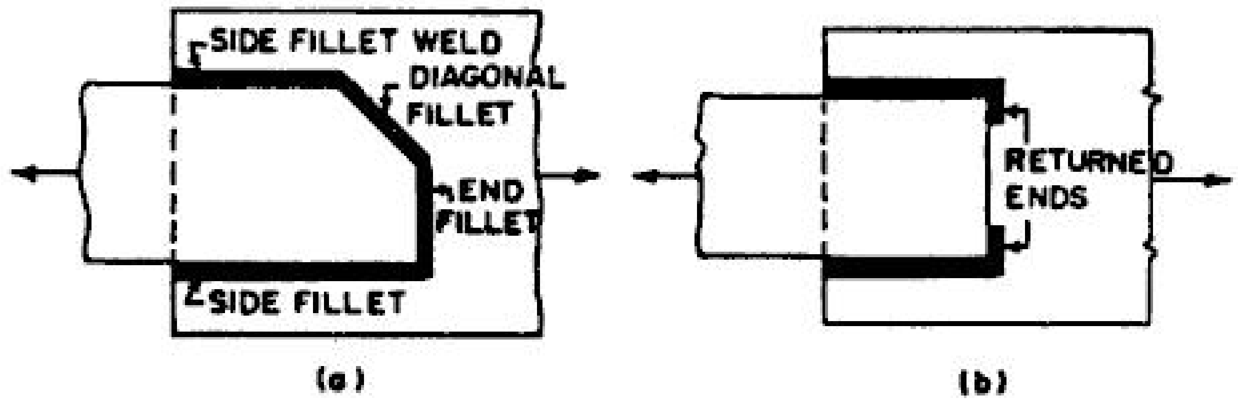


FIG. 5.13.

(iii) **Effective length of weld**: The effective length of the weld is taken as overall length minus twice the weld size. The effective length should not be less than four times the size of the weld, otherwise the weld size must be taken as one fourth of its effective length. If only the side welds are used, the length of each side fillet weld must not be less than the perpendicular distance between the two. When the ends are returned, as shown in Fig. 5.13 (*b*), the ends should be carried continuous for a distance not less than twice the size of the weld, specially when the joint is subjected to tensile force. *Only the effective length is shown on the drawing and the additional length (i.e. 2 × weld size) is provided by the welder.*

Special fillet welds may have *unequal* legs. In such a case the diagonal of the weld may make 30° and 60° with the two legs. Similarly, the face of fillet welds may be either *concave* or *mitre* (flat). Such welds are not very common.

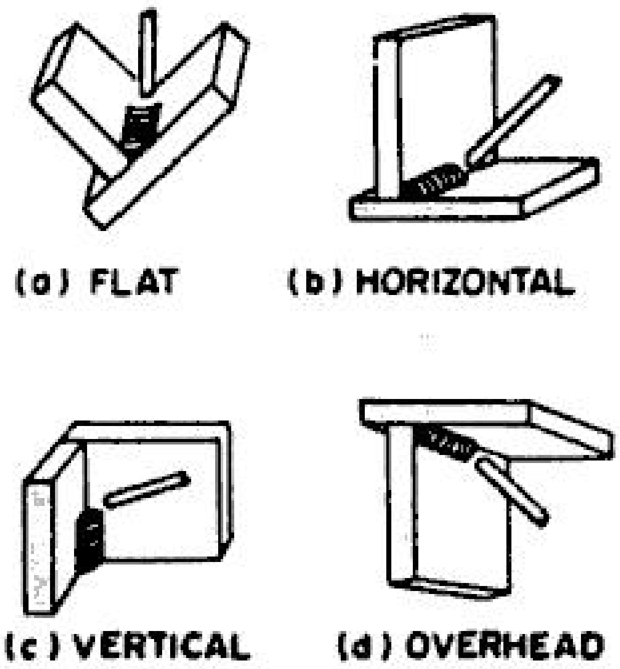


FIG. 5.14. POSITION FOR WELDING : FILLET WELDS.

Depending upon the *position*, fillet welds may be *flat, horizontal, vertical* or *over head*, as shown in Fig. 5.14.

5.7. DEFECTS IN WELDING

Welding is highly specialised technique of jointing, and it should be done carefully so that no *defects* or *imperfections* are left. The most important defects arising from the welding technique are as follows:

1. Undercutting
2. Overlap
3. Incomplete penetration
4. Lack of fusion
5. Slag inclusion
6. Porosity and gas inclusion
7. Edge melting

These defects have been shown diagrammatically in Fig. 5.15. *Under cutting* (Fig. 5.15 *a, b*) takes place due to excessive current and excessive length of arc, resulting in the formation of a groove in the base metal. When the weld metal overflows the groove, but does not fuse with base metal, and *overlap* is formed (Fig. 5.15 *c*). *Incomplete penetration* takes place when the weld metal does not penetrate up to the root of the joint because of faulty groove preparation (Fig. 5.15 *f, g*) or because of faulty technique used during welding. *Lack of fusion* (Fig. 5.15 *d, e*) takes place when the parent metal is coated with some foreign matter and when the groove is not clean. Due to this, there will be lack of union between two runs of weld metal. *Slag inclusion* (Fig. 5.15 *h*) takes place because of formation of oxides due to chemical reaction among the base metal, air and electrode coating, during welding. Some times, a group of *gas pores* may get entrapped in the weld, as shown in Fig. 5.15 (*i*). Such a defect of *gas inclusion* is also called *porosity*. *Edge melting off* occurs in fillet welds (Fig. 5.1 *j*) because of careless welding.

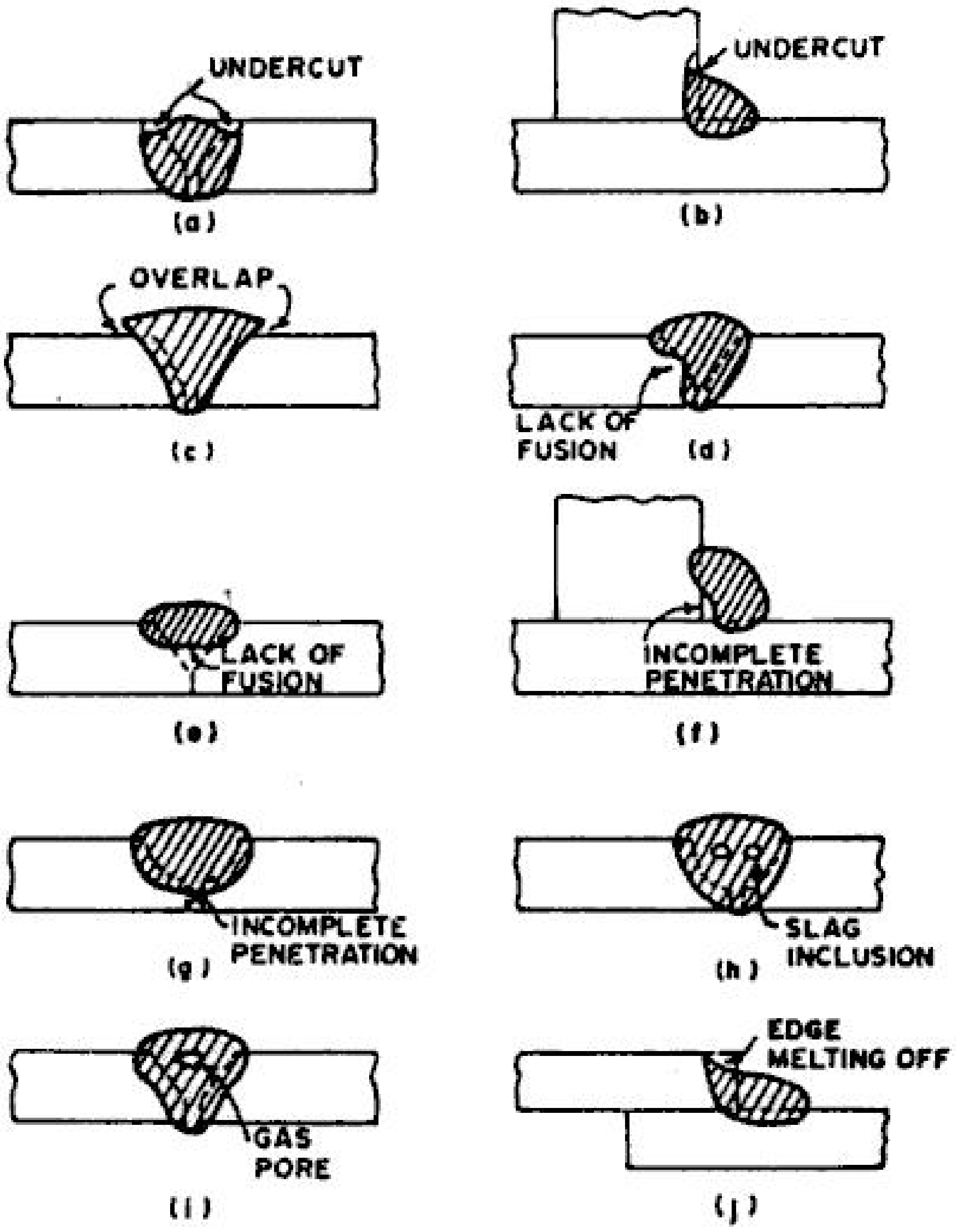


FIG. 5.15. IMPORTANT WELD DEFECTS.

5.8. WORKING STRESSES IN WELDS

Table 5.4 gives the permissible stresses in welds, as per IS : 816-1969, for mild steel conforming to IS : 226-1962 and electrodes conforming to IS : 814-1974.

TABLE 5.4 WORKING STRESSES IN WELDS (IS UNITS)

Kind of Stress	Max. Permissible Values	(Converted into SI units)
1. Tension or compression on section through throat of butt weld.	$0.6 f_y$	150 N/mm^2
2. Bending stress in tension or compression	$0.66 f_y$	165 N/mm^2
3. Shear on section through throat of butt or fillet weld	$0.44 f_y$	110 N/mm^2
4. Plug wled.	$0.44 f_y$	110 N/mm^2

Note 1. For welding done at the site (field), the maximum values in shear and tension given above are reduced to 80%.

2. When the effects of wind or earthquake forces are taken in the design, the above values are increased by 25%.

5.9. DESIGN OF FILLET WELDS FOR AXIAL LOADS

Fillet welds are approximately triangular in section, and may be of two types (*i*) standard fillet weld (Fig. 5.16 *a*) and (*ii*) speciai fillet weld. (Fig. 5.16 *b*). Standard fillet weld consists of isosceles triangle with 45° angles, and has equal legs. Special fillet weld has the side angles 30° and 60°, with unequal leg lengths.

Normally, a fillet weld has convex face, but sometimes, it may have concave face also. In each case, the minimum leg length and throat thickness are shown in Fig. 5.16 (*c*) and 5.16 (*d*) respectively.

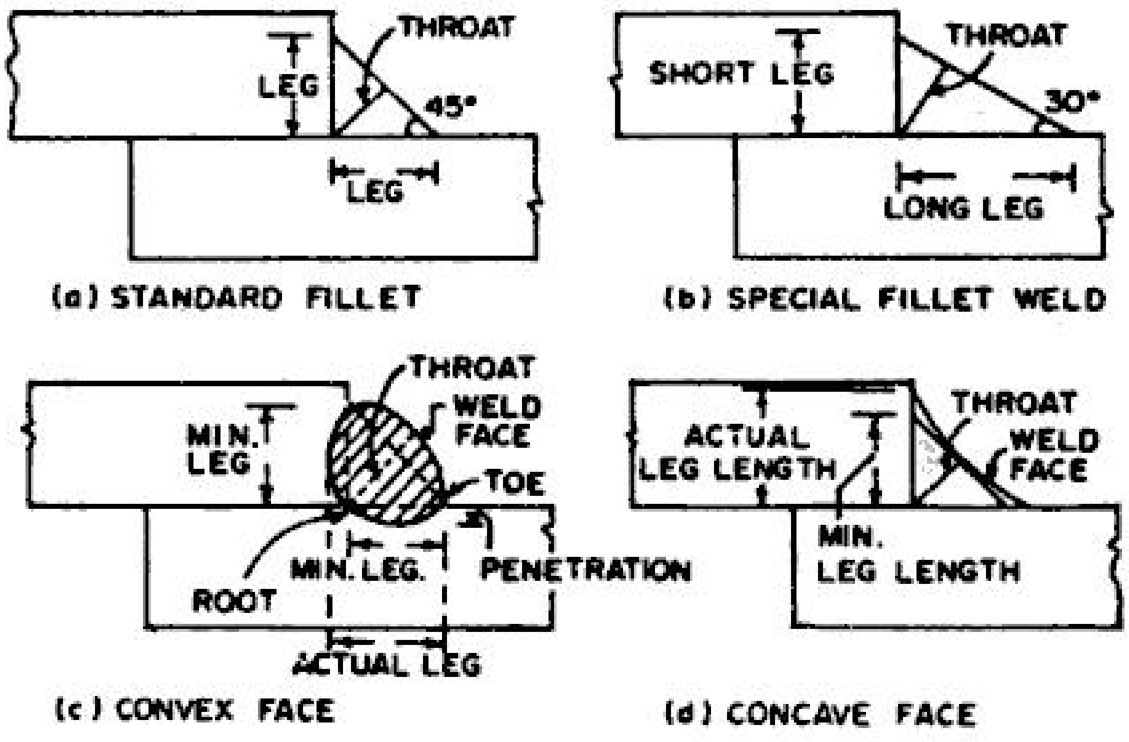


FIG. 5.16. ELEMENTS OF FILLET WELD.

Fig. 5.17 shows fillet weld subjected to axial load *P*. The strength of fillet weld is determined by its resistance against shear. The maximum stress will develop at the throat and failure will occur by shear along the throat. If we assume a uniform distribution of shear stress, we have:

$$P = \text{stress} \times \text{area.}$$

where Area = throat thickness × effective length

$$\therefore \quad P = t \, . \, L \, \tau_{vp} \qquad \text{...(5.2)}$$

or

$$P = 0.7\,h \, . \, L \, \tau_{vp} \qquad \text{...(5.3)}$$

where τ_{vp} = permissible maximum shear stress in weld.

t = effective thickness = $0.7h$

h = size of the weld.

L = effective length of the weld.

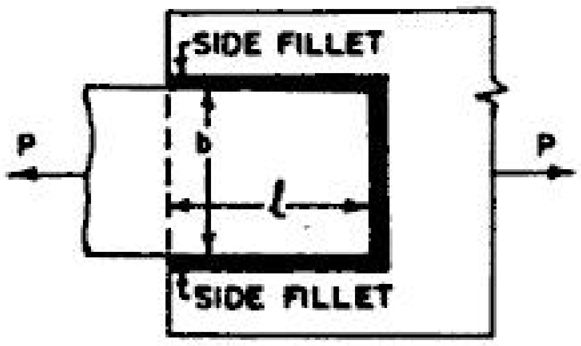


FIG. 5.17.

The specifications for fillet weld have already been given in § 5.6. When only side fillets are used, the effective length in each side is equal to the actual length minus twice the weld size. The effective length should not be less than four times the size of the weld. Also when only the side fillets are used, the length of each side fillet weld must not be less than the perpendicular distance between the two; thus in Fig. 5.17, l should not be less than b. Also, the perpendicular distance between the two side fillets should not exceed 16 times the thickness of thinner plate jointed. Thus, in Fig. 5.17, b should not be greater than $16\,t$. If this distance exceeds $16\,t$, intermediate plug or slot weld should be used so that buckling is prevented.

If *end fillet* is also used, the total length in Eq. 5.3 consists of lengths of side fillet plus the length of end fillet. In case when only end fillets are used, the overlap in a lap joint should not be less than five times the thickness of thinner plate jointed (Fig. 5.18).

Intermittent fillet wleds are used to transfer calculated stress across a joint when the strength required is less than that developed by a continuous weld of even the smallest allowable size for the thickness of parts joined. Any section (or length) of such intermittent fillet weld should not be less than *four times* the weld size, nor less than 40 mm. The clear spacing between the ends of consecutive effective lengths of intermittent fillet welds carrying

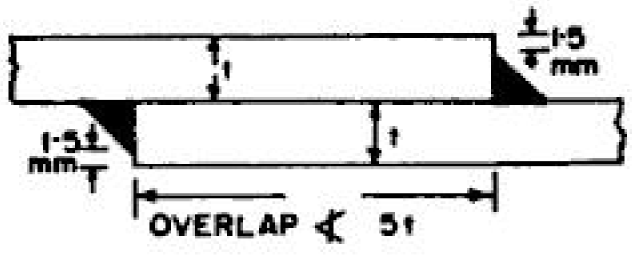


FIG. 5.18.

the calculated stress shall not exceed $12t$ for compression and $16t$ for tension and shall in no case be more than 200 mm, where t is the thickness of thinner part joined.

5.10. FILLET WELDING OF UNSYMMETRICAL SECTIONS : AXIAL LOAD

In case of unsymmetrical sections like angles and Tee, which are loaded along the axis passing through their centroid, the weld lengths are so arranged that the gravity axis of the weld lines coincides with the neutral axis. This will avoid eccentricity of loading and hence the bending moment.

Let us consider an angle section subjected to axial load P, welded to a gusset plate as shown in Fig. 5.19.

Let L_1 and L_2 be the required lengths of welds on the two faces, and P_1 and P_2 be the resisting forces exerted by the respective welds. These are assumed to act along the edges of the angle. Taking moments about the line of action of P_2, we obtain,

$$(a + b) P_1 = P a$$

$$\therefore \quad P_1 = \frac{Pa}{a + b} \quad ...(1)$$

Similarly, taking moments about the line of action of P_1

$$(a + b) P_2 = Pb$$

or $$P_2 = \frac{Pb}{a + b} \quad ...(2)$$

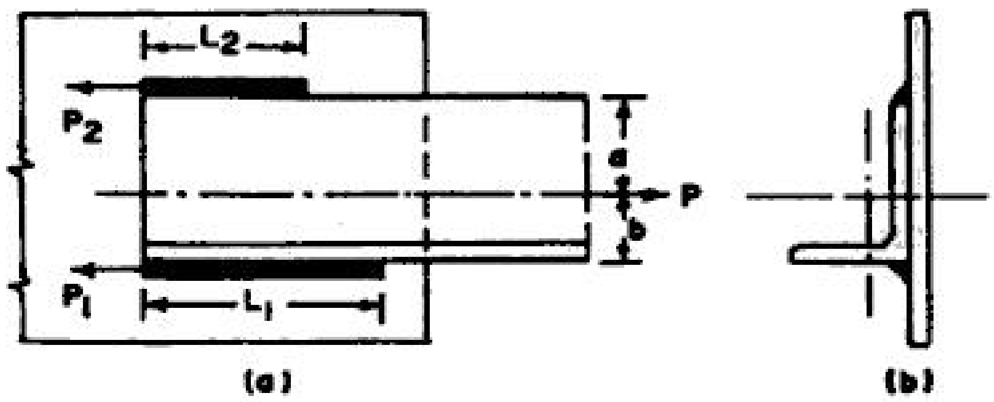


FIG. 5.19.

If s is the strength of the weld per unit length (per lineal length *i.e.* $s = \tau_{vp} \times 1 = \tau_{vp}$ N/mm) we have

$$L_1 = \frac{P_1}{s} = \frac{P \cdot a}{s(a + b)} \quad ...5.4\ (a)$$

and $$L_2 = \frac{P_2}{s} = \frac{P \cdot b}{s(a + b)} \quad ...5.4\ (b)$$

Sometimes, it is not possible to accommodate the required length of the weld on the sides of the section. In such cases, end fillets are also provided. The procedure of analysing such a case is similar to the one described above.

5.11. DESIGN OF BUTT WELDS

A butt weld is specified by the effective thickness of its throat, and its strength is taken equal to the strength of parts joined if full penetration of the weld metal is ensured. In the case of double-V, double-U, double-J and double bevel butt joints, full penetration is achieved and hence the *effective throat thickness* is taken equal to the thickness of the thinner parts joined. In the case of single V, U, J, and bevel joints, penetration is generally incomplete, and hence effective throat thickness is taken equal to $\frac{5}{8}$ times the thickness of thinner part connected, as per IS : 816-1969.

Where parts of unequal thickness are joined, the change in thickness should be gradual. A taper not exceeding 1 in 5 is provided when the difference in thickness of the parts exceeds 25% of the thickness of thinner part or 3 mm, whichever is greater.

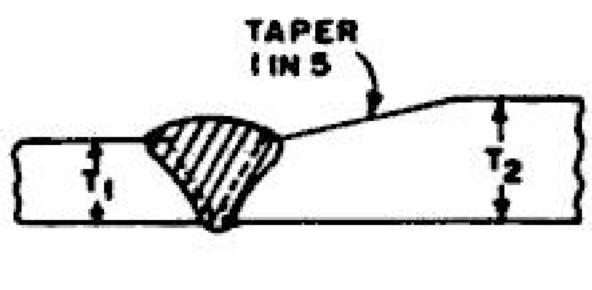


FIG. 5.20.

As per IS : 816-1969, permissible stress in butt weld is taken same as that of parent metal.

5.12. DESIGN OF PLUG AND SLOT WELDS

Plug weld (Fig. 5.6 *c*) and slot weld (Fig. 5.6 *d*) are used where ever sufficient space is not available for providing required length of the fillet. They are also used for equalising stress in plates and to prevent buckling in case of wide plates. For this purpose, a *slot* is cut in one of the over lapping plates, and the weld metal is filled in it. If the size of the slot is small, and is *filled completely* with weld metal, then it is known as a *plug weld*. However, if the slot is of big size and is fillet welded along its periphery, it is then known as *slot weld*. In either case, the slot may be either of circular or a rectangular shape.

IS : 816-1969 gives the following specifications for plug or slot weld :

1. The width or diameter should not be less than 3 times the thickness of the plate or 25 mm whichever is greater.
2. The corners at the enclosed ends of slots should be rounded with radius not less than $1\frac{1}{2}$ times the thickness or 12 mm whichever is greater.
3. The distance between the edge of the part and the edge of the slot or hole, or between the adjacent slots or holes shall not be less than twice the thickness and not less than 25 mm for holes.
4. The effective area of plug weld shall be considered as the nominal area of the hole in the plane of facing surface. The plug welds shall not be designed to carry stress.

Example 5.1. *Find the safe load that can be transmitted by the fillet welded joint shown in Fig. 5.21. The size of weld is 8 mm. Take the safe stress in the weld equal to 110 N/mm².*

Solution

Effective length of weld, $L = 80 + 60 + 80$
$= 220$ mm

Size of weld $= h = 8$ mm.

∴ Throat thickness $= t = 0.7h$
$= 0.7 \times 8 = 5.6$ mm.

∴ Safe load $P = t \, . \, L \, . \, \tau_{vp}$
$= 5.6 \times 220 \times 110$
$= 135520$ N $=$ **135.52 kN**

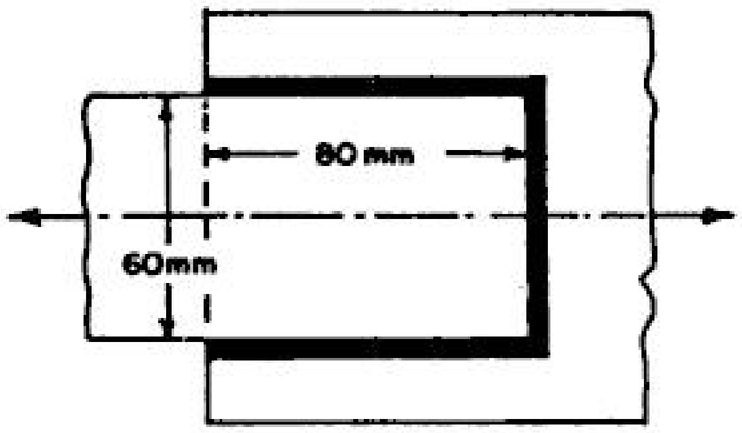


FIG. 5.21.

Example 5.2. *A 100 mm × 10 mm plate is to be welded to another plate 150 mm × 10 mm by the fillet welding on three sides. The size of the weld is 6 mm. Find out the necessary over lap of the plate, for full strength of the joint. Take allowable tensile stress in plate equal to 150 N/mm² and allowable stress in weld as 110 N/mm².*

Solution

The joint will be designed on the basis of the strength of small plate.

Total load taken by smaller plate $= 100 \times 10 \times 150 = 150000$ N

Throat thickness $= 0.7h = 0.7 \times 6 = 4.2$ mm.

∴ Allowable load per lineal mm $= 4.2 \times 110 = 462$ N

∴ Total length of weld required $= \dfrac{150000}{462} \approx 325$ mm.

Effective length of end fillet $= 100$ mm.

∴ Length to be provided in side fillets
$= 325 - 100 = 225$ mm

∴ Overlap $x = \dfrac{225}{2} = 112.5$ mm

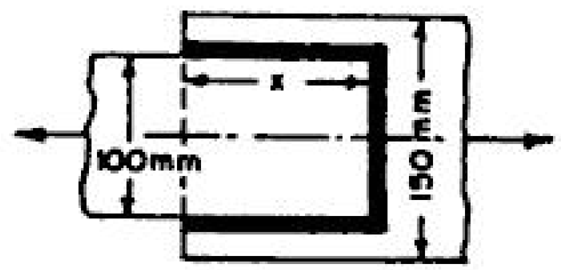


FIG. 5.22.

Provide an overlap of 115 mm, as shown in Fig. 5.22. This distance is more than the distance of 100 mm between the two side fillets.

Hence the design is O.K.

Example 5.3. *A tie bar 120 mm × 10 mm is to be connected to other of size 120 mm × 14 mm. If the tie bars are to be loaded by a pull of 160 kN, find out the size of end fillets such that the stresses in both the end fillets are same. Take permissible stress in weld = 110 N/mm².*

Solution

The portion of plates between the welds stretch by the same amount. Therefore, the strain and hence the stress in both the plates are same. The force carried by each plate will be proportional to its thickness. Thus, if the 10 mm thick plate carries a force P_1, the 14 mm thick plate will carry a force $1.4 P_1$. Therefore, to keep the stresses same in both the end welds, we must keep the size of the welds in proportion to the thickness of respective plates.

Let the size of lower weld $A = h$

∴ Size of the upper weld $B = 1.4h$

Length of weld in each case = 120 mm.

Strength of lower weld $= 0.7h \times 120 \times 110$
$= 9240\,h$ N

Strength of upper weld $= 0.7\,(1.4\,h)120 \times 110$
$= 12936\,h$ N

Total load carried by tie bar = 160 kN.

∴ $9240\,h + 12936\,h = 160 \times 1000$

From which $h = 7.22$ mm.

Keep $h = 7.5$ mm. ∴ Size of lower weld = 7.5 mm, which is less than the maximum permissible weld size of $10 - 1.5 = 8.5$ mm.

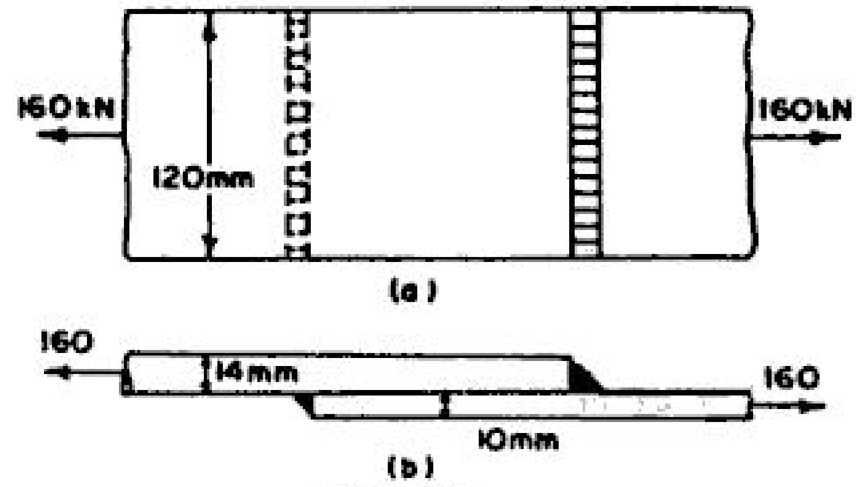


FIG. 5.23.

Also, size of upper weld $= 1.4h = 1.4 \times 7.5 =$ **10.5 mm**. This is also less than the maximum permissible size of $14 - 1.5 = 12.5$ mm.

Example 5.4. *A tension member consisting of two channels sections 200 mm × 75 mm @ 22.1 kg/m back to back is to be connected to gusset plate. Design the welded joint for the condition that the section is loaded to its full strength. Take A = 2821 sq. mm, thickness of flange = 11.4 mm and the thickness of the web = 6.1 mm, permissible stress in weld equal to 110 N/mm² and permissible stress in the section in axial tension = 150 N/mm².*

Solution

In the case of rolled section, the size of weld is limited to three fourth of the thickness.

∴ Max. size of weld $= \frac{3}{4} \times 6.1 = 4.6$ mm. We shall provide 4 mm weld.

Strength of weld per linear mm
$= 0.7 \times 4 \times 110 = 308$ N

The load to be carried by each channel
$= 2821 \times 150 = 423150$ N

∴ Total length of weld required for one channel $= \dfrac{423150}{308} \approx 1380$ mm

Length of the end weld = 200 mm

∴ Length of side welds
$= 1380 - 200 = 1180$ mm

∴ Overlap $= \dfrac{1180}{2} = 590$ mm.

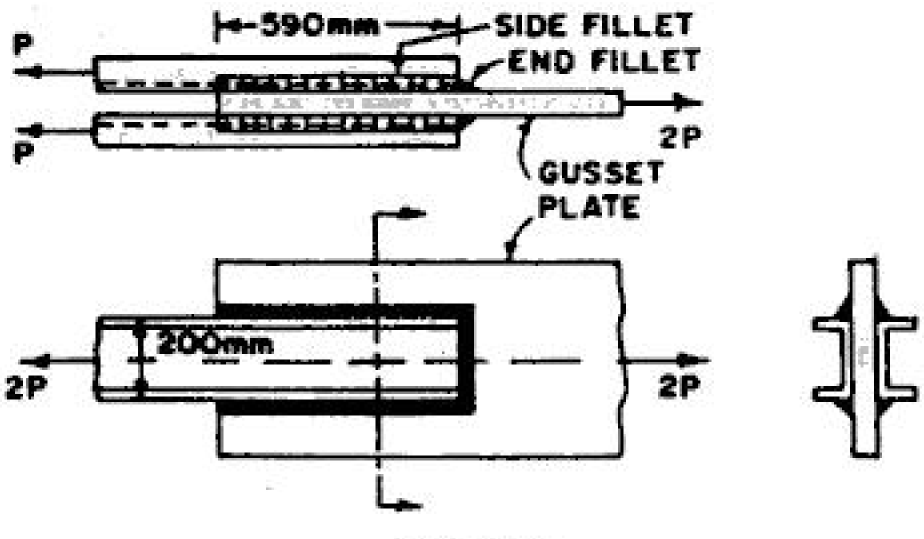


FIG. 5.24.

Example 5.5. *An I-section is built up by welding a 250 mm × 15 mm web plate to two 150 mm × 15 mm flange plates by 8 mm fillet welds. Find out maximum shearing force which may be permitted if the mean shearing stress in the web and the maximum shear stress in weld are not to exceed 100 N/mm².*

Solution

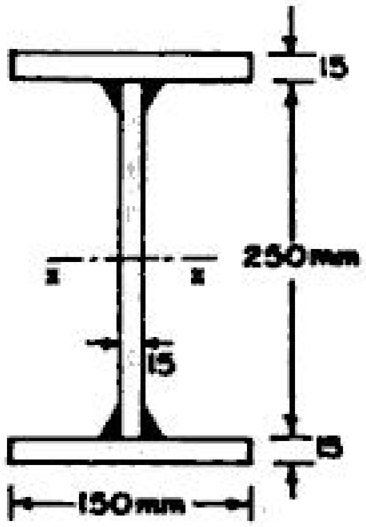


FIG. 5.25.

$$I_{xx} = \frac{1}{12}[\,150 \times 280^3 - 135 \times 250^3\,] = 98.62 \times 10^6 \text{ mm}^4$$

Shear stress q at the section passing through welds is given by :

$$q = \frac{F}{I_{xx}\,.\,b}(A\bar{y}) \qquad \text{where,}$$

b = effective thickness = 2 × throat thickness = 2 × 8 × 0.7 = 11.2 mm

$A\bar{y}$ = moment of the area about x-x

= 150 × 15 (125 + 7.5) = 298125 mm³

q = allowable shear stress = 100 N/mm²

$$\therefore \qquad 100 = \frac{F}{98.62 \times 10^6 \times 11.2}[298125]$$

From which F = 370497 N ...(1)

Maximum shear force limited on the web = 250 × 15 × 100 = 375000 N ...(2)

Hence maximum allowable shearing force = lesser of the above two values

= 370497 N ≈ **37.05 kN**

Example 5.6. *A welded plate girder is to be fabricated using web plates 1600 mm deep and 16 mm thick and flange plates 400 mm wide and 30 mm thick. The girder is to be used over a simply supported span of 20 m, carrying a load of 20 kN/m including its own weight. Design suitable welded connection between the web and the flange, taking permissible stress in weld as 110 N/mm².*

Solution : S.F. $F = \dfrac{20 \times 10^3 \times 20}{2} = 2 \times 10^5$ N

$$I = \frac{1}{12} \times 16\,(1600)^3 + 2\left[\frac{1}{12} \times 400\,(30)^3 + 400 \times 30\,(815)^2\right] = 2.1405 \times 10^{10} \text{ mm}^4$$

Horizontal shear, per mm length of plate girder, is given by

$$q = \frac{F}{I}(A\bar{y})$$

At the junction of flange and web,

$A\bar{y}$ = (400 × 30) (815) = 9.78 × 10⁶ mm³

$$\therefore \qquad q = \frac{2 \times 10^5}{2.1405 \times 10^{10}} \times 9.78 \times 10^6$$

≈ 91.4 N/mm

Vertical shear on the compression flange (loaded flange):

w = 20 kN/m = 20 N/mm

∴ Resultant shear

$$q_r = \sqrt{q^2 + w^2} = \sqrt{(91.4)^2 + (20)^2}$$

= 93.56 N/mm

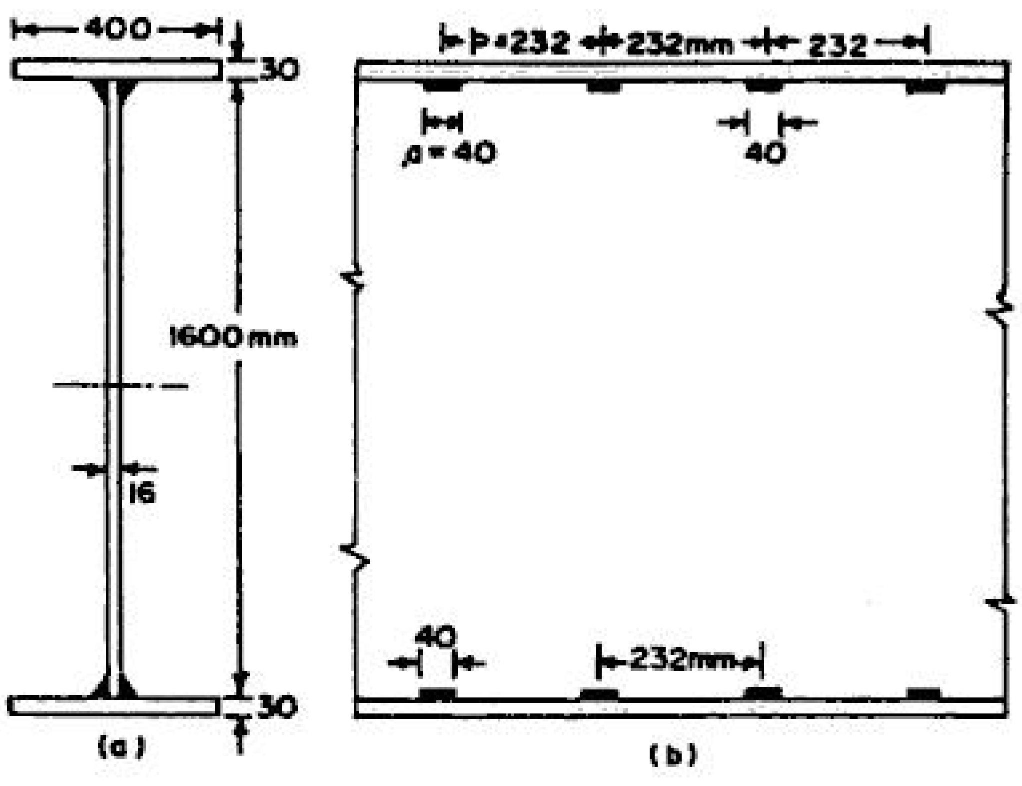


FIG. 5.26.

Let us first try *continuous weld* at each junction, on both sides.

Let h = size of weld.

∴ Strength of weld per lineal mm, $s = 2 \times 0.7h \times 110 = 154\,h$ N/mm.

Equating this to q_r, we get

$$h = \frac{93.56}{154} \approx 0.6 \text{ mm}$$

This is too small. Minimum weld size, as per IS : 816-1969 (Table 5.1), for 30 mm thickness of part (thicker plate) is 6 mm. Since the minimum thickness of weld is much more than h found above, we will use *intermittent weld*.

Let $h = 6$ mm.

The specifications for intermittent weld are as follows :

(*a*) *Effective length* of each intermittent weld should not be less than the following

(*i*) $4\,h = 4 \times 6 = 24$

or (*ii*) 40 mm.

Hence adopt effective length $a = 40$ mm.

(*b*) *Pitch* (*p*) *of the intermittent weld*: The clear spacing between intermittent weld should not exceed

(*i*) 12 × thickness of thinner part $= 12 \times 16 = 192$ mm

or (*ii*) 200 cm

Hence for 192 mm clear spacing between the weld,

$$p = 192 + a = 192 + 40 = 232 \text{ mm} \quad ...(i)$$

This is the *maximum spacing*. However, the spacing or pitch (p) should be such that the shear in the weld is not more than q_r.

Using 40 mm long intermittent welds on both sides of web plate, weld strength

$$= 2 \times 40 \times 0.7 \times 6 \times 110 = 36960$$

Thus, we have, $p \, . \, q_r \le 36960$

or $$p \le \frac{36960}{q_r} \le \frac{36960}{93.56} \le 395 \text{ mm.} \quad ...(ii)$$

Hence maximum permissible p is lesser of the two given by (*i*) and (*ii*) values

∴ Keep $p = 232$ mm. Clear spacing between the intermittent welds $= 232 - 40 = 192$ mm.

Example 5.7. *A tie member consisting of two channels ISMC 200 @ 22.1 kg/m back to back is to be connected to gusset plate 12 mm thick. Design the welded joint to develop full strength of the tie, given that the overlap is limited to 350 mm. Take permissible stress in weld equal to* $0.44 f_y$ *and permissible tensile stress in section equal to* $0.6 f_y$*, where* $f_y = 250 \text{ N/mm}^2$.

Solution

From ISI handbook, for ISMC 200 @ 22.1 kg/m, we have the following data :

Thickness of web = 6.1 mm

Thickness of flange = 11.4 mm

Sectional area $= 28.21 \text{ cm}^2 = 2821 \text{ mm}^2$

σ_{at} in section $= 0.6 \times 250 = 150 \text{ N/mm}^2$.

∴ Tensile strength of each channel section $= 150 \times 2821 = 423150$ N.

Maximum size of weld $= 6.1 - 1.5 = 4.6$ mm.

Provide 4 mm size weld.

Permissible stress in weld $= 0.44 \times 250 = 110 \text{ N/mm}^2$

∴ Strength of weld, per mm length $= 0.7 \times 4 \times 110 = 308$ N/mm

∴ Length of weld necessary, to connect one channel section.

$$L = \frac{423150}{308} \approx 1374 \text{ mm.} \quad ...(i)$$

The over lap is limited to 350 mm. Hence available length of weld, even providing end fillets, = 350 + 350 + 200 = 900 mm. This falls short of required total length. Hence let us provide additional fillets in two slots, each of length a.

The width of such a slot should not be less than 3 times the thickness = 3 × 6.1 = 18.3 mm. Hence provide two slots, each of 20 mm width. The edge distance of each slot should not be less than twice the thickness= 2 × 6.1 = 12.6 mm. Hence arrange the slots as shown in Fig. 5.27.

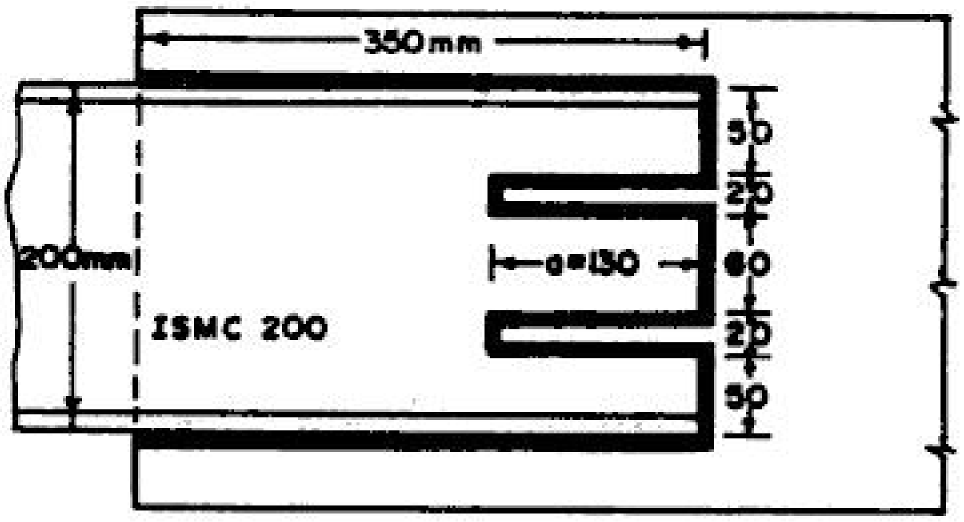


FIG. 5.27.

Total length of weld = (350 × 2) + 200 + 4 a − 2 × 20= 860 + 4 a ...(ii)

Equating this to required value of L we get

$$860 + 4a = L = 1374$$

$$\therefore \quad a = 128.5 \text{ mm.}$$

Hence provide 130 mm long slots for fillet welding, as shown in Fig. 5.27.

Example 5.8. *A circular penstock of mild steel, of 1.6 m diameter, is fabricated from 16 mm plate, lapping it and securing it by fillet welds of 12 mm size, provided on the inside and outside of the lapped ends, as shown in Fig. 5.28. Determine the safe internal pressure that can be allowed in the penstock. Take safe stress in weld equal to 110 N/mm².*

Solution.

Throat thickness $t = 0.7h = 0.7 \times 12 = 8.4$ mm.

Let p = safe internal pressure (N/mm²)

d = diameter of penstock (mm)

Internal force per unit length, causing bursting of pipe

$$= p\frac{d}{2} \quad ...(1)$$

Resistance offered by the weld, per unit length

$$= 2t \cdot \tau_{vp} \quad ...(ii)$$

Equating the two, we get

$$p\frac{d}{2} = 2t \cdot \tau_{vp}.$$

$$\therefore \quad p = \frac{4t\,\tau_{vp}}{d} \quad ...(5.5)$$

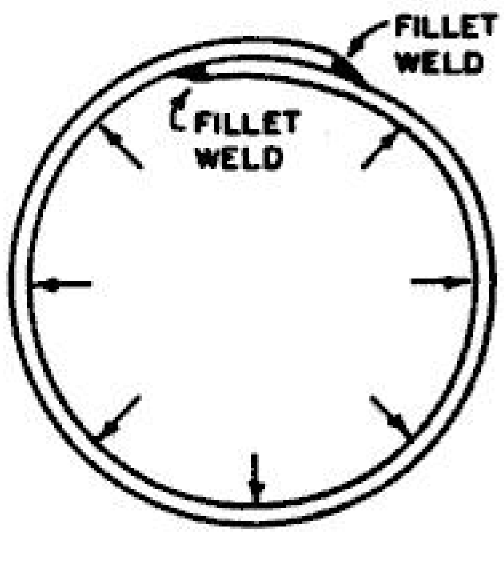


FIG. 5.28.

Substituting the given values, we get

$$p = \frac{4 \times 8.4 \times 110}{1600} = \mathbf{2.31\ N/mm^2}$$

Example 5.9. *An equal angle 65 mm × 65 mm @ 9.4 kg/m of thickness 10 mm carries a tensile load of 160 kN, applied along its centroidal axis. The angle is to be welded to a gusset plate. Find out the lengths of side fillet welds required at the heel and toe of the angle. Its C.G. is at 19.7 mm from its heel. Take permissible stress in the weld equal to 110 N/mm².*

Solution.

Taking the moments about line of action of P_2

$65\,P_1 = 160 \times 1000 \times 45.3$

$\therefore P_1 = 111508$ N

$\therefore P_2 = P - P_1$

$= 160000 - 111508$

$= 48492$ N

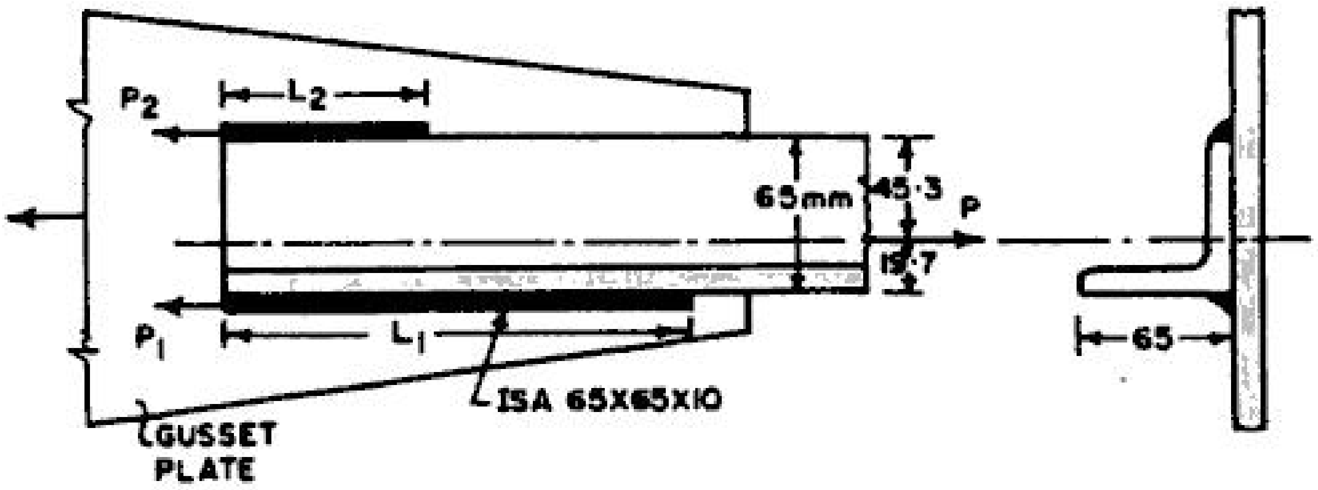


FIG. 5.29.

Size of weld

The maximum size of weld for a rounded edge at the toe of the angle $= \frac{3}{4} \times$ thickness $= \frac{3}{4} \times 10 = 7.5$ mm.

Strength of weld per lineal mm $= 0.7 \times 7.5 \times 110$ N/mm^2 $= 577.5$ N

$$\therefore \quad L_1 = \frac{P_1}{577.5} = \frac{111508}{577.5} = 193 \text{ mm}$$

and

$$L_2 = \frac{P_2}{577.5} = \frac{48492}{577.5} = 84 \text{ mm}$$

These values are *effective* and must be increased by twice the weld size, *i.e.* by $2 \times 7.5 = 15$ mm to get the actual length of the welds.

Example 5.10. *A tie bar consisting of a single angle 60 mm × 60 mm × 10 mm is to be welded to a gusset plate. The tie bar carries a load of 150 kN along its centroidal axis. Design the joint if both the side fillets and end fillets are to be provided. The centroidal axis of the angle lies at 18.5 mm from the heel of the angle.*

Solution

The maximum size of fillet weld at the end, along the square edge of the angle will be 1.5 mm less than the thickness of the angle. Therefore, the maximum size of the end fillet$=10-1.5=8.5$ mm.

The maximum size of side fillets, along the rounded edge

$= \frac{3}{4} \times 10 = 7.5$ mm

We shall provide 7.5 mm weld thoughout.

Strength of weld per mm length

$= 0.7 \times 7.5 \times 110 = 577.5$ N

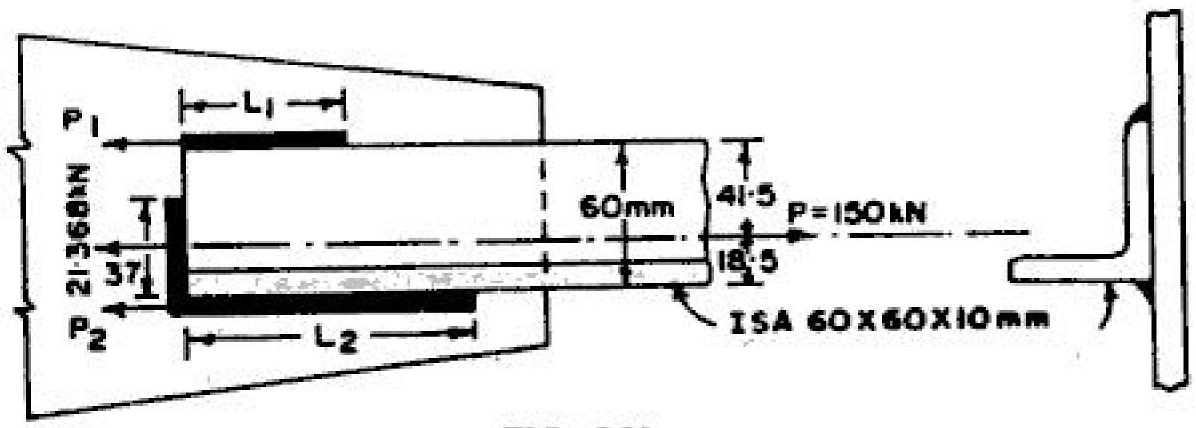


FIG. 5.30.

The end fillet weld will be placed symmetrical about the line of action of the load in order to avoid eccentricity. The maximum length of the end weld is, therefore equal to $2 \times 18.5 = 37$ mm.

The strength of end weld $= 577.5 \times 37 = 21368$ N$= 21.368$ kN

Taking moments about the line of action of force P_1 we get

$$60\,P_2 = 150 \times 41.5 - 21.368 \times 41.5$$

From which $\quad P_2 = 88.97$ kN

$\therefore \quad P_1 = 150 - 88.97 - 21.368 = 39.662$ kN.

$$\therefore \quad \text{Length } L_1 = \frac{P_1}{577.5} = \frac{39.662 \times 1000}{577.5} \approx 69 \text{ mm}$$

and

$$\text{Length } L_2 = \frac{P_2}{577.5} = \frac{88.97 \times 1000}{577.5} = 154 \text{ mm.}$$

The above values of L_1 and L_2 are *effective lengths*. Twice the size of the weld *i.e.* $2 \times 7.5 = 15$ mm must be added to above lengths to get the actual lengths of side fillets.

Thus, actual $L_1 = 69 + 15 = 84$ mm

and actual $L_2 = 154 + 15 = 169$ mm

Example 5.11. *Design the welded joint of a truss shown in Fig. 5.31. Take permissible shear stress in weld equal to 110 N/mm².*

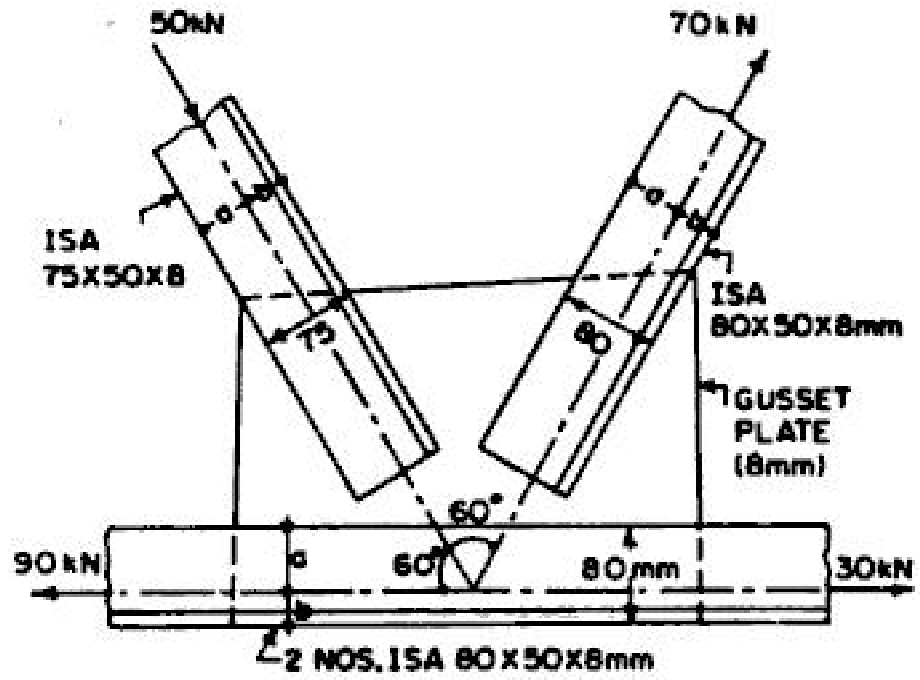


FIG. 5.31.

Solution

From ISI hand book, we get the following:

(*i*) For ISA 7550×8 mm, $b = 25.2$ mm; $a = 75 - 25.2 = 49.8$ mm ; $a + b = 75$ mm

(*ii*) For ISA 8050 × 8 mm, $b = 27.3$ mm; $a = 80 - 27.3 = 52.7$ mm ; $a + b = 80$ mm

(a) Member carrying 50 kN compressive load

Min. size of weld = 3 mm.

Max. size of weld $= \frac{3}{4} \times 8 = 6$ mm

Hence provide 6 mm weld.

Strength of weld per mm length,

$s = 0.7 \times 6 \times 110 = 462$ N

Provide only side welds of lengths L_1 and L_2. Referring to Fig. 5.19, we get

$$L_1 = \frac{Pa}{s\,(a+b)} = \frac{50 \times 1000 \times 49.8}{462 \times 75} \approx 72 \text{ mm}$$

and

$$L_2 = \frac{Pb}{s\,(a+b)} = \frac{50 \times 1000 \times 25.2}{462 \times 75} \approx 37 \text{ mm}$$

The above values are effective lengths.

$\therefore$ Actual $L_1 = 72 + 2 \times 6 = 84$ mm

Actual $L_2 = 37 + 2 \times 6 = 49$ mm

(b) Member carrying 70 kN tensile load

Keep 6 mm size weld, for which $s = 462$ N as before.

$$\therefore \quad L_1 = \frac{Pa}{s\,(a+b)} = \frac{70000 \times 52.7}{462 \times 80} \approx 100 \text{ mm}$$

$$L_2 = \frac{Pb}{s\,(a+b)} = \frac{70000 \times 27.3}{462 \times 80} \approx 52 \text{ mm}$$

$\therefore$ Actual $L_1 = 100 + 2 \times 6 = 112$ mm

Actual $L_2 = 52 + 2 \times 6 = 64$ mm.

(c) ***Horizontal tie***

The horizontal tie is a continuous member carrying a load of 90 kN to one side and 30 kN to the other side. Hence the joint will be designed for a net force= 90 − 30 = 60 kN. Since the tie consists of two angles, net force in each angle $= \frac{1}{2} \times 60 = 30$ kN.

Keep 6 mm size weld, for which $s = 462$ N, as before.

$$L_1 = \frac{Pa}{s(a+b)} = \frac{30000 \times 52.7}{462 \times 80} \approx 43 \text{ mm.}$$

Provide this in two halves, so that $\frac{1}{2}L_1 = 21.5 \approx 22$ mm. Also,

$$L_2 = \frac{Pb}{s(a+b)} = \frac{30000 \times 27.3}{462 \times 80} \approx 22 \text{ mm}$$

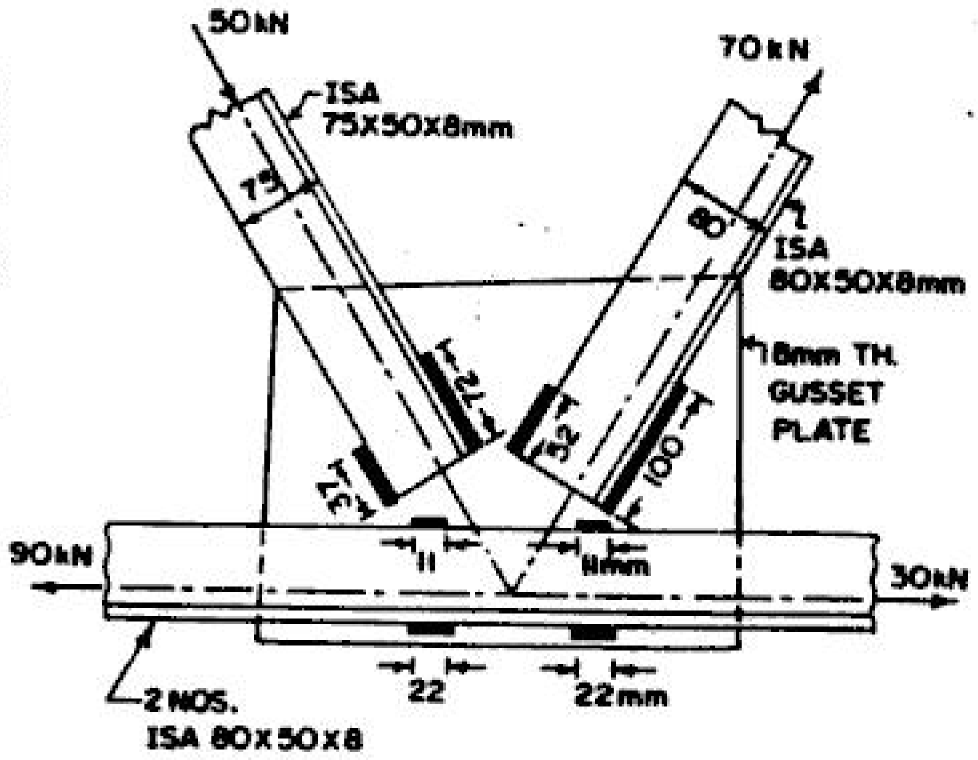


FIG. 5.32.

Provide this also in two halves, so that $\frac{1}{2}L_2 = 11$ mm. These are effective lengths.

Hence actual $\frac{1}{2}L_1 = 22 + 2 \times 6 = 34$ mm.

and actual $\frac{1}{2}L_2 = 11 + 2 \times 6 = 23$ mm.

The joint is shown in Fig. 5.32.

Example 5.12. *Two plates 160 mm × 10 mm and 160 mm × 12 mm are to be butt welded. Calculate the strength of the welded joint in tension, if (a) a single-V butt weld, (b) a double-V butt welded is used to connect them. Take permissible stress in weld as 150 N/mm².*

(a) ***Single-V butt joint***

Effective throat thickness $= \frac{5}{8} \times 10 = 6.25$ mm. Hence,

$$P = l \cdot t \cdot \tau_{vp} = 160 \times 6.25 \times 150$$
$$= 150000 \text{ N} = \mathbf{150\ kN}.$$

(b) ***Double-V butt joint***

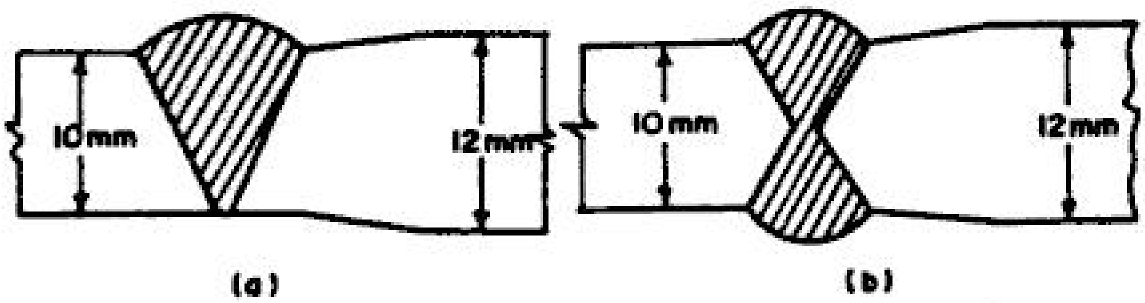


FIG. 5.33.

In the case of double-V butt joint, complete penetration of weld takes place. Hence effectiven throat thickness = thickness of thinner plate joined= 10 mm.

$$\therefore \quad P = l\,t\,\tau_{vp} = 160 \times 10 \times 150 = 240000 \text{ N} = \mathbf{240\ kN}.$$

The joints are shown in Fig. 5.33.

5.13. ECCENTRICALLY LOADED FILLET WELDED JOINTS

When the line of action of external force does not pass through the centroid of welded joint, the welds are subjected to both the axial load as well as the moment. We shall consider here two cases of fillet welded joints :

1. When the load does not lie in the plane of the welds.

and 2. When the load lies in the plane of the welds.

Case 1: When the load does not lie in in the plane of welds

This is the usual case when a plate bracket abuts against the flange of a stanchion, and is connected to it by fillet welding applied on both the sides of the plate bracket, as shown in Fig. 5.34.

The fillet weld is subjected to (*i*) vertical shear stress f_a due to axial load P, and (*ii*) horizontal shear stress f_b due to bending moment $M = P.e$.

Let the depth of the bracket be equal to d and fillet welds are applied to both the sides of the plate, for a complete length equal to the depth of plate.

Hence length of weld $L = 2d$.

Vertical shear stress in weld $= f_a = \dfrac{P}{L.t}$

or $$f_a = \frac{P}{2dt} \qquad ...(5.6)$$

where t = throat thickness of the weld.

Horizontal shear stress due to bending is

$$f_b = \frac{M}{I}.y$$

where $$I = 2\left(\frac{1}{12}t\,d^3\right) = \frac{td^3}{6} \quad \text{and} \quad y = \frac{d}{2}$$

$$\therefore \quad f_b = \frac{P.e}{t\,d^3/6} \times \frac{d}{2}$$

or $$f_b = \frac{3P.e}{t\,d^2} \qquad ...(5.7)$$

Resultant shear stress f_r is

$$f_r = \sqrt{f_a^2 + f_b^2} \qquad ...(5.8)$$

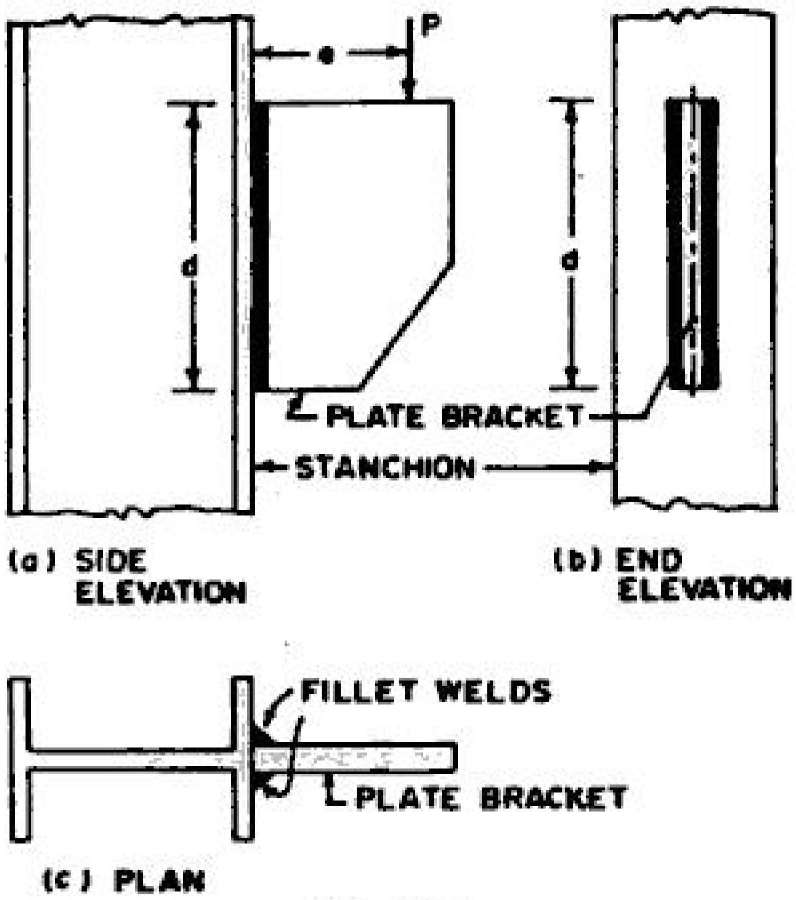


FIG. 5.34.

The resultant shear stress f_r should not exceed the permissible shear stress τ_{vp} in the weld.

For preliminary design, f_b can be assumed to be equal to f_r or τ_{vp}. In that case,

$$f_b = \tau_{vp} = \frac{3Pe}{td^2} \qquad \text{(from Eq. 5.7.)}$$

$$\therefore \quad d = \sqrt{\frac{3P.e}{t.\tau_{vp}}} \qquad ...(5.9)$$

From this, the depth of the bracket can be found.

Case 2. When the load lies in the plane of weld: Torsional stresses.

Consider a bracket connection shown in Fig. 5.35 in which a plate bracket is connected to the flange of a stanchion by way of fillet welds applied along the perimeter of the plate. In many cases, two plates are used, one attached to each face. Let P be the load acting on each plate, at an eccentricity e with respect to the centroid of the welds.

The weld element, at any point, will be subjected to two types of stresses: (*i*) vertical shearing stresses f_a due to axial load P and (*ii*) torsional shearing stress f_t due to a torsional moment $T = P.e$. The direction of vertical shear will be *vertical* while the direction of torsional shearing stress will be at right angles to the radius vector joining that point to the centroid of the weld.

Vertical shear stress,

$$f_a = \frac{P}{L\,t} \qquad ...(5.10)$$

where L = total effective length of the weld = $(2a + d)$

Torsional shear stress,

$$f_t = \frac{T\,.\,r}{J} \qquad ...(5.11)$$

where J = Polar moment of inertia of the weld.
$= I_{xx} + I_{yy}$

The resultant shear stress at any point of the weld is

$$f_r = \sqrt{f_a^2 + f_t^2 + 2 f_a\,.\,f_t \cos\theta} \qquad ...(5.12)$$

f_r will be maximum at a point for which f_t is maximum. This is induced at the farthest point such as point A (Fig. 5.35 c) for which r is maximum and θ is minimum simultaneously.

For a safe design, f_r should not exceed the maximum permissible value τ_{vp}.

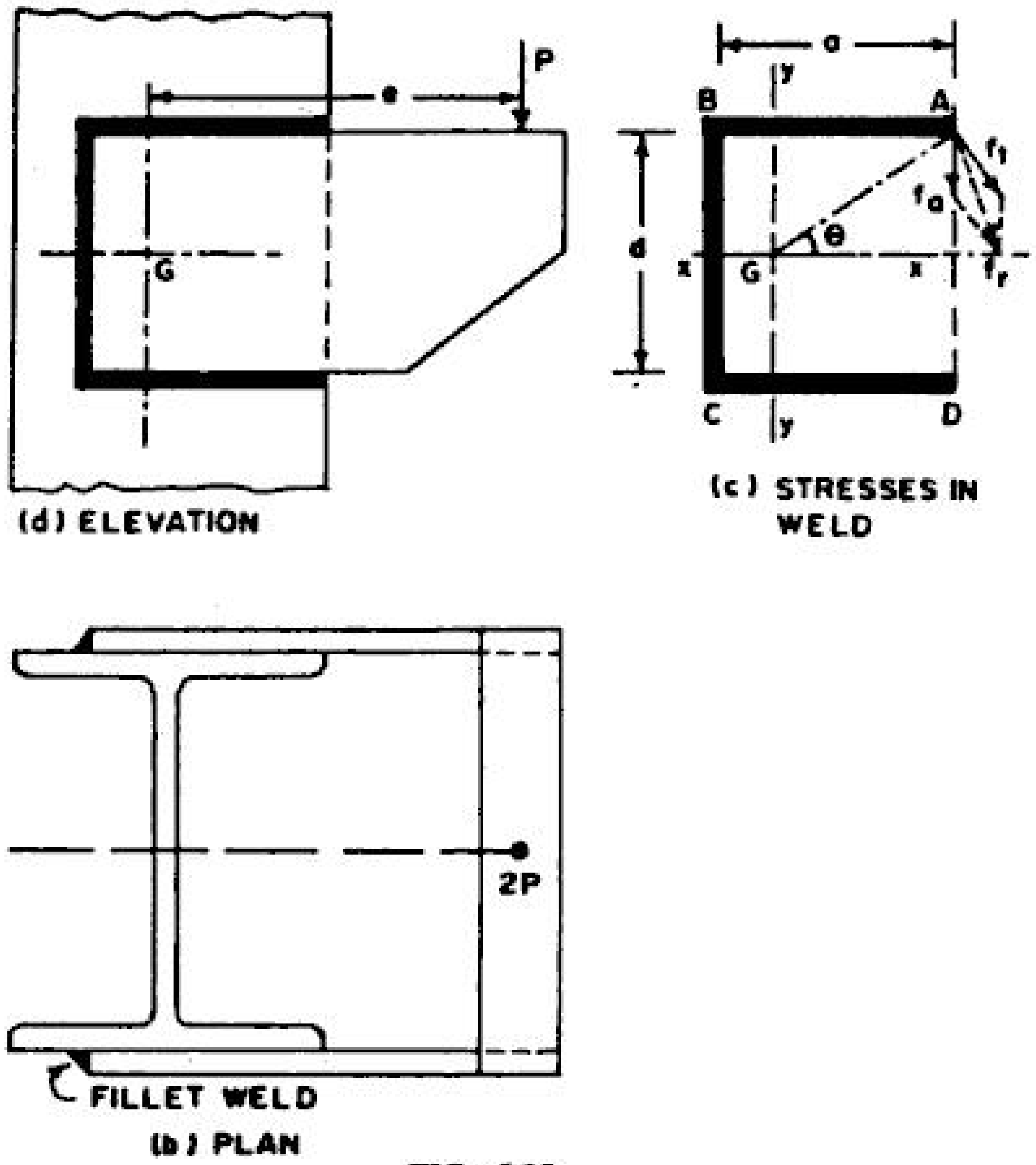


FIG. 5.35.

5.14. ECCENTRICALLY LOADED BUTT WELDED JOINTS

In the case of a butt welded joint subjected to eccentric loading, the load will never lie in the plane of welds. Fig. 5.36 shows a bracket connection in which the plate bracket has been welded to the flange of steel stanchion by way of a full penetration butt weld. The length (L) of the butt weld is equal to the height d of the plate bracket.

Let t = thickness of weld throat

Bending moment $M = P\,.\,e$.

The weld line will be subjected to two types of stresses : (i) Vertical shear stress f_a due to load P and (ii) Tensile or compressive stress f_b due to moment M.

$$f_a = \frac{P}{d\,.\,t} \qquad ...(5.13)$$

and $$f_b = \frac{M\,.\,y}{I_{xx}} = \frac{P\,.\,e\,.\,d/2}{\frac{1}{12} t d^3}$$

or $$f_b = \frac{6\,.\,P\,.\,e}{t d^2} \qquad ...(5.14)$$

As per IS : 816-1969, for a weld subjected to combined shear and bending stresses, the *equivalent stress* f_e is given by

$$f_e = \sqrt{f_b^2 + 3 f_a^2} \qquad ...(5.15)$$

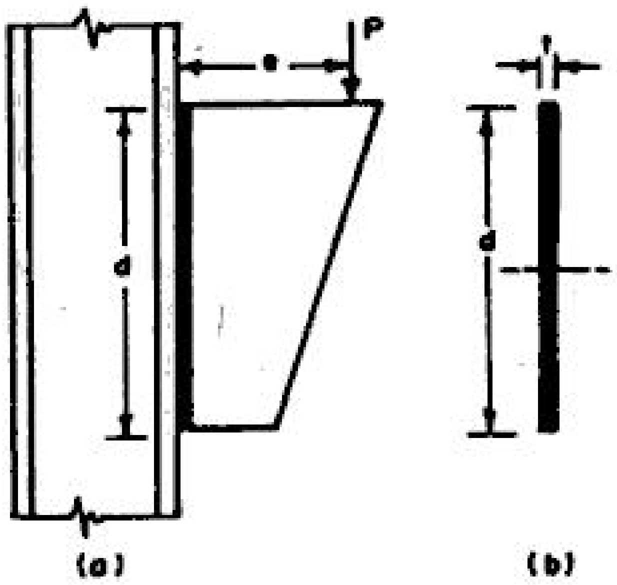


FIG. 5.36.

As per IS : 800-1984, the equivalent stress f_e should not exceed the max. permissible equivalent stress σ_e.

where σ_e = maximum permissible equivalent stress $= 0.9 f_y$

f_y = yield stress in parent metal.

For design purposes, the depth of bracket is given by

$$d = \sqrt{\frac{6.P.e}{t\,\sigma_b}} \quad ...(5.16)$$

where σ_b = permissible bending stress in the butt weld $= 0.66 f_y = 0.66 \times 250 = 165$ N/mm^2 for mild steel.

Example 5.13. *A plate bracket, carrying a load of 100 kN at an eccentricity of 120 mm, is connected to the face of a steel stanchion by fillet welds on both the sides of the plate, as shown in Fig. 5.37. Determine the size of the fillet weld.*

(b) If 8 mm fillet weld is used, determine the depth of the bracket.

(c) If 8 mm fillet weld is used with a bracket of 250 mm depth, calculate the resulting stress in the weld.

Solution

(*a*) Here, $d = 300$ mm. Let t be the throat thickness.

Vertical shear stress :

$$f_a = \frac{P}{2dt} = \frac{100 \times 1000}{2 \times 300\,t} = \frac{166.67}{t} \text{ N/mm}^2 \quad ...(1)$$

Horizontal shear stress :

$$f_b = \frac{3P.e}{td^2} = \frac{3 \times 100 \times 1000 \times 120}{t\,(300)^2} = \frac{400}{t} \text{ N/mm}^2 \quad ...(2)$$

$$\therefore \quad f_r = \sqrt{f_a^2 + f_b^2} = \sqrt{\left(\frac{166.67}{t}\right)^2 + \left(\frac{400}{t}\right)^2} = \frac{433.3}{t} \text{ N/mm}^2.$$

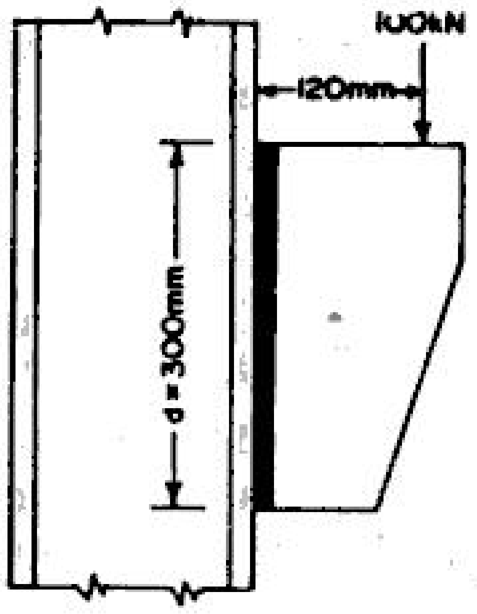


FIG. 5.37.

Equating this to permissible shear stress $\tau_{vp} = 110$ N/mm^2,

we get $$t = \frac{433.3}{110} = 3.94 \text{ mm}$$

$\therefore$ Size of weld $= \frac{t}{0.7} = \frac{3.97}{0.7} = 5.6$ mm. Hence provide 6 mm size weld.

(*b*) If size of weld is 8 mm, we have $t = 0.7 \times 8 = 5.6$ mm

Hence from Eq. 5.9,

$$d = \sqrt{\frac{3P.e}{t\,\tau_{vp}}} = \sqrt{\frac{3 \times 100 \times 1000 \times 120}{5.6 \times 110}} \approx \mathbf{242\ mm}$$

(*c*) Given: size of weld = 8 mm and $d = 250$ mm.

$$\therefore \quad t = 0.7 \times 8 = 5.6 \text{ mm}$$

$$f_a = \frac{P}{2dt} = \frac{100 \times 1000}{2 \times 250 \times 5.6} = 35.71 \text{ N/mm}^2$$

$$f_b = \frac{3Pe}{td^2} = \frac{3 \times 100 \times 1000 \times 120}{5.6\,(250)^2} = 102.85 \text{ N/mm}^2$$

$$\therefore \quad f_r = \sqrt{f_a^2 + f_b^2} = \sqrt{(35.71)^2 + (102.85)^2} = \mathbf{108.9\ N/mm^2 < 110\ N/mm^2}.$$

Example 5.14. *A bracket carrying a load of 120 kN is connected to column by means of two horizontal fillet welds, each of 150 mm effective length and 10 mm thick. The load acts at 80 mm from the face of the column as shown in Fig. 5.38. Find the throat stress.*

Solution

Fig. 5.38 shows the arrangement of the bracket.

$$t = 0.7 \times 10 = 7 \text{ mm}$$

$$f_a = \frac{P}{2Lt} = \frac{120 \times 1000}{2 \times 150 \times 7} = 57.14 \text{ N/mm}^2$$

Moment $= 120000 \times 80 = 96 \times 10^5$ N-mm.

The forces due to bending in the two welds will form a resisting couple $= (150 \times 7 \times f_b) \times 120$.

$$\therefore \quad 150 \times 7 \times f_b \times 120 = 96 \times 10^5$$

From which $f_b = 76.19 \text{ N/mm}^2$

$$\therefore \quad f_r = \sqrt{(57.14)^2 + (76.19)^2} = \mathbf{95.2 \text{ N/mm}^2}$$

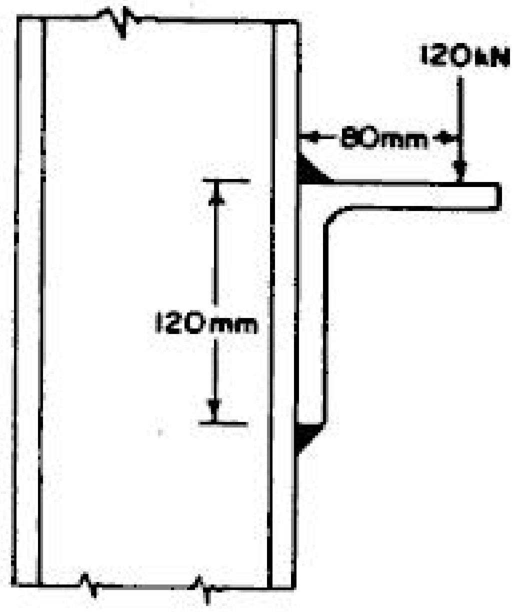


FIG. 5.38.

Example 5.15. *A bracket of I-section is welded to a steel stanchion, by using flange welds as well as web welds as shown in Fig. 5.39. The size of flange welds are double the size of web welds. Determine suitable weld size, taking a permissible shear stress of 110 N/mm² in the welds.*

Solution

Let t be the throat thickness of web welds and $2t$ be the throat thickness of flange welds. Let us also assume that the dimensions marked are *effective lengths* of the welds.

Area of flange welds, at throat $= 2 \times 200\,(2t) = 800\,t \text{ mm}^2$

Area of web welds, at throat $= 2 \times 300\,(t) = 600\,t \text{ mm}^2$

Total weld area $= 800\,t + 600\,t = 1400\,t$

$$\therefore \text{ Vertical shear stress in the welds } f_a = \frac{300 \times 1000}{1400\,t} = \frac{214.29}{t} \text{ N/mm}^2 \quad ...(1)$$

The bending stress at any point in the weld is given by $f_b = \frac{M}{I_{xx}} . y$. Here,

$$I_{xx} = 2\left[200 \times 2t \times 200^2 + \frac{1}{12} t\,(300)^3\right]$$

$$= 365 \times 10^5\,t \text{ mm}^4$$

Hence in the flange weld,

$$f_{bf} = \frac{300 \times 10^3 \times 180}{365 \times 10^5\,t} \times 200 = \frac{295.89}{t} \text{ N/mm}^2 \quad ...(ii)$$

and in the extreme fibre in web weld,

$$f_{bw} = \frac{300 \times 10^3 \times 180}{365 \times 10^5\,t} \times 150 = \frac{221.92}{t} \text{ N/mm}^2 \quad ...(iii)$$

Evidently, the horizontal shear stress due to bending, in the flange weld is more than

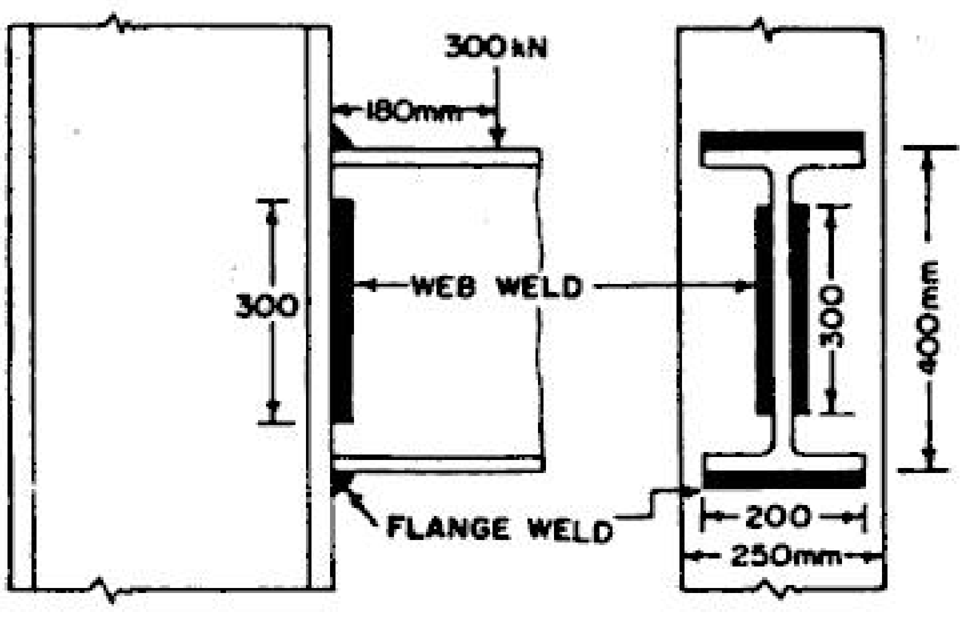


FIG. 5.39.

that in the web weld. Hence the flange weld becomes the governing criterion.

The resultant shear stress in flange weld is

$$f_{rf} = \sqrt{f_a^2 + f_{bf}^2} = \sqrt{\left(\frac{214.29}{t}\right)^2 + \left(\frac{295.89}{t}\right)^2} = \frac{365.34}{t}\ \text{N/mm}^2.$$

Equating this to permissible value $\tau_{vp} = 110\ \text{N/mm}^2$, we get

$$t = \frac{365.34}{110} = 3.32\ \text{mm}$$

$\therefore$ Size of web weld $= \dfrac{3.32}{0.7} = 4.74$ mm

$\therefore$ Provide 5 mm size weld in the webs and 10 mm size weld in the flanges.

Example 5.16. *A 12 mm thick plate bracket, carrying a load of 400 kN at a distance of 500 mm is to be welded to the face of steel stanchion, using intermittent fillet welds applied on both sides of the plate bracket. Design the joint.*

Solution

Let us use 10 mm size weld. The intermittent weld, shown in Fig. 5.40 is analogous to riveted joint subjected to moment when the load is not in the plane of the rivets.

The number of weld lengths, in each line, can be estimated by use of Eq. 3.22:

$$n = \sqrt{\frac{6M}{lpR}}$$

where l = no. of weld lines = 2 in the present case
p = pitch of intermittent welds
R = strength of each portion of intermittent weld.

(*i*) Min. effective length of intermittent weld = 4 × size of weld or 40 mm which ever is greater.

Hence using 10 mm size weld, keep *effective length* = 40 mm and *actual length* = 40 + 2 × 10 = 60 mm.

(*ii*) Clear spacing of the intermittent welds should not be greater than 16 times thickness of plate or 200 mm. Let us use pitch of 150 mm. Hence clear spacing between effective lengths = 150 − 40 = 110 mm, while the actual clear spacing will be = 150 − 60 = 90 mm only.

In the above equation, using $l = 2$, $p = 150$ mm

and R = strength of each weld
$= 0.7 \times 10 \times 40 \times 110 = 30800$ N,

we get

$$n = \sqrt{\frac{6\,(400 \times 1000 \times 500)}{2 \times 150 \times 30800}} = 11.4, \text{ say } 12$$

Hence provide 12 weld lengths on both sides, each of 60 mm actual length (or 40 mm effective length), at c/c spacing of 150 mm.

Total height of bracket

$= (11 \times 90) + (12 \times 60) = 1710$ mm.

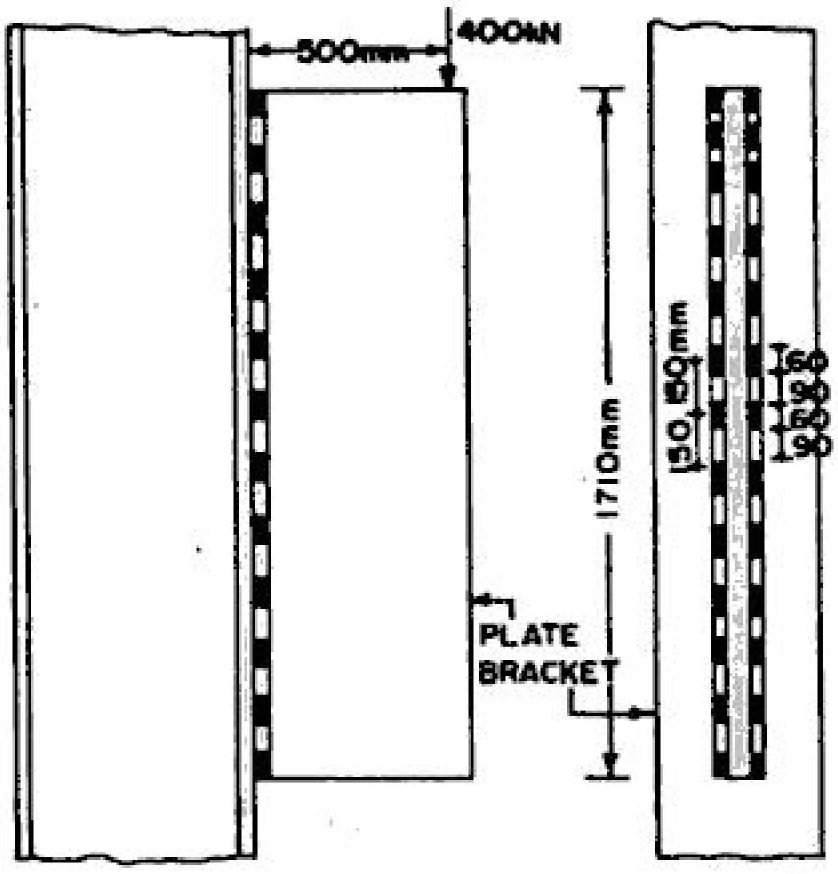


FIG. 5.40.

Example 5.17. *A circular shaft of diameter 120 mm is welded to a rigid plate by a fillet weld of size 6 mm. If a torque of 8 kN-m is applied to the shaft, find the maximum stress in the weld.*

Solution

We shall first derive the relation between the stress developed and the torque applied for such a shaft. Let d be its diameter and h be the size of the weld. Let t be the throat thickness $= 0.7h$. Here $t = 0.7 \times 6 = 4.2$ mm.

Considering a small area δa of the weld as shown in Fig. 5.41,

$$J \approx \Sigma \delta a \times \left(\frac{d}{2}\right)^2$$

$$\therefore \quad J = \frac{d^2}{4} \Sigma \delta a$$

where $\Sigma \delta a = \pi d . t$

$$\therefore \quad J = \frac{d^2}{4} . \pi d . t = \frac{\pi d^3}{4} t.$$

Torsional shear stress $f_t = \dfrac{T . r}{J} = T . \dfrac{d}{2} \times \dfrac{4}{\pi d^3 t}$

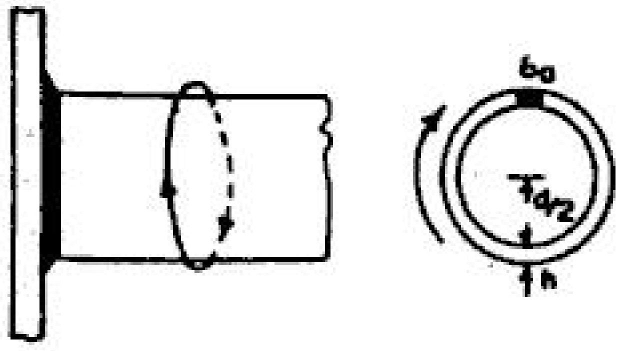


FIG. 5.41.

or $$f_t = \frac{2T}{\pi d^2 t} \qquad ...(5.17)$$

where $T = 8$ kN-m $= 8 \times 10^6$ N-mm. Substituting the values, we get

$$f_t = \frac{2 \times 8 \times 10^6}{\pi (120)^2 \times 4.2} = \mathbf{84.2\ N/mm^2}$$

Example 5.18. *A circular steel pipe 120 mm external diameter and 100 mm internal diameter is welded to a rectangular plate 12 mm thick by fillet weld around the perimeter. The pipe is subjected to a vertical point load of 12 kN acting at 500 mm from the welded end. It is also subjected to a twisting moment of 4 kN-m. Determine the size of the weld, taking permissible shear stress equal to 110 N/mm².*

Solution.

The weld section is subjected to *three* stresses.

(*i*) Vertical shearing stress f_a due to vertical force $P = 12$ kN

(*ii*) Horizontal shearing stress f_b due to moment

$M = P . e = 12 \times 10^3 \times 500 = 6 \times 10^6$ N-mm.

(*iii*) Torsional shearing stress f_t due to torsional moment

$T = 4$ kN-m $= 4 \times 10^6$ N-mm.

Let t be the throat thickness of the fillet weld.

(*i*) *Vertical shear stress* f_a *due to* S. F. P

$$f_a = \frac{P}{Lt} = \frac{P}{\pi d t} = \frac{12000}{\pi \times 120 \times t}$$

$$= \frac{31.8}{t} \text{ N/mm}^2$$

This stress acts vertically downwards, at all sections of the weld.

(*ii*) *Horizontal shear stress* f_b *due to* $M = P . e = 6 \times 10^6$ N-mm

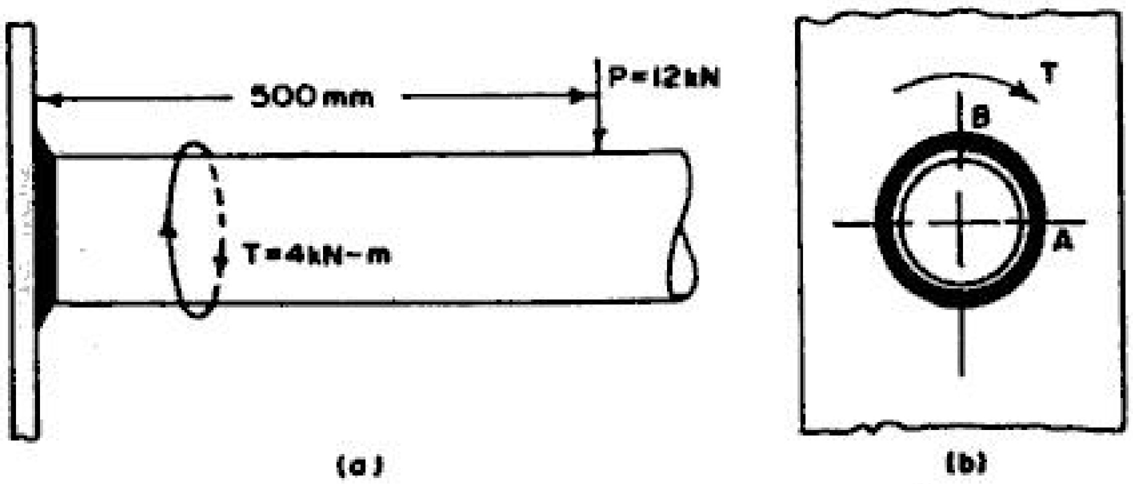


FIG. 5.42.

$$f_b = \frac{M}{I} \cdot y$$

where $I \approx \frac{\pi d^3}{8} t = \frac{\pi (120)^3}{8} t = 678584\, t$

and $y = \frac{d}{2} = 60$ mm

$$\therefore \quad f_b = \frac{6 \times 10^6}{678584\, t} \times 60 = \frac{530.5}{t} \text{ N/mm}^2$$

This stress acts perpendicular to the plane of the weld.

(*iii*) *Torsional shear stress* f_t *due to* $T = 4$ kN-m.

$$f_t = \frac{T}{J} \cdot r$$

where $J \approx \frac{\pi d^3}{4} t = \frac{\pi (120)^3}{4} t = 1357168\, t$

$$r = \frac{d}{2} = 60 \text{ mm}$$

$$\therefore \quad f_t = \frac{4 \times 10^6}{1357168\, t} \times 60 = \frac{176.8}{t} \text{ N/mm}^2$$

This stress acts in the plane of weld, along tangent (*i.e.* perpendicular to the radius vector).

Let us now consider the combined effect of the three.

At point A: f_b is zero while f_a and f_t are additive since both act vertically downwards. Hence the resultant stress is

$$f_{rA} = f_a + f_t = \frac{31.8}{t} + \frac{176.8}{t} = \frac{208.6}{t} \text{ N/mm}^2 \quad ...(i)$$

At point B : f_a and f_t act at right angles to each other, in the same plane (*i.e.* the plane of the weld) while f_b attains its maximum value and acts perpendicular to the plane of the weld. Hence the resultant stress is

$$f_{rB} = \sqrt{f_a^2 + f_t^2 + f_b^2} = \sqrt{\left(\frac{31.8}{t}\right)^2 + \left(\frac{176.8}{t}\right)^2 + \left(\frac{530.5}{t}\right)^2}$$

$$= \frac{560.1}{t} \text{ N/mm}^2 \quad ...(ii)$$

Thus maximum stress is induced at point *B*, which is the critical point. The stress at this point should not exceed permissible stress $\tau_{vp} = 110$ N/mm².

$$\therefore \quad \frac{560.1}{t} \le 110$$

From which $t \ge \frac{560.1}{110} = 5.09$

Hence size of weld $= \frac{t}{0.7} = \frac{5.09}{0.7} = 7.27$ mm

As per IS 816-1969, for the thickness= 12 mm of the thicker part, minimum size of weld is 5 mm. Hence provide **7.5 mm** size fillet weld.

Example 5.19. *A bracket is subjected to a load of 100 kN as shown in Fig. 5.43. The bracket is welded to a stanchion by means of three weld lines as indicated in Fig. 5.43. Find out the size of the welds so that the load is carried safely.*

Solution

Let t be the throat thickness of the weld. If $\bar{x}$ is the distance of centroid of weld area from AB, we have

$$\bar{x} = \frac{2 \times 120t \times 60}{2 \times 120t + 240t} = 30 \text{ mm}$$

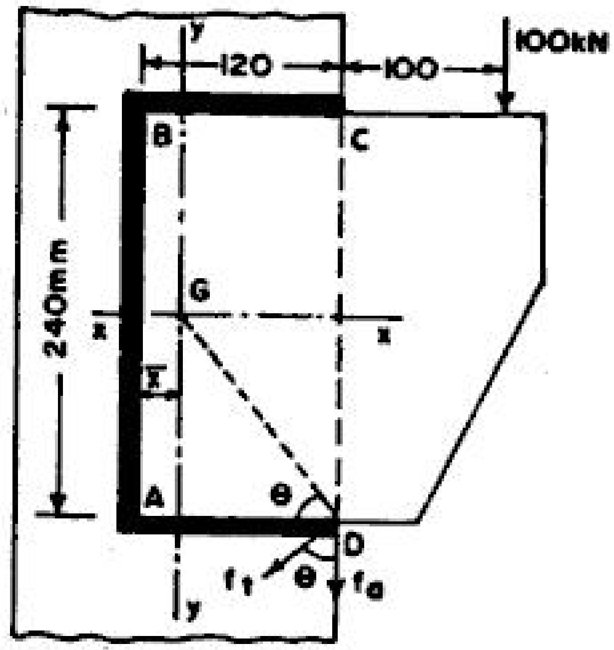


FIG. 5.43.

$\therefore$ Eccentricity of the load $= 100 + 120 - 30 = 190$ mm

$$I_{xx} = \frac{1}{12} t\,(240)^3 + 2 \times 120t\,(120)^2 = 460.8 \times 10^4 t \text{ mm}^4$$

$$I_{yy} = 240t\,(30)^2 + 2 \times \frac{1}{12} t\,(120)^3 + 2 \times 120t\,(60-30)^2$$

$$= 72 \times 10^4 t \text{ mm}^4$$

$$J = I_{zz} = I_{xx} + I_{yy}$$

$$= 460.8 \times 10^4 t + 72 \times 10^4 t = 532.8 \times 10^4 t \text{ mm}^4$$

$$\text{Area} = 2 \times 120t + 240t = 480t \text{ mm}^2$$

$$\text{Vertical shear } f_a = \frac{P}{A} = \frac{100 \times 1000}{480t} = \frac{208.3}{t} \text{ N/mm}^2 \quad ...(1)$$

The maximum stress due to torsion will occur either at C or D.

Length of radius vector for C or $D = \sqrt{(120)^2 + (90)^2} = 150$ mm

Maximum shear stress due to torsion,

$$f_t = \frac{T}{J} \cdot r = \frac{100000 \times 190}{532.8 \times 10^4 t} \times 150 = \frac{534.9}{t} \text{ N/mm}^2$$

Now $$\cos\theta = \frac{90}{150} = 0.6$$

Resultant stress is given by

$$f_r = \sqrt{f_a^2 + f_t^2 + 2 f_a f_t \cos\theta}$$

$$= \sqrt{\left(\frac{208.3}{t}\right)^2 + \left(\frac{534.9}{t}\right)^2 + 2\left(\frac{208.3}{t}\right)\left(\frac{534.9}{t}\right) \times 0.6} = \frac{680.6}{t} \text{ N/mm}^2$$

Equating this to the allowable shear stress of 110 N/mm², we get

$$t = \frac{680.6}{110} = 6.19 \text{ mm}$$

$\therefore$ Size of weld $= \frac{6.19}{0.7} = 8.84$ mm. Hence provide **9 mm** size weld.

Example 5.20. *A plate bracket, made of 10 mm thick plate is butt welded to the flange of a steel stanchion shown in Fig. 5.44. The bracket carries a load of 100 kN at an eccentricity of 110 mm from the face of the stanchion. Design the joint.*

Solution

Let is assume that the butt weld will have complete penetration. In that case the throat thickness will be equal to the thickness of plate of the bracket. Hence $t = 10$ mm

The depth of the bracket can be determined by the approximate expression:

$$d = \sqrt{\frac{6 . P . e}{t\,\sigma_b}} \quad ...(5.16)$$

Here $P = 100$ kN

$e = 110$ mm

$\sigma_b = 165$ N/mm^2.

$$\therefore \quad d = \sqrt{\frac{6 \times 100 \times 10^3 \times 110}{10 \times 165}} = 200 \text{ mm}$$

Let us provide a depth = 200 mm

Check for stresses :

Actual direct shear stress,

$$f_a = \frac{P}{dt} = \frac{100 \times 1000}{200 \times 10} = 50 \text{ N/mm}^2$$

Actual bending stress

$$f_b = \frac{6\,Pe}{td^2} = \frac{6 \times 100 \times 1000 \times 110}{10\,(200)^2} = 165 \text{ N/mm}^2$$

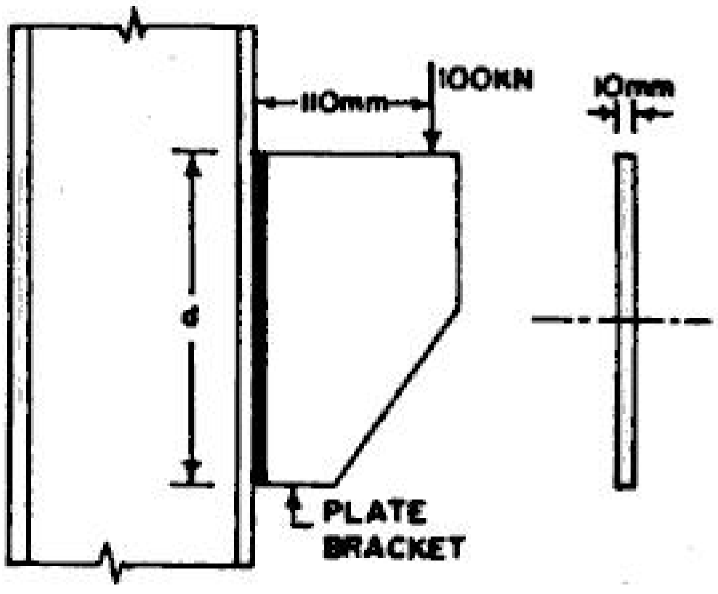


FIG. 5.44.

As per IS : 816-1969, the equivalent shear stress is given by

$$f_e = \sqrt{f_b^2 + 3f_a^2} = \sqrt{(165)^2 + 3\,(50)^2} = 186.35 \text{ N/mm}^2$$

As per IS : 800-1984, the equivalent stress f_e should be less than maximum permissible equivalent stress $\sigma_e = 0.9 f_y$

Taking $f_y = 250$ N/mm^2,

$\sigma_e = 0.9 \times 250 = 225$ N/mm^2.

Since $f_e < \sigma_e$, the design is safe.

PROBLEMS

1. A 150 mm × 16 mm plate is welded to other plate by two side welds 120 mm each and end fillet of 100 mm length. Find the safe axial load to which this joint may be subjected if the size of the weld is 7 mm.

2. A 100 mm × 10 mm plate is welded to other by means of two end fillets and two side fillets of 8 mm as shown in Fig. 5.45. If the plate is loaded to its full strength, find out the required overlap length.

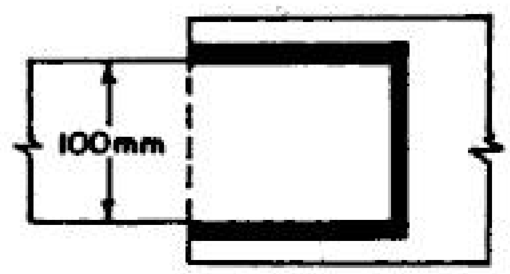


FIG. 5.45.

3. An equal angle 75 mm × 75 mm @ 11.0 kg/m is subjected to a load of 180 kN, whose line of action passes through the centroid of the section, which is at 22.2 mm from the heel. This angle is to be welded to a gusset plate. If the size of the weld is to be 8 mm, find the length of the side fillet welds.

4. An I-section is made up of a 200 mm × 10 mm thick web plate welded to two flange plates 120 mm × 10 mm thick by means of fillet welds to size 6 mm. Calculate the maximum shear force which this section can resist.

5. Fig. 5.46 shows a 10 mm angle bracket 100 mm wide welded to the flange of steel stanchion. It carries a vertical load of 240 kN. The connection consists of continuous 10 mm weld extending along the top and both sides and returned at the bottom of the bracket. Treating the 240 kN load as a vertical shear load (i.e. neglecting bending moment), calculate the depth of bracket, taking 110 N/mm^2 as the working stress in transverse weld and 79 N/mm^2 in the longitudinal weld.

(Based on U.L.)

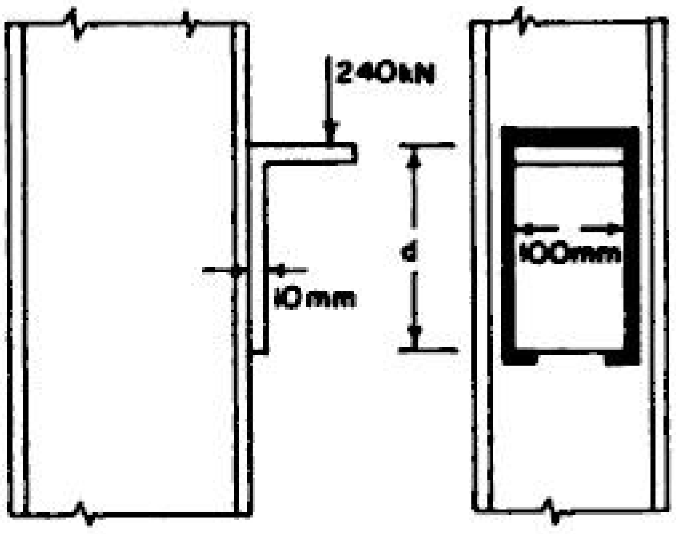


FIG. 5.46.

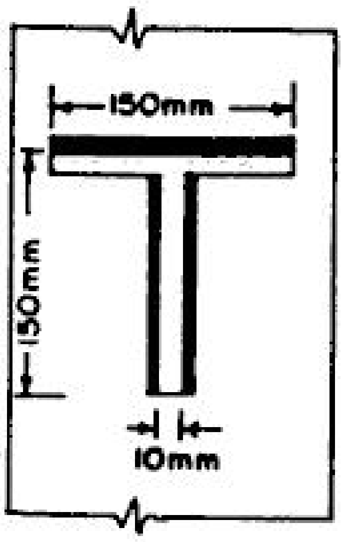


FIG. 5.47.

6. An I-section bracket carrying 120 kN load is connected by a column as shown in Fig. 5.47 by means of two side fillet welds 200 mm deep. The load is eccentric by 70 mm. Calculate the size of the fillet weld.

7. A bracket consisting of T-section 150 mm × 150 mm and 10 mm is connected to a column as shown in Fig. 5.47. The bracket carries 150 kN load at 80 mm eccentricity. If the size of the weld is 6 mm, find out the maximum throat thickness.

8. A bracket shown in Fig. 5.48 is welded to a stanchion by side fillet welds on three sides as indicated by heavy lines. Calculate the maximum force per mm of weld when the bracket carries a load of 200 kN acting as shown.

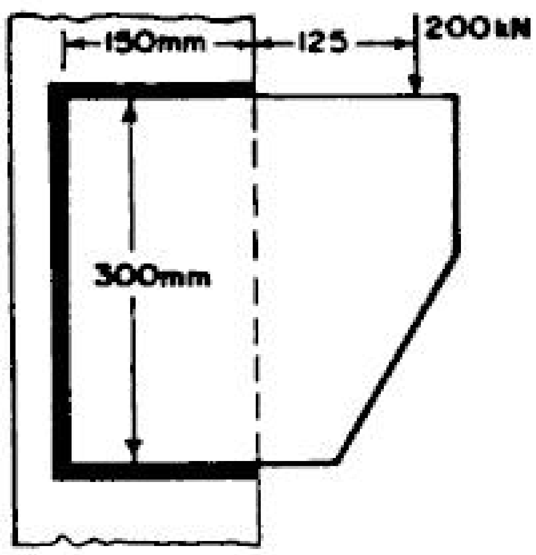


FIG. 5.48.

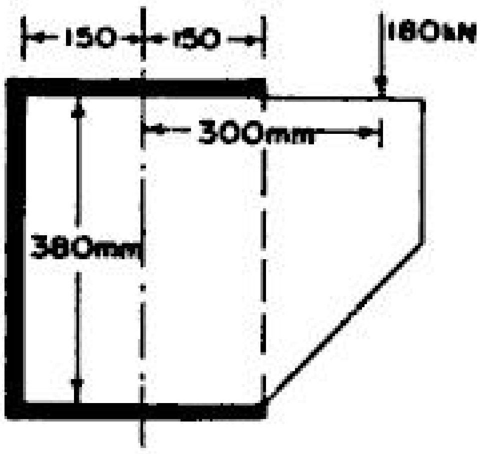


FIG. 5.49.

9. A bracket is welded to a stanchion by fillet welds having a throat thickness of 9 mm and a load of 180 kN is applied in the plane of the bracket, as shown in Fig. 5.49. The weld extends round three sides and has the given dimensions. Determine the maximum stress on the throat of the weld.

10. Determine the size of fillet weld required to join a plate bracket with flange of a stanchion, as shown in Fig. 5.50.

11. Determine the depth of the bracket of Fig. 5.50, if the plate is butt welded, using full penetration butt weld. The thickness of plate is 10 mm.

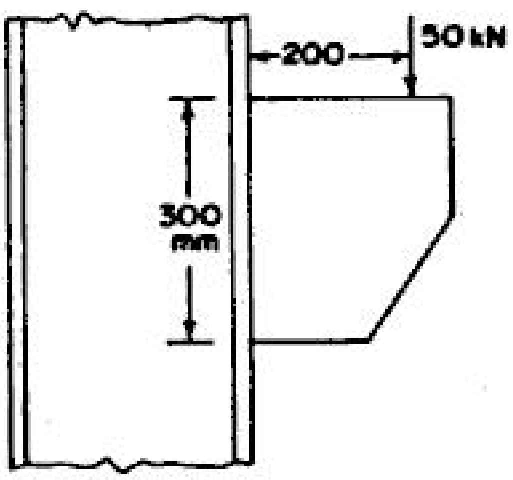


FIG. 5.50.

6

Design of Tension Members

6.1. INTRODUCTION

A tension member is the one which is intended to resist axial tension. Tension members are also called *ties* or *hangers*. In contrast to compression members, the disposition of material in a tie has no effect on its structural efficiency so that compact sections such as rods may be used without reduction in allowable stress. For tensile force to be axial, it is necessary that the load is applied through the centroid of the section of the member. However, inspite of all precautions, the fabrication work may cause some initial crookedness, due to which the so called axially tensioned members may have small eccentricity. The axial tensile force applied to the member has a tendency to straighten the member, which in turn reduces the initial eccentricity. Therefore, small unknown eccentricities are usually neglected in the design.

6.2. TYPES OF TENSION MEMBERS

In general, tension members can be divided into four groups:

(*i*) Wires and cables.

(*ii*) Rods and bars.

(*iii*) Single structural shapes and plates

and (*iv*) Built-up sections.

Typical tension members are shown in Fig. 6.1.

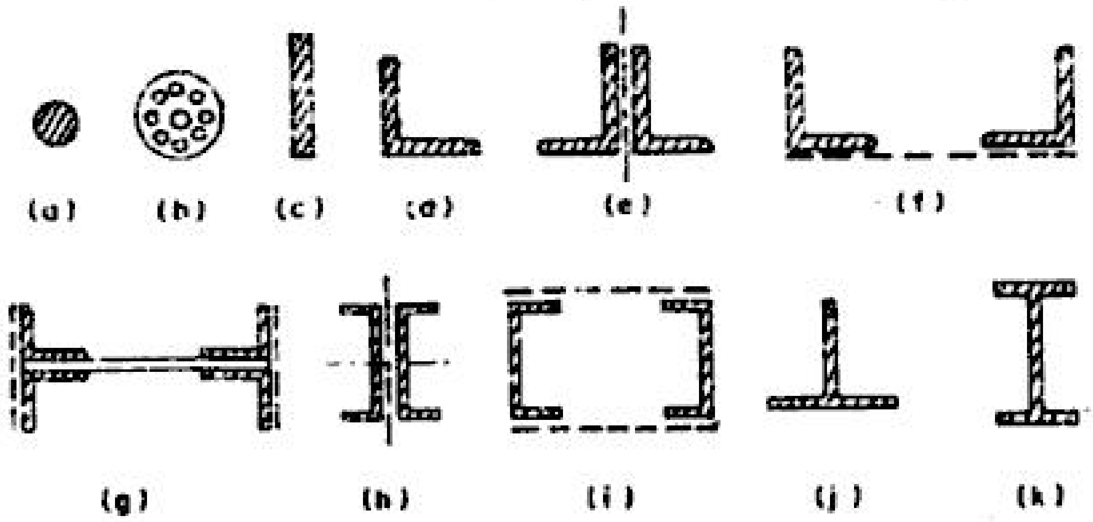


FIG. 6.1. TYPICAL TENSION MEMBERS.

1. Wires and cables: Wires used as structural tension members are usually cold-drawn from hot-rolled rods. Wire size is often specified by the gauge number rather than the diameter. Wire ropes or cables are used for hoists, derricks, rigging slings, guy wires, hangers for the suspension bridges etc. Wire rope is made by twisting wires into strands (generally 7 or 19 wires per strand), which are then twisted in opposite direction around a fiber or strand core to form a cable of several strands—usually six. The advantages of wire rope lie in its flexibility and strength. However, they require special fittings (such as end socket, eyebar or turn buckle etc) for proper end connections. These wires are glavanized or painted, when exposed to severe weathering.

2. Rods and bars: Members carrying tensile force of small magnitude may be made from hot-rolled square or round rods, or of flat bars. Round bars are threaded at the ends and are used with nuts. Because of slenderness of these members, their compressive strength is negligible. When bars are used, it is desirable to place the larger dimension in the vertical plane so that sagging is reduced. Flat bars may be welded, riveted or bolted to an adjoining part, or forged into loop or eyebar ends and connected to a pin. The major disadvantage of these sections is the *inadequate stiffeness* resulting in noticeable sag under their own weight, specially during erection.

3. Single structural shapes and plates: The common single structural shapes are angle sections, tee-sections and channel sections. Angle sections are considered more rigid than wires, ropes, rods and bars. However, one single objection to the single angle is the presence of eccentricity in the connection. Single angles are mainly used for bracing, for light truss tension members. They can also resist small compression if reversal of stresses takes place. Single channels can sometimes be effectively employed as tension members, because for the same cross-sectional area, the channel has lesser eccentricity than the angle. Occasionally, I-sections are also used as tension members as they have more rigidity. Plates are also used as tension members. *Structural tee*, shown in Fig. 6.1 (*j*) make excellent chord members for lightly loaded welded trusses since the stem may serve as gusset for attachment of single-angle or double angle web members. The I-members (Fig. 6.1 *k*) are used as tension members in heavier building or bridge trusses with double plane construction.

4. Built-up Section: When the member is required to take heavy tensile load, built-up section, consisting of two or more plates or shoes are used. Such a built-up section is made to provide (*i*) greater area which cannot be provided by a single rolled shape, (*ii*) greater rigidity, by way of greater moment of inertia, or (*iii*) suitable dimension to facilitate proper connection. The section shown in Fig. 6.1 (*e*), obtained by two angles placed back to back is extensively used in *single-plane* (single gusset) trusses. Fig 6.1 (*f*) and (*g*) show *two angle* and *four angle* members frequently used in light *double plane* (double gusset) riveted trusses. Fig 6.1 (*h*) shows a form of tension member sometimes used for single plane trusses when bending must also be resisted in addition to axial tension. Form shown in Fig. 6.1 (*i*) is used for heavier trusses, with their open sides provided with intermittent tie plates or lattice bars as shown by horizontal dotted lines.

6.3. NET SECTIONAL AREA : PLATES

When a tension member is spliced or joined to a gusset plate or any other member, by rivets or pins or bolts, its gross-sectional area is reduced by the holes of these fasteners. Hence a tension member is designed for its net sectional area at the joint.

(*a*) Chain riveting in plate section

Consider plates carrying tension, joined by chain riveting, as shown in Fig. 6.2.

If there are n number of rivet holes in each section, such as 1-1 or 2-2, we have

$$A_{net} = bt - n\,(d\,.\,t)$$

or $$A_{net} = t\,[b - nd] \qquad ...(6.1)$$

where t = thickness of the plate

d = diameter of the hole.

In Fig. 6.2 $n = 3$

$\therefore$ $A_{net} = t\,[b - 3d]$

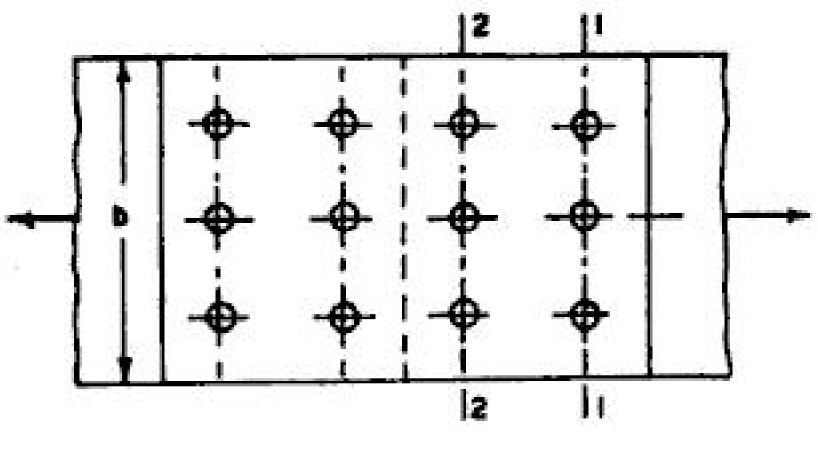


FIG. 6.2.

(b) Zig zag or staggered riveting

In the case of zigzag or staggered riveting, the net cross- sectional area along the chain of rivets is increased by an amount equal to $\frac{s^2 t}{4g}$.

where s = staggered pitch, *i.e.* the distance between any two consecutive rivets, measured parallel to the direction of force in the member.

g = gauge distance, *i.e.* the distance between the same two consecutive rivets in a chain, measured at right angles to the direction of force.

Thus, in Fig. 6.3 (*a*) *B*, *C* and *D* are not in the same line, but are staggered. Rivet *B* is staggered by a distance s_1 from rivet *C*, while rivet *D* is staggered by a distance s_2 from rivet *C*. Hence *net area* along section *ABCDE* is given by Steinman's formula:

$$A_{net} = t\left[(b - nd) + \left(\frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2} \right) \right] \qquad ...(6.2)$$

or Deduction = (sum of sectional area of holes, *B*, *C* and *D*) $- \left[\frac{s_1^2 t}{4g_1} + \frac{s_2^2 t}{4g_2} \right]$

In Fig. 6.3 (*a*), $n = 3$. The chain of lines should be so chosen that it produces maximum such *deduction*.

In general, we have Steinman's formula for uniform value of s:

$$A_{net} = t\left[(b - nd) + m\frac{s^2}{4g} \right] \qquad ...(6.3)$$

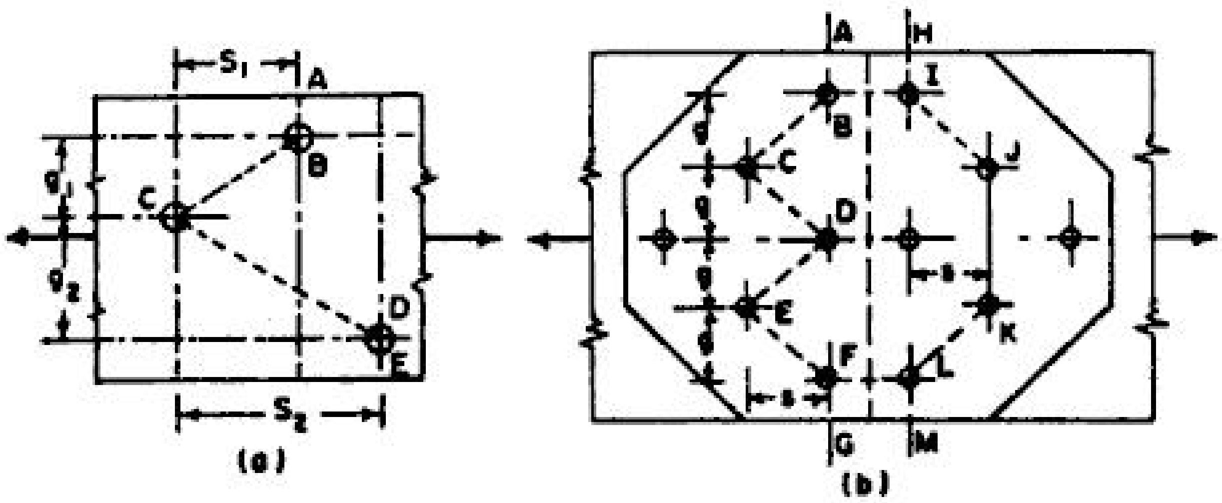


FIG. 6.3. STAGGERED RIVETING

where

n = no. of rivets in the section considered.

m = no. of zigzags or inclined lines.

Thus, in Fig. 6.3 (*b*)

(*i*) For line *ABCDEFG*, $n = 5$ and $m = 4$

$$\therefore \qquad A_{net} = t\left[(b - 5d) + 4\frac{s^2}{4g} \right]$$

(*ii*) Similarly, for line *HIJKLM*, $n = 4$ and $m = 2$

$$A_{net} = t\left[(b - 4d) + 2\frac{s^2}{4g} \right]$$

6.4. NET EFFECTIVE AREAS FOR ANGLES AND TEES IN TENSION

When an angle section is used as tension member, it is connected to the gusset plate by its flange. In both the cases, the rivet connecting the angle or T-section to the gusset plate does not lie on the line of action of load. This gives rise to an eccentric connection, due to which the stress distribution becomes non-uniform. The net cross-sectional area of such a section is *reduced* to account for this non-uniform stress distribution resulting from eccentricity. The reduced net sectional area of such a section is known as *net effective area*.

IS : 800-1984 gives the following recommendations for net effective area of angles and T-sections.

1. Single angle section connected by one leg only

In the case of single angle connected through leg, net effective sectional area is given by

$$A_{net} = A_1 + A_2 k \quad ...(6.4)$$

where A_1 = effective cross-sectional area of connected leg.

A_2 = the gross sectional area of unconnected leg.

$$k = \frac{3A_1}{3A_1 + A_2} \quad ...(6.5)$$

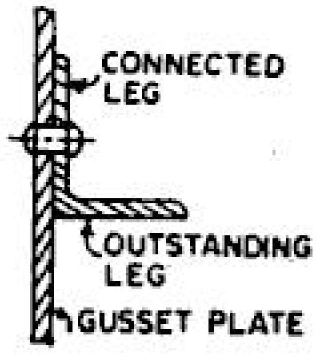


FIG. 6.4.

Where leg angles are used, the effective sectional area of the whole of the angle shall be considered.

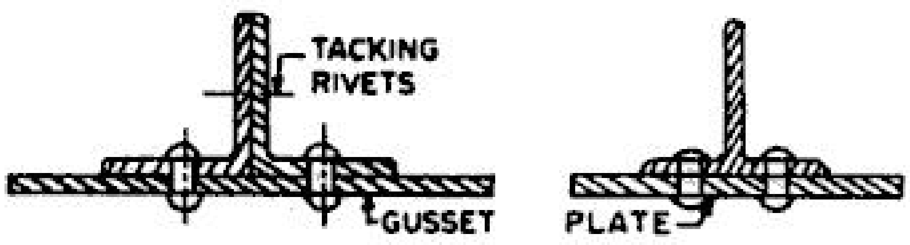
2. Pair of angles back-to-back (or a single tee) connected by one leg of the angle (or by the flange of the tee) to the same side of a gusset

$$A_{net} = A_1 + A_2 k \quad ...(6.6a)$$

where A_1 = effective cross-section area of connected legs (or flange of tee)

A_2 = Gross area of outstanding legs (or web of the tee)

$$k = \frac{5A_1}{5A_1 + A_2} \quad ...(6.6b)$$



(a) DOUBLE ANGLE SECTION (b) T-SECTION

FIG. 6.5.

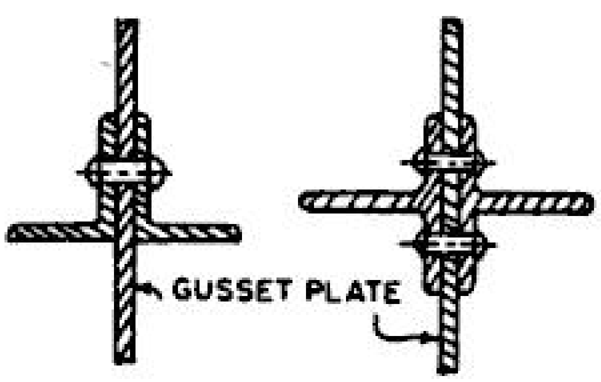
The angles shall be connected together (*i.e.* tacked) along their length so that they act as a single member. For that, the outstanding legs should be connected by taking *tacking rivets* at a pitch not exceeding 1.0 m.

3. Double angles or tees placed back to back and connected to each side of a gusset or to each side of part of a rolled section

Such members are shown in Fig. 6.6, for which the areas to be taken in computing the mean tensile stress shall be the effective area which will be taken equal to the gross area less the deduction of holes. This is subject to the condition that the angles or tees have been connected by tacking rivets along their length, at a pitch not exceeding one metre.

Note (*i*) Where the angles are back to back but are not tack riveted using a pitch not exceeding 1 m, the provision under (2) and (3) stated above shall not apply and each angle shall be designed as a single angle connected through one leg in accordance with provision under para (1).

(*ii*) Where two tees are placed back to back but not tack riveted using a pitch not exceeding 1 m, the provisions under para (3) stated above shall not apply and each tee shall be designed as single Tee connected to one side of a gusset in accordance with para (2).



(a) ANGLES (b) TEES

FIG. 6.6.

(*iii*) The area of the leg of an angle shall be taken as the product of the thickness and the length from the outer corner, minus half the thickness and area of the leg of a tee as the product of the thickness and the depth minus the thickness of the tee.

Hence for angle section shown in Fig 6.7 (*a*)

Gross area of connected leg $= (a - \frac{t}{2})\, t$

Area of outstanding leg $= (b - \frac{t}{2})\, t$

$\therefore \quad A_1 = (a - \frac{t}{2} - d)\, t$

and $\quad A_2 = (b - \frac{t}{2})\, t$

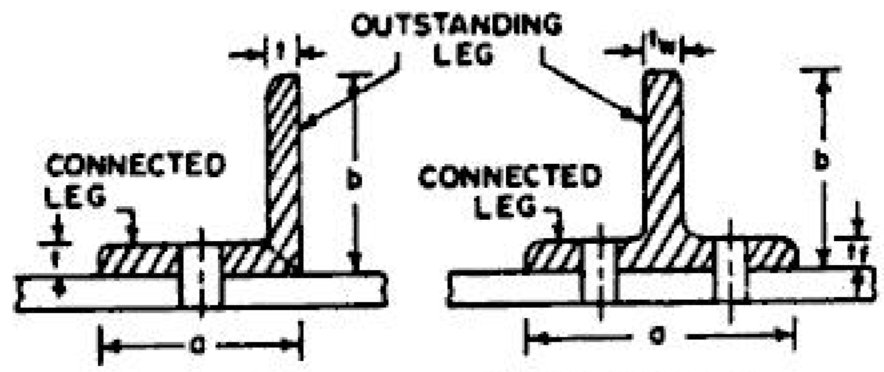


FIG. 6.7.

Similarly, for the T-section shown in Fig. 6.7(*b*).

Gross area of connected flange $= a\, t_f$

Gross area of outstanding leg $= (b - t_f)\, t_w$

$\therefore \quad A_1 = (a - 2d)\, t_f$

and $\quad A_2 = (b - t_f)\, t_w$

6.5. PERMISSIBLE STRESSES

As per IS : 800-1984, the permissible stress in axial tension, σ_{at}, on the net effective area of the section shall not exceed :

$$\sigma_{at} = 0.6 f_y \text{ N/mm}^2 \text{ (MP}_a\text{)}$$

where $\quad f_y$ = minimum yield stress in steel, in N/mm^2 (MP$_a$)

Table 6.1 gives the permissible stress σ_{at} for steel conforming to IS : 226–1975.

TABLE 6.1 ALLOWABLE VALUES OF σ_{at} FOR STEEL CONFORMING TO IS : 226-1975

	Form	*Thickness/ Diameter*	σ_{at} N/mm^2(MP$_a$)
1.	Plates, angles, tees, I-secions, channels and flats	(*i*) upto and including 20 mm (*ii*) 20 mm to 40 mm (*iii*) over 40 mm	150 144 138
2.	Bars (round, square, and hexogonal)	(*i*) upto and including 20 mm (*ii*) over 20 mm	150 144

6.6. DESIGN OF MEMBERS SUBJECTED TO AXIAL TENSION

The design of a member, subjected to axial tension, is done in the following steps.

Step 1. Knowing the axial pull (P) and permissible value of σ_{at}, calculate the net cross-sectional area required:

$$A_{net} = \frac{P}{\sigma_{at}} \qquad ...(6.7)$$

Step 2. Try a suitable section, making an allowance for rivet holes. The section should be so chosen that its gross area is about 25% to 50% more than the net area required. The following *guide lines* are useful:

(*i*) *For single angle*

$$A_{gross} = 1.35 \text{ to } 1.5 \text{ times } A_{net}.$$

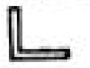
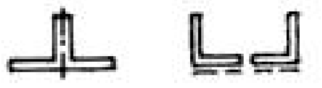
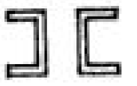
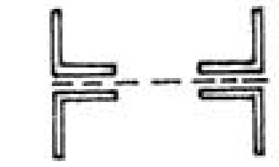
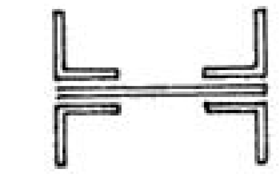
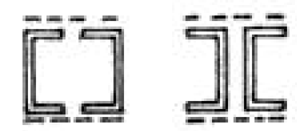
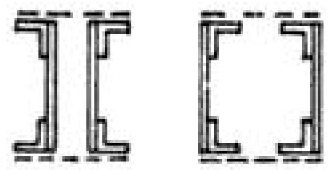
(*ii*) *For double angles*

Angles on same side of gusset plate : $A_{gross} = 1.35\, A_{net}$

Angles on either side of gusset plate : $A_{gross} = 1.25\, A_{net}$.

Step 3. Find actual A_{net} for the section selected in step 2 by making deductions for rivet holes. The number of holes to be deducted from a tension member depends upon the type of the section, and its end connections. However, guidance for the deduction for rivet holes may be taken from Table 6.2.

TABLE 6.2. GUIDE LINES FOR DEDUCTIONS

Form of tension member	*Shape & Arrangement*	*Deduction*
1. Single angle		One hole.
2. Double angles		One hole from each angle.
3. Double channels		Two holes from each web or one hole from each flange, whichever is greater.
4. Four angles (laced)		One hole from each angle.
5. Four angles with web.		One hole from each angle and two holes from web
6. Two channels (Laced)		Two holes from each channel web or one hole from each flange which ever is greater
7. Four angles with or without plates (Laced)		Two holes from each angle and one hole for every 150 mm width of the plate

Step 4. If the actual A_{net} calculated in step 3 is equal to or *slightly more* than the one found in step 1, the selected section is OK. If the actual A_{net} is less than required, try a heavier section. If the actual A_{net} is much more than the required, try a lighter section. Repeat the trial till a *suitable section* is obtained.

Step 5. Check for *slenderness ratio* when the reversal of load may occur. The recommendations of IS : 800-1984 for checking slenderness ratio are as follows :

(*a*) In any tension member in which a reversal of direct stress due to loads other than wind or seismic forces occurs, slenderness ratio should not exceed 180.

(*b*) In a member normally acting as a tie in a roof truss or a bracing system but subject to possible reversal of stress resulting from action of wind or earthquake forces, the slenderness ratio should not exceed 350. For other tension members (other than pretensioned members), the slenderness ratio should not exceed 400.

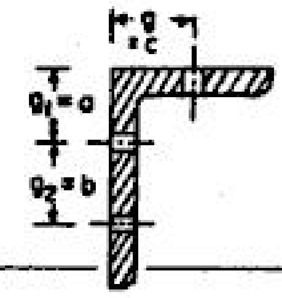
Step 6. Carryout the design of end connections. The number of rivets is given by :

$$n = \frac{P}{R}$$

where P axial tension and R= rivet value.

The rivets in angles should be located on gauge lines. The usual gauges on angles are given in Table 6.3, as per ISI recommendations. (Refer Table XXXI of ISI Hand Book for Structural Engineers)

TABLE 6.3. RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg size (mm)	*Double row of rivets* g_1 (= a) *(mm)*	*Double row of rivets* g_2 (= b) *(mm)*	*Single row of rivets* g (= c) *(mm)*	*Maximum rivets size for double row of rivets (mm)*
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	–	–	55	–
90	–	–	50	–
80	–	–	45	–
75	–	–	40	–
70	–	–	40	–
65	–	–	35	–
60	–	–	35	–
55	–	–	30	–
50	–	–	28	–
45	–	–	25	–
40	–	–	21	–
35	–	–	19	–
30	–	–	17	–
25	–	–	15	–
20	–	–	12	–

Example 6.1. *A mild steel flat of size 160 mm × 12 mm is used as a tension member in a roof truss. It is connected at its ends, to a gusset plate using 20 mm rivets by (a) chain riveting (b) zigzag riveting, as shown in Fig 6.8. Calculate the maximum tension which the flat can carry in each case. Which arrangement is stronger ? Take* $\sigma_{at} = 0.6 f_y$ *where* $f_y = 250\ N/mm^2$.

Solution

Formed dia. of rivets = 20 + 1.5 = 21.5 mm.

$$\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150\ \text{N/mm}^2$$

Case (a) Chain riveting

The most critical section is *abcd* (Fig. 6.8 *a*), where Eq. 6.1 is applicable:

$$A_{net} = t\,[b - nd] = 12\,[160 - 2 \times 21.5] = 1404\ \text{mm}^2$$

∴ Maximum tension in the flat $= \sigma_{at} \cdot A_{net} = 150 \times 1404 = 210600$ N = **210.6 kN**

***Case (b) Staggered riveting* (Fig. 6.8 *b*)**

We will try different sections.

(*i*) *Section efg*: Flat is weakened by only one rivet hole

$A_{net} = t\,[b - nd]$

$= 12\,[\,160 - 1 \times 21.5]$

$= 1662 \text{ mm}^2$ ...(*i*)

(*ii*) Section *efjk*: Applying equation 6.3,

$$A_{net} = t\left[\,(b - nd) + m\,\frac{s^2}{4g}\right]$$

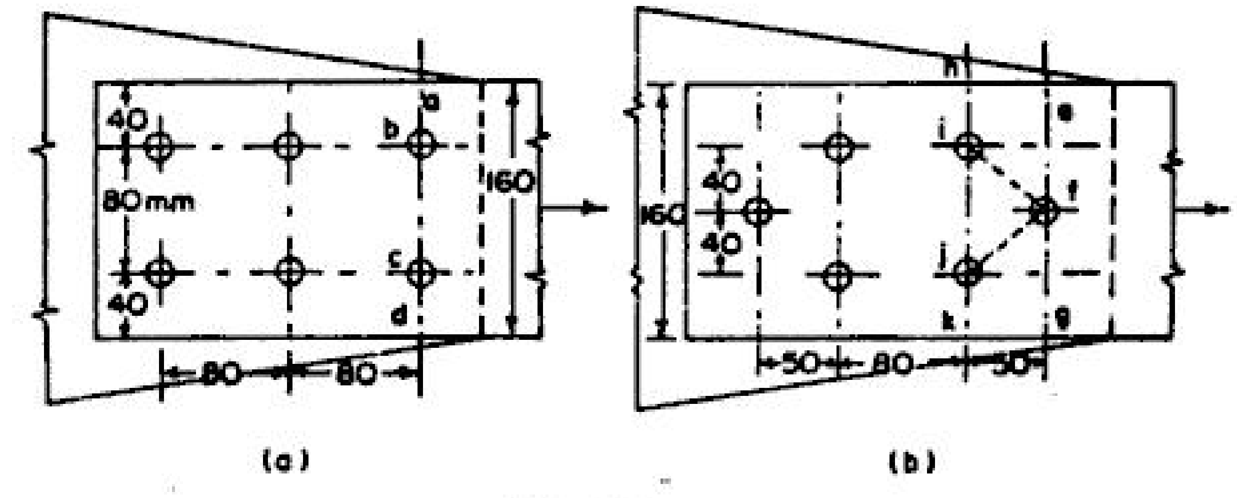


FIG. 6.8.

where

m = No. of inclined lines = 1; $n = 2$

$s = 50$ mm and $g = 40$ mm.

$$\therefore \qquad A_{net} = 12\left[(160 - 2\times 21.5) + 1\times\frac{(50)^2}{4\times 40}\right] = 1591.5 \text{ mm}^2 \qquad ...(ii)$$

(*iii*) *Section hifjk*:

Here $n = 3$; $m = 2$; $s = 50$ mm and $g = 40$ mm.

$$\therefore \qquad A_{net} = 12\left[(160 - 3\times 21.5) + 2\times\frac{(50)^2}{4\times 40}\right] = 1521 \text{ mm}^2 \qquad ...(iii)$$

Note: Section *hifjk* may not be critical as the strength of the rivet at *f* will be added to this.

$\therefore$ Most critical sectional area $= A_{net,\,min} = 1521 \text{ mm}^2$

$$\therefore \qquad P = \sigma_{at} \cdot A_{net} = 150 \times 1521 = 228150 \text{ N} = \mathbf{228.15 \text{ kN}}$$

Hence case *b* (zigzag riveting) is *stronger* than case (*a*).

Example 6.2. *A plate section, 300 mm × 12 mm has four staggered holes as shown in Fig. 6.9. The diameter of hole is 21.5 mm. Locate the critical section and find minimum net area.*

Solution

Here $s = 40$ mm, while g is different for different sections.

(*i*) *Chain abc* : $A_{net} = t\,[b - nd] = 12\,[300 - 2\times 21.5] = 3084 \text{ mm}^2$ ...(*i*)

(*ii*) *Chain abfgh*: There is only one zig-zag or inclined line

$$A_{net} = t\left[\,(b - nd) + m\,\frac{s^2}{4g}\right]$$

where $m = 1$, $n = 3$; $s = 40$ and $g = 50$

$$\therefore \qquad A_{net} = 12\left[\,300 - 3\times 21.5 + 1\times\frac{(40)^2}{4\times 50}\right] = 2922 \text{ mm}^2 \qquad ...(ii)$$

(*iii*) *Chain abfcd*: There are two zig-zag lines.

$$\therefore \qquad A_{net} = t\left[(b - nd) + \frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2}\right]$$

where $s_1 = s_2 = 40$ mm ; $g_1 = 50$ (for *bf*)

and $g_2 = 150$ (for *fc*) ; $n = 3$

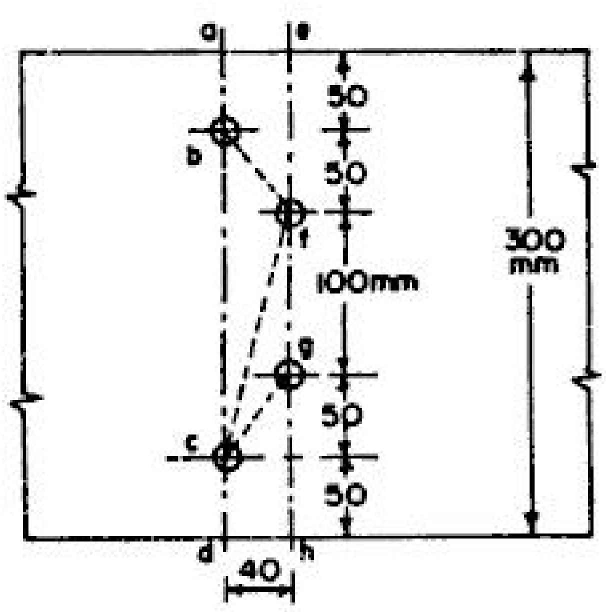


FIG. 6.9.

$$\therefore \quad A_{net} = 12\left[(300 - 3\times21.5)+\frac{(40)^2}{4\times50}+\frac{(40)^2}{4\times150}\right] = 2954 \text{ mm}^2 \quad ...(iii)$$

(*iv*) *Chain a b f g c d* There are two zigzag lines.

$$A_{net} = t\left[(b - nd) + \frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2}\right]$$

where $s_1 = s_2 = 40$ mm and $g_1 = g_2 = 50$; $n = 4$

$$\therefore \quad A_{net} = 12\left[(300 - 4 \times 21.5) + 2 \times \frac{(40)^2}{4 \times 50}\right] = 2760 \text{ mm}^2 \quad ..(iv)$$

Hence critical section is along chain *abfgcd* where the net sectional area = **2760 mm²**.

Example 6.3. *An angle section section ISA 8080 × 10 mm is used as a tension member and is connected to gusset plates by 16 mm dia. rivets through both the legs. The pitch of rivets on each leg is 60 mm. However, the rivets on the two legs are staggered by 30 mm as shown in Fig 6.10. Taking permissible stress of 150 N/mm² in axial tension, determine the allowable axial tension.*

Solution: For determining the net area, the angle is opened out as shown in Fig. 6.10 (*b*), and is looked upon as a plate of width equal to the sum of two legs of the angle *less* its thickness, with the thickness of the equivalent plate equal to the thickness of the legs.

∴ Width of equivalent plate = 80 + 80 − 10 = 150 mm

Distance between rivet holes = 45 + 45 − 10 = 80 mm.

Edge distance for each line = 35 mm.

Gross dia. of rivet holes = 16 + 1.5 = 17.5 mm.

(*i*) *Section abc*: There is only one rivet hole..

$$A_{net} = t\,[b - nd] = 10\,[150 - 1 \times 17.5] = 1325 \text{ mm}^2$$

(*ii*) *Section abde*: No. of inclined lines = 1

$$A_{net} = t\left[(b - nd) + m\,\frac{s^2}{4g}\right]$$

where $n = 2$; $m = 1$; $s = 30$ mm ; $g = 80$ mm

$$\therefore \quad A_{net} = 10\left[(150-2\times17.5)+1\times\frac{(30)^2}{4 \times 80}\right] = 1178.125 \text{ mm}^2$$

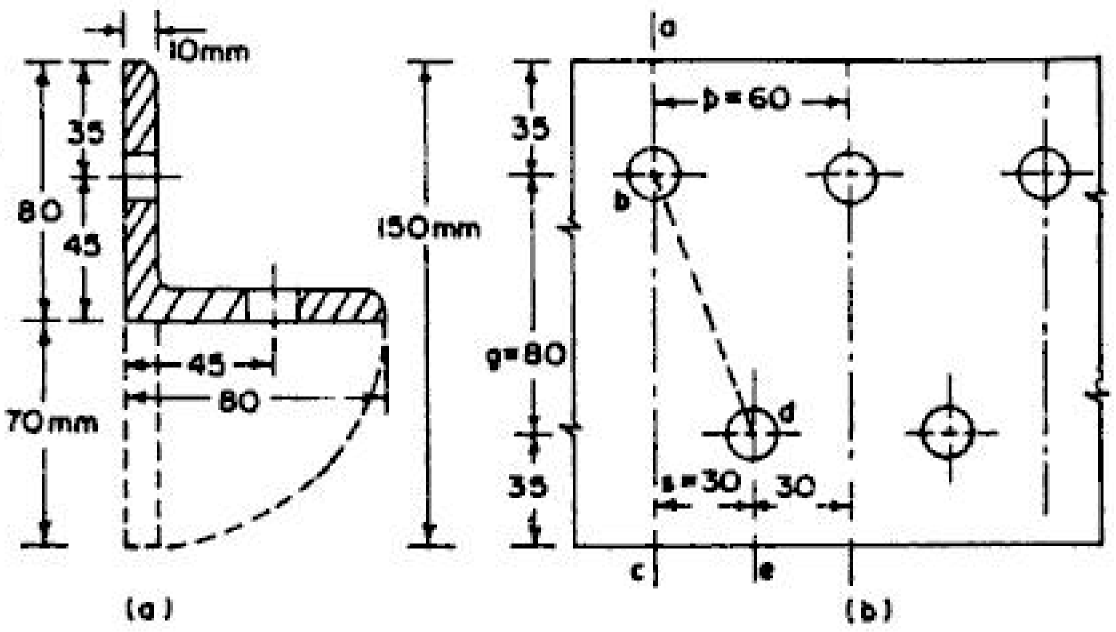


FIG. 6.10.

$$\therefore \quad A_{net,\,min} = 1178.125 \text{ mm}^2$$

$$\therefore \quad P = \sigma_{at} \times A_{net} = 150 \times 1178.125 = 176719 \text{ N} = \mathbf{176.719\ kN}$$

Example 6.4. *An angle section ISA 5030 × 60 mm is used as a tension member with its longer leg connected by 12 mm dia. rivets. Calculate its strength. What will be its strength if it is fillet welded? Take* $\sigma_{at} = 150$ *N/mm²*.

Solution

(a) *Riveted connection*

Dia. of rivet hole = 12 + 1.5 = 13.5 mm. From Eq. 6.4,

$$A_{net} = A_1 + A_2 k$$

where A_1 = net sectional area of connected leg

$$= 6(50 - \frac{6}{2} - 13.5) = 201 \text{ mm}^2$$

$$A_2 = \text{area of outstanding leg} = 6\left(30 - \frac{6}{2}\right) = 162 \text{ mm}^2$$

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 201}{3 \times 201 + 162} = 0.788$$

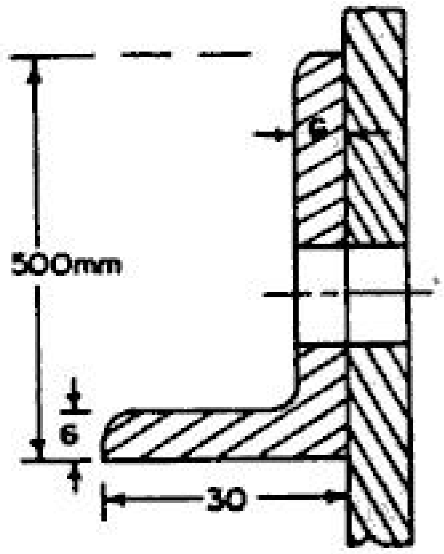


FIG. 6.11.

$$\therefore \quad A_{net} = 201 + 162 \times 0.788 \approx 328.7 \text{ mm}^2$$

$\therefore$ Strength of the member $= \sigma_{at} . A_{net}$ = 150 × 328.7 = 49304 N = **49.304 kN**

(b) *Welded Connection* : When the angle section is welded, no reduction in A_1 will be there since there is no rivet hole.

$$\therefore \quad A_1 = 6\left(50 - \frac{6}{2}\right) = 282 \text{ mm}^2$$

$$A_2 = 6\left(30 - \frac{6}{2}\right) = 162 \text{ mm}^2, \text{ as before}$$

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 282}{3 \times 282 + 162} = 0.839$$

$$\therefore \quad A_{net} = A_1 + A_2 k = 282 + 162 \times 0.839 = 417.96 \text{ mm}^2$$

$\therefore$ Strength of the member = 150 × 417.96 = 62695 N = **62.695 kN**

Example 6.5. *A tie of a roof truss consists of double angles ISA 10075 × 10 mm with its short legs back to back and long legs connected to the same side of a gusset plate, with 16 mm diameter rivets. Determine the strength of tie in axial tension, taking σ_{at} = 150 N/mm². Tack rivets have been provided at suitable pitch.*

Solution

Dia. of rivet hole = 16 + 1.5 = 17.5 mm. Each angle is weakened by one rivet hole. Hence this is case 2, where

$$A_{net} = A_1 + A_2 k, \quad \text{and}$$

$$k = \frac{5A_1}{5A_1 + A_2}$$

A_1 = Net area of connected legs

$$= 2\left[100 - \frac{10}{2} - 17.5\right] \times 10 = 1550 \text{ mm}^2$$

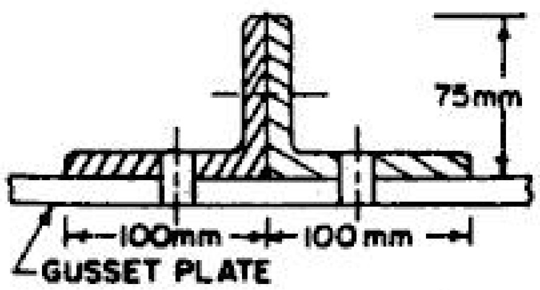


FIG. 6.12.

$$A_2 = \text{area of outstanding legs} = 2\left[75 - \frac{10}{2}\right] \times 10 = 1400 \text{ mm}^2$$

$$\therefore \quad k = \frac{5 \times 1550}{5 \times 1550 + 1400} = 0.847$$

$$\therefore \quad A_{net} = A_1 + A_2 k = 1550 + 1400 \times 0.847 = 2736 \text{ mm}^2$$

$\therefore$ Strength $= \sigma_{at} . A_{net}$ = 150 × 2736 = 410369 N = **410.369 kN**

Example 6.6. *A tension member, consisting of 4 ISA 110×110×10 mm is connected to a gusset plate by 20 mm diameter rivets, as shown in Fig. 6.13. Calculate the permissible tension in the member when*

(*a*) *No tacking rivets are used*

(*b*) *Tack riveting is done along C and D only*

(*c*) *Tack riveting is done along A and B only*

(*d*) *Tack riveting is done along A, B, C and D.*

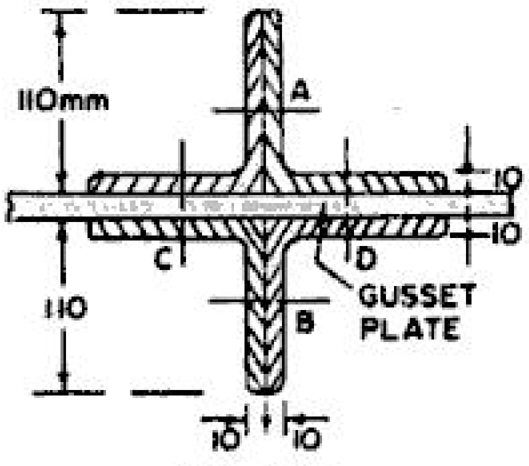


FIG. 6.13.

Solution

Diameter of rivet holes= 20 + 1.5= 21.5 mm

Gross area of each angle ISA110×110×10 is

$= 21.06\ \text{cm}^2 = 2106\ \text{mm}^2$

(a) When no tacking rivets are used

When no tack riveting is done, all angles will act individually, and hence Eq. 6.4 and 6.5 will be applicable (case 1 of § 6.4):

$$A_{net} = 4\,(A_1 + A_2 \,.\, k)$$

$$A_1 = \text{net area of connected leg} = 10(110 - \frac{10}{2} - 21.5) = 835\ \text{mm}^2$$

$$A_2 = \text{area of outstanding leg} = 10(110 - \frac{10}{2}) = 1050\ \text{mm}^2$$

$$\therefore \quad k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 835}{3 \times 835 + 1050} = 0.7046$$

$$\therefore \quad A_{net} = 4\,[835 + 1050 \times 0.7046] = 6299.5\ \text{mm}^2$$

$\therefore$ Permissible tension $= \sigma_{at} \,.\, A_{net} = 150 \times 6299.5 = 944924\ \text{N} =$ **944.924 kN**

(b) When tack riveting is done along C and D only

The angles will act in pairs on both sides of gusset plate. This is case 3 of § 6.4, wherein the net area will be equal to the gross-area minus deduction of area due to riveted holes.

$$\therefore \quad A_{net} = A_{gross} - \text{deduction for holes.} = 4 \times 2106 - 4 \times 21.5 \times 10 = 7564\ \text{mm}^2$$

$\therefore$ Permissible tension $= 7564 \times 150 = 1134600\ \text{N} =$ **1134.6 kN**

(c) When tack riveting is done along A and B only

In this case, the angles to the same side of the gusset plate will act in pair. This is case 2 of § 6.4, and Eq. 6.6 will be applicable.

$$\therefore \quad A_{net} = 4\,[A_1 + A_2 k] \quad \text{where } k = \frac{5A_1}{5A_1 + A_2}$$

Here A_1 = area of connected leg= 835 mm²

A_2 = area of outstanding leg= 1050 mm²

$$k_2 = \frac{5 \times 835}{5 \times 835 + 1050} = 0.799$$

$$\therefore \quad A_{net} = 4\,[835 + 1050 \times 0.799] = 6696\ \text{mm}^2$$

Permissible tension $= 6696 \times 150 = 1004397\ \text{N} \approx$ **1004.4 N**

(d) All angles are tack riveted

When tack riveting is done along all angles, the conditions are similar to case 3 of § 6.4, wherein the net area will be equal to gross-area minus deduction of area due to rivet holes.

$\therefore \quad A_{net} = 4 \times 2106 - 4 \times 21.5 \times 10 = 7564 \text{ mm}^2$

and $\quad P = 7564 \times 150 = 1134600 \text{ N} = \mathbf{1134.6 \text{ kN}}$ (*i.e.* same as for case *b* above)

It should be noted that if we compare the strength of the member in cases (*a*) to (*d*), proper tack riveting makes a difference of 15 to 20% in strength. Hence tack riveting is invariably done. It also increases the overall stiffness of the member.

Example 6.7. *The long leg of ISA 200 × 100 is connected to gusset plate by 22 mm diameter rivets in two rows, with gauge space of 75 mm and staggered pitch of 50 mm, as shown in Fig. 6.14. Determine suitable thickness of the angle to transmit a pull of 350 kN. Take* $\sigma_{at} = 150 \text{ N/mm}^2$.

Solution

Gross diameter of rivets = 22 + 1.5 = 23.5 mm

(*i*) For section along *abc* (Fig. 6.14 *b*), Deduction in width for hole

$= 1 \times 23.5 = 23.5$ mm

(*ii*) For section along *abde* (Fig. 6.14 *b*),

deduction in width for hole $= nd - \dfrac{s^2}{4g}$

$$= 2 \times 23.5 - \frac{50^2}{4 \times 75} = 38.67 \text{ mm}$$

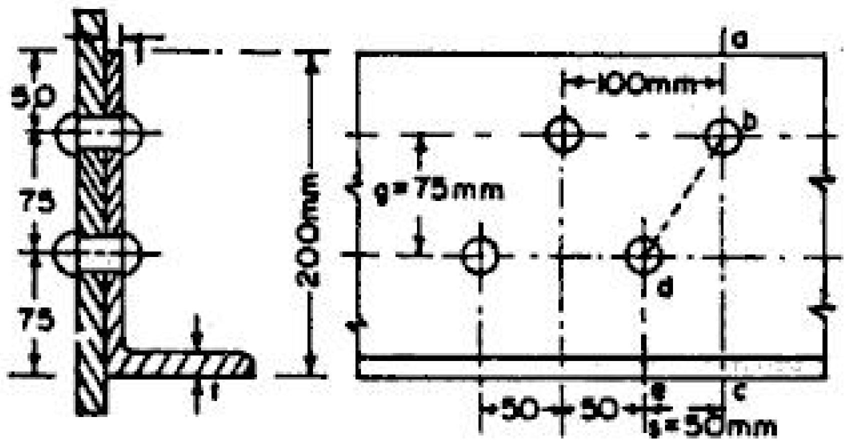


FIG. 6.14.

$\therefore$ Maximum deduction for hole, in 200 mm leg = 38.67 mm.

Now one leg (long leg) of the angle is connected to gusset plate.

Hence we get from Eqs. 6.4 and 6.5 (case 1 of § 6.4)

$$A_{net} = A_1 + A_2 k \qquad \text{where } k = \frac{3A_1}{3A_1 + A_2}$$

$$\therefore \quad A_1 = \text{net area of connected leg} = (200 - 38.67 - \frac{t}{2})\, t$$

$$A_2 = \text{Area of outstanding leg} = (100 - \frac{t}{2})\, t.$$

Assuming $t = 10$ mm for computation of net width of leg, we get

$$A_1 = \left(200 - 38.67 - \frac{10}{2}\right) t = 156.33\, t \text{ mm}^2$$

$$A_2 = (100 - \frac{10}{2})\, t = 95\, t \text{ mm}^2$$

$$k = \frac{3 \times 156.33\, t}{3 \times 156.33\, t + 95\, t} = 0.8316$$

$$\therefore \quad A_{net} = A_1 + A_2 k = 156.33\, t + 95\, t \times 0.8316 = 235.33\, t \text{ mm}^2$$

$$\therefore \quad P = \sigma_{at} \,.\, A_{net}.$$

or $\quad 350 \times 10^3 = 150 \times 235.33\, t$

From which $\quad t = 9.92$ mm.

Hence adopt $\quad t = 10$ mm.

Thus provide ISA 200100 × 10 mm angle section.

Example 6.8. *If in Example 6.3, the angle section is subjected to an axial pull of 180 kN, determine the staggered pitch of rivets.*

Solution Refer Fig. 6.10.

Let the staggered pitch be $= s$ mm

The section is weaker along line *abde*, for which

$$A_{net} = t\left[(b - nd) + \frac{s^2}{4g}\right]$$

where $n = 2$; $g = 80$ mm ; s is to be determined.

$$\therefore \quad A_{net} = 10\left[(150-2\times17.5)+\frac{s^2}{4\times80}\right] = \left(1150+\frac{s^2}{32}\right) \text{mm}^2$$

Now $P = \sigma_{at} \cdot A_{net}$

$$\therefore \quad 180 \times 10^3 = 150 \times \left(1150 + \frac{s^2}{32}\right) \qquad ...(1)$$

From which we get $s = \mathbf{40\ mm}$

Hence provide a staggered pitch of 40 mm. Actual pitch on each leg will be $= 2 \times 40 = 80$ mm.

Note: It is to be noted that A_{net} increases with increase in s. Hence while adopting final value of s, the value obtained from Eq. (1) above should be round off to the higher side and *not* to the lower side. For example, if we get $s = 41.4$ mm, we should adopt $s = 42$ mm and not $s = 40$ mm. Adoption of lower value of s will give lesser value of A_{net}, and hence the load capacity of the section will decrease.

Example 6.9. *A tie member in a roof truss is 1.75 m long and carries an axial load of 150 kN. Design a suitable single unequal angle section if (i) hand driven rivets are used, (ii) fillet weld is used at the joint.*

Solution. Assume that steel sections with thickness less than 20 mm will be used. Hence $\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150\ \text{N/mm}^2$.

(*a*) **Riveted member**

Step 1. Required $A_{net} = \dfrac{150 \times 1000}{150} = 1000\ \text{mm}^2$

Step 2. Choose a section having gross area equal to 40% more than the net area. Choose ISA 90 × 60 × 10 mm @ 11.0 kg/m having gross area equal to 1401 mm². This is connected by the longer leg, using 20 mm dia. rivets (say). Gross dia. of rivet $= 20 + 1.5 = 21.5$ mm.

Step 3. $A_{net} = A_1 + kA_2$

where $A_1 = \left(90 - 21.5 - \dfrac{10}{2}\right)10 = 635\ \text{mm}^2$

$A_2 = \left(60 - \dfrac{10}{2}\right) 10 = 550\ \text{mm}^2$

$k = \dfrac{3A_1}{3A_1 + A_2} = \dfrac{3 \times 635}{3 \times 635 + 550} = 0.776$

$\therefore \quad A_{net} = 635 + 0.776 \times 550 = 1061.78\ \text{mm}^2$

which is slightly more than the net area required.

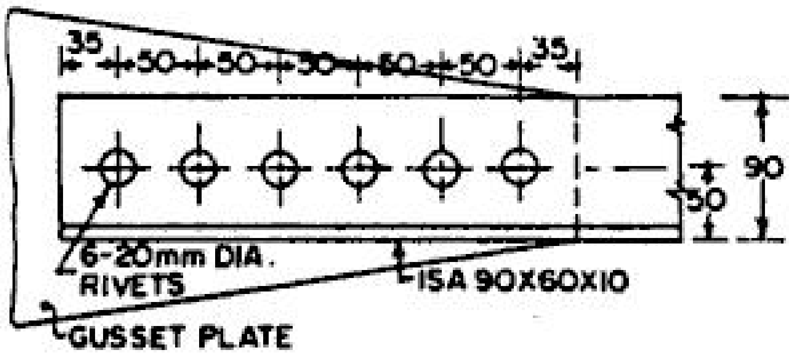


FIG.. 6.15.

Step 4. Load capacity of member $= 1061.78 \times 150 = 159267$N $= 159.27$ kN which is greater than 150 kN. Hence the section is OK.

Step 5. Check for slenderness ratio.

Since this is a tie number, there is no chance of reversal of stress.

However, as per code requirements, the slenderness ratio should not exceed 350. For the angle section, $r_{min} = 12.7$ mm.

$$\therefore \quad \lambda = \frac{l}{r_{min}} = \frac{1.75 \times 1000}{12.7} = 137.8 \not> 350$$

Step 6. Design of end connections

Strength of 20 mm dia. hand driven rivets:

$$\text{In shearing} = \frac{\pi}{4}(21.5)^2 \times 80 \times 10^{-3} = 29.04 \text{ kN}$$

$$\text{In bearing} = 21.5 \times 10 \times 250 \times 10^{-3} = 53.75 \text{ kN.}$$

$$\therefore \quad \text{Rivet value } R = 29.04 \text{ kN.}$$

$$\text{Hence No. of rivets} = \frac{150}{29.04} = 5.17$$

Provide 6 rivets at a pitch 2.5 × 20= 50 mm, and keep edge distance= 35 mm (see Table 3.2), as shown in Fig. 6.15.

(*b*) **Welded member**

Step 1.

$$\therefore \quad \text{Required } A_{net} = \frac{150 \times 10^3}{150} = 1000 \text{ mm}^2$$

Step 2. Choose an angle section having gross-sectional area equal to about 10% more than net area. Hence choose ISA 90×60×8 @ 8.9 kg/m having cross-sectional area of 1137 mm², $r_{min} = 12.8$ mm and $C_{xx} = 29.6$ mm. Let us weld longer leg to the gusset plate.

Step 3. Now $A_{net} = A_1 + kA_2$

where

$$A_1 = \left(90 - \frac{8}{2}\right) 8 = 688$$

$$A_2 = \left(60 - \frac{8}{2}\right) 8 = 448$$

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 688}{3 \times 688 + 448} = 0.822$$

$$\therefore \quad A_{net} = 688 + 0.822 \times 448 = 1056.1 \text{ mm}^2 > 1000 \text{ mm}^2$$

Hence the section is OK.

Step 4 . Check for slenderness ratio:

$$\lambda = \frac{1.75 \times 10^3}{12.8} = 136.3 \not> 350. \text{ Hence OK.}$$

Step 5 . Design of end connections

Min. size of weld = 3 mm (Table 5.1)

$$\text{Max. size of weld} = \frac{3}{4} \times 8 = 6 \text{ mm.}$$

Let us provide 6 mm size weld.

Let us take allowable stress in weld as 110 N/m².

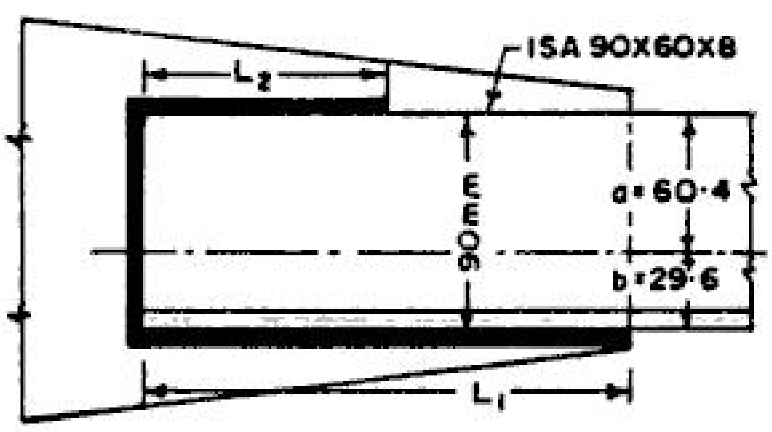


FIG. 6.16.

Strength of weld per lineal mm =0.7×6×110=462 N=0.462 kN.

$$\therefore \text{ Effective length of weld required} = \frac{150}{0.462} = 324.7 \text{ mm (say 330 mm).}$$

Let us distribute the weld in such a way that the *c.g.* of the weld coincides with the *c.g.* of the angle section, as shown in Fig. 6.16.

Hence $90 + L_1 + L_2 = 330$ mm ...(1)

Taking moments about L_2 edge,

$$0.462\left[L_1 \times 90 + 90 \times \frac{90}{2}\right] = 150 \times 60.4 \quad ...(2)$$

which gives $L_1 = 172.9$ mm

Keep $L_1 = 173$ mm.

Hence from (1), $L_2 = 330 - 90 - 173 = 67$ mm.

Example 6.10. *Design a T-section to carry an axial tension of 300 kN. Take $f_y = 250\ N/mm^2$. Also design the riveted joint at the end.*

Solution: $\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150\ N/mm^2$ for sections of less than 20 mm thickness.

Step1. Required $A_{net} = \dfrac{300 \times 10^3}{150} = 2000\ mm^2$

Step 2. Select a T-section which has a gross area of about $1.3 \times 2000 \approx 2600\ mm^2$. Hence choose ISHT 100 @ 20.0 kg/m, having sectional area equal to 2547 mm², $t_f = 9.0$ mm, $t_w = 7.8$ mm and $r_{min} = 27.6$ mm.

Choose 20 mm dia. power driven rivets, having gross dia. $= 20 + 1.5 = 21.5$ mm.

Step 3. $A_{net} = A_1 + kA_2$

where $A_1 = (200 - 2 \times 21.5)\,9.0 = 1413\ mm^2$

$A_2 = (100 - 9.0)\,7.8 = 709.8\ mm^2$

$$k = \frac{5A_1}{5A_1 + A_2} = \frac{5 \times 1413}{5 \times 1413 + 709.8} = 0.908$$

$\therefore$ $A_{net} = 1413 + 709.8 \times 0.908 = 2058\ mm^2 > 2000\ mm^2$

Step 4. ***Design of End Connections***

Strength of 20 mm dia. power-driven shop rivets.

In shearing $= \dfrac{\pi}{4}(21.5)^2 \times 100 \times 10^{-3} = 36.3$ kN

In bearing against 9 mm flange

$= 21.5 \times 9 \times 300 \times 10^{-3} = 58.05$ kN.

$\therefore$ Rivet value $= 36.3$ kN

No. of rivets required $= \dfrac{300}{36.3} = 8.26$

Provide 10 rivets in pairs, at a pitch $= 2.5 \times 20 = 50$ mm and keep edge distance equal to 35 mm (Table 3.2), as shown in Fig. 6.17.

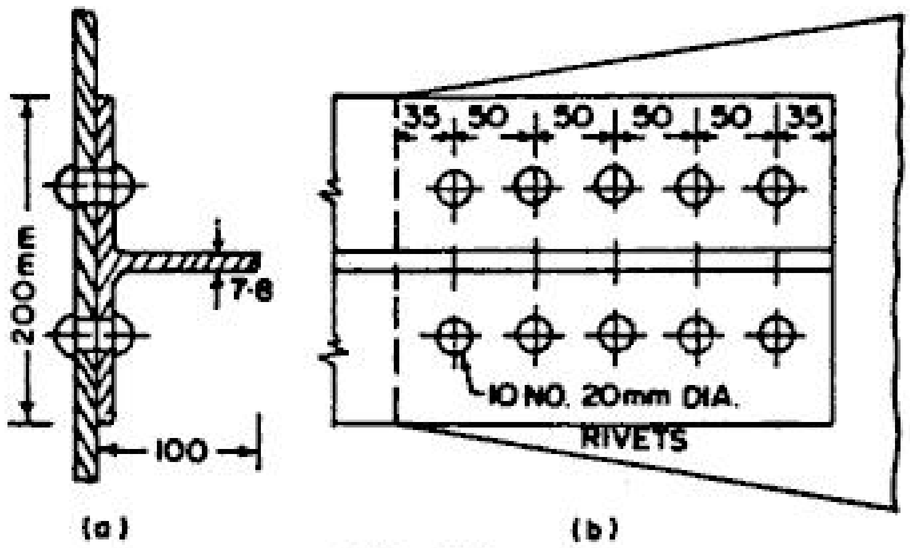


FIG. 6.17.

Example 6.11. *Design a channel section to carry an axial tension of 300 kN. Take $f_y = 250\ N/mm^2$. Also design the riveted joint at the end.*

Solution $\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150\ N/mm^2$.

Step 1. Required $A_{net} = \dfrac{300 \times 10^3}{150} = 2000\ mm^2$.

Step 2. Select a channel section which has a gross area about 30 to 40% more than the net area required. Let us choose ISLC 200 @ 20.6 kg/m, having a cross-sectional area of 2622 mm², $h = 200$ mm, $b = 75$ mm and thickness of web $= 5.5$ mm. Let us connect this to the gusset plate, using 18 mm dia. power driven rivets.

6.7. DESIGN OF MEMBERS SUBJECTED TO AXIAL TENSION AND BENDING

When a tension member is subjected to tensile force P which does not pass through the centroid of the section, bending moment $M = P.e$ is also induced, where e is the eccentricity of the load with respect to the centroid of the section. The section is thus subjected to both direct stress as well as bending stress. The combined stress at any point is given by.

$$f = \frac{P}{A} + \frac{M_{xx}}{I_{xx}} y \qquad ...(6.8)$$

where A = net cross-sectional area of the section.
I = moment of inertia of the section about xx.

When a member is subjected to axial tensile load and bending, the net cross-sectional area is given by

$$A = A_a + A_b = \frac{P}{\sigma_{at}} + \frac{My/r^2}{\sigma_{bt}}.$$

where σ_{at} = permissible stress in axial tension $= 0.6 f_y$
and σ_{bt} = permissible stress in bending tension$= 0.66 f_y$

$$\therefore \quad \frac{P/A}{\sigma_{at}} + \frac{M.y/Ar^2}{\sigma_{bt}} \leq 1.0$$

$$\frac{\sigma_{at,cal}}{\sigma_{at}} + \frac{\sigma_{bt,cal}}{\sigma_{bt}} \leq 1.0 \qquad ...(6.9)$$

When a tension member is subjected to both axial tension as well as uniaxial bending it should be so proportioned that

$$\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{bt,cal}}{0.66 f_y} \leq 1.0 \qquad ...(6.10)$$

When a section is subjected to both axial P and biaxial bending, the stress is given by

$$f = \frac{P}{A} + \frac{M_{xx}}{I_{xx}} y + \frac{M_{yy}}{I_{yy}} x \qquad ...(6.11)$$

A member subjected to both axial tension and biaxial bending should be so proportioned that

$$\frac{\sigma_{at,cal}}{\sigma_{at}} + \frac{\sigma_{btx,cal}}{\sigma_{btx}} + \frac{\sigma_{bty,cal}}{\sigma_{bty}} \leq 1.0 \qquad ...(6.12)$$

or

$$\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{btx,cal}}{0.66 f_y} + \frac{\sigma_{bty,cal}}{0.66 f_y} \leq 1.0 \qquad ...(6.13)$$

Design of section

A trial section is selected corresponding to an equivalent axial load (P_e) which is defined as the axial tensile load that produces average axial tensile stress in the section equivalent to maximum combined stress at the extreme fibre of the section. The equivalent axial load for uniaxial bending is given by

$$P_e = P + \frac{M}{I} . y . A$$

or

$$P_e = P + \frac{M.A}{(I/y)} = P + M\left(\frac{A}{Z}\right)$$

or

$$P_e = P + M.B_f \qquad ...(6.14)$$

where B_f = bending factor $= A/Z$

$$\therefore \quad \sigma_{at,cal} = \frac{P \times 10^3}{3522} \text{ N/mm}^2 \quad ...(1)$$

where P = axial load, in kN.

Self mass of angles = 2 × 15.8 = 31.6 kg/m

Mass of accessories etc. = 0.1 × 31.6 ≈ 0.4 kg/m (say)

∴ Total mass per m length of member = 32 kg/m.

∴ Self load = 32 × 9.81 N/m

$$M = \frac{12 \times 3}{4} + \frac{(32 \times 9.81 \times 10^{-3})\, 3^2}{8} = 9 + 0.353 = 9.353 \text{ kN-m} = 9.353 \times 10^6 \text{ N-mm}$$

$$\sigma_{bt,cal} = \frac{M}{I} \cdot y_t = \frac{9.353 \times 10^6}{2 \times 147.9 \times 10^4} \times 26.6 = 84.11 \text{ N/mm}^2 \quad ...(2)$$

Now, from Eq. 6.10, we have

$$\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{bt,cal}}{0.66 f_y} \le 1.0$$

$$\therefore \quad \frac{P \times 10^3}{3522 \times 0.6 \times 250} + \frac{84.11}{0.66 \times 250} \le 1.0$$

From which P = **259 kN**

Example 6.14. *The central bottom chord member of roof truss is 4 m long, carries an axial tension 250 kN and supports a moveable differential chain hoist of 4 kN capacity including impact. The member is composed of 2 angles 100 mm × 75 mm × 10 mm @ 13 kg/m with the longer legs up and placed back on either side of 10 mm gusset plates. The outstanding shorter legs at the bottom provide necessary track for the hoist. The angles are tack riveted at 1 m centers with 20 mm dia. rivets. Check the efficiency of the section provided. The permissible tensile stress in axial tension and bending are 150 MPa and 165 MPa respectively. The permissible stress in compression for flangeless beams unsupported laterally may be taken as* $\frac{3300}{l/b}$ *N/mm², subject to a maximum of 140 N/mm², where l = unsupported length of the member and b is the combined thickness of the angles.*

Solution

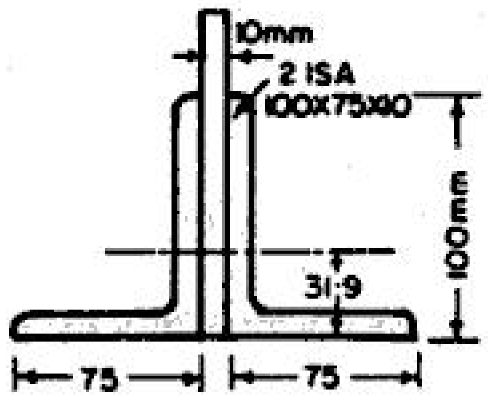


FIG. 6.22.

For each angle, $A = 1650 \text{ mm}^2$;

$C_{xx} = 31.9$ mm ; $I_{xx} = 160.4 \times 10^4 \text{ mm}^4$

Net area = 2 [1650 − 21.5 × 10] = 2870 mm²

$I_{xx} = 2 \times 160.4 \times 10^4 = 320.8 \times 10^4 \text{ mm}^4$.

$y_t = C_{xx} = 31.9$ mm ; $y_c = 100 - 31.9 = 68.1$ mm.

Self load = 2 × 13 × 9.81 = 255 N/m

Wt. of accessories = 0.1 × 255 ≈ 25 N/m (say)

Total weight = 255 + 25 = 280 N/m.

$$M = \frac{4 \times 4}{4} + \frac{280 \times 10^{-3} (4)^2}{8} = 4 + 0.56 = 4.56 \text{ kN-m} = 4.56 \times 10^6 \text{ N-mm}$$

$$\sigma_{at,cal} = \frac{P}{A_{net}} = \frac{250 \times 10^3}{2870} = 87.1 \text{ N/mm}^2$$

$$\sigma_{bt,cal} = \frac{M}{I_{xx}} \cdot y_t = \frac{4.56 \times 10^6}{320.8 \times 10^4} \times 31.9 = 45.3 \text{ N/mm}^2$$

Also,

$$\sigma_{bc,cal} = \frac{M}{I_{xx}} \cdot y_c = \frac{4.56 \times 10^6}{320.8 \times 10^4} \times 68.1 = 96.8 \text{ N/mm}^2$$

$$\sigma_{at} = 150 \text{ MPa} = 150 \text{ N/mm}^2 \text{ (given)}$$

$$\sigma_{bt} = 165 \text{ MPa} = 165 \text{ N/mm}^2 \text{ (given)}$$

$$\sigma_{bc} = \frac{3300}{l/b}, \text{ where } l=1 \text{ m}=1000 \text{ mm} ; b=2\times10=20 \text{ mm}.$$

$$\sigma_{bc} = \frac{3300}{1000/20} = 66 \text{ N/mm}^2$$

Now $\frac{\sigma_{at,cal}}{\sigma_{at}} + \frac{\sigma_{bt,cal}}{\sigma_{bt}} = \frac{87.1}{150} + \frac{45.3}{165} = 0.581 + 0.275 = 0.856 < 1$. Hence safe.

Also, max. compressive stress $= \sigma_{bc,cal} - \sigma_{at,cal}$

$= 96.8 - 87.1 = 9.7 \text{ N/mm}^2 < 66 \text{ N/mm}^2$ Hence safe.

Hence the section provided is suitable.

Example 6.15. *Design a double angle tension member carrying axial tensile force of 350 kN. In addition to this, it is also subjected to a U.D.L. of 4 kg/m of its length, including the self weight. The centre to centre distance between the end connections is 2.5 m.*

Solution (Fig. 6.23)

Step 1. On the basis of axial force only,

$$A_{net} = \frac{350 \times 10^3}{150} = 2333 \text{ mm}^2.$$

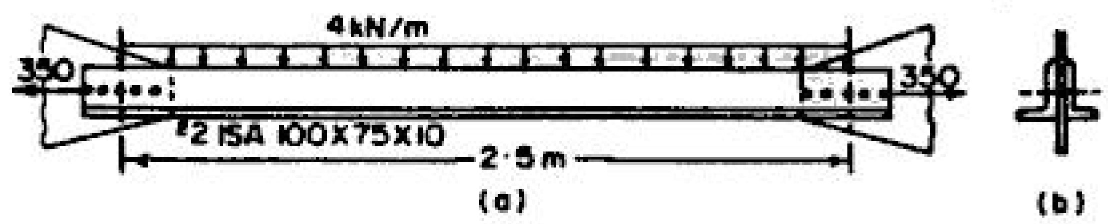


FIG. 6.23.

Step 2. Select a section such that the total gross area$=1.4 \times 2333 \approx 3270 \text{ mm}^2$. Choose two ISA 100 × 75 × 10 @ 13.0 kg/m, each having an area of 1650 mm^2 and place them on either side of the gusset plate with longer side vertical as shown in Fig. 6.23 (*b*). Each angle section has $I_{xx} = 160.4 \times 10^4 \text{ mm}^4$ and $C_{xx} = 31.9$ mm.

Step 3. Using 20 mm dia. rivets, both for end connection as well as for tacking,

$$A_{net} = 2[1650 - 21.5 \times 10] = 2870 \text{ mm}^2$$

$$\therefore \quad \sigma_{at,cal} = \frac{350 \times 10^3}{2870} = 121.95 \text{ N/mm}^2$$

$$\sigma_{at} = 0.6 \times 250 = 150 \text{ N/mm}^2.$$

$$\therefore \quad \frac{\sigma_{at,cal}}{\sigma_{at}} = \frac{121.95}{150} = 0.813$$

Step 4. $$M = \frac{4(2.5)^2}{8} = 3.125 \text{ kN-m} = 3.125 \times 10^6 \text{ N-mm}.$$

$$\sigma_{bt,cal} = \frac{M}{I} y_t = \frac{3.125 \times 10^6}{2 \times 160.4 \times 10^4} \times 31.9 = 31.07 \text{ N/mm}^2.$$

$$\sigma_{bt} = 0.66 f_y = 0.66 \times 250 = 165 \text{ N/mm}^2.$$

$$\therefore \quad \frac{\sigma_{bt,cal}}{\sigma_{bt}} = \frac{31.07}{165} = 0.188.$$

Step 5. ***Check:***

$$\frac{\sigma_{at,cal}}{\sigma_{at}} + \frac{\sigma_{bt,cal}}{\sigma_{bt}} \le 1$$

or $$0.813 + 0.188 \le 1$$

or $$1.001 \ge 1$$

Since the difference is negligible, no further revision is necessary and the section may be taken to be safe.

The end connection may now be designed for an axial tensile load of 350 kN.

6.8. TENSION SPLICES

When the required length of a tension member is less than available length, or when two lengths of a tension member have different cross-sections, tension splices are provided to join the two lengths of the member. Tension splices are provided on both the sides of member jointed, in the form of cover plates, so as to form a butt joint. The required number of rivets, to transfer the load, are placed on each side of the joint. When tension members of different thickness are to be jointed, *filler plates* or *packing* may be used to bring the member in level. According to Indian Standard, additional rivets are to be provided if the thickness of filler plate (or packing) is more than 6 mm. For each 2 mm thickness of packing, the number required by normal calculations should be increased by 2.5 %. For double shear connections packed on both sides, the number of additional rivets or bolts shall be determined from the thickness of thicker packing. The additional rivets or bolts should preferably be placed in an extension of the packing.

Example 6.16. *Design a tension splice to connect two plates of size 250 mm × 20 mm and 220 mm × 12 mm, for a design load of 250 kN.*

Solution. Since both the plates, carrying a tensile load of 250 kN, are of different thickness, a packing plate of thickness = 20−12 = 8 mm will be required, in addition to the splice plates.

Let us use 10 mm thick splice plates on both the sides of the joint, as shown in Fig. 6.24 (*a*). We will use 20 mm dia. field rivets.

$$\text{Strength of rivets in double shear} = 2 \times \frac{\pi}{4}(21.5)^2 \times 90 \times 10^{-3} = 65.35 \text{ kN.}$$

$$\text{Strength of rivets in bearing against 10 mm plate} = 21.5 \times 10 \times 270 \times 10^{-3} = 58.05 \text{ kN.}$$

∴ Rivet value = 58.05 kN.

No. of rivets required on each side of joint $= \frac{250}{58.05} = 4.3$

Provide 5 rivets.

Since the thickness of filler plate is more than 6 mm, additional rivets required at the rate of 2.5% for each 2 mm thickness of packing

$$= \frac{8}{2} \times 2.5 = 10\%.$$

∴ Additional rivets = 5 × 0.1 = 0.5 rivets (or say 1 rivets).

Hence provide this additional rivet in the extension of this packing plate, as shown in Fig. 6.24 (*b*.) Spacing between the rivets = 3 × 20 = 60 mm, and edge distance = 2 × 20 = 40 mm.

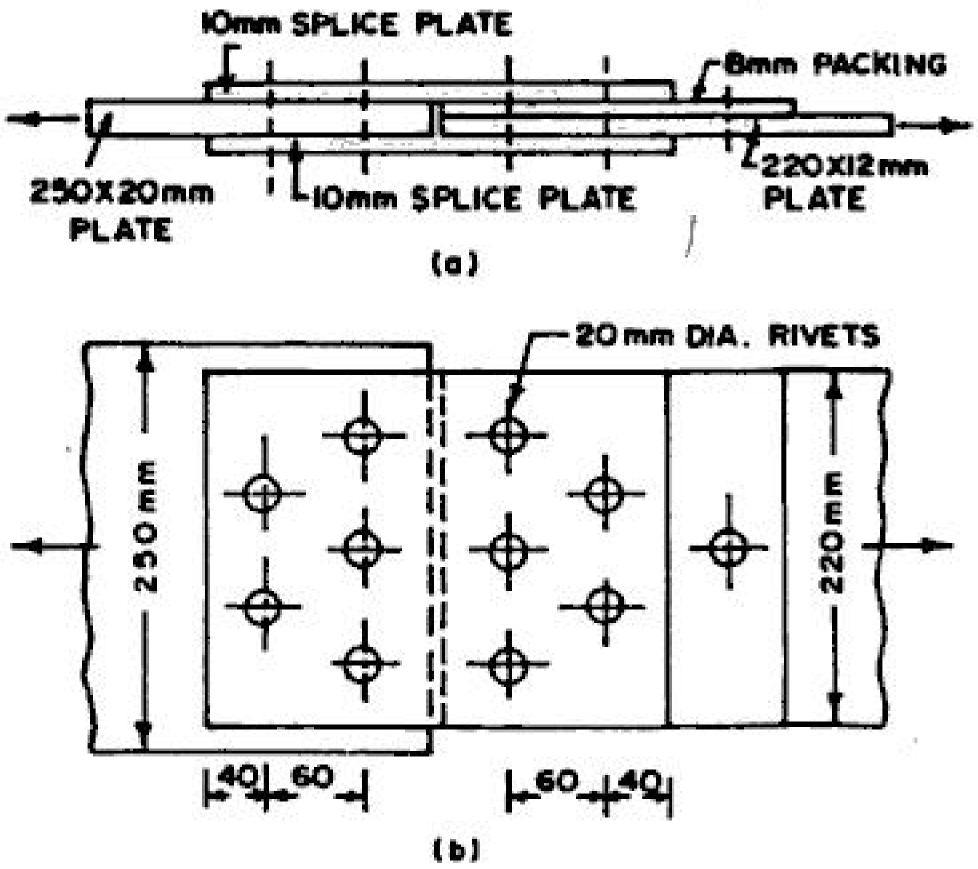


FIG. 6.24.

6.9. LUG ANGLE

When a tension member is connected to a gusset plate at its end, a large number of rivets are required, specially when the tensile load is large, necessitating in the provision of a big size gusset plate. The size of the gusset plate can be decreased by use of a lug angle. A lug angle is a short length of an angle section, which is attached to the main tension member at the connecting end, to provide extra gauge lines for accommodating the required number of rivets, as shown in Fig. 6.25. Lug angles are not very common because of following reasons : (1) they produce eccentric connection because of the rivets placed along the lug angle (*ii*) stress distribution in the rivets of lug angle is not uniform, and (*iii*) rivets on the lug angle are not as efficient as other rivets.

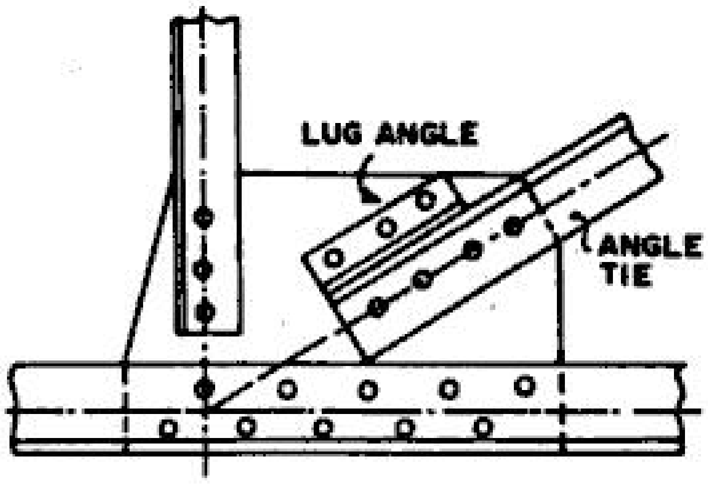


FIG. 6.25. LUG ANGLE..

IS : 800-1984 lays down following specifications for the design of lug angles :

1. Lug angles connecting a channel shaped member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

2. In case of angle members, the lug angles and their connections to the gusset or other supporting member shall be capable of developing a strength not less than 20% in excess of the force in the outstanding leg of the angle, and the attachment of the lug angle to the angle member shall be capable of developing 40% in excess of that force.

3. In the case of channel members and the like, the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10% in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20% in excess of that force.

4. In no case shall fewer than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

5. The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.

6. Where lug angles are used to connect an angle member the whole area of the member shall be taken as effective, *i.e.*.

$$A_{net} = \text{gross area} - \text{deduction for holes.}$$

Example 6.17. *Using a lug angle, design a suitable riveted end connection for an angle ISA 100 × 65 × 10 mm thick, using 16 mm diameter shop rivets.*

Solution

$$\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150 \text{ N/mm}^2$$

$$\text{Strength of rivets in shear} = \frac{\pi}{4}(17.5)^2 \times 100 \times 10^{-3} = 24.05 \text{ kN.}$$

$$\text{Strength of rivets in bearing} = 17.5 \times 10 \times 300 \times 10^{-3} = 52.5 \text{ kN.}$$

$\therefore$ Rivet value = 24.05 kN

Gross area of angle = 1551 mm²

$\therefore$ $A_{net} = 1551 - 17.5 \times 10 = 1376 \text{ mm}^2$

$\therefore$ Strength of member $= 1376 \times 150 \times 10^{-3} = 206.4$ kN.

Assuming that the 100 mm leg (longer leg) is connected to the gusset plate,

$$\text{Area of connected leg} = \left(100 - \frac{10}{2}\right) 10 = 950 \text{ mm}^2$$

$$\text{Area of outstanding leg} = \left(65 - \frac{10}{2}\right) 10 = 600 \text{ mm}^2$$

$$\therefore \quad \text{Strength of connected leg} = \frac{950}{950+600} \times 206.4 = 126.50 \text{ kN}$$

$$\text{Strength of outstanding leg} = \frac{600}{950+600} \times 206.4 = 79.90 \text{ kN}$$

No. of rivets required for connecting the angle to the gusset plate

$$= \frac{126.50}{24.05} = 5.26$$

Hence provide 6 rivets.

Design of lug angle

Strength outstanding leg = 79.90 kN

$\therefore$ Required strength of lug angle $= 1.2 \times 79.90 = 95.88$ kN

$$\therefore \quad A_{net} = \frac{95.88 \times 10^3}{150} = 639.2 \text{ mm}^2$$

Gross area will be about 20 to 25% more than the above to account for the rivet holes. Let us try ISA $55 \times 55 \times 8$ mm @ 6.4 kg/m, having area = 818 mm².

$\therefore$ Available $A_{net} = 818 - 17.5 \times 8$
$= 678 \text{ mm}^2 > 639.2 \text{ mm}^2$.

Hence OK.

Number of rivets required to connect lug angle to main angle $= 1.4 \times \dfrac{79.90}{24.05} = 4.65$

Hence provide 5 rivets.

All these rivets will be spaced at a pitch of $16 \times 3 \approx 50$ mm c/c, as shown in Fig. 6.26.

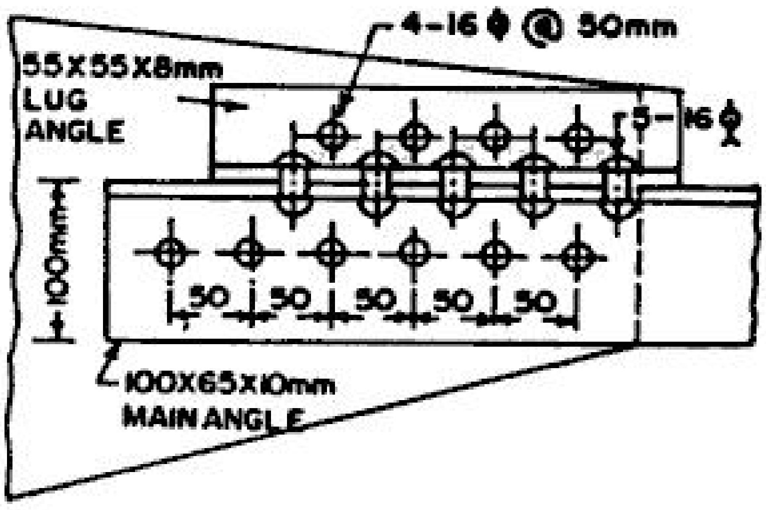


FIG. 6.26.

PROBLEMS

1. Explain how net effective area of various sections used as tension members are computed.
2. In a roof truss, a diagonal consists of an ISA 60×60×8 mm and it is connected to gusset plate by one leg only, using 16 mm dia. shop rivets in one chain along the length of the member. Determine the tensile strength of the members.

3. A tension member consists of an ISA 150 × 115 × 10 mm. Determine the safe axial pull it can carry if (*i*) it is connected by sufficient number of 20 mm rivets at the end, (*ii*) if it is connected by suitable weld at each end.

4. A tie member in a roof truss consists of two ISA 90 ×60× 8 mm. Determine the safe load carrying capacity of the member if (*a*) the angles are on the same side of the gusset plate (*b*) the angles are on either side of the gusset plate. 16 mm rivets are used for connection at the ends and the members are suitably tracked along their length.

5. A tension member consists of 4 angles ISA 100×100×8 mm arranged as shown in Fig. 6.27, and connected by 20 mm dia. rivets at the ends. Determine the safe load carrying capacity of the member if

(*a*) tacking rivets are provided along the lines *a-a* and *b-b*.

(*b*) tacking rivets are provided along *c-c* only.

(*c*) tacking rivets are provided along *c-c* and *d-d*.

(*d*) No tacking rivets are provided.

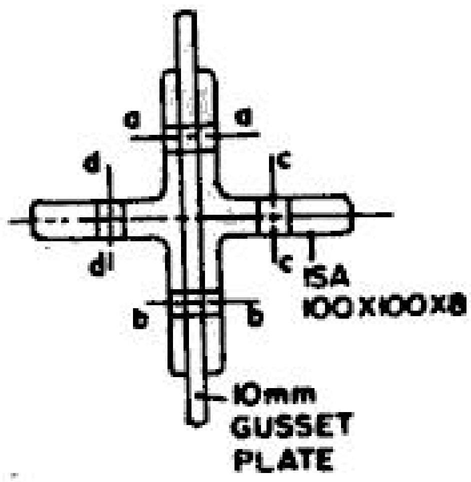


FIG. 6.27.

6. Long leg of an angle ISA 150 × 75 mm is connected to gusset plate using 20 mm dia. rivets in two rows, using a staggered pitch of 40 mm and a gauge space of 55 mm. Determine the thickness of the angle to transmit a pull of 240 kN. Take $f_y = 250$ N/mm^2.

7. An angle ISA 75 × 75 × 10 mm thick angle is connected to gusset plates by 18 mm dia. rivets through both the legs, with a pitch of 80 mm. The rivets in one leg are staggered by 40 mm with respect to those in the other leg as shown in Fig. 6.28. Determine the allowable tensile load on the angle, taking $\sigma_{at} = 150$ N/mm^2.

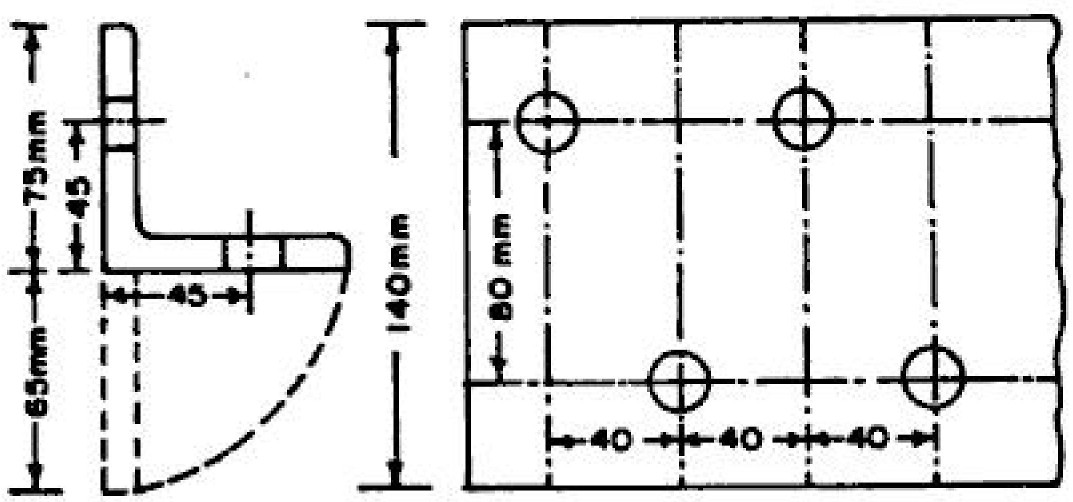


FIG. 6.28.

8. A tension member is subjected to a pull of 250 kN. Design suitable section if (*a*) it consists of a single angle connected by one leg, (*b*) it consists of double angle on either side of gusset plate and tacked along the length.

9. Design a single angle tension member subjected to axial pull of 250 kN and connected by suitable weld at the ends. Also design the welded connections at the ends. The effective length of the member is 3 m and it is subjected to possible reversal of stress due to action of wind.

10. A bridge truss diagonal carries an axial pull of 320 kN and it consists of two mild steel flats 250 mm × 10 mm and 260 mm × 18 mm jointed together. Design a suitable splice at the joint.

11. Design a riveted end connection for the full strength of angle section ISA 110 × 110 × 10 mm, using a lug angle.

12. Design and connection for an angle 125 × 75 × 10 mm, using lug angle with 20 mm dia. shop rivets.

13. The bottom tie of a roof truss is 4 m long. In addition to an axial tension of 100 kN, it has to support at its centre a shaft load of 4 kN. The member is composed of two angels 100 mm × 75 mm × 10 mm with the longer legs turned down and placed back to back on either side of 10 mm thick gusset plate, as shown in Fig. 6.29. The angles are tack riveted at 90 cm centres with 20 mm dia. rivets. Assuming pin jointed connections at the nodes, check the efficiency of the section provided.

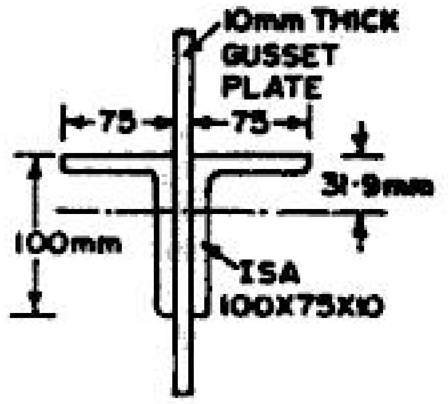


FIG. 6.29.

7.2. MODES OF FAILURE OF A COLUMN

A column, subjected to axial load, may fail under the following modes:

(1) crushing
(2) buckling
(3) mixed mode of buckling and crushing

If a *short length* of a bar or column is subjected to a compressive force P, a uniform compressive stress p is induced, p being equal to P/A. As the value of P is increased, p will also increase, and ultimately, failure will occur due to *crushing* or *yielding* of the material. In the case of *long columns* subjected to axial loading, *buckling* occurs due to *elastic instability*. Every such column has a *critical load* which causes elastic instability, due to which column suddenly fails due to excessive bending stresses on the section. Columns of intermediate length fail in the mixed mode of buckling and crushing. Most of the practical columns fail due to this mixed mode.

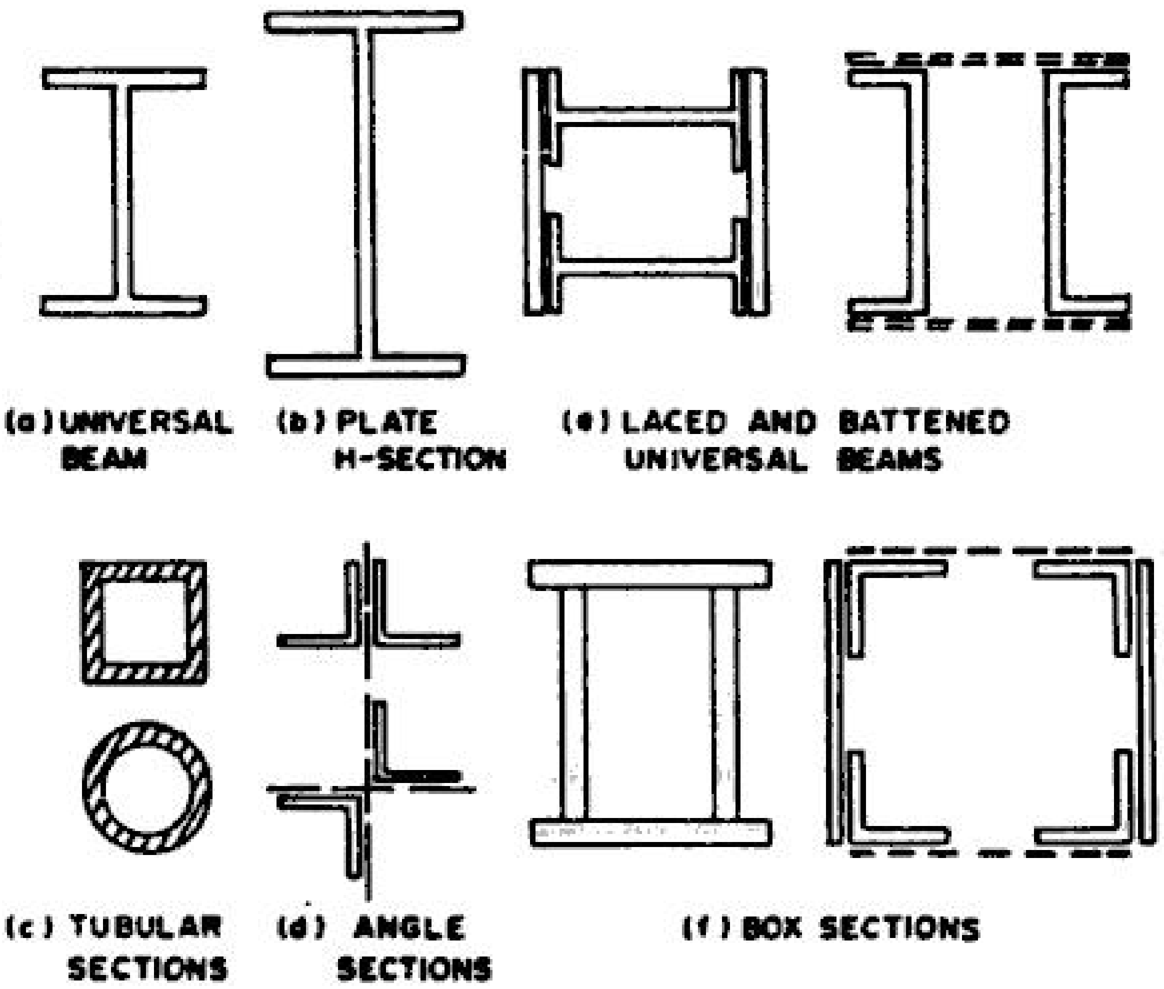


FIG. 7.2. STANCHION SECTIONS

Critical Equilibrium: Elastic Instability : Equilibrium of an absolutely rigid body may be *stable*, *neutral* and *unstable*. For example, a ball resting on a concave surface is in *stable equilibrium* and if it is given a small displacement from its position, it will return to its initial position (Fig 7.3 *a*). A ball resting on horizontal surface is in *neutral equilibrium* and if it is given a small displacement, it does not return to its initial position but its motion stops [Fig. 7.3 *b*]. Lastly, a ball resting on a convex surface is in a condition of *unstable equilibrium*; on being displaced from its original position, it continues to move further away (Fig. 7.3 *c*).

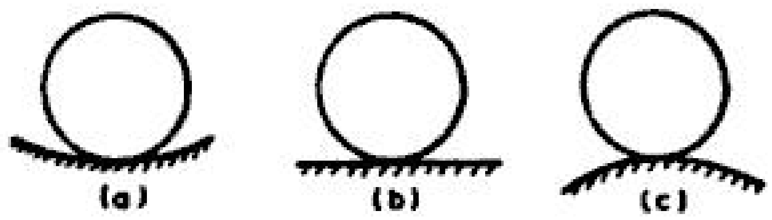


FIG. 7.3.

The stability of a long column is also of similar nature. If a column is subjected to an axial force P (less than a certain critical load P_{cr}), it will compress a little and will remain straight. The column will be in a condition of *stable equilibrium*. If it is deflected slightly by some lateral load δQ and then the load is removed, the column will become straight again and will assume the original configuration of equilibrium (Fig 7.4 *a*). The same cycle may be repeated with the force P increasing in magnitude until a critical force P_{cr} is reached at which the column remains in a slightly deformed position after the application and removal of the lateral force δQ. This instantaneous condition is in a state of *neutral equilibrium* (Fig. 7.4*b*). Finally, if force P exceeds the critical one, the straight configuration of equilibrium of the column will become unstable (Fig. 7.4 *c*). At a load greater than critical, a curvilinear configuration is stable and it can be shown by exact differential equations of elastic curve that a mere increase of 1.5% in P_{cr} causes as maximum sideways deflection equal to 22% of the length of the column, provided the material remains elastic. Such large deflections cannot be tolerated in engineering practice, since large stresses are induced in the column by the resulting eccentricity of the applied force and the column collapses. Therefore, the determination of *critical force* P_{cr} is of utmost importance, since it represents the ultimate capacity of an ideal structure. Investigations show that *instability* has been the cause of many accidents and structural damages.

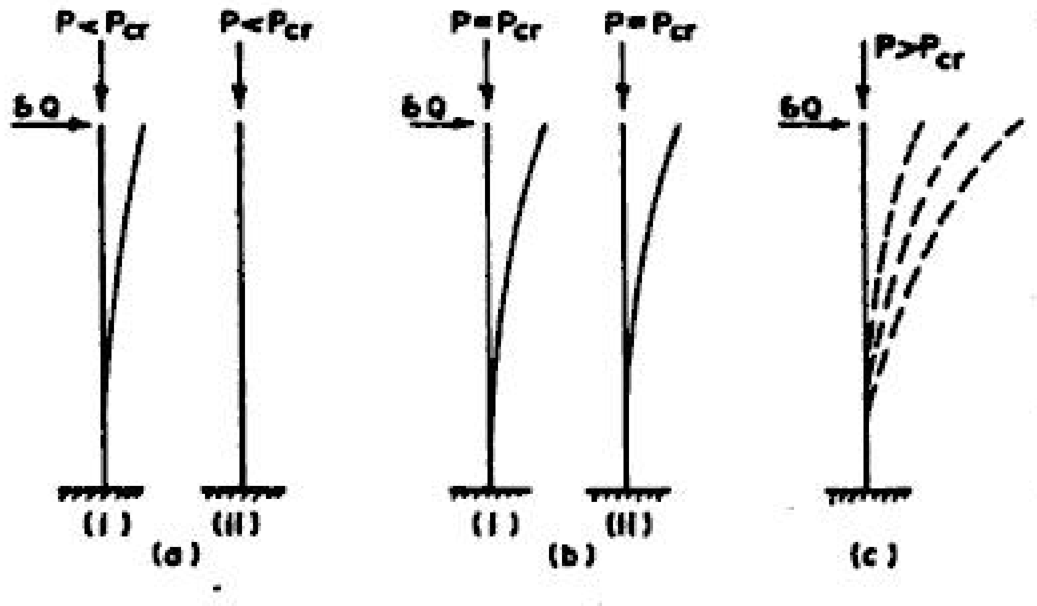


FIG 7.4.

Buckling of columns

The curved configuration of the column, under an axial load, is called *buckled shape.* Due to this the distribution of stress over the section will not be uniform, and the resulting eccentricity, however slight, will cause a bending moment. This bending moment produces bending stresses which are referred as *buckling stresses* simply to prevent confusion with bending stresses produced by eccentrically applied loads. *Thus the buckling and bending stresses are the same except that the former, by definition result from axial loads while the latter from applied eccentric load.* However, it should be clearly noted that the buckling stresses are in addition to the direct compressive stress due to applied load.

In general, the buckling tendency of a column varies with the ratio of the length to least lateral dimension. This ratio is known as *slenderness ratio.* For tall slender columns, this ratio is large and if failure occurs, it will be entirely due to buckling (mode 2). When this ratio is very small, failure occurs due principally to *yielding* or crushing (mode 1). Between these extremes are the so called intermediate columns where failure will be due to combination of buckling and crushing (mode 3).

7.3. BUCKLING FAILURE : EULER'S THEORY

Let us take a column which is very long in proportion to its cross-sectional dimensions. The column is assumed to be perfectly straight and homogeneous in quality, and the compressive load is perfectly axially applied. The strength to resist buckling is greatly affected by conditions of the ends, whether fixed or free. To start with, we will take the standard case of a column, hinged at both the ends.

At *critical load* causing *neutral equilibrium*, stated in the previous article, a compression member may buckle in any direction, if its moment of inertia I is same about the axes. Generally, a compression member does not possess equal flexural rigidity in all directions. The significant flexural rigidity EI of a column depends on the minimum I and at the critical load, a column buckles either to one side or to the other, in that plane about which I is minimum.

Consider a column with its ends free to rotate around frictionless pins. The buckled shape shown in Fig. 7.5 is possible only at a *critical load* called *buckling load, crippling load* or *Euler load,* as prior to this load, the column remains straight. *The smallest force at which a buckled shape is possible is called critical force.*

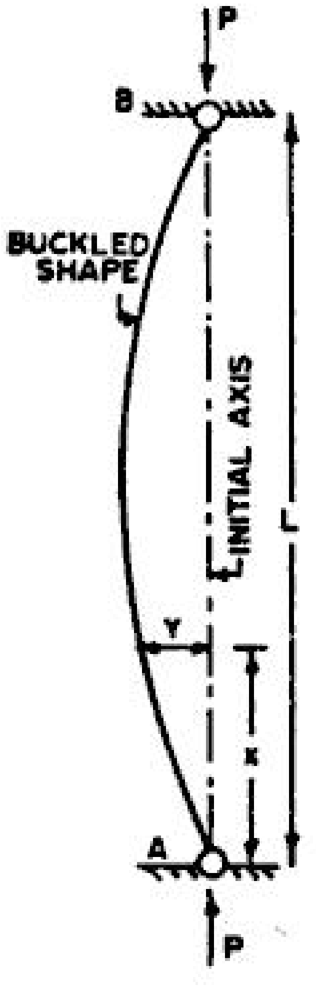


FIG. 7.5.

The effect of imperfection is to convert the strut problem from a problem of stability to a problem of stress, of which Euler's theory takes no account.

The critical stress p_E which is defined as an average stress over the cross-section is given by

$$p_E = \frac{P_E}{A} = \frac{\pi^2 EI}{AL^2} = \frac{\pi^2 EAr^2}{AL^2} = \frac{\pi^2 E}{(L/r)^2} \quad ...(7.1\ a)$$

for the standard case.

In the above expression, $\frac{L}{r}$ is known as *slenderness ratio.* The graphical interpretation of Eq. 7.1 (*a*) is shown by the curve *ACB* in Fig. 7.6, where the critical stress p_E is plotted versus the slenderness ratio for mild steel ($E = 2.0 \times 10^5$ N/mm²). It will be appreciated that the curve is entirely defined by the magnitude of *E* and is independent of its ultimate strength. A long column has greater $\frac{L}{r}$ and losses its strength at very small compressive stress. This condition cannot be improved by taking a steel of higher strength, since the modulus of steel does not vary much with alloy and heat treatment and remains practically constant.

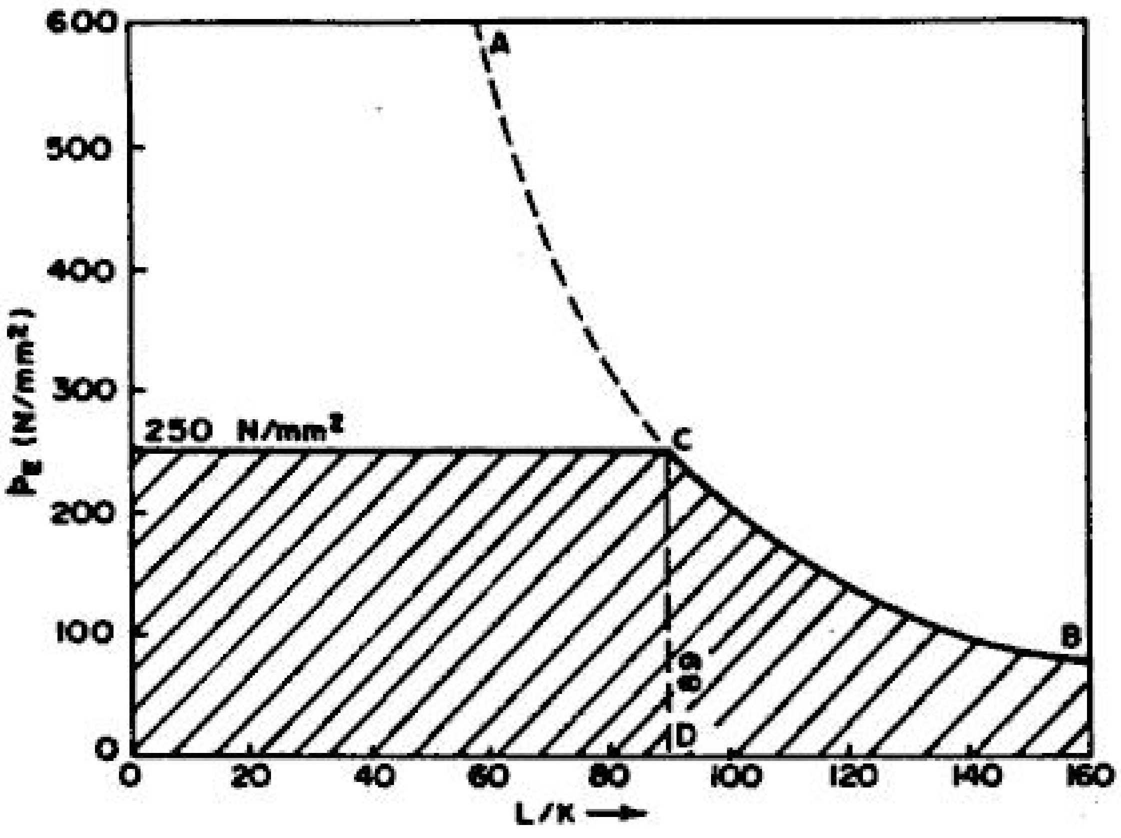


FIG. 7.6.

In Fig. 7.6, let *OY* represent the yield stress of material. It is clear that Euler formula cannot possibly apply if $\frac{L}{r}$ is less than *OD*, since for stress greater than *OY*, the material becomes plastic and no longer follows Hooke's law. Taking yield point stress at 250 N/mm², we have

$$\frac{L}{r} = OD = \sqrt{\frac{\pi^2 \times 2.06 \times 10^5}{250}} \approx 89.$$

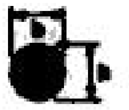
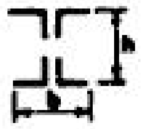
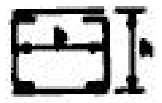
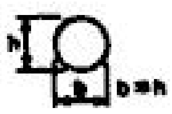
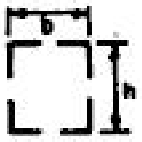
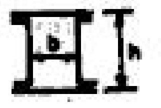
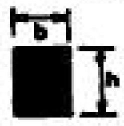
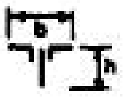
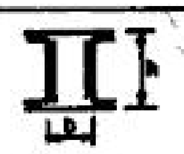
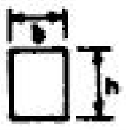
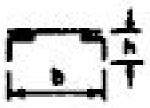
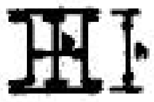
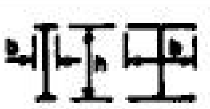
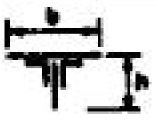
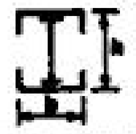
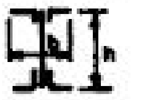
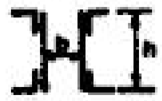
Hence Euler's formula is applicable only if $\frac{L}{r}$ is greater than 89 for steel column hinged at both the ends.

7.4. IDEAL END CONDITIONS AND EFFECTIVE LENGTH

A column may have two *types* of end conditions

(*i*) Position restraint (*ii*) Direction restraint.

An end has a *position restraint* when it is not free to change its position (or is not free to translate), but is free to rotate. The *hinged end* of a column is a typical example of position restraint. In the direction restraint, the end of the column is not free to change its direction. It may be free to change its position, or it may not be free to change its position. However, an end of a column may have both the position retraint as well as direction restraint.

Section	r_x/h	r_y/b	Section	r_x/h	r_y/b	Section	r_x/h	r_y/b
	0.250	0.250		0.40 0.45	0.19 0.22		0.40 0.44	0.38 0.42
	0.350	0.350		0.40 0.45	0.40 0.45		0.40 0.44	0.50 0.54
	0.294	0.294		0.25 0.30	0.19 0.22		0.37 0.40	0.48 0.52
	$0.5\sqrt{\frac{3b+h}{3(b+h)}}$	$0.5\sqrt{\frac{3h+b}{3(h+b)}}$		0.27 0.30	0.35 0.38		0.40 0.44	0.25 0.30
	0.38 0.42	0.22 0.25		0.20 0.25	0.20 0.25		0.39 0.44	0.30 0.33
	0.20	0.40		0.38 0.40	0.19 0.23		0.33 0.36	0.52 0.54

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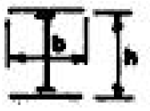
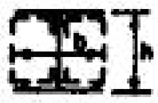
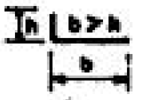
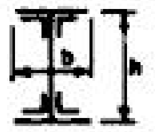
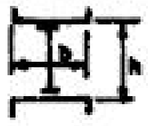
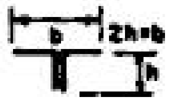
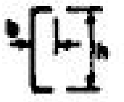
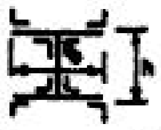
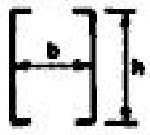
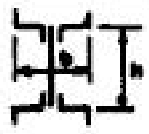
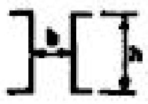
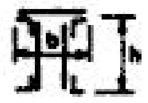
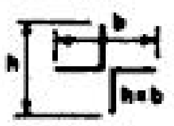
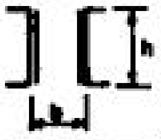
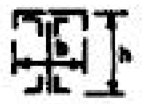
	0.30 0.32	0.30 0.32		0.38 0.43	0.19 0.22		0.42 0.46	0.27 0.29
	0.28 0.31	0.31 0.33		0.40 0.45	0.20 0.25		0.49 0.52	0.30 0.33
	0.30 0.32	0.21 0.22		0.35 0.35	0.20 0.25		0.49 0.52	0.27 0.29
	0.31 0.33	0.19 0.21		0.35 0.37	0.42 0.46		0.46 0.49	0.27 0.30
	0.28 0.30	0.22 0.25		0.35 0.37	0.54 0.56		0.36 0.40	0.52 0.56
	0.21 0.22	0.21 0.22		0.33 0.36	0.52 0.54		0.36 0.40	0.23 0.26

FIG. 7.11. APPROXIMATE VALUES OF RADII OF GYRATION FOR COMPRESSION MEMBERS

7.6. RADIUS OF GYRATION AND SLENDERNESS RATIO

The *radius of gyration* of a section is given by

$$r = \sqrt{I/A}$$

For every section, the values of radius of gyration about principal axes are required so that least radius of gyration (r_{min}) may be obtained to find slenderness ratio. The values of radius of gyration of individual sections are given in ISI handbook No. 1. However, for built-up sections, the computations of radius of gyration is extremely tedius. These values are therefore selected from ready made tables available for various types of built-up sections. Fig. 7.11 gives the values of radius of gyration about principal axes, for some common forms of individual as well as built-up sections. These values may be adopted in the preliminary design involving trial sections.

The *slenderness ratio* (λ) of a compression member is defined as the ratio of its effective length to the appropriate radius of gyration:

$$\lambda = \frac{L_E}{r} = \frac{l}{r}$$

For finding the maximum value of slenderness ratio of given member, the least value of its radius of gyration should be selected. As per Indian Standard IS : 800–1984, the maximum slenderness ratio should not exceed the values given in Table 7.3.

TABLE 7.3 MAXIMUM SLENDERNESS RATIOS

	Member	*Maximum slenderness ratio* λ
1.	A member carrying compressive loads resulting from dead load and imposed loads.	180
2.	A member subjected to compressive forces resulting from wind/earthquake forces, provided the deformation of such member does not adversely affect the stress in any part of the structure.	250
3.	A member normally acting as a tie in a roof truss or a bracing system but subject to possible reversal of stresses resulting from the action of wind or earthquake forces	350

7.7. VARIOUS COLUMN FORMULAE

We have seen in § 7.3 that Euler's formula for critical load is applicable only when the slenderness ratio is more than 89. For values less than this, it ceases to apply. Even for values higher than 89, it does not give satisfactory results. In fact, Euler's formula is suitable for *large* slenderness ratio ($L/r > 200$). Apart from this limitation, a column is never *ideal* — it may have small imperfections in its geometry due to which small eccentricity to axial load may be induced. In order to solve the problem of columns of small L/r ratios, and small eccentricity of loads, various formulae were evolved, partly on the basis of theoretical approach and partly on the basis of experimental results. The following column formulae are of practical importance:

1. Rankine's formula.
2. Rankine-Gorden formula.
3. Johnson's straight line and parabolic formula.
4. Secant formula.
5. Merchant Rankine formula : I.S. code formula.

1. Rankine's formula :

Euler's formula, though valid for L/r ratio higher than 89 (for mild steel), does not take into account the direct compressive stress and hence gives correct results only for very long columns. For columns intermediate between very long and short ones, Euler's formula, though applicable, does not give correct results.

Rankine proposed an empirical formula for columns which covers all cases ranging from very short to very long struts. He proposed the relation

$$\frac{1}{P} = \frac{1}{P_C} + \frac{1}{P_E}$$

where $P_C = f_c \, . \, A$ = ultimate load for a strut.

$P_E = \frac{\pi^2 EI}{l^2}$ = Eulerian crippling load for the standard case.

In the above relation, $\frac{1}{P_C}$ is constant for a material. For short column, P_E is very large and hence $\frac{1}{P_E}$ is small in comparison to $\frac{1}{P_C}$, thus making crippling load P approximately equal to P_C. For long columns, P_E is extremely small and hence $\frac{1}{P_E}$ is large as compared to $\frac{1}{P_C}$, thus making the crippling load approximately equal to P_E. Thus the value of P obtained from the above relation covers all cases ranging from short to long columns or struts.

The above relation can be re-arranged as :

$$P = \frac{P_C \, . \, P_E}{P_E + P_C} = \frac{P_C}{1 + P_C/P_E} = \frac{f_c \, . \, A}{1 + \frac{f_c A L^2}{\pi^2 EI}}$$

or

$$P = \frac{f_c \, . \, A}{1 + \left(\frac{f_c}{\pi^2 E}\right)\left(\frac{L}{r}\right)^2} = \frac{f_c \, . \, A}{1 + a\left(\frac{L}{r}\right)^2} \quad ...(7.7)$$

In the above equation, f_c is the crushing stress for the material and a is the Rankine's constant for the material. Though the formula is empirical, the values of constant a is not calculated from the values of f_c, π^2 and E, but is determined experimentally. Table 7.4 gives the values of f_c, and a for the materials commonly used for columns and struts.

TABLE 7.4 RANKINES CONSTANTS.

	Material	f_c		a *(For hinged ends)*
		kg/cm²	*N/mm²*	
1.	Wrought Iron.	2550	255	$\frac{1}{9000}$
2.	Cast Iron	5670	567	$\frac{1}{1600}$
3.	Mild Steel	3300	330	$\frac{1}{7500}$
4.	Strong Timber	500	50	$\frac{1}{750}$

Eq. 7.7 is the Rankine's formula for the standard case of column hinged at ends. For columns of other end conditions, the values of constants will be changed accordingly. However,. since a is a constant for a particular material used as a hinged column, it is not convenient

to remember the values of a for other end conditions. It is therefore better to modify the Rankine's formula and remember in the form :

$$P = \frac{f_c . A}{1 + a\left(\frac{L_E}{r}\right)^2} = \frac{f_c . A}{1 + a\left(\frac{l}{r}\right)^2} \quad ...(7.8)$$

in which a is the Rankine's constant for a particular material and for the standard case of column, and L_E or l is the equivalent length of the column. Since equivalent length is generally known, it can be substituted in the above formula to get the Rankine's formula for a particular case of end conditions. *It should be noted that Eqs. 7.7 and 7.8 do not include the factor of safety.*

2. Rankine Gorden formula It is expressed in the form :

$$\frac{P}{A} = \frac{C_1}{1 + C_2\left(\frac{l}{r}\right)^2} \quad ...(7.9)$$

A.I.S.C. adopted this formula in 1949 for design of secondary members having l/r ranging between 120 to 200, taking a value of $C_1 = 1260$ kg/cm^2 and $C_2 = 1/18000$. Similarly, Building Code of New York adopted this formula for main members having l/r between 60 to 120, and for secondary members having L/r ratio ranging between 60 to 200.

3. Johnson's straight line and parabolic formula

Johnson proposed the following two formulae:

$$P = A\left(f - n\frac{l}{r}\right) \quad ...(7.10)$$

and

$$P = A\left(f - n'\frac{l^2}{r^2}\right) \quad ...(7.11)$$

Eq. 7.10 is known as *straight line formula*, sometimes written in the form

$$\frac{P}{A} = C_1 - C_2\frac{l}{r} \quad ...(7.10\ a)$$

For mild steel, C_1 is 1125 kg/cm^2 and $C_2 = 5.625$ kg/cm^2. The Chicago Building Code adopted this formula, taking $C_1 = 1120$ kg/cm^2 and $C_2 = 4.8$ kg/cm^2, for member having l/r ratio ranging between 30 and 120. Similarly, American Bridge Co. adopted this formula with a maximum value of permissible stress as 915 kg/cm^2, taking $C_1 = 1335$ kg/cm^2 and $C_2 = 7.03$ kg/cm^2.

Eq. 7.11 is known as *parabolic formula*, sometimes written in the form

$$\frac{P}{A} = C_3 - C_4\left(\frac{l}{r}\right)^2 \quad ...(7.11\ a)$$

AISC adopted this formula for members having $l/r < 120$, taking $C_3 = 1190$ kg/cm^2 and $C_4 = 0.0334$ kg/cm^2. A.R.E.A. and A.A.S.H.O. adopted this formula for members having $l/r < 140$, taking the following values :

$C_3 = 1090$ kg/cm^2 and $C_4 = 0.0175$ kg/cm^2, for riveted structures.

and $C_3 = 1050$ kg/cm^2 and $C_4 = 0.0233$ kg/cm^2, for welded structures.

4. Secant formula

The secant formula is also based on the same assumptions as the Euler's formula. However, in the secant formula, load is not axial, but has small eccentricity e. According to this formula, average stress *at failure* is given by

(*a*) Flanges and plates in compression with unstiffened edges : $\frac{256\,t_1}{\sqrt{f_y}}$, subject to a maximum of $16\,t_1$.

(*b*) Flanges and plates in compression with stiffened edges : $20\,t_1$ to the innermost face of the stiffening

(*c*) Flanges and plates in tension : $20\,t_1$

Note 1. Stiffened flanges shall include flanges composed to channels or I-sections or of plates with continuously stiffened edges.

Note 2. t_1 denotes the thickness of the flanges of a section or of a plate in compression, or the aggregate thickness of plates, if connected appropriately.

Note 3. The width of the outstand of members referred above shall be taken as follows:

	Types	*Width of outstand*
(*i*)	Plates	Distance from the free edge to the row of rivets or welds.
(*ii*)	Angles, channels, Z-sections and stems of Tee-sections	Nominal width
(*iii*)	Flange of beam and tee sections	Half the nominal width.

(*B*). Where a plate is connected to other parts of a built up member along lines generally parallel to the longitudinal axis of the member, the width between any two adjacent lines of connections or supports shall not exceed the following :

For plates in uniform compresion : $\frac{1440\,t_1}{\sqrt{f_y}}$ subject to max. of $90\ t_1$

However, where the width exceeds : $\frac{560\,t_1}{\sqrt{f_y}}$ subject to a max. of $35\,t_1$ for welded plates which are not stress relieved or $\frac{800\,t_1}{\sqrt{f_y}}$, subject to max. of $50\,t_1$, for other plates, the excess width shall be assumed to be located centrally and its sectional area shall be neglected when calculating the effective geometrical properties of the section.

5. Angle struts (Table 7.6 and Fig. 7.13)

(i) Single angle struts: discontinuous:

(*a*) Single angle discontinuous struts connected by a single rivet or bolt may be designed for axial load only, provided the compressive stress (σ_{ac}) does not exceed 80 percent of the values given in Table 7.5, in which the effective length l of the strut shall be taken as centre to centre of intersection at each end and r is the minimum radius of gyration. In no case, however, shall the ratio of slenderness of such single angle struts exceed 180.

(*b*) Single angle discontinuous struts connected by a weld or by two or more rivets or bolts in line along the angle at each end may be designed for axial load only, provided the compressive stress does not exceed the values given in Table 7.5, in which the effective length l shall be taken as 0.85 time the length of the strut, centre to centre of intersection at each end and r is the minimum radius of gyration.

(ii) Double angle struts: discontinuous

(*a*) For double angle discontinuous struts, back to back connected to both sides of the gusset or section by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be required as applied axially. The effective length l in the plane of end gusset shall be taken as between 0.7 to 0.85 times the distance

$$r_{min} = r_{yy} = \sqrt{\frac{26.17 \times 10^6}{9755}} \approx 51.8 \text{ mm}$$

$$\lambda_{max} = \frac{l}{r_{min}} = \frac{5000}{51.8} = 96.53.$$

$$\therefore \quad \sigma_{ac} = 90 - 6.53\left(\frac{10}{10}\right) = 83.47 \text{ N/mm}^2$$

$$\therefore \quad P = A\,\sigma_{ac} = 9755 \times 83.47 \times 10^{-3} = \mathbf{814.25\ kN}$$

(b) Plates attached to the flange (Fig. 7.17)

$$I_{xx} = 5131.6 \times 10^4 + 2\left[\frac{1}{12} \times 250\,(10)^3 + 2500\,(125+5)^2\right]$$

$$= 135.86 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 334.5 \times 10^4 + 2 \times \frac{1}{12} \times 10\,(250)^3 = 29.387 \times 10^6 \text{ mm}^4$$

$$A = 4755 + 2\,(250 \times 10) = 9755 \text{ mm}^2$$

$$\therefore \quad r_{min} = r_y = \sqrt{\frac{29.387 \times 10^6}{9755}} = 54.89$$

$$\lambda_{max} = \frac{5000}{54.89} = 91.1$$

$$\therefore \quad \sigma_{ac} = 90 - 1.1\left(\frac{10}{10}\right) = 88.9 \text{ N/mm}^2$$

Hence $P = 88.9 \times 9755 \times 10^{-3} = \mathbf{876.22\ kN}$

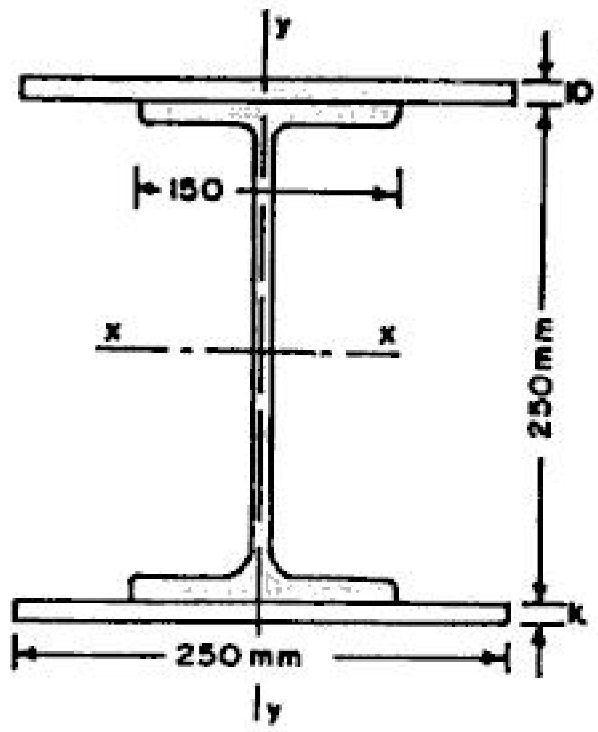


FIG. 7.17.

Example 7.6. *A built-up column shown in Fig. 7.18 (a) consists of two ISMC 250 @ 30.4 kg/m. Determine the maximum allowable load for the column if the channels are so placed as to give the column equal resistance to bending about either axis. Take effective length of column as 4.5 m. What will be the load capacity of the column if the channels are arranged as shown in Fig. 7.18 (b)? Comment on this arrangement. Take $f_y = 250$ N/mm². The channels are braced appropriately in either case.*

Solution. For ISMC 250 @ 30.4 kg/m, we have:

$I_{xx} = 3816.8 \times 10^4$;

$I_{yy} = 219.1 \times 10^4$;

$C_{yy} = 23.0$ mm ;

$a = 3867$ mm²; $b = 80$ mm

Case (a) : Channels arranged as shown in Fig. 7.18 a

Let the back-to-back distance between the channels be x mm. For equal resistance, $I_X = I_Y$. Now,

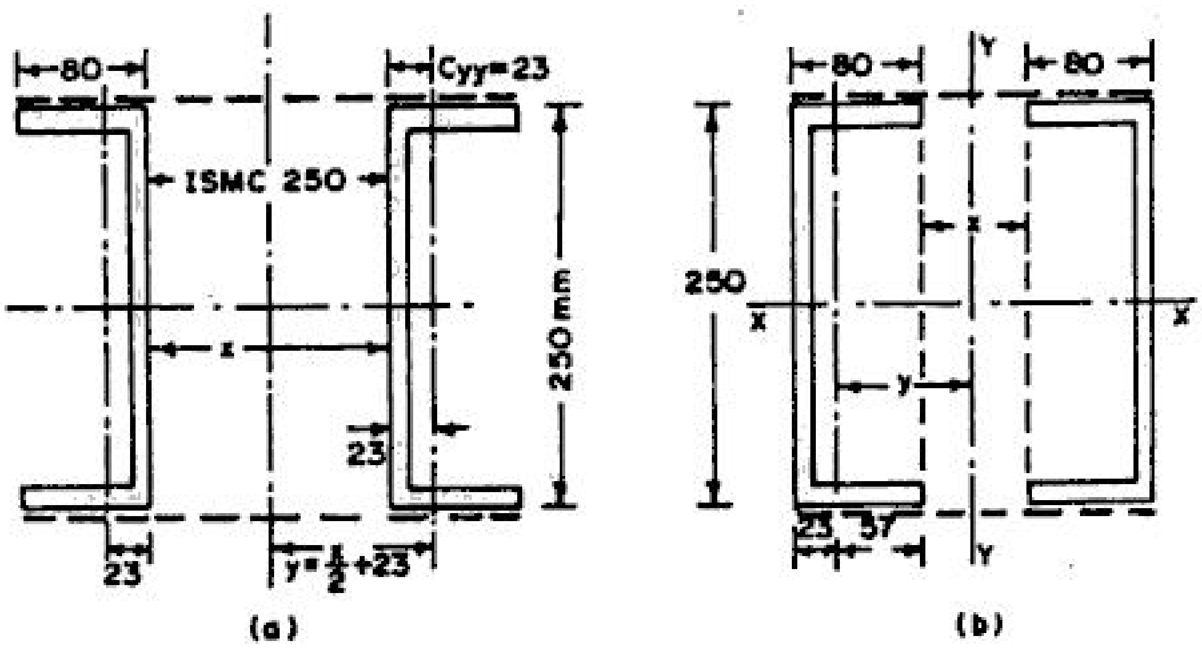


FIG. 7.18.

$$I_X = 2I_{xx} = 2 \times 3816.8 \times 10^4$$

$$= 7633.6 \times 10^4 \text{ mm}^4$$

$$I_Y = 2\,[I_{yy} + ay^2] = 2\left[219.1 \times 10^4 + 3867\left(\frac{x}{2} + 23\right)^2\right] = 438.2 \times 10^4 + 7734\,(0.5x + 23)^2$$

Now $I_X = I_Y$, for equal strength.

$\therefore$ $7633.6 \times 10^4 = 438.2 \times 10^4 + 7734\,(0.5x + 23)^2$

From which $x = 146.91$ mm.

Hence keep the distance between the channels = 146.91 mm. In that case,

$$I_X = I_Y = I = 7633.6 \times 10^4 \text{ mm}^4.$$

$$r = \sqrt{I/A} = \sqrt{(7633.6 \times 10^4)/2 \times 3867} = 99.34 \text{ mm}.$$

$$\lambda = \frac{l}{r} = \frac{4.5 \times 1000}{99.34} = 45.29$$

$$\therefore \quad \sigma_{ac} = 139 - (139 - 132)\frac{5.29}{10} = 135.3 \text{ N/mm}^2$$

$$\therefore \quad P_{safe} = A\,.\,\sigma_{ac} = 2 \times 3867 \times 135.3 \times 10^{-3} \approx \mathbf{1046.4\ kN}$$

(b) When the channels are arranged as shown in Fig. 7.18 (b)

Let the distance between the inner tips of the two channels be x mm.

$$y = 0.5x + 57$$

$$I_Y = 2\,[I_{yy} + ay^2] = 2\,[219.1 \times 10^4 + 3867\,(0.5x + 57)^2\,]$$

$$= 438.2 \times 10^4 + 7734\,(0.5x + 57)^2$$

For equal strength, $I_Y = I_X$.

$\therefore$ $438.2 \times 10^4 + 7734\,(0.5x + 57)^2 = 7633.6 \times 10^4$

From which $x = 78.91$ mm

In that case, $I_X = I_Y = 7633.6 \times 10^4$ mm^2

Hence, r, λ, σ_{ac} and P will be the same as before.

Comments: In this arrangement, the distance between the outer faces of channels will be $= 2 \times 80 + 78.91 = 238.91$ mm, as against a distance of ($2 \times 80 + 146.91 = 306.91$ mm). Hence the material required for bracing in the second case will be less. *However, the arrangement for the second case is unsatisfactory from the point of view of fabrication, inspection and maintenance.*

Example 7.7. *A built-up column consists of ISMB 300 @ 44.2 kg/m, having two plates, each of 10 mm thickness, attached to each flange so as to have equal resistance about either axis. Determine the load the column can safely carry over an effective length of 4.5 m. Take $f_y = 250$ N/mm^2.*

Solution For ISMB 300 @ 44.2 kg/m, we have:

$I_{xx} = 8603.6 \times 10^4$ mm^4 ; $I_{yy} = 453.9 \times 10^4$ mm^4.

$a = 5626$ mm^2. Width of flange = 140 mm.

Let the width of plates be b. This width should be such that we get $I_X = I_Y$ so that the column may have equal resistance in either direction.

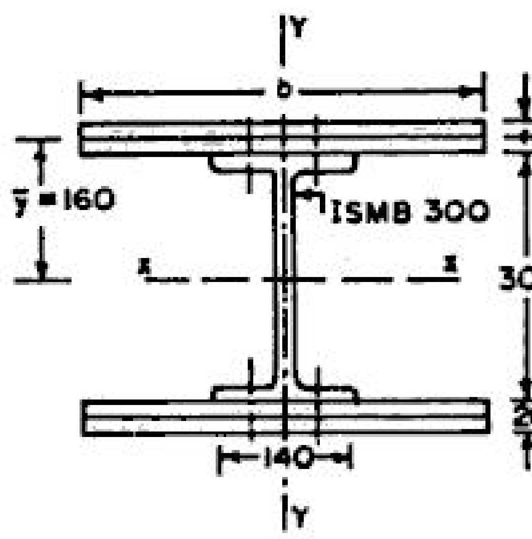


FIG. 7.19.

$$I_X = I_{xx} + 2\left[\frac{1}{12}bt^3 + bt\,\bar{y}^2\right]$$

$$= 8603.6 \times 10^4 + 2\left[\frac{1}{12}b\,(20)^3 + b \times 20\,(160)^2\right]$$

$$= 8603.6 \times 10^4 + 102.53 \times 10^4\,b \qquad \text{...(1)}$$

$$A = \frac{\pi}{4}(D_1^2 - D_2^2) = \frac{\pi}{4}[(150)^2 - (100)^2]$$

$$= 9817.477 \text{ mm}^2$$

$$\therefore \; r = \sqrt{\frac{I}{A}} = \sqrt{\frac{19.9418 \times 10^6}{9817.477}} = 45.069 \text{ mm.}$$

Alternatively, for a hollow section,

$$r = \sqrt{\frac{D_2^2 + D_1^2}{16}} = \sqrt{\frac{(150)^2 + (100)^2}{16}} = 45.069$$

$$\sigma_{ac} = \frac{P}{A} = \frac{800 \times 1000}{9817.477} = 81.49 \text{ N/mm}^2.$$

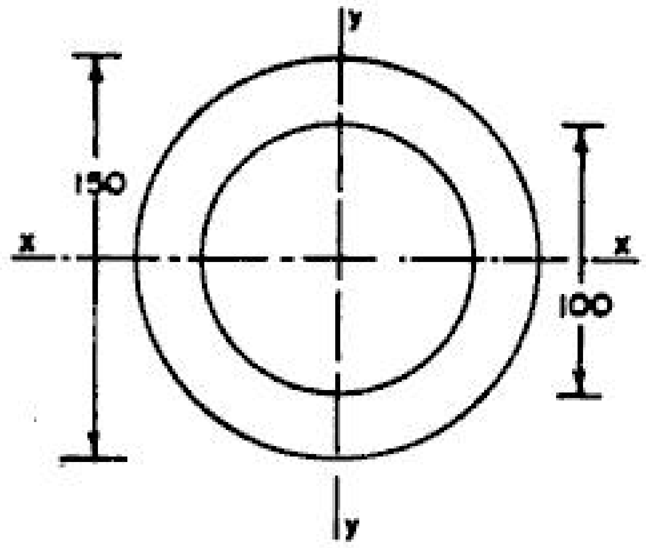


FIG. 7.24.

Corresponding to this value of $\sigma_{ac} = = 81.49$ and $f_y = 250$ N/mm², we get,

$$\lambda = 100 - (100 - 90) \times \frac{81.49 - 80}{90 - 80} = 98.51$$

But $\lambda = \frac{l}{r}$

Hence $l = \lambda . r = 98.51 \times 45.069 \approx 4440$ mm $=$ **4.44 m**

7.12. DESIGN OF COMPRESSION MEMBERS

The design of a compression member is basically a trial and error procedure. For a column of given length, end conditions and load, the gross area of cross-section depends upon the permissible stress σ_{ac} which, in turn, depends upon l/r ratio. Since the section is not known before hand, the radius of gyration is unknown, and hence l/r ratio is also not known. The design is therefore, done in the following steps:

Step 1. Given the actual length and the end conditions, find the effective length l.

Step 2. *Assume* suitable value of slenderness ratio of the member, as under:

(*a*)	for single struts	:	Assume λ between 120 to 150
(*b*)	For double angle strut	:	Assume λ between 100 to 120
(*c*)	For channel section strut	:	Assume λ between 80 to 100
(*d*)	For I-section stanchions	:	Assume λ between 60 to 90
(*e*)	For built-up sections	:	Assume λ between 30 to 50 (with lower value for I-sections and higher value for channel sections).

Step 3. Select the value of σ_{ac}, corresponding to the assumed value of λ, from Table 7.5 (or, from Table 5.1 of the code)

Step 4. Compute the gross-area by the relation:

$$A = \frac{P}{\sigma_{ac}}$$

Step 5. From the section tables, select a suitable section having the above area. Find the minimum radius of gyration r_{min}. for this section.

Step 6. Compute $\lambda = l/r$. If this matches with the assumed value of λ, the design is ok. Otherwise repeat steps 3 to 6.

Example 7.13. *Design a column of rolled steel I-section to carry an axial load of 500 kN. The column is 4 m long and is effectively held in position at both ends but restrained against at one end only. Take yield stress in steel as 250 N/mm².*

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position, without sharing any axial load. However, when the column deflects, the lateral system carries the transverse shear force.

The following *lateral systems* (Fig. 7.28) are used for compound columns:

(*i*) Lacing or latticing
(*ii*) Battening or batten plates
(*iii*) Perforated cover plates.

These are discussed in the subsequent articles.

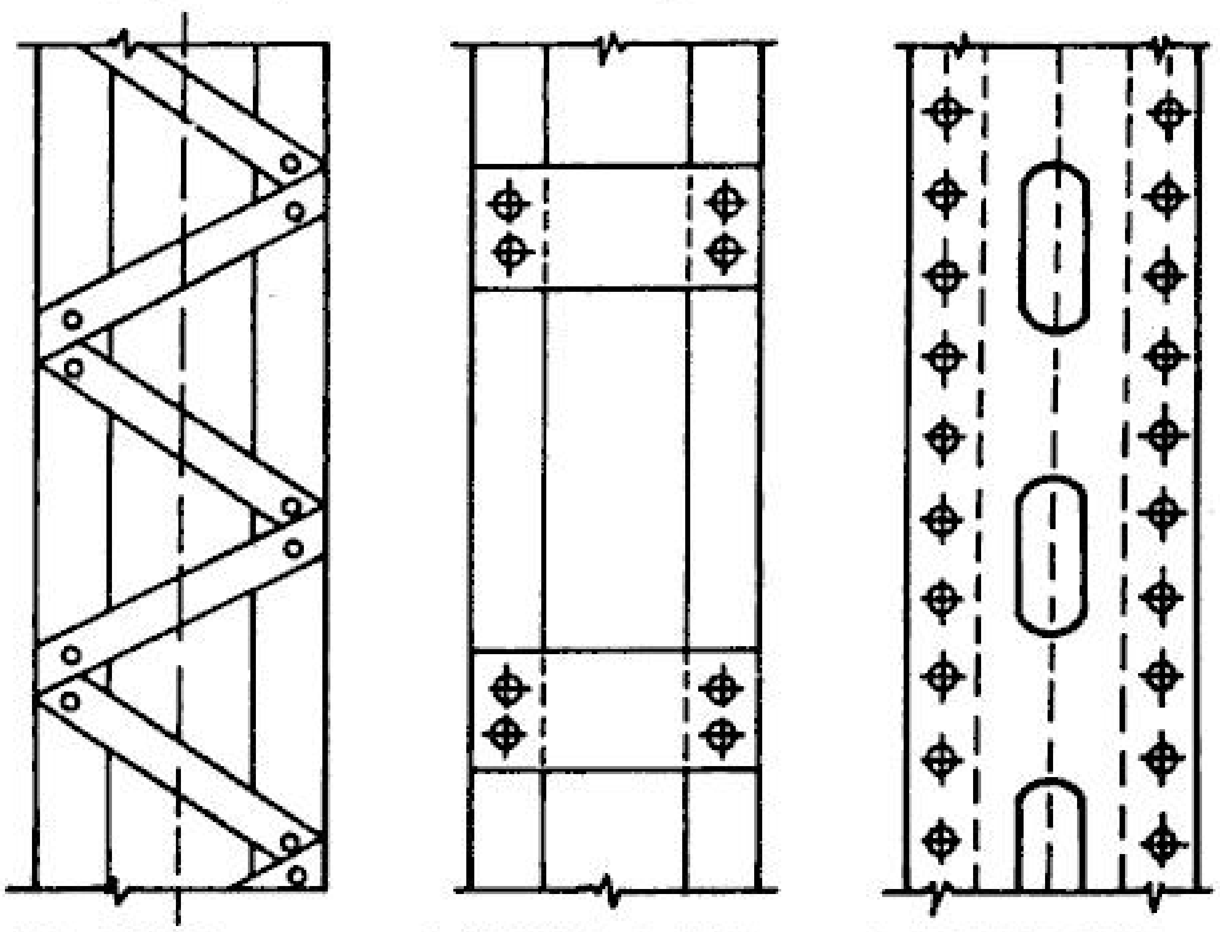
Design procedure

The design of a built-up or compound column is carried out in the following steps:

Step 1. Given the actual length and end conditions, find the effective length *l*.

Step 2. Assume a suitable value of slenderness ratio λ ranging from 30 to 50 (see § 7.12).

Step 3. Select the value of σ_{ac} corresponding to this assumed value of λ, from Table 7.5. Alternatively, σ_{ac} may be assumed as any value ranging between 130 to 145 N/mm^2.



(a) LACINGS (b) BATTEN PLATES (c) PERFORATED COVER PLATES

FIG. 7.28. LATERAL SYSTEMS.

Step 4. Find the gross area required:

$$A = \frac{P}{\sigma_{ac}}$$

Step 5. Try elemental sections such that their total area is more than *A* required above. Arrange the elements in such a way that we get $I_X \approx I_Y$ for the composite section.

Step 6. For the above arrangement find r_{min}. Hence compute maximum slenderness ratio $\lambda = l/r_{min}$. In the case of battened columns, the effective length should be *increased* by 10 %.

Step 7. For this value of λ, compute σ_{ac} (from Table 7.5) and hence load capacity (*P*) of the column. This should be equal to the actual load. If the load capacity comes out to be *less* than the actual load, *revise* the section. Similarly, if the load capacity of the column is different from actual load by a larger margin, revise the section.

7.14. LACINGS

Lacing or *latticing* is the most commonly used lateral system. The common sections used as lacings are: flats, angles, and channels. Tubular sections may also be used. Lacings may be of two types:

(*a*) Single lacing (*b*) Double lacing.

General Requirements: IS : 800-1984 lays down the following general requirements:

1. Compression members comprising of two main components laced and tied should, where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis in the plane of lacing. Thus in Fig. 7.29 (*a*), $r_Y \nless r_X$.

$$V = \frac{2.5}{100} \times P \qquad \text{...(7.18)}$$

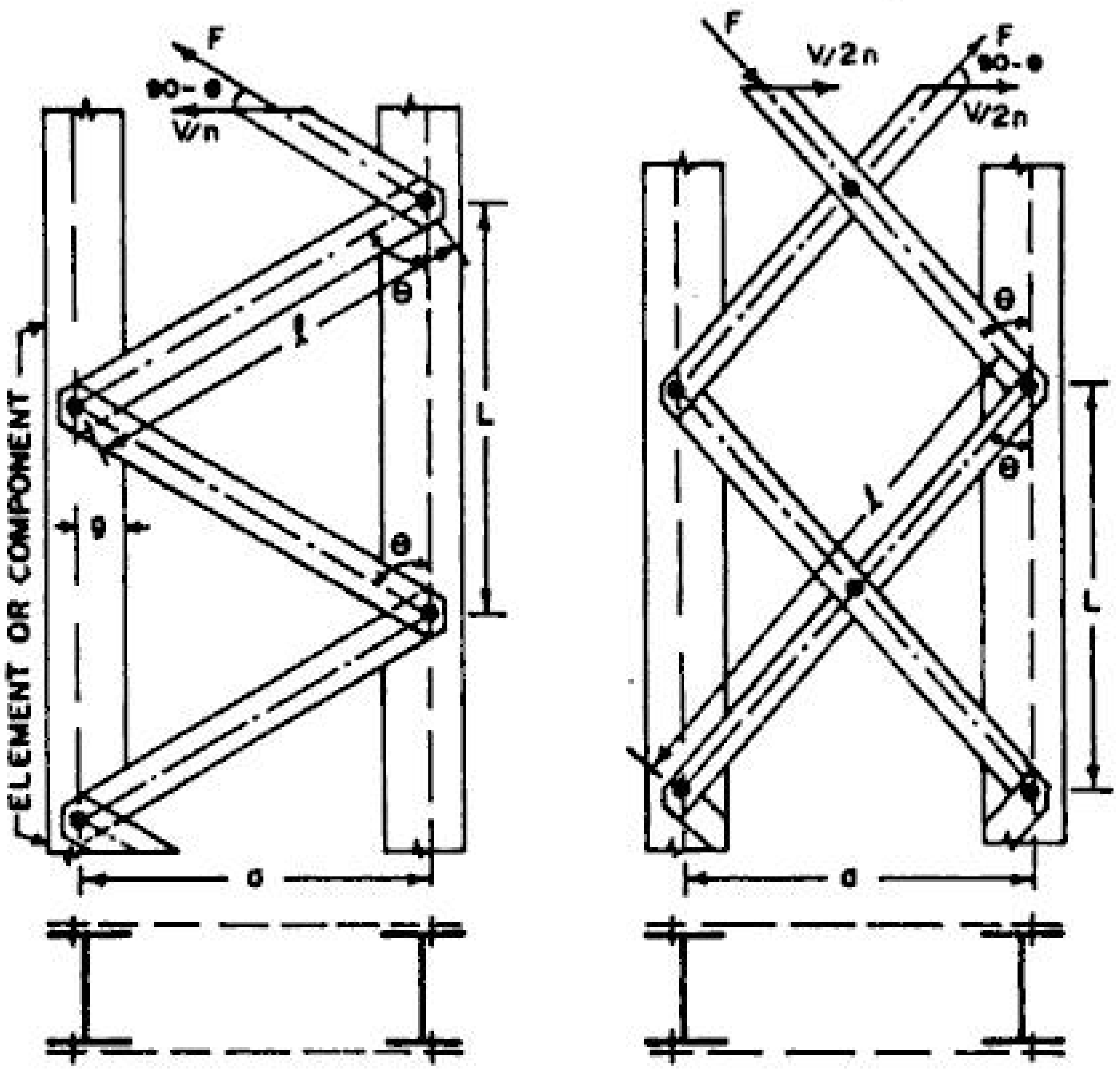
where P is the axial force in the member.

For single lacing system shown in Fig. 7.31 (*a*), the transverse shear force in each lacing is $\frac{V}{n}$, wher n = number of transverse system in parallel planes (= 2 for the present case). Hence axial force F in each lacing is

$$F = \frac{V}{n \sin \theta} \qquad \text{...(7.19 a)}$$

where θ is the inclination of the lacing with vertical. This force will be tensile in one lacing and compressive in the next lacing, meeting at a point.

For double lacing system shown in Fig. 7.31 (*b*), there are two lacings cut by any horizontal plane. Hence shear force in each lacing is $\frac{V}{2n}$, where n = number of transverse system in parallel planes (= 2 for the present case). Hence the axial force F in each lacing is



(a) SINGLE LACING (b) DOUBLE LACING

FIG. 7.31. FORCE IN LACING.

$$F = \frac{V}{2n \sin \theta} \qquad \text{...(7.19 b)}$$

This force will be compressive in one lacing bar and tensile in the other lacing bar. For flat lacing bar having width b, thickness t and rivet diameter d, we have

$$\text{Compressive stress in each lacing bar} = \frac{F}{b \times t} \ngtr \sigma_{ac} \qquad \text{...7.20 (a)}$$

$$\text{Tensile stress in each lacing bar} = \frac{F}{(b - d)\,t} \ngtr \sigma_{at} \qquad \text{...7.20 (b)}$$

2. Angle of Inclination : Lacing bars, whether in double or single systems, shall be inclined at an angle not less than 40° nor more than 70° to the axis of the member.

3. Slenderness ratio and effective length : The slenderness ratio λ (= l/r) of the lacing bars for compression members shall not exceed 145.

For flat bars of width b and thickness t, used for lacing, we have

$$\frac{l_e}{r} = \frac{l_e}{\sqrt{I/A}} = \frac{l_e}{\sqrt{(\frac{1}{12} b t^3) / (b \times t)}}$$

or

$$\frac{l_e}{r} = \frac{l_e \sqrt{12}}{t} \ngtr 145 \qquad \text{...(7.21)}$$

The effective length l_e of the lacing bars should be taken follows :

Type of Lacing	*Effective length* (l_e)
1. Single lacing system (Riveted at ends)	Length between inner end rivets of the lacing bar. Thus, in Fig. 7.31 (*a*), $l_e = l$
2. Double lacing system (Riveted at ends and at intersection)	0.7 times length between the inner end rivets of the lacing bar effectively riveted at intersection. Thus in Fig. 7.31 (*b*), $l_e = 0.7\,l$
3. Single or double lacing system, welded	0.7 times the distance between the inner ends of welds connecting the lacing bars to the member.

4. Maximum spacing: The maximum spacing of lacing bars, whether connected by riveting or welding, shall be such that the slenderness ratio $\lambda_e\ (= L/r_e)$ of the components or elements of the member between consecutive connections is not greater than 50 or 0.7 times the most unfavourable slenderness of the member as a whole, whichever is less, where L is the distance between centres of connection of the lattice bars to each component (as shown in Fig. 7.31).

The slenderness ratio of the components within the lacing spacing is

$$\lambda_{element} = \lambda_e = \frac{L}{r_e} \qquad ...(7.22)$$

where $L=$ spacing of lacing and r_e is the minimum radius of gyration of elements. This clause is aimed at avoiding any local buckling of the component.

It should be noted that if a is the centre to centre distance (horizontal) between the rivet lines of the two elements (Fig. 7.31), the spacing (L) of the lacing is given by

$$L = 2a \cot\theta \text{ for single lacing system}$$

and $$L = a \cot\theta \text{ for double lacing system}$$

Also, $$l = \frac{L}{2} \sec\theta = a \operatorname{cosec}\theta \text{ for single lacing}$$

and $$l = L \sec\theta = a \text{ coses}\,\theta \text{ for double lacing.}$$

where l is the length of the lacing between inner end rivets.

5. Width of Lacing bars: In riveted construction, the minimum width of lacing bars shall be follows :

Nominal Rivet Dia. (mm)	*Width of lacing bars (mm)*
22	65
20	60
18	55
16	50

6. Thickness of Lacing bars: The thickness of flat lacing bars shall be not less than one-fortieth of the length betweeen the inner end rivets or welds for single lacing, and one-sixtieth of the length for double lacing riveted or welded at inerstections.

Thus $$t \nless l/40 \text{ single lacing}$$

and $$t \nless l/60 \text{ for double lacing.}$$

It should be noted that width b and thickness t should be such that the compressive or tensile stresses (Eq. 7.20) induced in the lacing, due to force F (Eq. 7.19), do not exceed the permissible values. Rolled sections or tubes of equivalent strength may be used instead of flats.

For double lacing system shown in Fig. 7.31 (*b*),

$$L = \frac{a}{\tan\theta} = a\cot\theta \qquad \text{...7.2 } (b)$$

Also, $$l = a \operatorname{cosec}\theta \qquad \text{...7.26 } (c)$$

for both single lacing as well as double lacing.

Step 3. Find the slenderness ratio of each element (or component), between the lacings, by Eq. 7.22:

$$\lambda_{element} = \lambda_e = \frac{L}{r_e}$$

where r_e = minimum radius of gyration of the element.

See that λ_e is less than 50 or 0.7 times slenderness ratio (λ) of the composite column.

Step 4. Compute the length (l) of each lacing between the inner end rivets, and hence determine effective length l_e of the lacing. For single lacing system with riveted ends, $l_e = l$ and for double lacing system riveted at ends, $l_e = 0.7l$. Similarly, for welded lacings, $l_e = 0.7\,l$.

Now $l = a \operatorname{cosec}\theta$ for both single lacing as well as double lacing.

Step 5. Choose rivet diameter, and for that fix minimum value of width b of the flat (The rivet diameter may be so chsoen that Eq. 7.23 is satisfied by using one rivet)

Step 6. Choose thickness t of lacing consisting of flats. For single lacing system, $t \nless \frac{l}{40}$, while for double lacing system, $t \nless \frac{l}{60}$. Determine maximum slenderness ratio of lacing, given by Eq. 7.21.

$$\lambda_{max.} = \frac{l_e}{r} = \frac{l_e\sqrt{12}}{t}, \text{ which should be less than 145.}$$

Step 7. For the above value of λ, find σ_{ac} from Table 7.5.

Step 8. Determine transverse shear $V = 0.025P$. Determine force F in each lacing from Eq. 7.19. Hence compute compressive and tensile stresses in the lacing. These should be less than the corresponding permissible values σ_{ac} and σ_{at}.

Step 9. Design the end connections for the lacing. For this, first determine the rivet value (R). Hence determine the number of rivets from Eq. 7.23. Generally, only one rivet is used at each end. Hence the rivet diameter should be so chosen that Eq. 7.23 is satisfied. That is

$$R > 2F\cos\theta \text{ (for single lacing of Fig 7.32 } b)$$

and

$$R > \frac{F}{2} \text{ (for double lacing).}$$

Example 7.17. *Design a built-up column composed of two channel sections placed back to back, carrying an axial load of 1345 kN. Effective length of column is 5.95 m. Take $f_y = 250\ N/mm^2$.*

Solution

For built-up column section, λ may be assumed between 30 to 50. Let us assume $\lambda = 40$, corresponding to which $\sigma_{ac} = 139\ \text{N/mm}^2$ (Table 7.5).

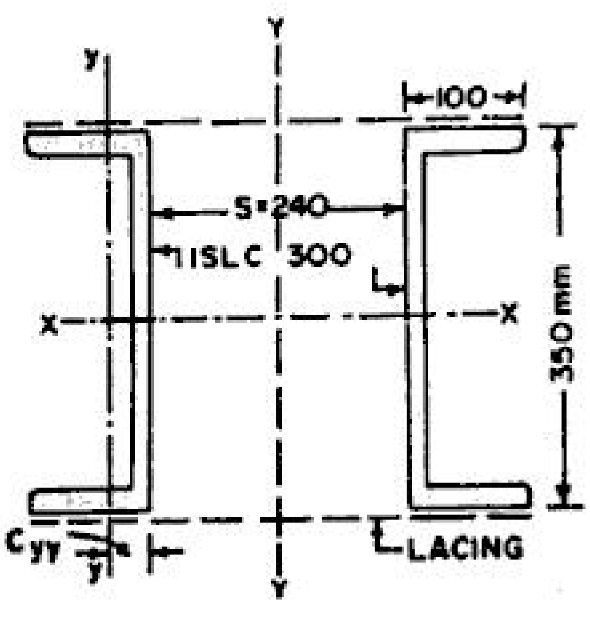


FIG. 7.34.

$$\therefore \quad A = \frac{P}{\sigma_{ac}} = \frac{1345 \times 10^3}{139} = 9676.2\ \text{mm}^2.$$

Also, $$r_{min} = \frac{l}{\lambda} = \frac{5950}{40} = 148.75\ \text{mm}$$

$$\lambda_e = 30.36 = 25.532 \cot\theta.$$

or $$\cot\theta = \frac{30.36}{25.532} = 1.189 \text{ or } \tan\theta = \frac{1}{1.189} = 0.841$$

which gives $$\theta \approx 40.06°$$

Hence the value of angle θ should not be less than the above value. However we shall keep $\theta = 45°$ (> 40.06°) for the sake of convenience in construction.

Now, for single lacing, $$l = \frac{L}{2}\sec\theta = \frac{L}{2}\sqrt{2} \text{ (Fig. 7.31 } a\text{)} = \frac{720}{2}\sqrt{2} \approx 509 \text{ mm}$$

$$\text{(or } l = a \operatorname{cosec}\theta = 360 \operatorname{cosec} 45° = 360\sqrt{2} \approx 509 \text{ mm)}$$

∴ Effective length of lacing, $l_e = l = 509$ mm

Minimum thickness of lacing, $t = \frac{l}{40} = \frac{509}{40} = 12.7$ mm

Hence keep $t = 14$ mm.

Using 20 mm diameter rivets, minimum width (b) of flat = 60 mm. Hence use 60 mm × 14 mm flats for lacing, connected by 20 mm dia. power driven shop rivets.

$$\text{Maximum slenderness ratio} = \frac{l_e\sqrt{12}}{t} \text{ (Eq. 7.21)} = \frac{509\sqrt{12}}{14} = 125.9 < 145.$$

Hence satisfactory. For this value of λ, the permissible compressive stress is given by (Table 7.5).

$$\sigma_{ac} = 64 - (64 - 57)\frac{5.9}{10} = 59.87 \text{ N/mm}^2$$

Now, $V = 0.025 \times 1345 = 33.625$ kN

For single system, the force in lacing bar is given by Eq. 7.19(a):

$$F = \frac{V}{n\sin\theta} = \frac{33.625}{2\sin 45°} = 23.776 \text{ kN}$$

Compressive stress in each lacing bar

$$= \frac{23.776 \times 1000}{60 \times 14} = 28.3 \text{ N/mm}^2 < \sigma_{ac}. \text{ Hence safe.}$$

Tensile stress in each lacing bar

$$= \frac{23.776 \times 1000}{(60 - 21.5)14} = 44.1 \text{ N/mm}^2 < 150. \text{ Hence safe.}$$

Design of end connections

Using 20 mm dia. power driven rivets, strength of each rivet

(i) In single shear : $P_s = \frac{100}{1000} \times \frac{\pi}{4}(21.5)^2 = 36.3$ kN

(ii) In bearing : $P_b = \frac{300}{1000} \times 21.5 \times 14 = 90.3$ kN

∴ Rivet value $= R = 36.3$ kN.

For the rivet system shown in Fig. 7.32(b)

$$\text{No. of rivets} = \frac{2F\cos\theta}{R} = \frac{2 \times 23.776 \cos 45°}{36.3} = 0.93 \approx 1$$

Hence the lacing will consist of 60 mm × 14 mm flats, connected by one power driven shop rivet of 20 mm dia. at each end. The arrangement is shown in Fig.. 7.35.

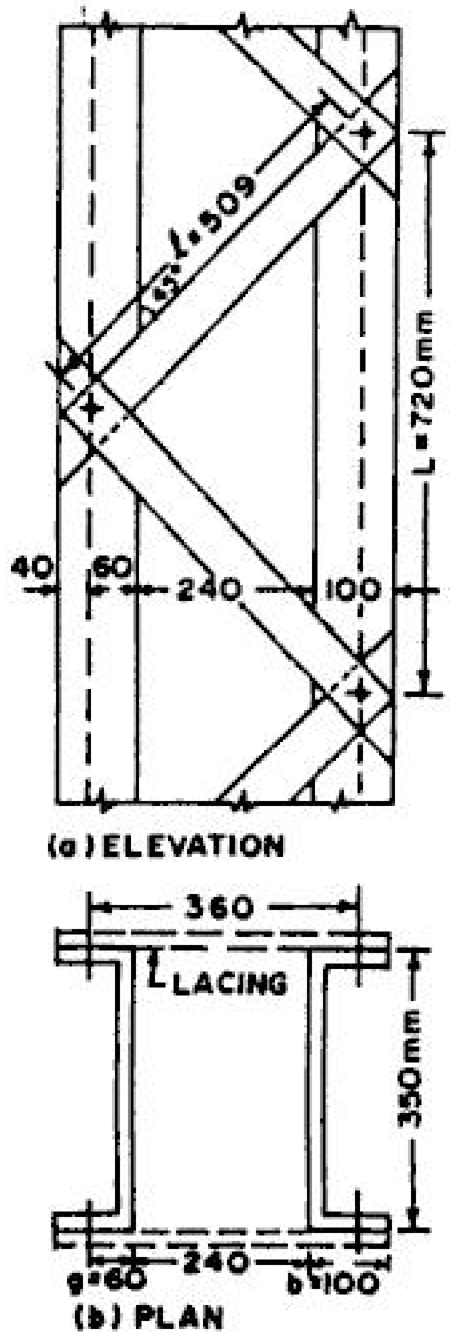


FIG. 7.35.

Example 7.19. *Design double lacing system for the composite column of example 7.17.*

Solution: Let θ be the angle of inclination of the lacings with vertical. This should be as small as possible to take full advantage of double lacing. Smallest permissible value of θ is 40°. For ISLC 350, distance of rivet lines from the channel face $= g = 60$ mm.

Thus $$a = 60 + 240 + 60 = 360 \text{ mm.}$$

$\therefore$ $$L = a \cot\theta \text{ (for double lacing system)} = 360 \cot\theta.$$

$$\lambda_e = \frac{L}{r_e} \text{ where } r_e = r_{min} \text{ for each component} = r_{yy} = 28.2 \text{ mm}$$

$\therefore$ $$\lambda_e = \frac{360 \cot\theta}{28.2} = 12.766 \cot\theta \quad ...(1)$$

As per code requirements, λ_e should be equal to or less than 0.7λ ($= 0.7 \times 43.37 = 30.36$) or 50 whichever is less.

$\therefore$ $$\lambda_e = 30.36 \quad ...(2)$$

Equating (1) and (2), we get

$$12.766 \cot\theta = 30.36$$

$$\tan\theta = \frac{12.766}{30.36} = 0.4205$$

From which $$\theta = 22.8°.$$

However, minimum permissible value of $\theta = 40°$

Hence keep $$\theta = 40°$$

Accordingly, $$L = a \cot\theta = 360 \cot 40° = 429 \text{ mm}$$

$$\lambda_e = \frac{L}{r_e} = \frac{429}{28.2} = 15.2 \quad \text{(which is less than } 0.7\lambda = 30.36\text{).}$$

Again, for double lacing system,

$$l = L \sec\theta \text{ (or} = a \operatorname{cosec}\theta) = 429 \sec 40° \approx 560 \text{ mm.}$$

$$\text{(Check : } l = \sqrt{a^2 + L^2} = \sqrt{(360)^2 + (429)^2} = 560 \text{ mm.)}$$

Also, for double lacing, $$l_e = 0.7l = 0.7 \times 560 = 392 \text{ mm}$$

For double lacing system, $$t = \frac{l}{60} = \frac{560}{60} = 9.33 \text{ mm}$$

Adopt $$t = 10 \text{ mm.}$$

$$\text{Maximum slenderness ratio} = \frac{l_e\sqrt{12}}{t} = \frac{392\sqrt{12}}{10} \approx 136 < 145.$$

For this value of λ, the permissible compressive stress is given by (Table 7.5).

$$\sigma_{ac} = 57 - (57 - 51)\frac{6}{10} = 53.4 \text{ N/mm}^2.$$

Now, $$V = 0.025 \times 1345 = 33.625 \text{ kN}$$

Force in lacing bar, for double lacing system is given by Eq. 7.19 (*b*).

$$F = \frac{V}{2n\sin\theta} = \frac{33.625}{2 \times 2 \sin 40°} = 13.078 \text{ kN.}$$

Using one rivet at each end, we get from Eq. 7.23(*b*)

$$n = 1 = \frac{2F\cos\theta}{R}$$

(b) Check for local buckling of column angles : The arrangement for single laced system is shown in Fig 7.39. Refer Fig. 7.31 also. Keeping the distance of rivet lines from the angle face= 60 mm, we have

$$a = 350-(60+60)=230 \text{ mm.}$$

Hence spacing (L) of lacing bars is given by

$$L = 2a \cot \theta = 2 \times 230 \cot 45° = 460 \text{ mm}$$

r_{min} for each component=19.4 mm

Hence for the component of the column,

$$\lambda_e = \frac{L}{r_{min}} = \frac{460}{19.4} = 23.7$$

As per code requirement, λ_e should be equal to or less than 0.7λ ($= 0.7 \times 53.7 = 37.6$) or 50 whichever is less.

Hence the column angles are safe against local buckling.

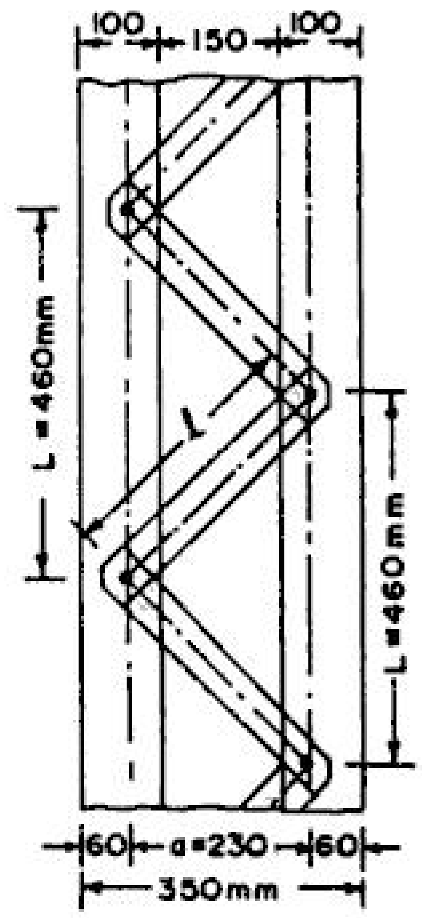


FIG. 7.39.

(c) Check for local buckling of lacing

$$l = \frac{L}{2}\sqrt{2} = \frac{460}{2}\sqrt{2} = 325.3 \text{ mm}$$

∴ Effective length $l_e = l = 325.3$ mm (for single lacing system.)

$$r_{min} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.89 \text{ mm}$$

$$\therefore \quad \lambda = \frac{l}{r_{min}} = \frac{325.3}{2.89} = 112.7 < 145 \text{ Hence safe.}$$

(d) Strength of lacing bars in tension and compression

Total transverse shear, $V = 0.025\,P = 0.025 \times 1159.3 \approx 28.98$ kN

Hence, for single lacing system, axial force in each lacing is

$$F = \frac{V}{n \sin\theta} = \frac{V}{2 \sin\theta} = \frac{28.98}{2 \sin 45°} = 20.49 \text{ kN.}$$

This force will be compressive in one lacing and tensile in the next lacing.

(i) Strength of flat in tension

Rivet diameter = 20 mm

Gross diameter = 20 + 1.5 = 21.5 mm mm.

∴ Net area of lacing = $(60 - 21.5) \times 10 = 385 \text{ mm}^2$

Hence tensile stress in each lacing bar $= \frac{F}{A_n} = \frac{20.49}{385} \times 1000 = 53.22 \text{ N/mm}^2$.

Permissible tensile stress, $\sigma_{at} = 0.6 f_y = 0.6 \times 250 = 150 \text{ N/mm}^2$

Hence safe

(ii) Strength of flat in compression

For the flat, $\lambda = \frac{l}{r_{min}} = 112.7$

Hence, from Table 7.5, for $f_y = 250 \text{ N/mm}^2$ and $\lambda = 112.7$,

$$\sigma_{ac} = 72 - (72 - 64) \times \frac{2.7}{10} = 69.84 \text{ N/mm}^2$$

Fig. 7.42 (*a*) shows distorted shape of a battend column. The points of contraflexure will be midway between batten plates. Fig 7.42 (*b*) shows the free body diagram. The transverse shear force to each side of point of contraflexure will be $\frac{1}{2}\frac{V}{N}$, where N is the number of parallel planes of battens. If C is the centre to centre spacing of battens, the bending moments arising at each end of battens plate, due to this transverse shear will be

$$M = M_1 = M_2$$

$$= \left(\frac{1}{2}\frac{V}{N}\right) \times C = \frac{VC}{2N} \quad ...(7.27)$$

The longitudinal shear induced due to these moments is

$$V_1 = \frac{1}{S}(M_1 + M_2) = \frac{VC}{NS} \quad ...(7,28)$$

where S = minimum transverse distance between the centroids of the rivet group/welding.

Battens shall be of plates, angles channels or I-sections and shall be riveted or welded to the main components so as to resist simultaneously a longitudinal shear $V_1 = \frac{VC}{NS}$ and moment $M = \frac{VC}{2N}$.

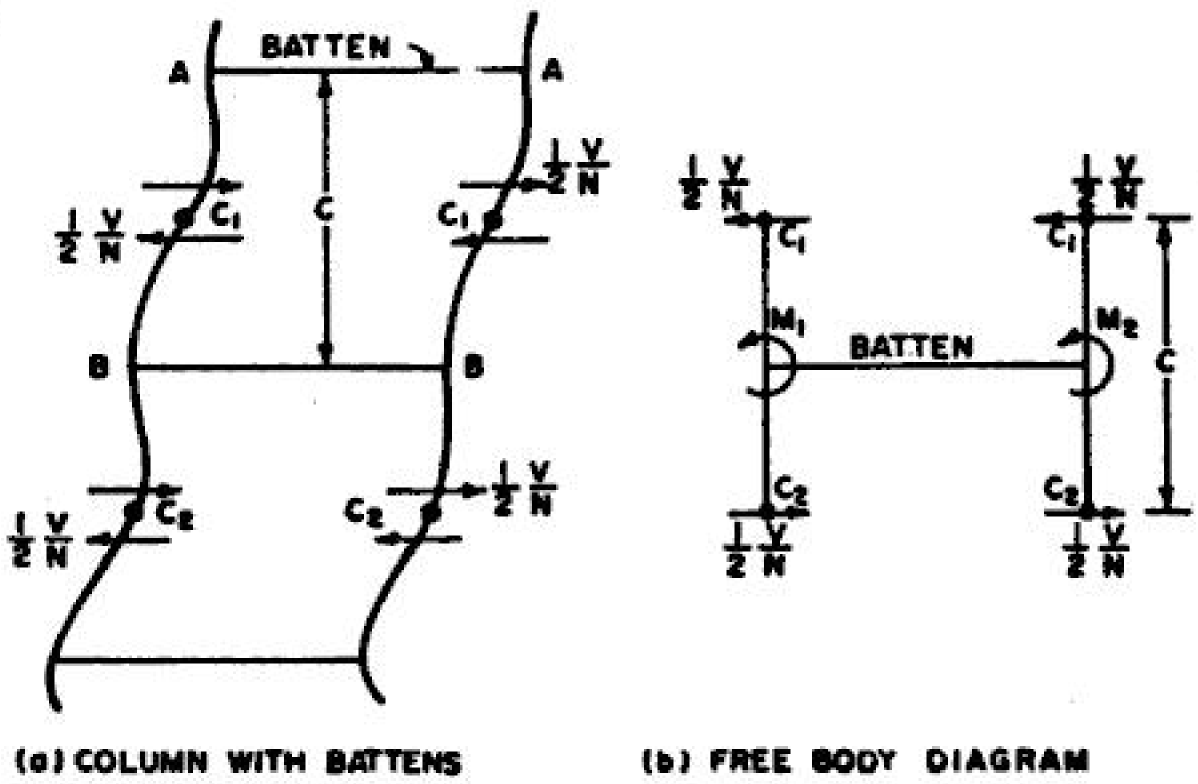


FIG. 7.42. MOMENTS AND SHEARS IN BATTEN PLATE

6. The longitudinal shear stress in the batten will be equal to $\frac{V_1}{D.t}$ where D is the overall depth of the batten plate and t is the thickness.

Hence $$\frac{V_1}{D.T} \not> \tau_{va} \quad ...(7.29)$$

where τ_{va} = permissible average shear stress = 100 N/mm^2 for steel conforming to IS : 226–1975

7. The bending stress in the batten is

$$\frac{M}{Z} = \frac{M}{\frac{1}{6}D^2 t} \not> \sigma_{bc} \text{ or } \sigma_{bt} \quad ...(7.30)$$

where σ_{bc} or σ_{bt} = permissible bending stress = 165 N/mm^2 for steel conforming to IS : 226-1975.

Example 7.23. *Design the battens of built-up column of Example 7.20.*

Solution

1 ***Spacing of battens*** : The spacing C of battens is given by

$$\frac{C}{r_{c,min}} \not> 50 \text{ or } 0.7 \text{ times } \frac{l}{r} \text{ of composite column.}$$

Here, $r_{c,min}$ = minimum radius of gyration of component = 53.4 mm

$$\text{Strength of rivet in bearing} = 21.5 \times 8 \times 300 \times 10^{-3} = 51.6 \text{ kN}$$

$\therefore$ Rivet value = 36.3 kN

$$\text{Shear force in each rivet} = \frac{V_1}{6} = \frac{103.75}{6} = 17.292 \text{ kN}$$

$$\text{Force due to moment, in end rivets} = \frac{M \cdot r}{\Sigma r^2}$$

where $r = [(70)^2 + (225)^2]^{\frac{1}{2}} = 235.64$ mm

$$\Sigma r^2 = 2(70)^2 + 4\,(235.64)^2 = 231900$$

$\therefore$
$$\text{Force due to moment} = \frac{15562.5 \times 235.64}{231900} = 15.813 \text{ kN}$$

$$\cos\theta = \frac{70}{235.64} \approx 0.2971$$

$\therefore$
$$\text{Resultant force on end rivet} = [(17.272)^2 + (15.813)^2 + 2 \times 17.272 \times 15.813 \times 0.2971]^{\frac{1}{2}}$$
$$= 26.66 \text{ kN} < \text{Rivet value. Hence safe.}$$

Example 7.24. *Design a built-up column composed of two channel sections, placed back to back, carrying an axial load of 1345 kN. The column, having a length of 7.5 m, is effectively held in position at one end, but restrained against rotation at one end only. Design the batten plates also. Take $f_y = 250$ N/mm².*

Solution

The problem is similar to the one solved in Example 7.17 except that battens are used in place of lacings.

(*a*) **Design of column**

For battened column, effective length is increased by 10%.

$\therefore$ Effective length $l = 1.1(0.8L)$.

$= 1.1 \times 0.8 \times 7.5 = 6.6$ m $= 6600$ mm

For built-up column section composed of channels, λ may be assumed between 30 to 50. Let us assumed $\lambda = 40$ corresponding to which $\sigma_{ac} = 139$ N/mm².

$\therefore$
$$A = \frac{P}{\sigma_{ac}} = \frac{1345 \times 10^3}{139} = 9676.2 \text{ mm}^2.$$

Also,
$$r_{min} = \frac{l}{\lambda} = \frac{6600}{40} = 165 \text{ mm.}$$

Let us choose 2 ISLC 400 @ 45.7 kg/m each, having the following properties.

$$a = 5825 \text{ mm}^2$$
$$r_{xx} = 15.50 \text{ cm} = 155.0 \text{ mm}$$
$$r_{yy} = 2.81 \text{ cm} = 28.1 \text{ mm}$$
$$C_{yy} = 2.36 \text{ cm} = 23.6 \text{ mm}$$

Total $A = 2a = 2 \times 5825 = 11650$ mm²

The channels are placed back to back, at a spacing S, given by Eq. 7.17 (*a*) :

$$S \approx 2\,[r_{xx} - C_{yy}] = 2\,[155 - 23.6] = 262.8 \text{ mm}$$

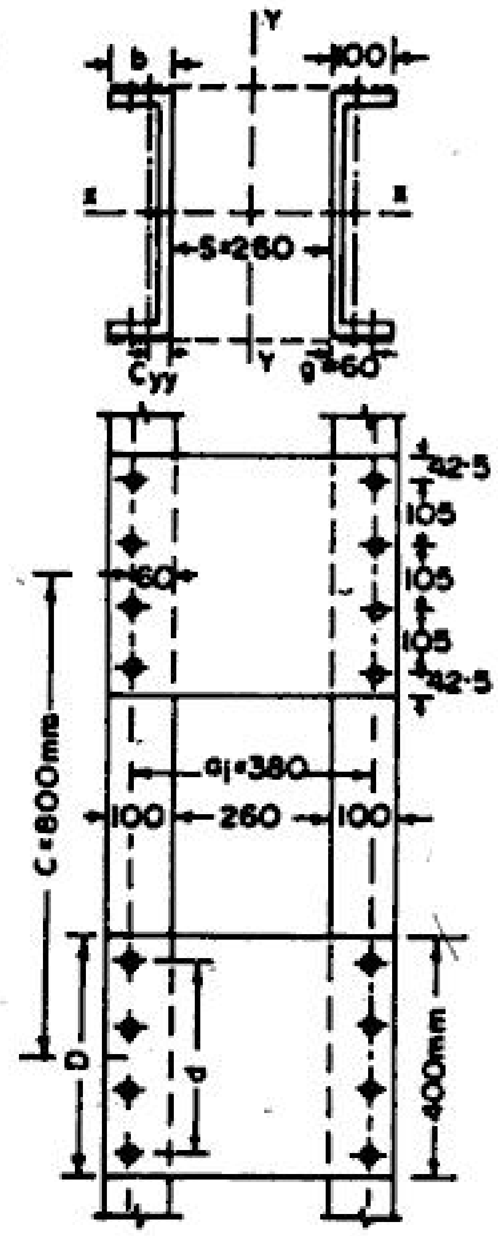


FIG. 7.44.

It should be noted that Eq. 7.17 has been derived on the premise that the column has equal values of radii of gyration in both the directions. ISI Handbook No. 1 [SP: 6(1)-1964] gives properties of such a composite column, consisting of two channel sections, in Table XXXVII, for various values of spacing S. From this table, we observe that when $S = 260$ mm, $r_Y = 15.62$ cm which is slightly more than r_{xx}. Hence adopt $S = 260$ mm, for which $r_Y = 156.2$ mm for the composite section. Hence $r_{min} = r_X = r_{xx} = 155.0$ mm.

$$\therefore \quad \lambda = \frac{6600}{155.0} = 42.58$$

From Table 7.5, for $f_y = 250$ and $\lambda = 42.58$, get

$$\sigma_{ac} = 139 - (139 - 132)\frac{2.58}{10} = 137.19 \text{ N/mm}^2$$

$$\therefore \quad P = A.\sigma_{ac} = 11650 \times 137.19 \times 10^{-3} = 1598.3 \text{ kN} > 1345$$

Hence satisfactory.

Note. The column has about 11% more capacity. The students may redesign the section with 2 ISLC 350 @ 38.8 kg/m, with $S = 240$ mm (as was done in example 7.17). However, since effective length l is more in this case, λ will come out to be more and hence the section will be have carrying capacity of 1319 kN only, against a required value of 1345 kN.

(*b*) **Design of battens**

(*i*) ***Spacing of battens***

The spacing C of the battens is given by

$$\frac{C}{r_{c,min}} \not> 50 \text{ or } 0.7 \text{ times } \frac{l}{r} \text{ of composite section.}$$

$$0.7 \times \frac{l}{r} \text{ of composite section} = 0.7 \times 42.58 = 29.81$$

Hence $\quad \dfrac{C}{r_{c,min}} < 29.81$

or $\quad C < 29.81 \times r_{c,min}$

Here $\quad r_{c,min} = r_{yy} = 28.1$

$\therefore \quad C < 29.81 \times 28.1 < 837.5$

Keep $\quad C = 800$ mm.

(*ii*) ***Thickness of batten***

$$t > \frac{a_i}{50} \quad \text{(Refer Fig. 7.41)}$$

From Table II of handbook, $g = 60$ mm, for this section. Hence $a_i = 260 + 2(60) = 380$ mm.

$$\therefore \quad t > \frac{380}{50} > 7.6 \text{ mm.}$$

Keep $\quad t = 8$ mm

(*iii*) ***Depth of batten*** (Fig 7.41) :

$$a = 260 + 2\,(23.6) = 307.2 \text{ mm}; \; b = 100 \text{ mm}$$

For intermediate battens, $d > \dfrac{3}{4}a = \dfrac{3}{4} \times 307.2 = 230.4$ mm

For end battens, $d > a = 307.2$ mm

For any batten, $d > 2b = 2 \times 100 = 200$ mm

Hence effective depth $= 307.2$ mm

Using 20 mm dia. rivets with an edge distance of 40 mm, total depth D of the batten is

$$D = 307.2 + 2\,(40) = 387.2 \text{ mm.}$$

Hence Try 400 mm × 8 mm battens at spacing $(C) = 800$ mm.

(iv) Check for shearing and bending stresses.

$$\text{Transverse shear } V = 0.025P = 0.025 \times 1345 = 33.625 \text{ kN}$$

Here $S = a_i = 380$

$$\therefore \quad \text{Longitudinal shear } V_1 = \frac{VC}{NS} = \frac{33.625 \times 800}{2 \times 380} = 35.395 \text{ kN}$$

$$\text{Moment } M = \frac{VC}{2N} = \frac{(33.625 \times 1000) \times 800}{2 \times 2} = 6.725 \times 10^6 \text{ N-mm}$$

$$\therefore \quad \text{Longitudinal shear stress} = \frac{V_1}{D \times t} = \frac{35.395 \times 10^3}{400 \times 8} = 11.06 \text{ N/mm}^2 < 100$$

Hence safe.

$$\text{Bending stress} = \frac{M}{\frac{1}{6}D^2 . t} = \frac{6.725 \times 10^6}{\frac{1}{6} \times 400^2\,(8)} = 31.5 \text{ N/mm}^2 < 165 \text{ N/mm}^2.$$

Hence safe.

(v) Design of connections

Try 20 mm dia. rivets.

$$\text{Strength of rivet in single shear} = \frac{\pi}{4}(21.5)^2 \times 100 \times 10^{-3} = 36.3 \text{ kN}$$

$$\text{Strength of rivet in bearing} = 21.5 \times 8 \times 300 \times 10^{-3} = 51.6 \text{ kN}$$

$$\therefore \quad \text{Rivet value} = 36.3 \text{ kN}$$

Assume 4 rivets in a vertical row, @ 105 mm centre to centre with an edge distance of 42.5 mm.

$$\text{Shear force in each rivet} = \frac{V_1}{4} = \frac{35.395}{4} = 8.849 \text{ kN}$$

$$\text{Force due to moment on end rivets} = \frac{M . r}{\Sigma r^2} = \frac{6.725 \times 10^3 \times 157.5}{2\,[(52.5)^2 + (157.5)^2]} = 19.214 \text{ N}$$

$$\therefore \quad \text{Resultant force in the rivet} = \sqrt{(8.849)^2 + (19.214)^2} = 21.15 \text{ kN} < \text{Rivet value.}$$

Hence safe.

7.16. PERFORATED COVER PLATES

Perforated cover plates are mostly used for box sections consisting of four angle sections (Fig. 7.45), so that the interior of column remains accessible for painting and inspection. The cover plates consist of continuous plates, running throughout the height of the column, with width (B) equal to width of the composite section and with perforations at a regular interval. The perforations are done axially, in elliptical shape. The cover plates have three advantages:

(*i*) They add to the sectional area of column and the portions beyond the perforations share axial load to the extent of their effective area.

1(*a*) Where the ends of compression members are faced for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist any tension when bending is present.

(*b*) The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

2. Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which they are subjected.

3. Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members jointed in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress shall be provided for.

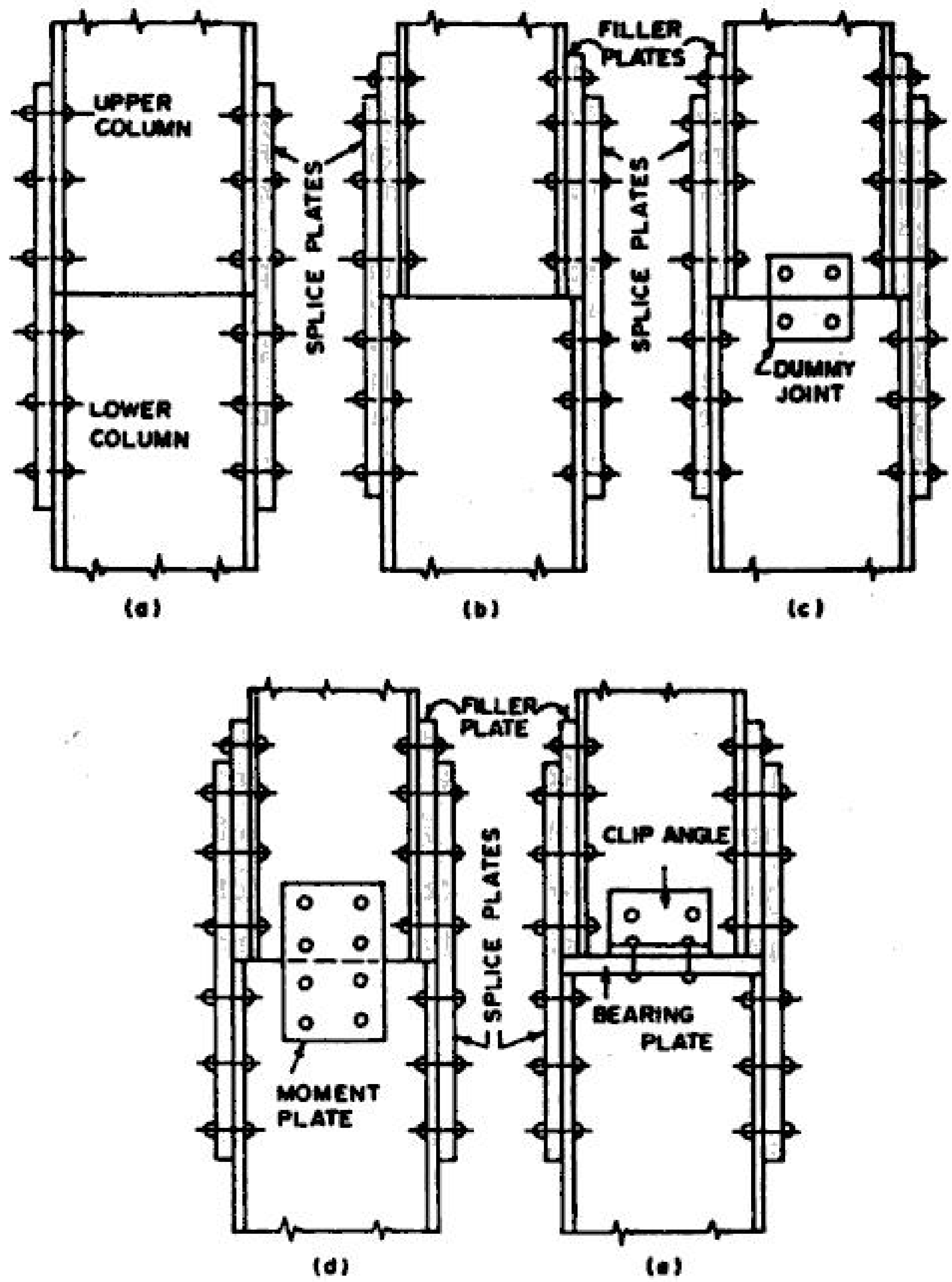


FIG. 7.46. RIVETED COLUMN SPLICES

Design steps

1. Assume width of splice plate equal to the width of the flange.

2. When the ends of the columns are not faced, the splices take the full axial load of the column. If the ends are perfectly faced for bearing over the whole area, splices simply hold the two columns in position. However, in this case, it is usual practice to design the splices to transmit 50% of the axial load of the column.

3. Assume column splice to act as a *short column*, having zero slenderness ratio, and adopt σ_{ac} (for $\lambda = 0$), for the type of steel used. Hence find the thickness of splice plate.

4 Assume suitable rivet diameter, compute the rivet value and determine the number of rivets required to transmit the axial load.

5. Where upper column does not have full bearing over the lower column, provide a bearing plate of suitable thickness *t*. In such a case, it is assumed that the axial load of the column is taken by flanges alone, and the load taken by web is neglected. This assumption

gives rise to a moment $M = \frac{P}{2} \times a$. If b is the width of bearing plate (equal to the width of the flanges),

$$z = \frac{1}{6} b t^2$$

and moment of resistance of the bearing plate $= \frac{1}{6} b\, t^2 . \sigma_{bc}$

$$\therefore \quad M = \frac{1}{6} b\, t^2 . \sigma_{bc}$$

or

$$t = \sqrt{\frac{6M}{b\, \sigma_{bc}}} \quad ...(7.31)$$

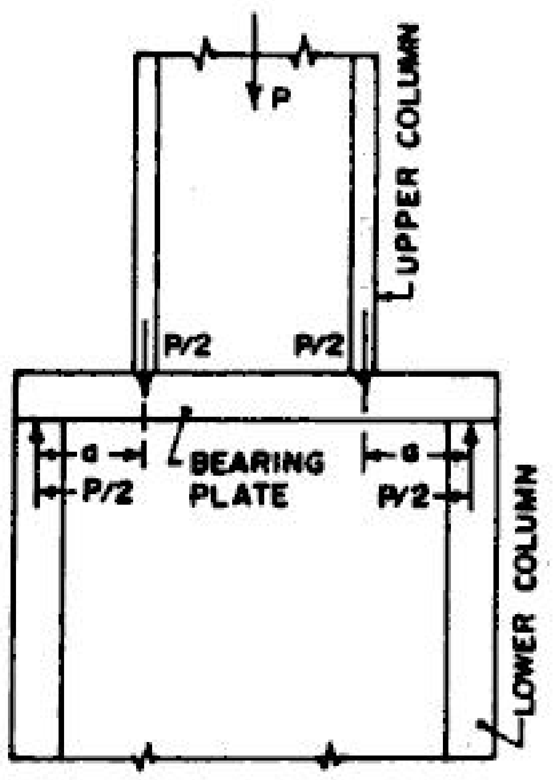


FIG. 7.47.

6. When the column is also subjected to bending moment and shear force, in addition to the axial load, the splice plates attached to the flanges are designed for axial load and bending moment, while the splice plates attached to web are designed to resist shear force. The web splice plates are also known as *shear plates*.

Example 7.25. *A column consisting of ISHB 300 @ 63 kg/m carries an axial load of 500 kN. Design the splices, assuming that (a) ends of the columns are cut by ordinary method and are not milled, and (b) ends of the columns are milled and faced for bearing. Take $f_y = 250\ N/mm^2$.*

Solution : For the splices, taking $\lambda = 0$, $\sigma_{ac} = 0.6 f_y = 0.6 \times 250 = 150\ \text{N/mm}^2$.

(a) When the ends are not milled

The splices transmit the full axial load of 500 kN. Hence the load transmitted by each *flange splice* $= 0.5 \times 500 = 250$ kN.

$\therefore$ Area of flange splice plate

$$= A = \frac{250 \times 1000}{150} = 1666.7\ \text{mm}^2$$

For ISHB 300 @ 63 kg/m, $h = 300$ mm, $b = 250$ mm, $t_f = 19.6$ mm

Hence thickness of flange splice plate $= \frac{1666.7}{250} = 6.67$ mm

Provide $t = 8$ mm.

Let us use 22 mm dia. power driven rivets.

Strength of rivet in single shear

$$= 100 \times \frac{\pi}{4} (23.5)^2 \times 10^{-3}$$

$$= 43.37\ \text{kN}.$$

Strength of rivet in bearing

$$= 300 \times (8 \times 23.5) \times 10^{-3}$$

$$= 56.4\ \text{kN}$$

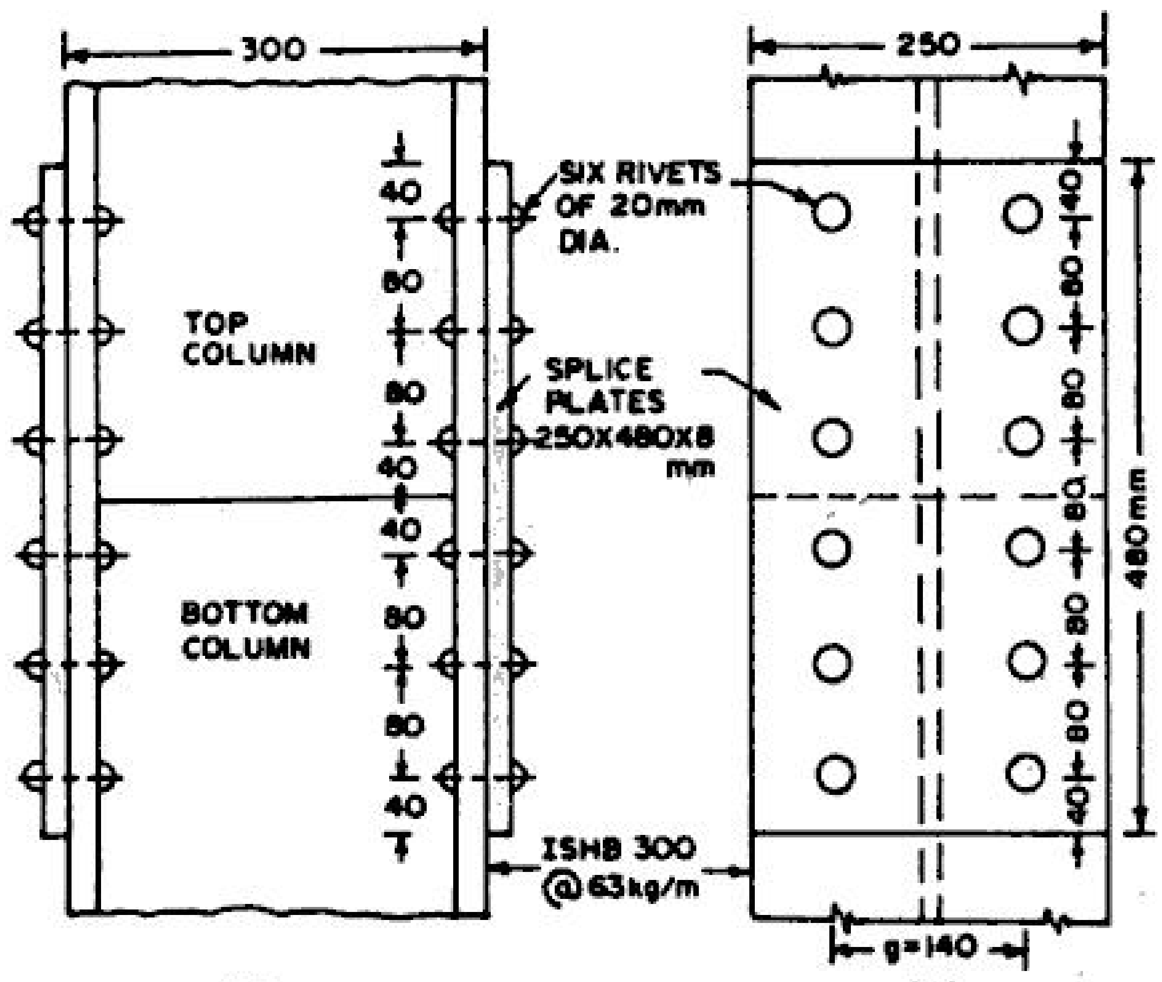


FIG. 7.48.

$\therefore$ Rivets value $= 43.37$ kN

$\therefore$ No. of rivets required $= \dfrac{250}{43.37} = 5.76$

Use 6 rivets in two lines of 3 each on either side of the joint. Keeping 80 mm pitch, with 40 mm edge distance, the length of splice plate will be $= 5 \times 80 + 2 \times 40 = 480$ mm. Distance between two rows of rivets $= g = 140$ mm.

The arrangement of rivets etc. are shown in Fig. 7.48.

(b) Ends of the columns machined and milled.

In this case, it may be *assumed* that half the load is transmitted through the splices, while other half is transmitted by bearing, though as per IS code whole of load is transmitted through bearing and the splices simply hold the two portions in position.

$\therefore$ Load on splices

$= 0.5 \times 500 = 250$ kN

$\therefore$ Load on each splice plate

$= \frac{1}{2} \times 250 = 125$ kN.

Area of splice plate

$= \dfrac{125 \times 1000}{150} = 833.35 \text{ mm}^2$

$\therefore$ Thickness of plate

$= \dfrac{833.35}{250} = 3.33$ mm

This is too small. Provide a minimum thickness of 6 mm.

Let us use 20 mm dia. power driven rivets.

Strength of rivets in single shear $= 100 \times \frac{\pi}{4}(21.5)^2 \times 10^{-3}$

$= 36.3$ kN

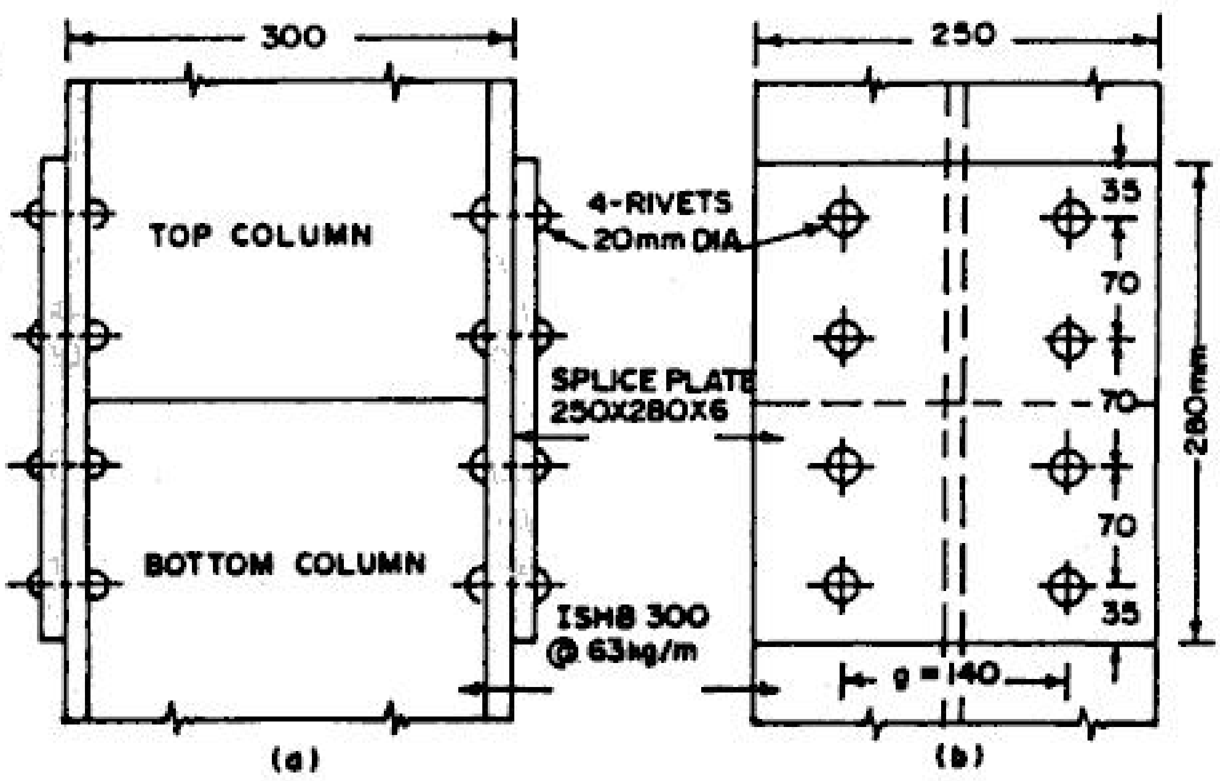


FIG. 7.49

Strength of rivets in bearing $= 300 \times (21.5 \times 6) \times 10^{-3} = 38.7$ kN

$\therefore$ Rivet value $= 36.3$ kN.

$\therefore$ No. of rivets required $= \dfrac{125}{36.3} = 3.44$

Use 4 rivets in two lines of 2 each on either side of joint. Keeping 70 mm pitch and 35 mm edge distance, length of splice plate $= (3 \times 70) + (2 \times 35) = 280$ mm. The arrangement is shown in Fig. 7.49.

Example 7.26. *Redesign the splices of column of Example 7.25, if the column is also subjected to a moment of 18 kN-m, in addition to the axial load of 500 kN. Assume the ends of the column to be milled.*

Solution

As the ends of the columns are milled for complete bearing, let us assume that 50% of axial load is transmitted through splices, and remaining 50% through direct bearing. However, the splices will transmit full moment, due to which, they will be subjected to additional axial load. Since there is no shear force, only flange splices will be used.

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The *section modulus* represents the strength of the section. Greater the value of Z, stronger will be the section. *The strength of the section does not therefore, depend on the sectional area but depends on the disposition of this area with respect to the centroidal axis.*

It should be noted that the total compressive force C, above the N.A., is equal to the total tensile force T below the N.A., for the beam to be in equilibrium. There two forces acting in opposite directions, as marked in Fig. 8.2 (*b*), and form a couple $= C(\text{or } T) \times$ lever arm. This couple, also known as the *moment of resistance* (M_r), resists the external bending moment.

Maximum permissible bending stress

For *laterally supported beams*, the permissible bending stress in tension (σ_{bt}) or in compression (σ_{bc}) are given by

$$\sigma_{bt} \text{ or } \sigma_{bc} = 0.66 f_y \qquad ...(8.5)$$

For *laterally unsupported beams*, σ_{bt} is taken equal to 0.66 f_y, but σ_{bc} is given by :

$$\sigma_{bc} = 0.66 \frac{f_{cb} \cdot f_y}{\left[(f_{cb})^n + (f_y)^n \right]^{1/n}} \qquad ...(8.6)$$

where n is assumed as 1.4. In the above expression, f_{cb} is the *elastic critical stress*, and the method of its computation is discussed in § 8.10. The values of σ_{bc} as derived from Eq. 8.6, for some Indian standard structrual steels are given in Table 8.4.

8.4. SHEAR STRESS

When a beam is loaded transversely, it is subjected to both *bending moment* as well as *shearing force.* For a simply-supported beam, with uniformly distributed load, maximum B.M. usually occurs at mid-span, while maximum shear force is induced at the supports. In general, every section of the beam is subjected to both B.M. (M) as well as shear force (V). The shear force causes *shearing stress* at the section, the magnitude of which varies across the depth of the beam, at that section.

On any layer, at height y from N.A., the intensity of transverse shear stress (τ_v) is given by :

$$\tau_v = \frac{V}{Iz} (A \bar{y}) \qquad ...(8.7\,a)$$

where

V = Transverse S.F. at the section.

I = Moment of inertia of the section, about the bending axis.

z = Width of the section at which τ_v is being computed.

$A\bar{y}$ = First moment of the outer area, above the point where τ_v is being computed, about the N.A.

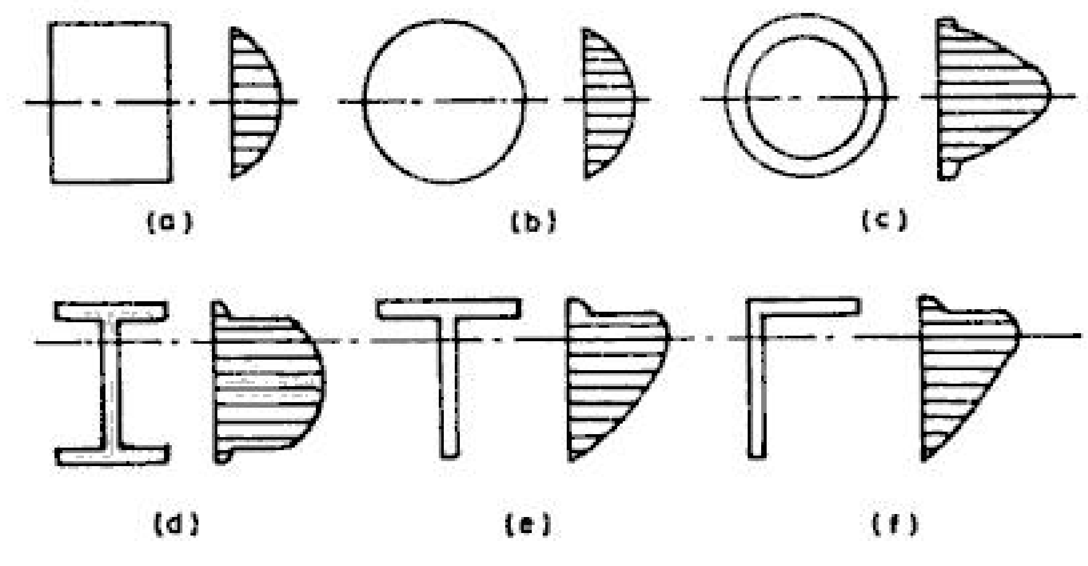


FIG. 8.3.

It is to be noted that τ_v does not vary uniformly across the depth of the section. Fig. 8.3 shows the shear stress distribution for some typical sections. For a rectangular section, the maximum shearing stress occurs at the N.A., and its magnitude is given by

If a symmetrical I-beam is subjected to couples M at the ends (Fig 8.7), and the ends are restrained against torsion, lateral buckling will take place when M attains a *critical* value M_{cr}. The value of critical bending moment M_{cr} depends on the properties of material, shape and dimensions of beam, effective length of compression flange, support conditions and type of loading. The corresponding *elastic critical stress*(f_{cb}) is equal to M_{cr}/Z_{xx}. The method of computing f_{cb} is given in § 8.10(*b*) and the method of computing corresponding permissible compressive stress σ_{bc} is given in § 8.10 (*c*).

(*a*) Effective length of compression flange

The recommendations of IS : 800-1984 for computing the effective length of compression flanges are as follows :

1. For simply supported beams and girders, where no lateral restraint for the compression flanges is provided, but where each end of the beam is restrained against rotation, the effective length l of the compression flanges, to be used in Table 8.5, shall be taken as follows :

(a)	With ends of compression flanges unrestrained against lateral bending (that is, free to rotate in plan at the bearings)	l = span
(b)	With ends of compression flanges partially restrained against lateral bending (that is, not free to rotate in plan at the bearings)	$l = 0.85 \times$ span.
(c)	With ends of compression flanges fully restrained against lateral bending (that is, not free to rotate in plan at the bearings)	$l = 0.7 \times$span.

Restraint against torsion can be provided by :

(*i*) Web or flange cleats, or

(*ii*) bearing stiffeners acting in conjunction with the bearing of the beam, or

(*iii*) Lateral end frames or other external supports to the ends of the compression flanges,

or (*iv*) their being built into walls.

Where the ends of the beams are not restrained against torsion, or where the load is applied to the compression flange and both the load and flange are free to move laterally, the above values of the effective length shall be increased by 20 per cent.

Note: The end restraint element shall be capable of safely resisting in addition to wind and other applied external forces, a horizontal force acting at the bearing in a direction normal to the compression flange of the beam at the level of the centroid of the flange and having a value equal to not less than 2.5 percent of the maximum force occurring in the flange.

2. For beams which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end restraint required in para 1 above, the effective length of the compression flange shall be taken as the maximum distance, centre to centre, of the restraint members.

3. For cantilever beams of projecting length L, the effective length l to be used in Table 8.5 shall be taken as follows :

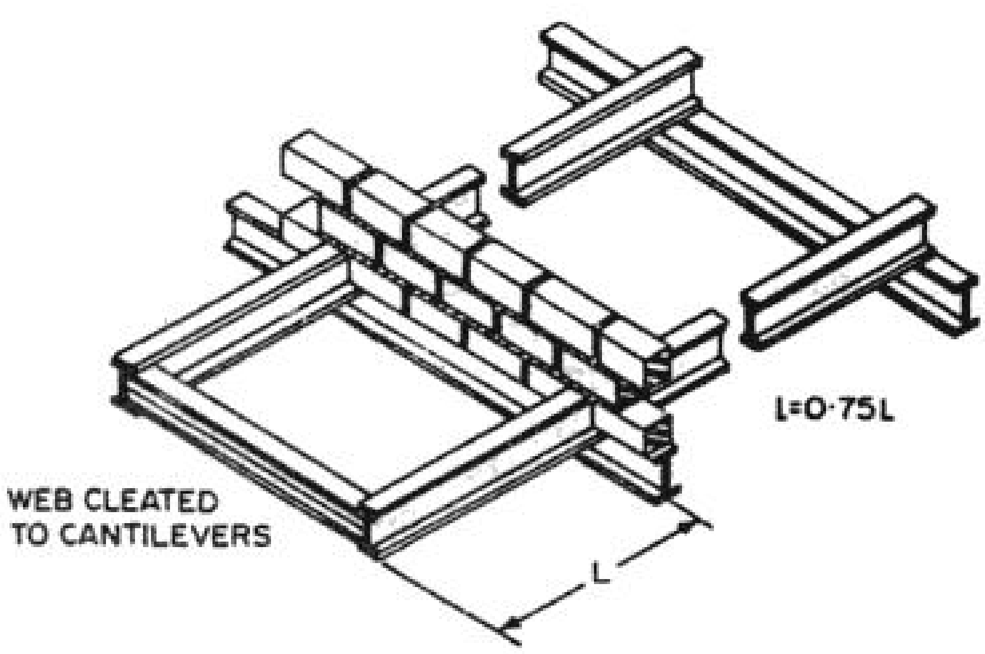


FIG. 8.8. CANTILEVER BUILT-IN AT SUPPORT, RESTRAINED AGAINST TORSION AT THE END

$$\sigma_{bc} = 0.66 \frac{f_{cb} \cdot f_y}{\left[(f_{cb})^n + (f_y)^n \right]^{1/n}} \qquad ...(8.6)$$

where n is assumed as 1.4. The values of σ_{bc} as derived from the above formula for some of Indian Standard Structural Steels are given in Table 8.4.

TABLE 8.4 : VALUES OF σ_{bc} CALCULATED FROM f_{cb} FOR DIFFERENT VALUES OF f_y (N/mm² or MPa)

f_y → / f_{cb} ↓	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
20	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13
30	19	19	19	19	19	19	19	19	19	19	19	19	19	19	20	20	20
40	25	25	25	25	25	25	25	25	26	26	26	26	26	26	26	26	26
50	30	30	31	31	31	31	31	31	31	32	32	32	32	32	32	32	32
60	36	36	36	36	36	37	37	37	37	37	38	38	38	38	38	38	38
70	41	41	41	41	42	42	42	43	43	43	43	44	44	44	44	44	44
80	45	46	46	46	47	47	48	48	48	49	49	49	49	50	50	50	50
90	50	50	51	51	51	52	53	53	54	54	54	55	55	55	56	56	56
100	54	54	55	55	56	57	57	58	59	59	60	60	60	61	61	62	62
110	58	58	59	60	60	61	62	63	64	64	65	65	66	66	67	67	67
120	61	62	63	64	64	65	67	67	68	69	70	70	71	71	72	72	73
130	65	66	67	67	68	70	71	72	73	74	74	75	76	76	77	78	78
140	68	69	70	71	72	73	75	76	77	78	79	80	80	81	82	83	84
150	71	72	73	74	75	77	79	80	81	82	83	84	85	86	87	88	89
160	74	75	77	78	79	81	82	84	85	87	88	89	90	91	92	93	94
170	77	78	80	81	82	84	86	88	89	91	92	93	94	95	97	98	99
180	79	81	82	84	85	87	89	91	93	94	96	97	98	100	101	102	103
190	82	84	85	87	88	90	93	95	97	98	100	102	102	104	106	107	108
200	84	86	88	89	91	93	96	98	100	102	103	115	106	108	110	111	113
210	86	88	90	92	93	96	99	101	103	105	107	109	110	112	114	116	117
220	89	90	92	94	96	99	102	104	106	109	111	112	114	116	118	120	121
230	90	93	94	96	98	101	104	107	110	112	114	116	118	120	122	124	126
240	92	94	97	99	100	104	107	110	113	115	117	119	121	124	126	128	130
250	94	96	99	101	103	106	110	113	115	118	120	122	124	127	130	132	134
260	96	98	100	103	105	108	112	115	118	121	123	126	128	131	133	136	138
270	97	100	102	104	107	111	114	118	121	124	126	129	131	134	137	139	142
280	99	101	104	106	108	113	116	120	123	126	129	132	134	137	140	143	145
290	100	103	165	108	110	115	119	122	126	129	132	135	137	141	144	147	149
300	102	104	107	110	112	116	121	125	128	131	135	137	140	144	147	150	153
310	103	106	108	111	114	118	123	127	130	134	137	140	143	147	150	153	156
320	104	107	110	113	115	120	125	129	133	136	140	143	146	150	153	157	160
330	105	108	111	114	117	122	126	131	135	138	142	145	148	152	156	160	163
340	106	110	113	115	118	123	128	133	137	141	144	148	151	155	159	163	166
350	108	111	114	117	120	125	130	134	139	143	147	150	153	158	162	166	169
360	109	112	115	118	121	126	131	136	141	145	149	152	156	161	166	169	172
370	110	113	116	119	122	128	133	138	143	147	151	155	158	163	168	172	175
380	111	114	117	120	123	129	135	140	144	149	153	157	160	166	170	174	178
390	111	115	118	121	125	130	136	141	146	151	155	159	163	168	173	177	181
400	112	116	119	122	126	132	137	143	148	152	157	161	165	170	175	180	184
420	114	118	121	124	128	134	140	146	151	156	160	165	169	175	180	185	189
440	115	119	123	126	130	136	142	148	154	159	164	169	173	179	185	190	195

TABLE 8.5 B : MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} N/mm^2 (MPa) IN EQUAL FLANGE-BEAMS OR CHANNELS WITH f_y = 250 N/mm^2, $\frac{T}{t} \leq 2.0$ and $\frac{d_1}{t} \leq 85$

D/T → l/r_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	161	161	160	160	160	160	160	159	159	159	159	159	159	159	159
45	161	160	159	159	158	158	158	157	157	157	157	157	157	157	157
50	160	158	158	157	156	156	156	155	155	155	154	154	154	154	154
55	159	157	156	155	154	154	153	153	152	152	152	151	151	151	151
60	158	156	154	153	152	152	151	150	149	149	149	148	148	148	148
65	156	154	153	151	150	149	148	147	146	146	145	145	144	144	144
70	155	153	151	149	149	147	146	144	143	142	142	141	141	140	140
75	154	152	149	147	146	144	143	141	140	139	138	137	137	136	136
80	153	150	148	145	143	142	140	138	136	135	134	133	132	132	132
85	152	149	146	143	141	139	138	135	133	131	130	129	128	127	127
90	151	147	144	141	139	137	135	131	129	127	126	125	124	123	123
95	150	146	142	139	137	134	132	128	126	124	122	121	120	119	118
100	149	145	141	137	134	132	129	125	122	120	118	116	115	114	113
110	147	142	137	133	130	127	124	119	115	113	111	108	107	105	105
120	144	139	134	129	126	122	119	113	109	106	104	101	99	97	96
130	142	136	131	126	121	118	114	108	103	99	97	94	91	89	88
140	140	133	128	122	118	113	110	103	97	94	91	87	85	82	81
150	138	131	124	119	114	109	105	98	92	88	85	81	78	76	74
160	136	128	121	115	110	106	101	93	87	83	80	75	73	70	68
170	134	126	119	112	107	102	98	89	83	79	75	70	68	64	63
180	131	123	116	109	104	99	94	85	79	74	71	66	63	60	58
190	129	121	113	106	101	95	91	82	75	71	67	62	59	55	54
200	127	118	111	104	98	92	88	79	72	67	63	58	55	51	50
210	125	116	108	101	95	90	85	76	69	64	60	55	52	48	46
220	123	114	106	99	92	87	82	73	66	61	57	52	49	45	43
230	122	112	103	96	90	84	80	70	63	58	55	49	46	42	40
240	120	110	101	94	87	82	77	68	61	56	52	47	43	40	38
250	118	108	99	92	85	80	75	65	59	54	50	44	41	37	35
260	116	106	97	89	83	77	73	63	57	52	48	42	39	35	33
270	114	104	95	87	81	75	71	61	55	50	46	41	37	33	31
280	113	102	93	85	79	73	69	59	53	48	44	39	35	32	30
290	111	100	91	84	77	72	67	58	51	46	42	37	34	30	28
300	109	98	89	82	75	70	65	56	49	45	41	36	32	29	27

TABLE 8.5 F : MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} N/mm^2 (MPa) IN EQUAL FLANGE I-BEAMS OR CHANNELS WITH f_y = 400 N/mm^2, $\frac{T}{t} \leq 2.0$ and $\frac{d_1}{t} \leq 67$

D/T → l/r_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	253	252	250	249	249	248	248	247	247	246	246	246	246	246	246
45	251	248	246	245	244	243	243	242	241	241	240	240	240	240	239
50	248	245	242	240	239	238	237	235	234	234	233	233	233	232	232
55	245	241	238	236	234	232	231	229	227	227	226	225	225	225	224
60	242	237	234	231	228	226	225	222	220	219	218	217	217	216	216
65	239	234	229	225	222	220	218	215	212	211	210	209	208	207	207
70	236	230	225	220	217	214	212	207	205	203	202	200	199	198	198
75	233	226	220	215	211	208	205	200	197	195	193	191	190	189	188
80	230	223	216	210	206	202	199	193	189	186	185	182	181	180	179
85	227	219	212	205	200	196	192	186	181	178	176	174	172	171	170
90	225	215	207	201	195	190	186	179	174	171	168	165	164	162	161
95	222	212	203	196	190	185	180	172	167	163	161	157	155	153	152
100	219	208	199	191	185	179	175	166	160	156	153	150	148	145	144
110	213	202	191	183	176	169	164	154	148	143	140	135	133	130	129
120	208	195	184	175	167	160	154	144	136	131	127	123	120	117	115
130	203	189	177	167	159	152	146	134	126	121	117	111	108	105	103
140	198	183	171	160	152	144	138	126	117	111	107	102	98	95	93
150	193	178	165	154	145	137	131	118	109	103	99	93	89	86	84
160	188	172	159	148	139	131	124	111	102	96	92	85	82	78	76
170	183	167	154	142	133	125	118	105	96	90	85	79	75	71	69
180	179	162	149	137	127	119	112	99	90	84	79	73	69	65	63
190	175	158	144	132	122	114	108	94	85	79	74	68	64	60	58
200	171	153	139	128	118	110	103	90	81	75	70	63	60	55	53
210	167	149	135	123	114	105	99	86	77	70	66	59	55	51	49
220	163	145	131	119	110	102	95	82	73	67	62	56	52	48	45
230	159	141	127	115	106	98	91	79	70	64	59	53	49	44	42
240	156	138	123	112	102	94	88	75	67	61	56	50	45	42	39
250	152	134	120	108	99	91	85	72	64	58	53	47	43	39	37
260	149	131	117	105	96	88	82	70	61	55	51	45	41	37	34
270	146	128	114	102	93	85	79	67	59	53	49	43	39	35	32
280	143	125	111	99	90	83	77	65	57	51	47	41	37	33	30
290	140	122	108	97	88	80	74	63	55	49	45	39	35	31	29
300	137	119	105	94	85	78	72	61	53	47	43	37	33	29	27

The thickness of plate is given by

$$t_p = \sqrt{\frac{3 f_p n^2}{\sigma_{bc}}}, \text{ where } \sigma_{bc} = 0.66 f_y = 0.66 \times 250 = 165 \text{ N/mm}^2$$

$$\therefore \quad t_p = \sqrt{\frac{3 \times 4.95\,(85.5)^2}{165}} = 25.65 \text{ mm.}$$

Hence provide $t_p = 26$ mm.

Example 8.6. *A simply supported beam has on effective span of 6 m and consists of ISMB 500 @ 86.9 kg/m section. Determine the uniformly distributed load the beam can support, in addition to its own weight, if (a) the beam is connected to columns at the ends by framed connections, and (b) there is a lateral support at midpoint also. Take $f_y = 250\ N/mm^2$ and $E = 2 \times 10^5\ N/mm^2$.*

Solution

From steel tables, for ISMB 500 @ 86.9 kg/m, we have the following :

$$Z_{xx} = 1808.7 \times 10^3 \text{ mm}^3$$

$$I_{xx} = 45213.3 \times 10^4 \text{ mm}^4$$

$$h = 500 \text{ mm}; \; b = 180 \text{ mm}$$

$$t_f = 17.2 \text{ mm} = T$$

$$t_w = 10.2 \text{ mm} = t$$

$$h_2 = 37.95 \; ; \; r_y = 35.2 \text{ mm.}$$

(a) Beam connected to columns at the ends

In such a case, the ends of the beam are assumed to provide *full torsional restraint.* However, there is no restraint against lateral bending at the ends

Hence effective length of compression flange, $= l = L = 6$ m.

$$\frac{T}{t} = \frac{t_f}{t_w} = \frac{17.2}{10.2} = 1.69 < 2.0$$

$$d_1 = h - 2t_f = 500 - 2 \times 17.2 = 465.6 \text{ mm}$$

$$\frac{d_1}{t} = \frac{d_1}{t_w} = \frac{465.6}{10.2} = 45.6 < 85.$$

Hence Table 8.5 (*B*) will be applicable.

Now $$\frac{D}{t} = \frac{h}{t_f} = \frac{500}{17.2} = 29.07$$

and $$\frac{l}{r_y} = \frac{6 \times 1000}{35.2} = 170.4$$

Hence from Table 8.5 (*B*), we have the following data :

D/T → l/r_y ↓	25	30	29.07
170	89	83	84.12
180	85	79	80.12

$\therefore$ Allowable reaction, $R_a = \sigma_{ac} \cdot t_w \cdot B$

where $B = b' + \frac{1}{2}h = 300 + \frac{350}{2} = 475$ mm.

$\therefore$ $R_a = 147.25 \times 7.4 \times 475 \times 10^{-3} = 517.6$ kN $>> R$.

Hence safe.

(b) Design of Main beams

Effective span $= 9 + 0.3 = 9.3$ m.

The beam will be subjected to two point loads, each of value P, as shown in Fig. 8.16.

Value of $P = wL = 20.89 \times 6 = 125.34$ kN.

Assume self weight

$= \frac{\text{Total load}}{300} = \frac{2 \times 125.34}{300} \approx 0.85$ kN/m.

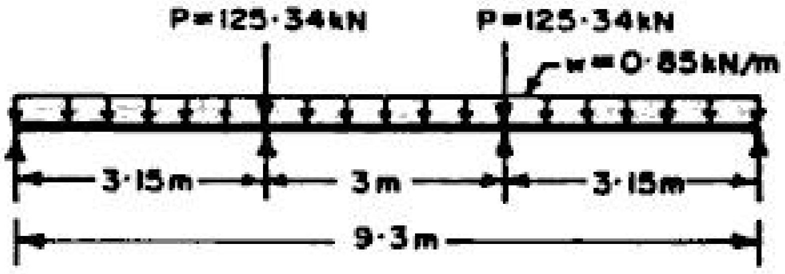


FIG. 8.16

$$M_{max} = 125.34 \times 3.15 + \frac{0.85(9.3)^2}{8} = 394.82 + 9.19 = 404.01 \text{ kN-m}$$

$$V = 125.34 + \tfrac{1}{2} \times 0.85 \times 9.3 \approx 129.3 \text{ kN}$$

Because of intermittant lateral supports from secondary beams, the effective length of compression flange = spacing of secondary beams =3 m. The beam is assumed to be restrained against torsion at the ends.

To start with, assume $\sigma_{bc} = 0.66 f_y = 0.66 \times 250 = 165$ N/mm^2

$\therefore$ Z required $= \frac{404.01 \times 10^6}{165} = 2449 \times 10^3$ mm^3.

Since effective length of the compression flange is about $\frac{1}{3}$ of the span, take trial value of $Z = 1.3 \times 2449 \times 10^3 \approx 3184 \times 10^3$ mm^3.

Hence try ISWB 600 @ 133.7 kg/m having the following properties :

$$Z_{xx} = 3540.0 \times 10^3 \text{ mm}^3 ; I_{xx} = 106198.5 \times 10^4 \text{ mm}^4$$
$$h = 600 \text{ mm} ; b = 250 \text{ mm} ; h_2 = 42.90 \text{ mm}$$
$$h_1 = 514.2 \text{ mm} ; r_{yy} = 52.5 \text{ mm}$$
$$t_f = 21.3 \text{ mm} = T$$
$$t_w = 11.2 \text{ mm} = t$$

Now $\frac{T}{t} = \frac{t_f}{t_w} = \frac{21.3}{11.2} \le 2$

$$d_1 = h - 2t_f = 600 - 2 \times 21.3 = 557.4$$

$\therefore$ $\frac{d_1}{t} = \frac{d_1}{t_w} = \frac{557.4}{11.2} = 49.77 < 85$

Hence Table 8.5 (B) will be applicable.

Now $\frac{D}{t} = \frac{h}{t_f} = \frac{600}{21.3} = 28.17$

and $\frac{l}{r_y} = \frac{3000}{52.5} = 57.14$

of 16 t_p for compression flange, and 20 t_p for tension flange. Knowing B, t_p can be found. Plates commonly available are not thicker than 14 mm. Hence if t_p is larger, two or more plates of smaller thickness can be used.

(*b*) **Design of rivets**

The rivets are designed for horizontal shear. The shear stress at any layer of a section is given by Eq. 8.7 (*a*):

$$\tau_v = \frac{V}{Iz}(A\bar{y})$$

where z = width of section at which τ_v is being computed $= B$

$A\bar{y}$ = first moment of the *outer area*.

Referring to Fig. 8.19,

$$A = B \,.\, t_p \text{ and } \bar{y} = \frac{h}{2} + \frac{t_p}{2}$$

Let q = shear force per unit length, at the level of rivets $= \tau_v \,.\, z$

$$\therefore \quad q = \frac{V}{I}(A\bar{y}) = \frac{V}{I}(B\,t_p)\left(\frac{h}{2} + \frac{t_p}{2}\right)$$

or

$$q = \frac{V B\, t_p (h + t_p)}{2I} \qquad \text{...(8.30)}$$

where I = moment of inertia of built-up section

If there are two rows of rivets, at a pitch p, the force in each rivet is equal to $\frac{1}{2} q\,p$. This should be equal to rivet value R.

Hence

$$R = \frac{V B\, t_p (h + t_p)}{4I} p \qquad \text{...(8.31)}$$

or

$$p = \frac{4IR}{V B\, t_p (h + t_p)} \qquad \text{...(8.32)}$$

It is to be noted that pitch is inversely proportional to shear force, and hence pitch will decrease at the end, where S.F. is maximum.

As per IS : 800, the maximum pitch in compression flange should not exceed $12t$ or 200 mm whichever is less, while the maximum pitch in tension should not exceed $16t$ or 200 mm, whichever is less. Here t is the thickness of the plate or the flange, whichever is less. Generally, the rivets are staggered on both sides, as shown in Fig. 8.19, so that the section is weakened by one rivet hole only. The diameter of the rivet is found by Unwins formula, $d = 6.05\sqrt{t}$, where t is the thickness of the thickest member, in mm.

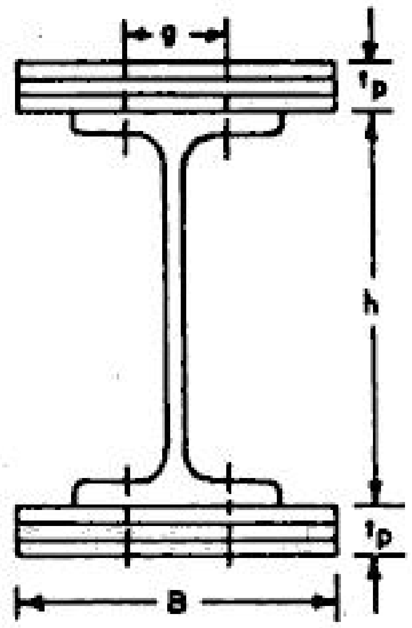


FIG. 8.20.

(*c*) **Curtailment of Plates**

The biggest advantage of a plated beam is that the flange plates can be curtailed at a section where these are no longer required from B.M. point of view, and thus the design becomes economical.

Let L = effective span of beam

Z_i = Section modulus of rolled section

Z_1 = Section modulus of built-up section, with one plate on each flange

Z_2 = Section modulus of built-up section, with two plates on each flange

Z_3 = Section modulus of built-up section with three plates on each flange.

Step 7. ***Check for deflection***

$$y_{max} = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{(60 \times 1000)(9)^4}{2 \times 10^5 \times 97598.5 \times 10^4} \times (1000)^3 = 26.3 \text{ mm.}$$

$$y_{allow.} = \frac{L}{325} = \frac{9000}{325} = 27.7 \text{ mm. Hence safe.}$$

Step 8. ***Check for web crippling***

Reaction $R = V = 270$ kN.

b = width of support = 200 mm,

$h_2 = 33.0$ mm.

$\therefore$ Web crippling stress $= \dfrac{R}{(b + h_2\sqrt{3})\, t_w} = \dfrac{270 \times 1000}{(200 + 33\sqrt{3})\, 9.2} = 114 \text{ N/mm}^2$

Permissible value $= \sigma_p = 0.75 f_y = 0.75 \times 250 = 187.5 \text{ N/mm}^2$. Hence safe.

Step 9. ***Check for web buckling***

$$\lambda_w = \frac{h_1}{t_w\sqrt{3}}, \text{ where } h_1 = 384 \text{ mm}$$

$$\therefore \quad \lambda_w = \frac{384}{9.2\sqrt{3}} = 24.1$$

Hence for $f_y = 250$ and $\lambda = 24.1$, $\sigma_{ac} = 148 - (148 - 145)\dfrac{4.1}{10} = 146.77$

$\therefore$ Allowable reaction, $R_a = \sigma_{ac}\, t_w \,.\, B$,

where $B = b + \frac{1}{2}h = 200 + \dfrac{482}{2} = 441$ mm

$$\therefore \quad R_a = 146.77 \times 9.2 \times 441 \times 10^{-3} = 595.5 \text{ kN} \gg (R = 270 \text{ kN})$$

Hence safe.

Step 10. ***Design of riveted joint***

The flange plates will be riveted to the flange by two rows of rivets, with staggered pitch. Using 20 mm dia. power driven rivets:

Strength of rivet in single shear $= 100 \times \frac{\pi}{4}(21.5)^2 \times 10^{-3} = 36.3$ kN

Strength of rivet in bearing $= 300 \times 21.5 \times 15.4 \times 10^{-3} = 99.33$ kN

$\therefore$ Rivet value $R = 36.3$ kN.

The pitch of rivets is given by Eq. 8.32 :

$$p = \frac{4IR}{VBt_p(h + t_p)}$$

where $I = 97598.5 \times 10^4$; $R = 36.3$ kN; $B = 360$ mm

$t_p = 16$ mm; $h = 450$ mm; $V = 270$ kN

$$\therefore \quad p = \frac{4 \times 97598.5 \times 10^4 \times 36.3}{270 \times 360 \times 16\,(450 + 16)} = 195.5 \text{ mm.}$$

Maximum pitch in compression flange $= 12t = 12 \times 16 = 192$ mm, or 200 mm whichever is lower. Maximum pitch in tension flange $= 16t = 16 \times 16 = 256$ mm or 200 mm, whichever is lower.

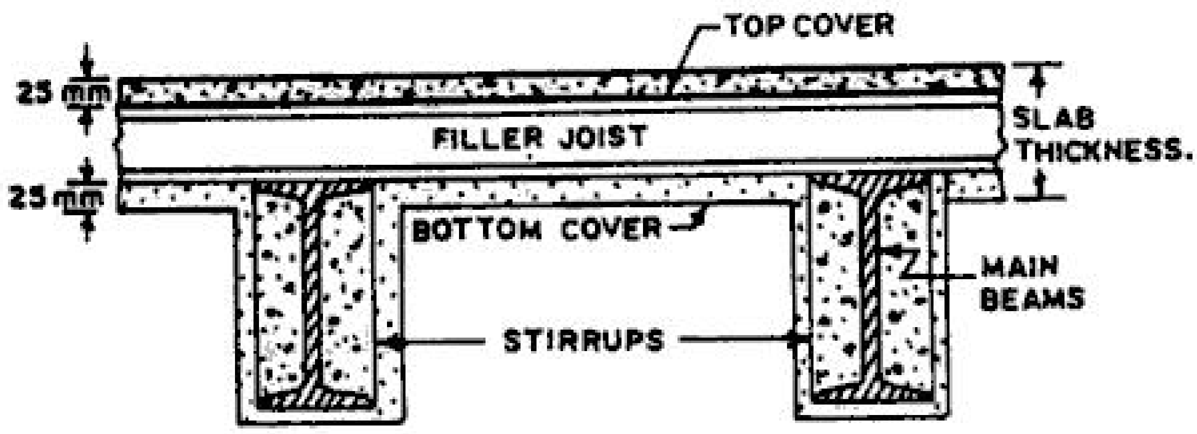
in discontinuous filler joist floor is smaller than that of continuous filler joist construction. The bottom of filler joist remain flush with the concrete slab, in the case of discontinuous fillers.

The *specifications* for design of filler joists are given below:

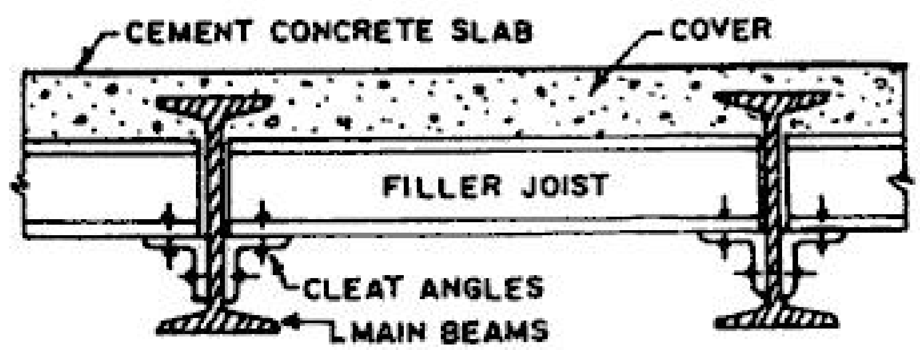
1. **Bending moment** : The bending moments on slabs of which filler joists form part, shall be calculated to satisfy the following :

(*a*) As slabs spanning continuously over supports and subjected to those combinations of dead and live loads producing the maximum positive and negative moments.

(*b*) In the case of three or more approximately equal spans of continuous filler joists, as slab designed for uniformly distributed loading satisfying the moment values given Table 8.6.



(a) CONTINUOUS FILLERS WITH BOTTOM COVER



(b) DISCONTINUOUS FILLER

FIG. 8.33. FILLER JOIST FLOORS.

The spans are considered approximalety equal when the longest span does not exceed the shortest span by more than 15%.

TABLE 8.6. B.M. VALUES FOR FILLER JOISTS.

Near middle of end span	*At support next to end support*	*At middle of interior spans*	*At other interior supports*
$+\frac{wL^2}{10}$	$-\frac{wL^2}{10}$	$+\frac{wL^2}{12}$	$-\frac{wL^2}{12}$

where w = the dead load plus live load per unit length of span

and L = the span centre of supports.

Note : For single spans, the slab shall be assumed to be freely supported.

2. **Moment of resistance**: The moment of resistance of the slab shall be calculated from the section properties of the filler joists only, except that where the filler joists are completely embedded in a solid concrete slab, the slab may alternatively be calculated as composite reinforced concrete section.

3. **Spacing** : The spacing of filler joists centre to centre shall not exceed n times the minimum thickness of the structural concrete slab as given in Table 8.7, unless the concrete is reinforced to span as slab or function as an arch between the filler joists.

Example 8.16. *Design a jack arch roof for a room 4.5 m × 8.75 m. Assume total uniformly distributed load of 10 kN/m², including the self weight, and allowable stress in masonry as 0.3 N/mm². Take $f_y = 250$ N/mm².*

Solution Provide R.S. Joists at regular interval of 1.75 m. The span of the joists will thus be 4.5 m, and span of jack arch will be 1.75 m.

Rise of arch has to be between $\frac{L}{8}\left(=\frac{1.75}{8}=0.219\text{ m}\right)$ to $\frac{L}{12}\left(=\frac{1.75}{12}=0.1458\text{ m}\right)$.

$$\text{Hence provide rise } R = 0.175\text{ m} = 175\text{ mm}$$

$$\text{Load per metre width of arch} = W = 1.75 \times 1 \times 10 = 17.5\text{ kN}$$

$$\text{Horizontal thrust, } T = \frac{WL}{8R} = \frac{17.5 \times 1.75}{8 \times 0.175} = 21.875\text{ kN/m.}$$

On the end joist, vertical reaction $V = \frac{W}{2} = \frac{17.5}{2} = 8.75$ kN/m

$$\text{Horizontal reaction } T = 21.875\text{ kN/m}$$

$$\therefore \quad \text{Resultant} = \sqrt{(8.75)^2 + (21.875)^2} = 23.56\text{ kN/m}$$

(a) Design of jack arch.

Provide 100 mm thick arch.

Hence $t = 100$ mm; width $b = 1$ m $= 100$ mm.

$$\therefore \quad \text{Stress} = \frac{T}{t \times b} = \frac{21.875 \times 1000}{100 \times 1000} \approx 0.22\text{ N/mm}^2$$

Hence minimum thickness of 100 mm will be sufficient.

(b) Design of beam

Clear span = 4.5.

Providing 0.3 m bearing at the end, effective span = 4.5 + 0.3 = 4.8 m. Load on central beam = W = 17.5 kN/m

$$\therefore \quad M_{max} = \frac{17.5\,(4.8)^2}{8} = 50.4\text{ kN-m} = 50.4 \times 10^6\text{ N-mm}$$

$$V = \frac{17.5 \times 4.8}{2} = 42\text{ kN}$$

$$\therefore \quad \sigma_{bc} = 0.66 f_y = 0.66 \times 250 = 165\text{ N/mm}^2$$

$$\therefore \quad \text{Required } Z = \frac{50.4 \times 10^6}{165} = 305.5 \times 10^3\text{ mm}^3$$

Use ISLB 275 @ 33.0 kg/m, having $Z = 392.4 \times 10^3$ mm³ and $I = 5375.3 \times 10^4$ mm⁴.

(c) Check for deflection.

$$y = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{(17.5 \times 1000)\,(4.8)^4}{(2 \times 10^5)\,(5375.3 \times 10^4)} \times (1000)^3$$

$$= 11.25\text{ mm.}$$

$$\text{Permissible } y = \frac{L}{480} = \frac{4.8 \times 1000}{480} = 10\text{ mm.}$$

9.2. LOADS ACTING ON GANTRY GIRDER

A gantry girder, having no lateral support in its length, has to withstand vertical loads from the weight of the crane, hook load and impact and horizontal loads from crane surge. Various forces acting on a gantry girder are given below.

1. Vertical loads : The wheels of the crane bridge moving over the gantry girder transfer vertical loads to the gantry girder. There are two wheels to each end of the crane bridge. The distance between the two wheels is known as *wheel base*. The total vertical loads consist of

(*i*) weight of crane girders

(*ii*) weight of trolly or crab car

(*iii*) self weight of girder and rails.

The weight W_c of the crane girder is transferred to gantry girders through four wheels (two wheels at each end). Hence weight W_1 transferred to gantry girder, due to crane, is given by

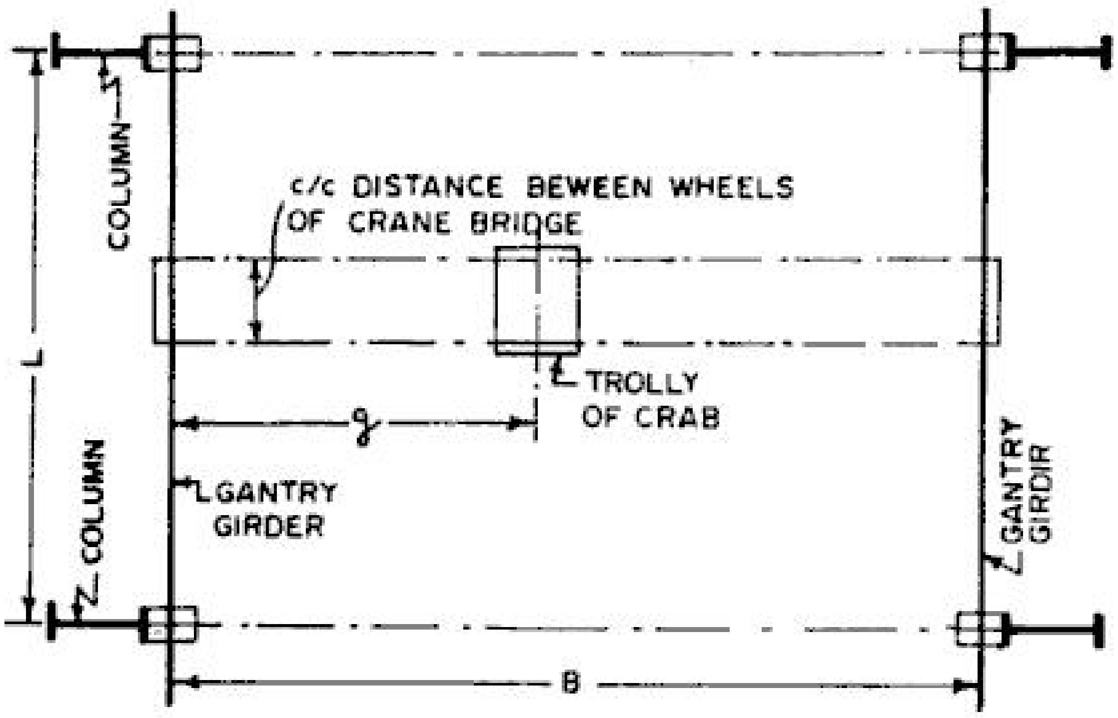


FIG. 9.2.

$$W_1 = \frac{W_c}{4} \qquad ...(9.1)$$

where W_1 is wheel load due to crane girder weight.

The weight W_t of the trolly is transferred to crane wheels as axle loads. The load to the gantry is maximum when the trolly wheels are closest to the girders, and its magnitude is given by

$$W_2 = \frac{W_t\,(B-g)}{2B} \qquad ...(9.2)$$

where B = distance between gantry girders (Fig. 9.2)

g = distance between the C.G. of the trolly and gantry

W_2 = load transferred to gantry through each wheel of the bridge, there being two wheels at each end of the bridge.

2. Lateral forces : Lateral forces are caused due to sudden stopping and starting of the crab load when moving over crane bridge. The intensity of lateral load is a function of the weight of the trolly, lifting load and the acceleration of the trolly. Lateral forces are also caused when the crane is dragging weights across the floor of the shop.

3. Longitudinal force (Drag force) : This is caused due to the starting and stopping of the crane bridge moving over the crane rails, as the crane moves longitudinally, *i.e.* in the direction of gantry girders.

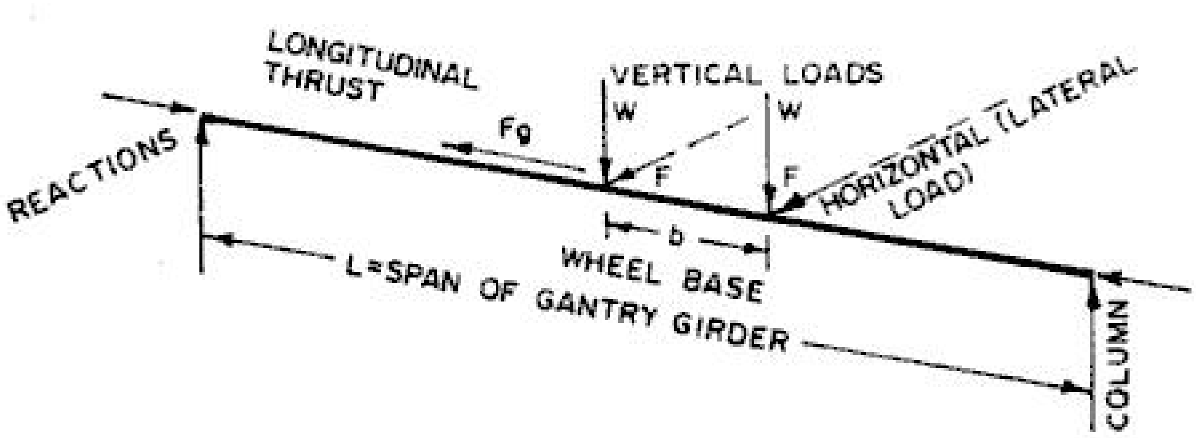


FIG. 9.3. FORCES ACTING ON GANTRY GIRDER

Similarly, for the *horizontal bending*, $(\sigma_{bc,cal})_H = (\sigma_{bt,cal})_H = \dfrac{M_Y}{Z_Y}$ ...(9.4)

where Z_Y is the modulus of section for the top flange only (see Fig. 9.8 *b*).

The combined bending compressive stress is taken as the sum of the two calculated compressive stresses, and this combined value should not exceed allowable compressive stress, increased by 10%. $(\sigma_{bc,cal})_V + (\sigma_{bc,cal})_H = 1.1\,\sigma_{bc}$ ...(9.5)

where σ_{bc} is the permissible bending compressive stress, determined by taking lateral instability into account.

The gantry girders should also be checked for web shear stress, buckling, crushing and combined stresses, as usual.

9.7. CONSTRUCTION DETAILS

The gantry girders are supported on brackets attached to columns. Surge girder is also provided to transmit the horizontal load from the top flange to the crane stanchion. Alternatively, the gantry girders are suitably connected to the crane stanchion (Fig. 9.9 (*a*). Fig. 9.9 gives the typical details.

9.8. DESIGN PROCEDURE

The design of a gantry girder is carried out in the following steps:

Step 1. ***Determination of loads***

(*i*) Determine weight (W_c) of crane bridge, excluding trolly. This weight will act as U.D.L. The wheel load W_1 due to the total weight W_c will be

$W_1 = \dfrac{W_c}{4}$ (there being four wheels, two at each end) ...(9.1)

(*ii*) Determine maximum wheel load due to trolly and lifted load. The vertical reaction on each wheel of the crane would be maximum when trolly is at the nearest distance$=a=$min. hook approach, as marked in Fig. 9.6 (*a*). Find wheel load W_2 in this position :

$$W_2 = \frac{W_t\,(B-g)}{2B} = \frac{W_t\,(B-a)}{2B} \qquad ...(9.2\ a)$$

(*iii*) Find total load on each wheel of crane bridge$= W_1 + W_2$.

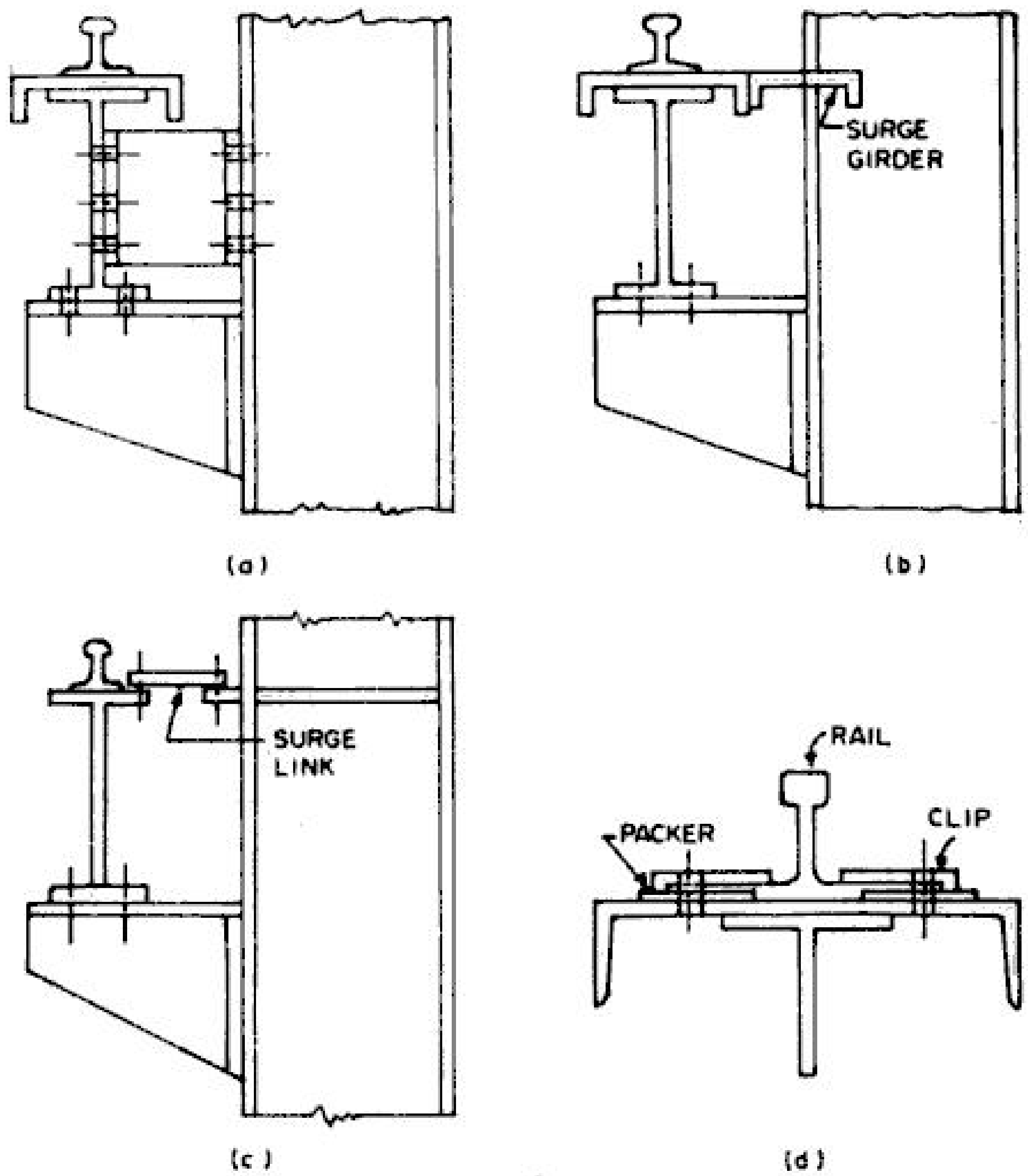


FIG. 9.9. CONSTRUCTION DETAILS

Step 3. ***Selection of section for gantry girder***

Take $\sigma_{bt} = 0.66 f_y = 0.66 \times 250 = 165 \text{ N/mm}^2$

$$\therefore \quad Z = \frac{M_X}{\sigma_{bt}} = \frac{505.735 \times 10^6}{165} = 3065.1 \times 10^3$$

Trial $Z = 1.15 \times 3065.1 \times 10^3 \approx 3525 \times 10^3 \text{ mm}^3$

Try ISWB 600 @ 133.7 kg/m as the basic beam having the following properties (Fig. 9.12 *a*).

$Z_{xx} = 3540.0 \times 10^3 \text{ mm}^3$,
$I_{xx} = 106198.5 \times 10^4 \text{ mm}^4$
$I_{yy} = 4702.5 \times 10^4 \text{ mm}^3$
$a_b = 17038 \text{ mm}^2$
$h = 600$ mm
$b = 250$ mm
$t_f = 21.3$ mm
$t_w = 11.2$ mm

Provide ISMC 300 @ 35.8 kg/m on the top flange of the beam, having the following properties (Fig. 9.12*b*).

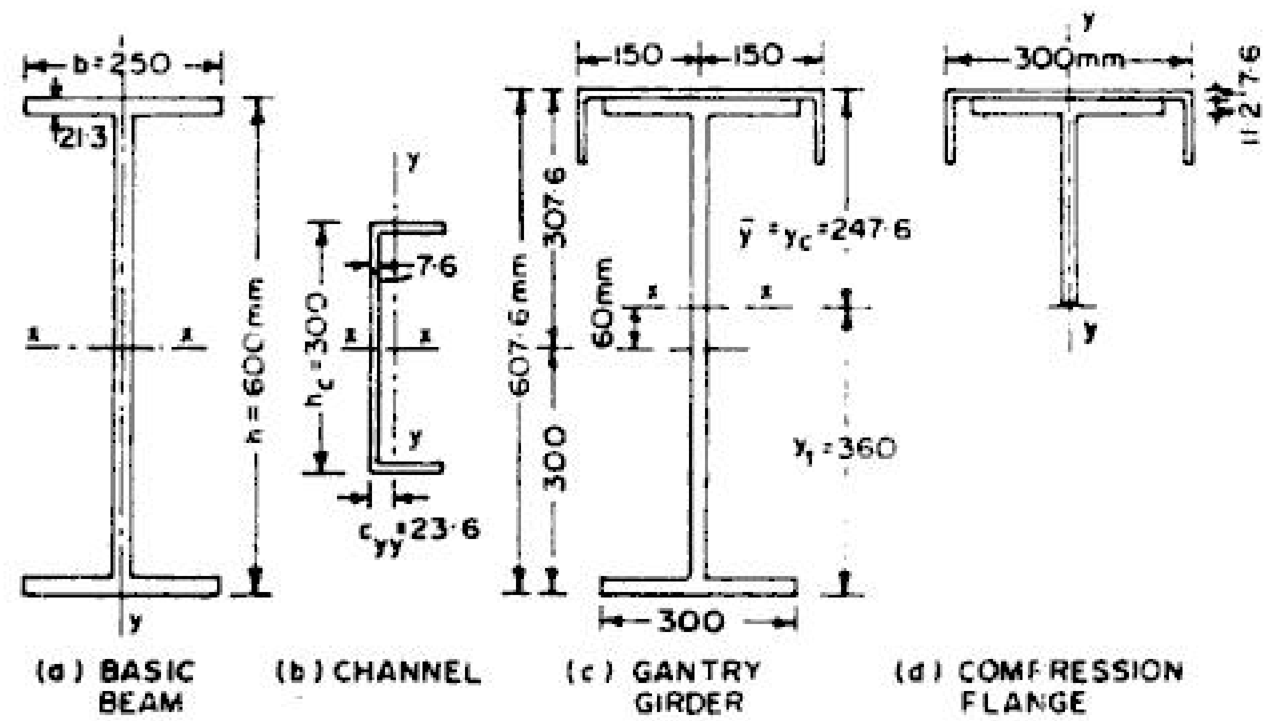


FIG. 9.12.

$(I_{yy})_c = 310.8 \times 10^4 \text{ mm}^3$, $(I_{xx})_c = 6362.6 \times 10^4 \text{ mm}^4$
$a_c = 4564 \text{ mm}^2$; $C_{yy} = 23.6$ mm, $t_{wc} = 7.6$ mm

Step 4. ***Determination of N.A. and I_{xx}***

The composite section is shown in Fig. 9.12 (*c*). The N.A. (*i.e. x-x* axis) is situated at $\bar{y}$ below the top flange, where

$$\bar{y} = \frac{a_c C_{yy} + a_b \left(\frac{h}{2} + t_{wc}\right)}{a_c + a_b} = \frac{(4564 \times 23.6) + 17038\,(300 + 7.6)}{4564 + 17038}$$

or $\bar{y} = y_c = 247.6$ mm

$\therefore \quad y_t = (h + t_{wc}) - y_c = 600 + 7.6 - 247.6 = 360$ mm

$$I_X = \left[106198.5 \times 10^4 + 17038\,(307.6 - 247.6)^2\right] + \left[310.8 \times 10^4 + 4564\,(247.6 - 23.6)^2\right]$$
$$= 112332 \times 10^4 + 23211 \times 10^4 = 135543 \times 10^4 \text{ mm}^4$$

$$(\sigma_{bt,cal})_V = \frac{M_X}{I_X} \cdot y_t = \frac{505.735 \times 10^6}{135543 \times 10^4} \times 360 = 134.32 \text{ N/mm}^2$$

Permissible $\sigma_{bt} = 0.66 f_y = 0.66 \times 250 = 165 \text{ N/mm}^2$. Hence safe.

Step 5. ***Determination of σ_{bc}***

I_Y for the whole section $= (4702.5 \times 10^4) + 6362.6 \times 10^4 = 11065.1 \times 10^4$

Moment of inertia of compression flange about *y-y* axis is

$$I_{YCF} = \left(\frac{1}{2} \times 4702.5 \times 10^4\right) + 6362.6 \times 10^4 = 8713.85 \times 10^4 \text{ mm}^4$$

2. Redesign the gantry girder of problem 1, if the wheel base is 3 m.

3. Design a gantry girder to carry an electrically operated head crane for the following data :

(*i*)	Span of gantry girder	: 10 m
(*ii*)	Crane capacity	: 350 kN
(*iii*)	Distance between centres of gantry girder	: 18 m
(*iv*)	Weight of crab	: 80 kN
(*v*)	Minimum approach distance of crane hook	: 1.2 m
(*vi*)	Weight of crane girder	: 190 kN
(*vii*)	Wheel base	: 4.0 m
(*viii*)	Height of rails	: 80 mm
(*ix*)	Mass of rail section	: 30 kg/m

Take $f_y = 250 \text{ N/mm}^2$

4. A gantry girder is composed of ISMB 600 @ 122.6 kg/m and a channel section ISLC 300 @ 33.1 kg/m placed on the top of beam with its flange down. Compute (*i*) maximum compressive stress (*ii*) maximum bending tensile stress, and (*iii*) max. shear stress, given the following :

(*i*)	Span of gantry girder	: 6.0 m
(*ii*)	Crane capacity	: 300 kN
(*iii*)	Distance between centres of gantry girder	: 15 m
(*iv*)	Weight of crane girder	: 130 kN
(*v*)	Weight of crab	: 70 kN
(*vi*)	Minimum approach of crane hook	: 1 m
(*vii*)	Distance between centres of wheels	: 3.8 m
(*viii*)	Mass of rail section	: 30 kg/m

Are the above stresses within safe limits ?

$$M_x = \left(\frac{M_2 - M_1 \cos kL}{\sin kL}\right) \sin kx + M_1 \cos kx \qquad ...(12)$$

Also, the value of max. B.M. is given from Eq. 9 :

$$M_{x.\,max} = \sqrt{\left(\frac{M_2 - M_1 \cos kL}{\sin kL}\right)^2 + M_1^2}$$

or

$$M_{x.\,max} = M_2 \sqrt{\frac{1 - 2(M_1/M_2)\cos kL + (M_1/M_2)^2}{\sin^2 kL}} \qquad ...(13)(10.2)$$

or

$$M_{x.\,max} = M_2 \times F_m = M_0 \times F_m \qquad ...(10.2\ a)$$

Hence we find here that the maximum moment (M_0) in the beam (which was equal to M_2 in absence of axial load P) has been *magnified* by the presence of axial load. The *magnification factor* for this case is

$$F_m = \sqrt{\frac{1 - 2(M_1/M_2)\cos kL + (M_1/M_2)^2}{\sin^2 kL}} \qquad ...10.2\ (b)$$

Case 2 : Transverse uniform loading : No end moments

Fig. 10.5 shows the case of a beam column subjected to U.D.L. acting transversely, along with the axial load P. Here, the primary moment M_i can be expressed by the equation

$$M_i = -\frac{wL}{2}x + \frac{wx^2}{2} = -\frac{w}{2}x(L - x) \qquad ...(14)$$

Here, we observe that $\frac{d^2 M_i}{dx^2} = +w$, and hence $f_1(x) \neq 0$.

Let $\quad f_1(x) = a_1 + a_2 x$

$$\therefore \quad \frac{d^2[f_1(x)]}{dx^2} = 0$$

Since $f_1(x)$ = value of M_x satisfying Eq. 4, we get from Eq. 4

$$\frac{d^2[f_1(x)]}{dx^2} + k^2 f_1(x) = \frac{d^2 M_i}{dx^2}$$

or $\quad 0 + k^2(a_1 + a_2 x) = +w$

From the above we get

$$a_2 = 0 \text{ and } a_1 = +\frac{w}{k^2}$$

Hence $\quad f_1(x) = a_1 + a_2 x = +\frac{w}{k^2}$

Substituting in Eq. 5

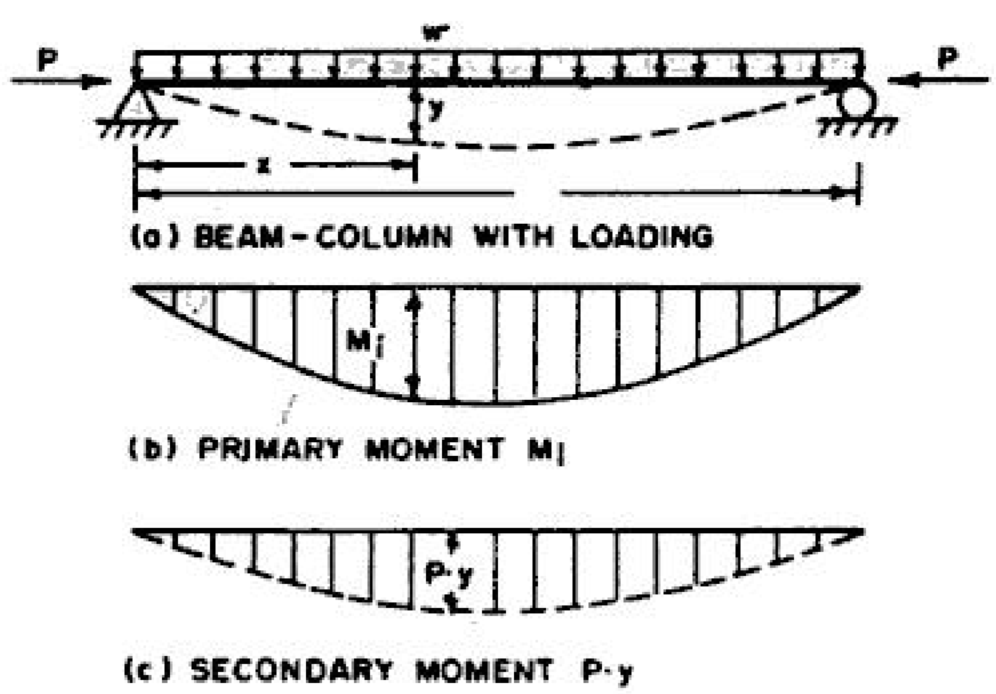


FIG. 10.5.

$$M_x = c_1 \sin kx + c_2 \cos kx + \frac{w}{k^2} \qquad ...(15)$$

Let us now apply the boundary conditions.

At $\quad x = 0,\ M_x = 0$ Hence $c_2 = -\frac{w}{k^2}$

Also, at $\quad x = L,\ M_x = 0 = c_1 \sin kL - \frac{w}{k^2}\cos kL + \frac{w}{k^2}$

Example 10.2. *A beam column is subjected to axial load P and a transverse load W at the mid-span. Compute the magnification factors for deflection, and moment, using simplified approach, and compare the two values of the magnification factors.*

Solution As proved in § 10.3, the maximum deflection under any type of transverse loading is given by Eq. 10.5 :

$$y_{max} = \frac{y_0}{1-\alpha}.$$

Hence magnification factor for deflection

$$= \frac{1}{1-\alpha} \quad \text{...(a)}$$

Again, the magnification factor for moment is given by Eq. 10.6 (*b*)

$$F_m = \frac{C_m}{1-\alpha} \quad \text{...(10.6 a)}$$

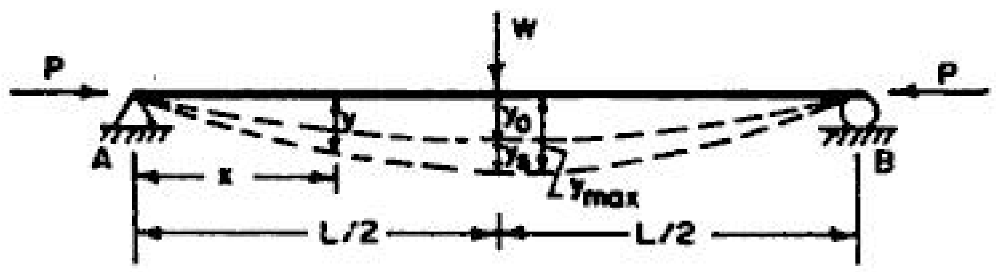


FIG. 10.8.

where $$C_m = 1 + \left(\frac{\pi^2 EI y_0}{M_0 L^2} - 1\right)\alpha \quad \text{...(10.6 c)}$$

Here $$M_0 = \frac{WL}{4}\ ;\ y_0 = \frac{WL^3}{48\,EI}$$

∴ $$\frac{y_0}{M_0} = \frac{1}{12}\frac{L^2}{EI}$$

∴ $$C_m = 1 + \left(\frac{\pi^2 EI}{L^2} \times \frac{1}{12}\frac{L^2}{EI} - 1\right)\alpha = 1 - 0.1775\,\alpha$$

∴ $$F_m = \frac{1 - 0.1775\,\alpha}{1-\alpha} \quad \text{...(10.7)}$$

Thus we observe that the magnification factor for deflection $\left(= \frac{1}{1-\alpha}\right)$ is the same for all types of transverse loading while the magnification factor (F_m) for the moment depends upon the type of loading.

The values of magnification factors for deflection and moments, for various values of α are tabulated below :

α	*Magnification factor for deflection* $= 1/(1-\alpha)$	*Magnification factor for moment* $=(1-0.1775\,\alpha)/(1-\alpha)$	*% Difference*
0.1	1.111	1.091	1.8
0.2	1.250	1.206	3.6
0.3	1.429	1.353	5.6
0.4	1.667	1.548	7.7
0.5	2.000	1.822	9.8
0.6	2.500	2.234	11.9
0.7	3.333	2.919	14.2
0.8	5.000	4.290	14.6
0.9	10.00	8.402	19.0

Since the values of the two factors are very near, the magnification factor for deflection can be taken as the magnification factor for moment, as an approximation.

10.4. COMPRESSIVE LOAD WITH UNIAXIAL BENDING : INTERACTION EQUATION

Neglecting the secondary moment, an approximate expression for the combined compressive stress for a *short* beam-column subjected to an axial load and bending moment with respect to one axis only may be expressed as

$$f_{max} = \frac{P}{A} + \frac{M y_c}{I}$$

or

$$f_{max} = \sigma_{ac,cal} + \sigma_{bc,cal}$$

Dividing both sides by f_{max}, we get

$$\frac{\sigma_{ac,cal}}{f_{max}} + \frac{\sigma_{bc,cal}}{f_{max}} = 1$$

The above expression may further be modified by substituting the applicable allowable stresses in place of f_{max} term.

Hence

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma_{bc,cal}}{\sigma_{bc}} \le 1 \qquad ...(10.9)$$

where $\sigma_{ac,cal}$ = calculated average axial compressive stress $= \frac{P}{A}$

$\sigma_{bc,cal}$ = calculated bending compressive stress in extreme fibre.

σ_{ac} = permissible axial compressive stress in the member subject to axial compressive load only.

σ_{bc} = permissible bending compressive stress in extreme fibre.

The above expression does not take into account the effect of additional moment (secondary moment) due to axial load interacting with the deflections. It was adopted in design specifications in IS : 800 – 1962. It applies to members when $\sigma_{ac,cal}/\sigma_{ac}$ is less than or equal to 0.15. When $\sigma_{ac,cal}/\sigma_{ac} > 0.15$, the secondary moment due to the member deflection may be of a significant magnitude. However, in order to take into effect the secondary moment (or the magnification of maximum moment), Eq. 10.9 can be modified as under :

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma'_{bc,cal}}{\sigma_{bc}} \le 1 \qquad ...(10.10)$$

where $\sigma'_{bc,cal}$ = calculated maximum bending compressive stress, taking into account the magnified moment.

$$= \frac{\sigma_{bc,cal}}{1 - P/P_E} = \frac{\sigma_{bc,cal}}{1 - \frac{n\,\sigma_{ac,cal}}{f_{cc}}}$$

where $f_{cc} = \frac{P_E}{A}$ = average compressive stress under Euler load commonly known as *elastic critical stress* in compression $= \frac{\pi^2 E}{\lambda^2}$

$\lambda = \frac{l}{r}$ = slenderness ratio in the plane of bending.

Substituting these values in Eq. 10.10, we get

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma_{bc,cal}}{\sigma_{bc}\left(1 - \frac{n\,\sigma_{ac,cal}}{f_{cc}}\right)} \le 1 \qquad ...(10.11)$$

Example 10.6. *A beam-column of effective length of 6 m carries an axial load of 450 kN and equal end moments of 50 kN-m each about the major axis. Design the H-section of the column. Assume that the frame falls under case (b) of Table 10.1 and the column bends either in single or in double curvature.*

Solution

Step 1. Computation of equivalent axial load

To start with, let $P_{EQ.} = (P + 0.75P) = 1.75P$, for uniaxial bending.

$\therefore \quad P_{EQ} = 1.75 \times 450 = 787.5$ kN.

From steel tables, we find that average value of r_y is around 50 mm for ISHB sections. Hence $\lambda = l/r_y = 6000/50 = 120$. Corresponding to this value of λ, $\sigma_{ac} = 64$ N/mm².

$$\therefore \quad \text{Area required} = \frac{787.5 \times 10^3}{64} = 12305 \text{ mm}^2$$

Select ISHB 450 @ 92.5 kg/m (the maximum available section), having $A = 11789$ mm², $Z = 1793300$ mm³ and $r_{yy} = 50.8$ mm

$$B_f = \frac{A}{Z} = \frac{11789}{1793300} = 6.57 \times 10^{-3}$$

Hence from Eq. 10.16, taking $\sigma_{ac}/\sigma_{bc} = 0.6$

$$P_{EQ} = P + M\,B_f\left(\frac{\sigma_{ac}}{\sigma_{bc}}\right)$$

or

$$P_{EQ} = 450 + 50 \times 10^3 \times 6.57 \times 10^{-3}\,(0.6) = 450 + 197.1 = 647.1 \text{ kN}$$

Step 2. Selection of section $\quad \lambda = l/r_y = 6000/50.8 = 118$

Hence $\quad \sigma_{ac} = 65.6$ N/mm²

$$\therefore \quad \text{Area required} = \frac{647.1 \times 10^3}{65.6} = 9864 \text{ mm}^2$$

Try ISHB 400 @ 82.2 kg/m having the following properties:

$A = 10466$ mm² ; $h = 400$ mm ; $b = 250$ mm,
$t_f = 12.7$ mm, $t_w = 10.6$ mm $r_x = 166.1$ mm ;
$r_y = 51.6$ mm ; $Z_x = 1444.2 \times 10^3$ mm³ ;
$d_1 = h - 2t_f = 400 - 2 \times 12.7 = 374.6$ mm.

Step 3. Determination of σ_{ac}

$$\lambda = l/r_y = \frac{6000}{51.6} = 116.3. \quad \text{Hence } \sigma_{ac} \approx 66.96 \text{ N/mm}^2$$

Step 4 Determination of σ_{bc}

$$\frac{T}{t} = \frac{t_f}{t_w} = \frac{12.7}{10.6} = 1.2 < 2 ; \frac{d_1}{t} = \frac{d_1}{t_w} = \frac{374.6}{10.6} = 35.34 < 85$$

Hence Table 8.5 (*b*) will be applicable.

Now $$\frac{D}{T} = \frac{h}{t_f} = \frac{400}{12.7} = 31.5 \text{ and } l/r_y = \frac{6000}{51.6} = 116.3$$

Hence from Table 8.5 (*b*), we get $\sigma_{bc} = 112.1$ N/mm²

Step 5. Determination of critical buckling stress

$$f_{cc} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 \times 2.0 \times 10^5}{(6000/166.1)^2} = 1512.7 \text{ N/mm}^2$$

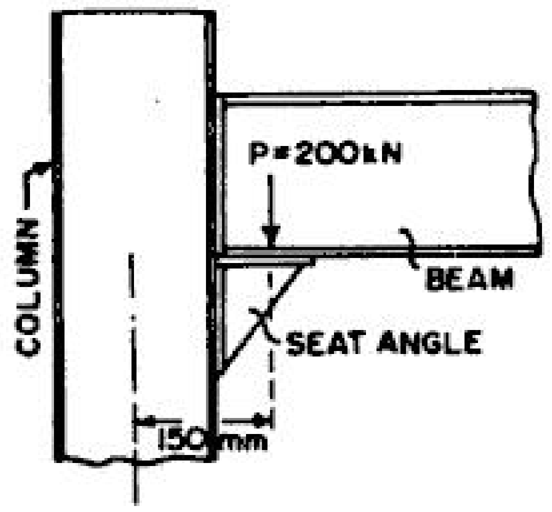


FIG. 10.13.

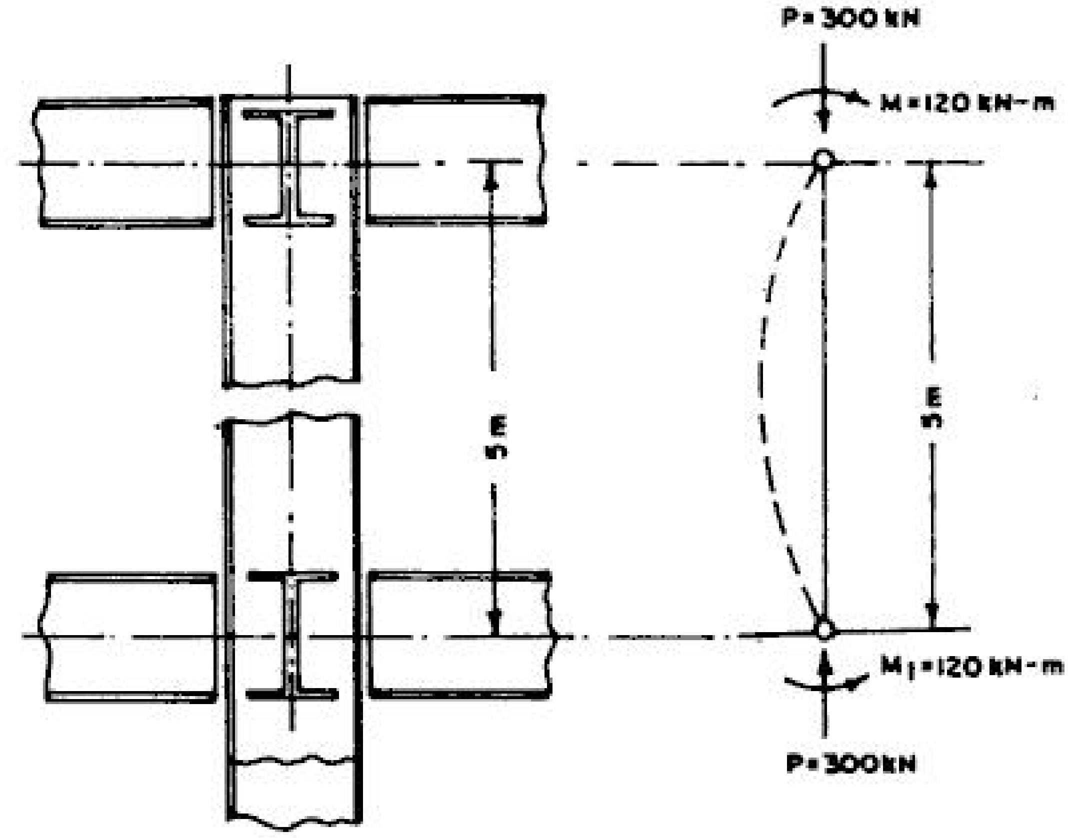


FIG. 10.14.

8. A H-section column supports beams framing into it as shown in Fig. 10.14. The connections are moment connections. The column supports an axial load of 300 kN, which includes the beam reactions at its top. Due to unbalanced floor loading, moments of 120 kN-m each are applied in opposite directions at top and bottom as shown. Side sway is prevented by a bracing system. The effective length factors K_x and K_y are estimated to be 0.9 and 1. Design the section.

9. Fig. 10.15 shows a corner column of 5.5 m length, receiving loads of $P_1 = 150$ kN and $P_2 = 140$ kN. Design a suitable column section, selecting either a wide flange section or H-section.

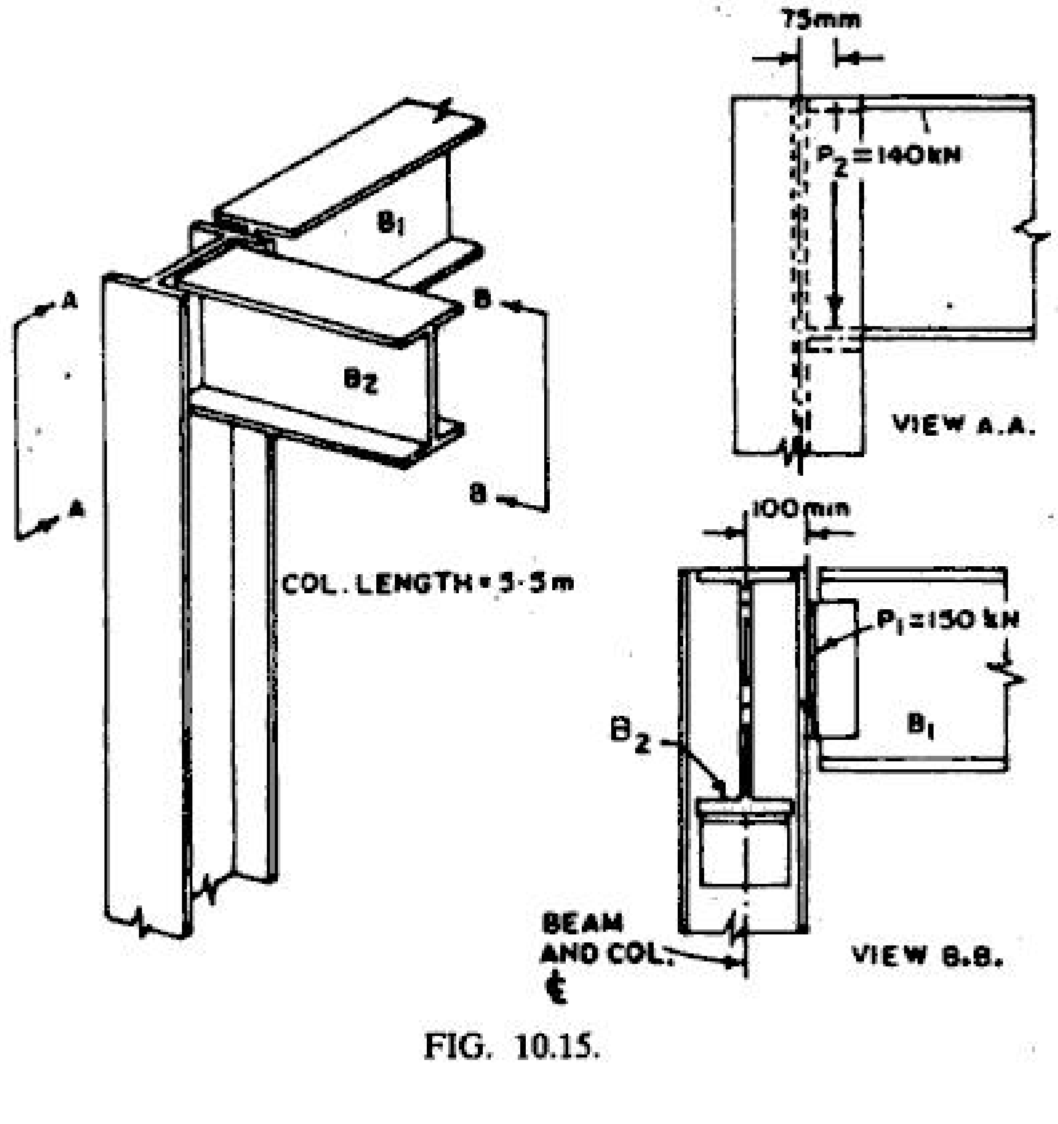


FIG. 10.15.

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The maximum stress at the farthest point B is given by

$$p_{max} = \frac{P}{A} + \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}} \qquad ...(11.16)$$

The above stress should not exceed the allowable bearing stress (w) for the concrete. Several trials may be necessary to establish a suitable size of the base.

Though the anchor bolts are not required to take any tension (or resist B.M.), nominal size anchor bolts are provided to keep the column in position.

Case (*b*) Compression over part of the base and tension in the holding down bolts

This happens when eccentricity is large, so that the resultant stress at the other end is tensile. Only a part of the base plate is in contact with the concrete base. The tensile stresses are taken by anchor bolts.

The base shown in Fig. 11.17 is subjected to an axial load P and moments M_{xx} and M_{yy} about xx and yy axes respectively. Fig. 11.17 (*b*) shows the direction of resultant moment vector M_R. The neutral axis of the section will be at right angles to the plane of action of M_R. Fig. 11.17 (*c*) shows the stress distribution, where p is the maximum compressive stress and t is the maximum tensile stress. The base plate is in contact with the concrete base for a length N only ; in other words, N is the depth of *N.A.*

The forces in concrete and steel can be expressed in terms of stress p in concrete and the depth N of the *N.A.* This can be solved for N. However the shape of the contact area (shown shaded in Fig. 11.17 *a*) will cause difficulty. Fig. 11.18 (*a*) and (*b*) show two alternative positions of the *N.A.*, with the expressions for the value and position of various components of compressive forces corresponding to the complex stress block.

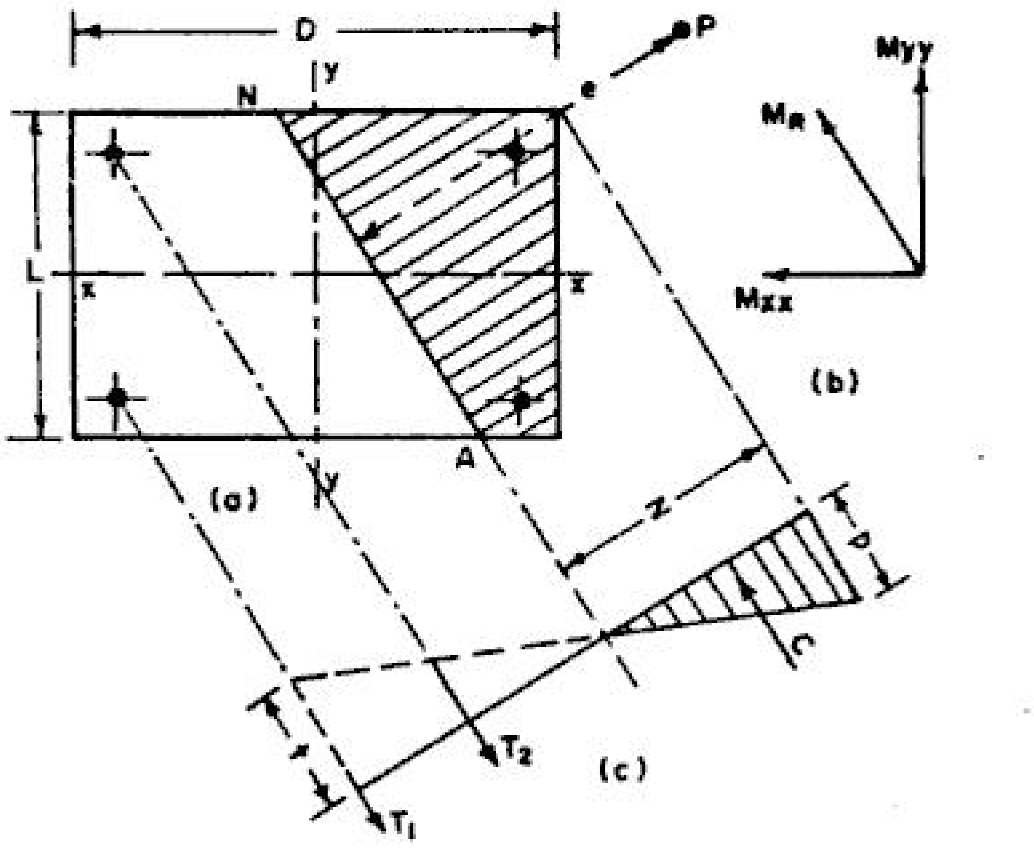

FIG. 11.17.

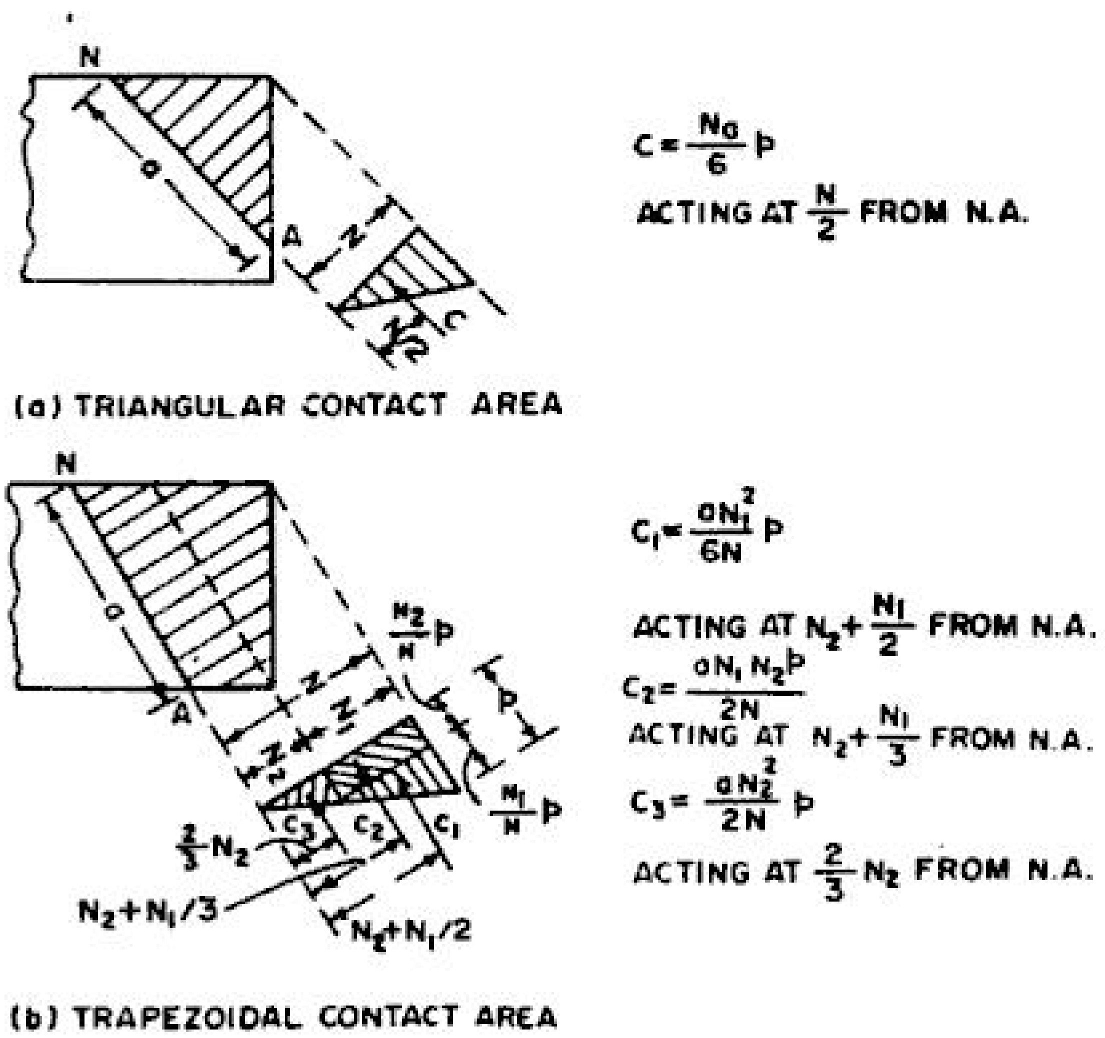

FIG. 11.18. COMPONENTS OF STRESS BLOCK.

ength L. The intensity of load transferred by the base plate to the top tier $= P/l$. Similarly the intensity of bearing pressure from the lower tier$= P/L$. Fig. 11.23 (*b*) and (*c*) show the *B.M.D.* and *S.F.D.* for the upper tier. The maximum *B.M.* occurs at the centre of the beam, its value being

$$(M_{max}) = \frac{P}{L} \times \frac{L}{2} \times \frac{L}{4} - \frac{P}{l} \times \frac{l}{2} \times \frac{l}{4} = \frac{P}{8}(L - l) \qquad ...(11.17)$$

The bending moment at the edge of the plate is

$$(M_{edge})_{top\ tier} = \frac{P}{8L}(L - l)^2 \qquad ...(11.18)$$

The maximum S.F. occurs at the edge of the base plate, its value being:

$$(V_{max})_{top\ tier} = \frac{P}{L}\left(\frac{L - l}{2}\right)$$

$$= \frac{P}{2L}(L - l) \qquad ...(11.19)$$

The load P of the column applied to the base plate of length l is assumed to be dispersed through the flanges and fillets at an angle of 30° to the horizontal, beyond the base plate. Hence this load is assumed to be dispersed on a length equal to $l + 2\sqrt{3}h_2$, where h_2 is the distance from extreme fibre to the bottom of fillet between the web and the flange.

∴ Bearing stresses

$$= \frac{P/n}{(l + 2\sqrt{3}h_2)t_w} \leq \sigma_{br} \qquad ...(11.20)$$

where n is the number of beams in the tier and σ_{br} is the permissible bearing stress, equal to $1.33(0.75 f_y) = f_y = 250\ \text{N/mm}^2$.

Similarly, in the transverse direction, if the width of the base plate is b and length of beams of tier is B, the max. B.M. and S.F. for the bottom tier are given by

$$(M_{max})_{bottom\ tier} = \frac{P}{8}(B - b) \qquad ...(11.21)$$

and

$$(V_{max})_{bottom\ tier} = \frac{P}{2B}(B - b) \qquad ...(11.22)$$

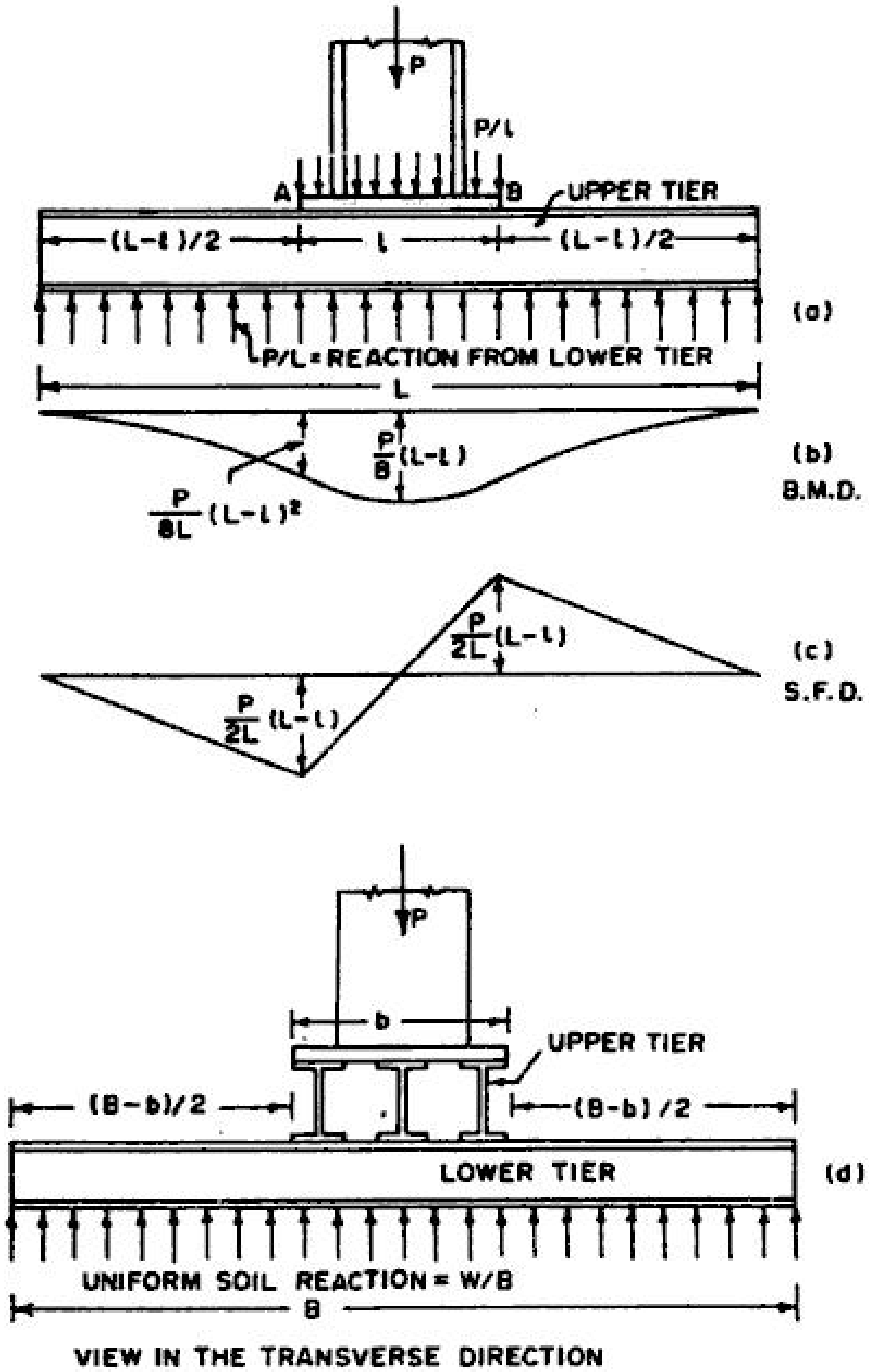

FIG. 11.23.

The bending moment at the edge of the plate is

$$(M_{edge})_{bottom\ tier} = \frac{P}{8B}(B - b)^2 \qquad ...(11.23)$$

Example 11.10. *Design a grillage foundation for a compound column consisting of ISHB 400 @ 82.2 kg/m with flange plates 350 mm × 20 mm one on each flange, and carrying on load*

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is it the full continuity moment as assumed in elastic rigid-frame analysis. Because of difficulty of evaluating the degree of restraint, semi-rigid connections are not used in plastic design and are rarey used in working stress design. *Actual connections* are neither completely rigid nor completely flexible, and can be *classified* on the basis of the *ratio of the moment developed by the connections to the full moment capacity of the connected member, expressed as a percentage.* As stated above, the approximate percentages for *simple connections* are from 0 to 20, for a semi-rigid connection from 20 to 80, and for a *rigid connection* from 80 to 90. The percentage of a particular connection must be determined by actual tests.

12.2. SIMPLE BEAM END CONNECTIONS

Steel beams are supported at their ends by (*i*) masonry walls or piers, (*ii*) steel columns, or (*iii*) heavier beams and girders running in transverse directions. In the case where the beam is supported on *steel supports*, the beam reactions are borne by the *end connections* of the beam with the supporting member. As discussed above, in the case of *simple framing*, the original angle between the members may change upto 80% of the amount it would theoretically change if *frictionless hinged connections* could be used. For beams, such a connection provides only *shear transfer* at the ends.

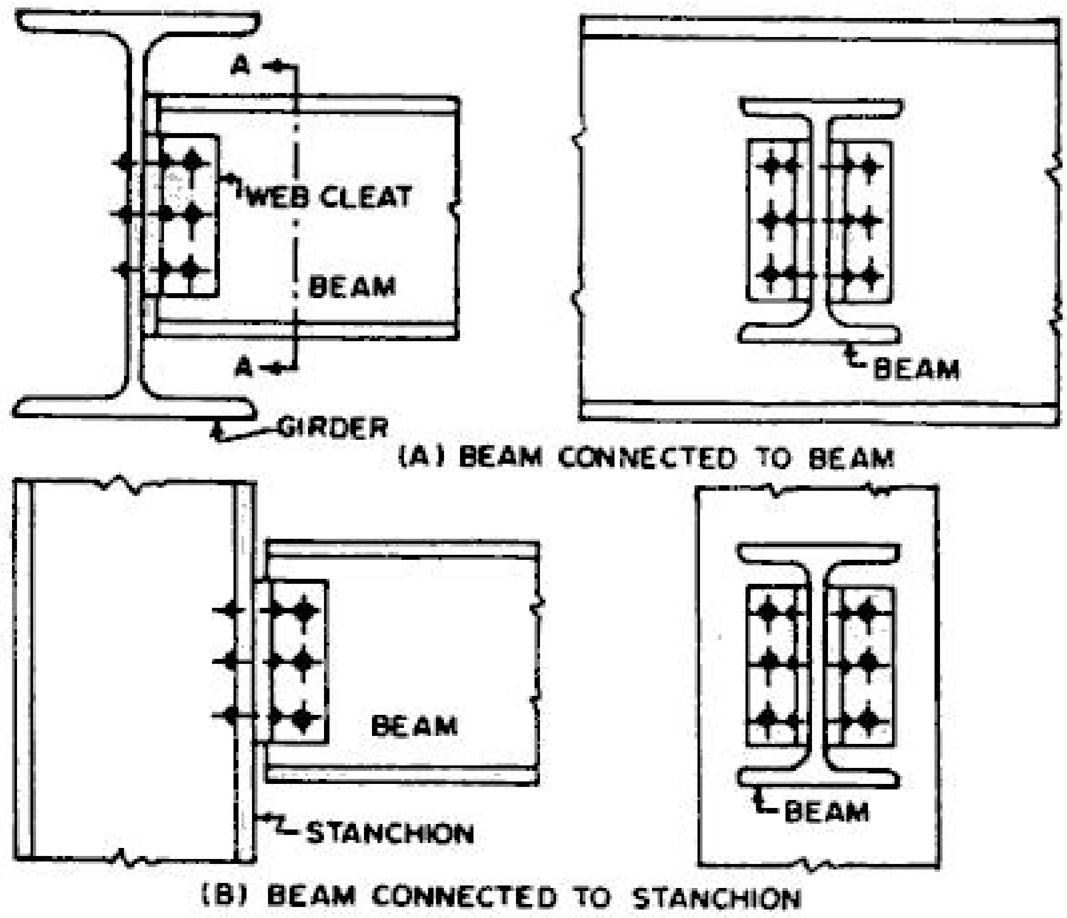


FIG. 12.2. FRAMED BEAM CONNECTIONS

Simple beam end connections are of two types :

1. Framed connections (Fig. 12.2, 12.3)
2. Seated connections

(*a*) Unstiffened seat connection (Fig. 12.4)

(*b*) Stiffened seat connection (Fig. 12.5)

Framed Connections :

A *framed connection* is the one when a beam is connected to a girder or a stanchion by means of two angles placed on the two sides of the web of the beam, as shown in Fig. 12.2.

When the beams intersect and are attached to other beams so that flanges of both are at the same elevation, as in Fig. 12.3 (*a*), (*b*), the beams framing-in have their flanges *coped* or *cut away*. The loss of section is primarily loss of flange that carries little shear, so that *normally a cope results in little loss of strength.*

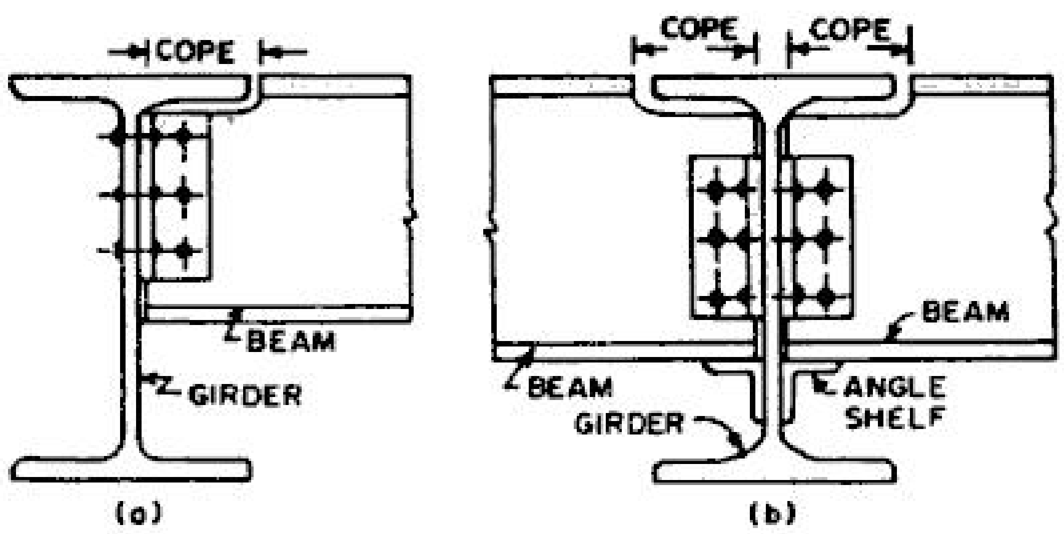


FIG. 12.3. BEAM FRAMING AT THE SAME LEVEL

$$\therefore \qquad b = \frac{V}{\sigma_p\, t_w} - h_2\sqrt{3}$$

Provided that $\qquad b \nless \frac{1}{2}B$

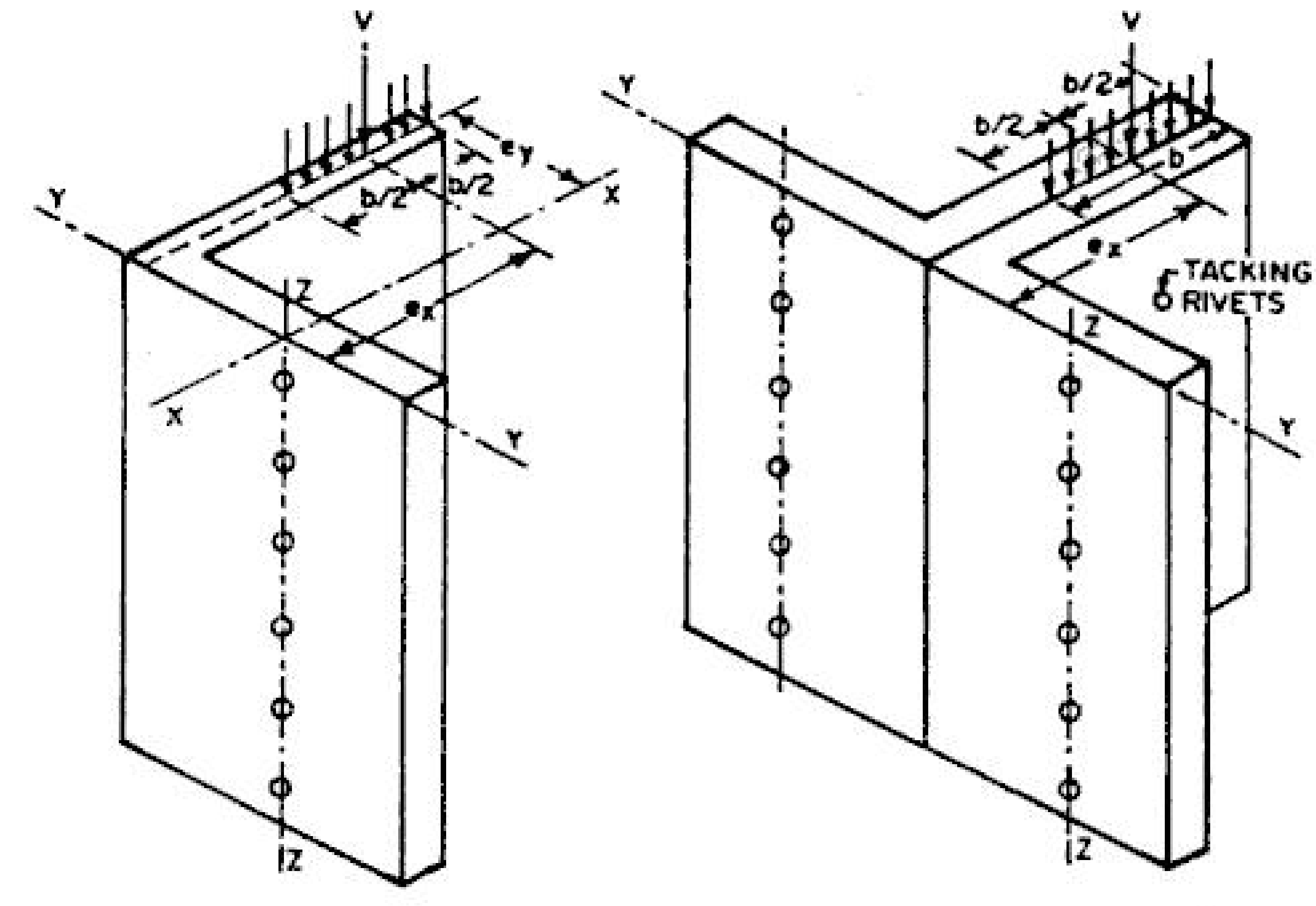
$$\therefore \qquad b = \left(\frac{V}{\sigma_p\, t_w} - h_2\sqrt{3}\right) \quad \nless \frac{1}{2}B \qquad \text{...(12.10)}$$

Knowing b, and providing an end clearance of 12 mm, the size of the horizontal seat angle is found. The width of outstanding leg of the stiffening angle, which is kept slightly less than that of the seat angle is thus known.

$$\text{The bearing area of stiffening leg} = \frac{V}{\sigma_p} \qquad \text{...(12.13)}$$

Hence the total thickness of the outstanding leg of the stiffening angle is found. The thickness of the stiffening leg should not be less than the web thickness of the supporting beam. Also, to avoid local buckling, the ratio of the *outstand* to its thickness should not exceed 16. The length of the stiffening angle will depend upon the space required for rivets on column (or supporting girder). These rivets are designed for direct shear and bending stresses.

Fig. 12.16(a) shows only one stiffener angle, where the reaction V has an eccentricity e_x along xx-axis and e_y along yy-axis. Due to this, the rivets are subjected to bending moment $M_y = V \,.\, e_x$ and twisting moment $V \,.\, e_y$, in addition to the direct shear. The twisting moment acts in a plane parallel to Y–Z plane (*i.e.*, the plane in which the rivets are provided), and due to this the rivets are subjected to additional shear.



(a) SINGLE STIFFENER ANGLE

(b) DOUBLE STIFFNER ANGLES

FIG. 12.16.

Fig. 12.16 (b) shows two stiffener angles, suitably tack riveted along their length in vertical direction. Due to this the two stiffener angles act as one unit. The reaction V has an eccentricity e_x along the x-x axis, and due to this the rivets are subjected to bending moment $V \,.\, e_x$ in the *XZ-plane.*

Example 12.5. *A beam ISWB 600 @ 133.7 kg/m transmits an end reaction of 300 kN to the web of a stanchion ISHB 250 @ 54.7 kg/m. Design a stiffened seat connection, using double angle stiffeners.*

Solution

For ISWB 600 @ 133.7 kg/m, we have $t_w = 11.2$ mm; $h_2 = 42.9$ mm and $b_f = 250$ mm

Using $p = 75$ mm and noting that $l = 1$, we have

$$n = \sqrt{\frac{6 \times 17696}{1 \times 75 \times 43.37}} = 5.7$$

Try 8 rivets, since there is a twisting moment M_x also, in addition to the bending moment M_y.

Check for stresses in rivets

Shear force P_1 due to $P = P/n = 200/8 = 25$ kN

$$\Sigma y^2 = 2\left[(37.5)^2 + (112.5)^2 + (187.5)^2 + (262.5)^2\right] = 236250 \text{ mm}^2$$

For the extreme rivet, $y = 262.5$ mm (Fig. 12.19).

$\therefore$ Shear force P_2 due to $M_x = \dfrac{M_x \cdot y}{\Sigma y^2}$

$$= \frac{13000 \times 262.5}{236250} = 14.44 \text{ kN}$$

$\therefore$ Resultant S.F.

$$= \sqrt{P_1^2 + P_2^2} = \sqrt{(25)^2 + (14.44)^2}$$

$= 28.87$ kN. Hence,

$$\tau_{vf,cal} = \frac{28.87 \times 10^3}{A} = \frac{28.87 \times 10^3}{433.736}$$

$= 66.57 \text{ N/mm}^2$

$$\sigma_{tf,cal} = \frac{6 M_y}{l p n^2 A},$$

$$= \frac{6 \times 17696 \times 10^3}{1 \times 75 (8)^2 \times 433.736}$$

$= 51.00 \text{ N/mm}^2$

$$\therefore \quad \frac{\sigma_{tf,cal}}{\sigma_{tf}} + \frac{\tau_{vf,cal}}{\tau_{vf}} = \frac{51.00}{100} + \frac{66.57}{100}$$

$= 1.176 < 1.4.$ Hence OK.

Provide top cleat of size ISA 100 × 75 × 8 mm with its 100 mm leg horizontal and joint it with 2 rivets of 22 mm dia. on each leg. The details of the connection are shown in Fig. 12.20.

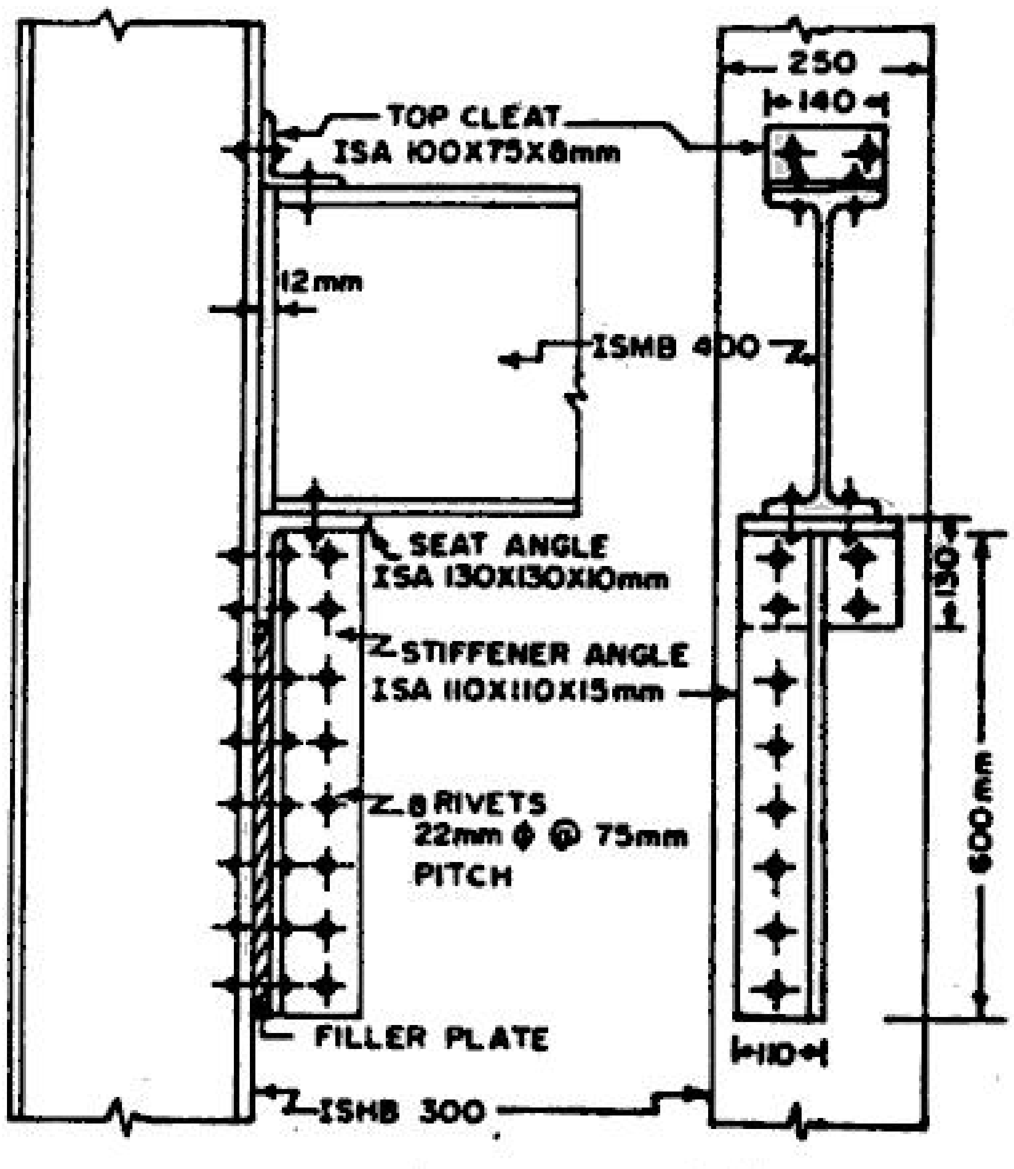


FIG. 12.20.

12.6. MOMENT RESISTANT CONNECTIONS

Upto this stage, we have discussed simple connections which transfer shear only, and permit full rotation of the beam. Some times, it may be required to transmit moments also, in addition to shears, such as in building frames. As discussed earlier, there are two types of constructions which permit the transfer of moments, either fully or partially :

(*i*) Rigid construction and (*ii*) Semi-rigid construction.

In both the above types of construction, the connections or joints are so designed that they permit the transfer of moment in addition to shear. Based on the magnitude of moment to be transferred, these may be *two types* of connections :

1. Small moment resistant connections 2. Large moment resistant connections

We shall discuss both the types of connections separately in a greater detail.

The permissible tensile force in rivet $= P_{max}$

Permissible shear force in rivet = strength rivet in single shear $= P_{sp}$.

Hence the following interaction should be satisfied :

$$\frac{P_t}{P_{max}} + \frac{P_s}{P_{sp}} \leq 1.4 \qquad ...(12.20)$$

(iii) **Design of rivets along *BB*** : The resultant horizontal force (H) in the rivets A_1A_1, situated above the top flange of the beam (*i.e.*, in the top bracket) acts at some distance e above the rivet line *BB*. Hence the rivets along *BB* are subjected to a B.M. $M_b = e\,H$, acting in their own plane. The number or rivets (n_b) is found from Eq. 3.20 for this B.M. (M_b) taking $l = 1$:

$$n_b = \sqrt{\frac{6\,M_b}{p\,R}} \qquad ...(12.21)$$

where R is the rivet value for these rivets.

Actual force in rivets due to M_b is

$$F_b = \frac{M_b \,.\, y_{max}}{\Sigma y^2} \qquad ...(12.22\ a)$$

Actual force in rivets due horizontal shear force H is

$$F_s = \frac{H}{n_b} \qquad ...(12.22\ b)$$

∴ Resultant force $F = \sqrt{F_b^2 + F_s^2}$ ...(12.22)

which should be less than the rivet value R, for these rivets.

(iii) **Design of rivets along *CC*** : The rivets along line *CC*, connecting flange angles to the flanges of the beam, are provided in two rows (one row for each angle to the either side). The rivets along lines *CC* are subjected to (*i*) horizontal force H (as for the rivets along *BB*) and (*ii*) B.M. $= H\,.\,e'$, acting in a plane perpendicular to the plane of the rivet line. The number of rivets n_c is given by

$$n_c = \sqrt{\frac{6\,M}{p\,R_c}} \qquad ...(12.23)$$

where R_c is equal to the max. permissible tensile stress (P_{max}) in the rivet given by Eq. 12.18. Actual stresses in the rivets are checked in the same manner as for rivets along A_2A_2.

(iv) **Design of bracket plate** : The bracket plates (top and bottom), along with the web of the supported beam, acts as a *rectangular beam.* Hence the thickness of bracket plate is kept equal to the thickness of the web of the beam. This equivalent rectangular beam (Fig. 11.25 *b*) must be capable of resisting tensile stress due to horizontal force in the outer most rivet of line A_1A_1. If F_b is the maximum horizontal force in extreme rivet in A_1A_1, due to B.M., the maximum tensile stress in the gusset plate is

$$f_{max} = \frac{F_b}{(l - \phi)\,t} \qquad ...(12.24)$$

where l = pitch length of rivets along A_1A_1, ϕ is the rivet hole dia. and t is the thickness of gusset plate.

Depending upon the direction of applied moment at the joint, the inclined edge of one gusset plate will be subjected to tension while that of the other gusset plate will be subjected to compression. In the latter case, *lateral buckling* of the inclined edge of the gusset plate will take place. This lateral buckling may be reduced either by (*i*) providing greater thickness of the gusset plate, or (*ii*) providing stiffening angles along the inclined edge (Fig. 11.24 *a*).

Hence to have the advantage of end moments (developed by the connections) in the design of the beam, one should consider an end moment M_y/F instead of M_W as working load.

Again, curve *OB* corresponds to *rigid connection.* The beam line *I* intersects this curve at point *b*, which gives a moment which is lesser than full fixing moment $M_0\left(=\frac{wL^2}{12}\right)$. Due to this, moments will *increase* at the centre of the span, and the section at mid-span will be overstressed unless this is taken into account to start with.

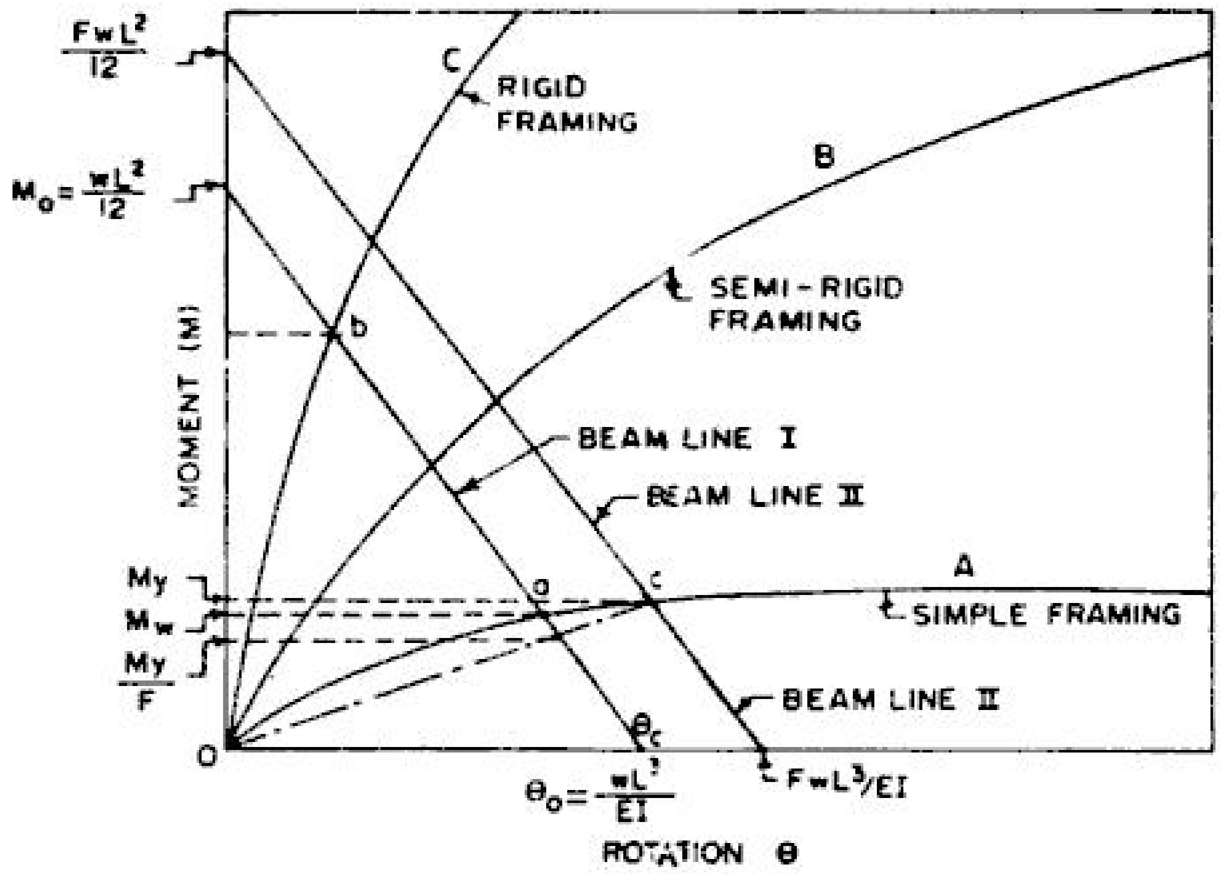


FIG. 12.31.

It is important to note that in *simple framing* (type -A connections), the mid-span moment is much bigger than the end moment while in *rigid framing*, the end moments are much bigger than the mid-span moment. In a semi-rigid connection (denoted by curve *OC*), the difference between the end moment and mid-span moment decreases. If the *M-θ* curve can be controlled at our will, we can make both the moments equal and *the beam can be designed for the minimum moment.*

PROBLEMS

1. A beam ISLB 300 @ 37.7 kg/m transmits an end reaction of 200 kN to the web of the girder ISMB 500 @ 86.9 kg/m. Design a framed connection, using power driven shop rivets for the web of ISLB 300 and hand driven field rivets for the web of ISMB 500.
2. A beam ISMB 200 @ 25.4 kg/m transmits an end reaction of 150 kN to the flange of a column ISHB 300 @ 58.8 kg/m. Design a framed connection, using power driven shop rivets.
3. Redesign the connection of problem 2, providing an unstiffened seat connection.
4. A beam ISMB 500 @ 86.9 kg/m transmits an end reaction of 320 kN to the flange of ISHB 300 @ 58.8 kg/m. Design a stiffened seat connection, using double angle stiffeners.
5. Redesign the joint of problem 4, using single angle stiffener, if the end reaction is 250 kN.
6. A beam ISLB 300 @ 37.7 kg/m transmits a vertical reaction of 100 kN and an end moment of 20 kN-m to a column ISHB 300 @ 58.8 kg/m. Design a clip angle connection.
7. A beam ISMB 400 @ 61.6 kg/m transmits a vertical reaction of 150 kN and an end moment of 40 kN-m to a column ISHB 300 @ 58.8 kg/m. Design a split beam connection.
8. A beam ISMB 500 @ 103.7 kg/m transmits an end reaction of 200 kN and an end moment of 160 kN-m to a column ISHB 400 @ 82.2 kg/m. Design a bracket connection.

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Depth of ISMB 450 is 450 mm. Providing a top tension plate,

$$\text{Force in plate} = \frac{130 \times 10^6}{450} = 288889 \text{ N.}$$

Taking value of $\sigma_{at} = 165 \text{ N/mm}^2$,

$$\text{Area of top plate} = \frac{288889}{165} = 1751 \text{ mm}^2$$

The width of the top plate should be less than the width of top flange ($b_f = 150$ mm) of the beam. Hence provide 130 mm wide top plate.

$$\therefore \quad \text{Thickness of plate} = \frac{1751}{130} = 13.5 \text{ mm.}$$

Adopt thickness = 14 mm.

Adopt fillet weld of 10 mm size. The strength of weld per lineal mm = 0.7 × 10 × 110 = 770 N/mm.

∴ Length of weld required
= 288889/770 = 375 mm.

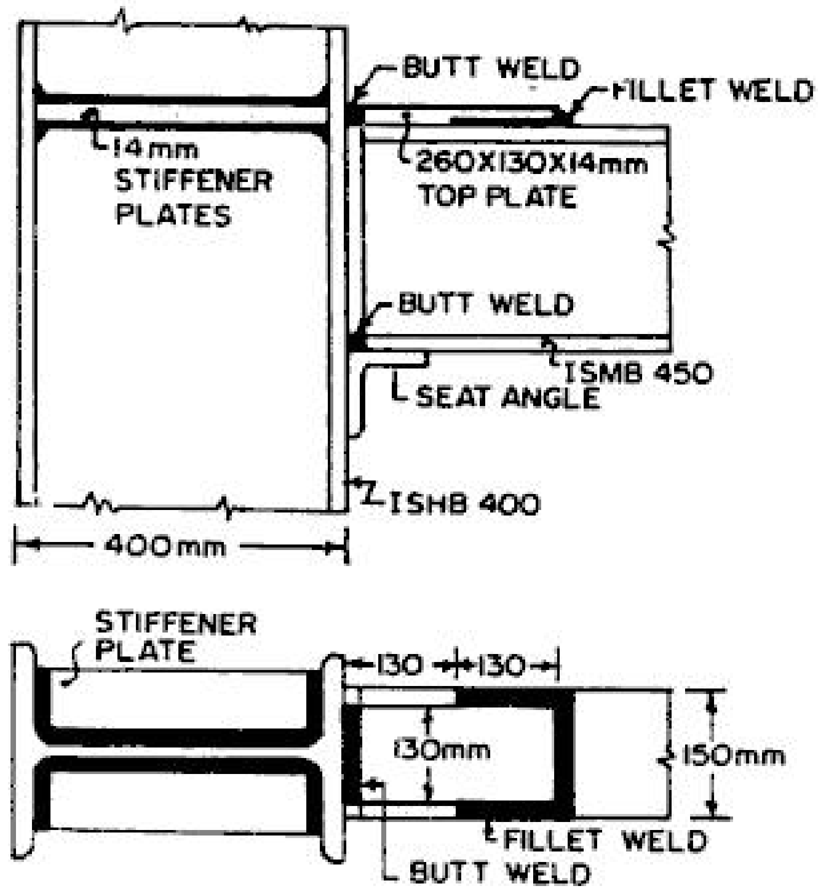

FIG. 13.13.

Providing weld length on the sides equal to width of plate (=130 mm), and also providing welds at the end, total length of weld available = 2 × 130 + 130 = 390 mm >375 mm. Hence OK. Also, keeping an unwelded length equal to the width (=130 mm) of the plate, total length of the plate = 130 + 130 = 260 mm.

Hence provide 260 mm × 130 mm × 14 mm top plate and weld it to the top flange of the beam by fillet weld. Also, connect it to the flange of the column with the help of full penetration butt weld. In order that the supporting flange of the column does not deform, provide 14 mm thick stiffener plates to both the sides of the web of the column, as shown in Fig. 13.13.

The bottom flange may be directly connected to the flange of the column and in addition, be supported on an unstiffened seat. Design of the seat angle will be the same as done in Example 13.5 for an end shear of 150 kN. The details of the connection are shown in Fig. 13.13.

PROBLEMS

1. A beam ISLB 350 @ 49.5 kg/m carries a total U.D.L. of 180 kN over a span of 6 m. Design a connection for connecting it to the flange of a column ISHB 250 @ 54.7 kg/m by providing direct fillet welds on both the sides of the web of the beam.
2. Redesign the connection for data of problem 1, using two plates.
3. Redesign the connection for data of problem 1, using framing angles.
4. A beam ISLB 350 @ 49.5 kg/m transmits an end reaction of 120 kN to the flange of a column ISHB 300 @ 58.8 kg/m. Design unstiffened welded seat connection.
5. A beam ISMB 400 @ 61.6 kg/m transimits an end reaction of 200 kN to the flange of a column ISHB 300 @ 63.0 kg/m. Design a stiffened welded seat connection.
6. A beam ISLB 500 @ 75.0 kg/m transmits an end reaction of 120 kN and a moment of 90 kN-m to the flange of a column ISHB 400 @ 82.2 kg/m. Design a suitable welded connection.

(*iii*) ***Clear depth*** (d_c) : It is the vertical distance between legs of angles of compression and tension flange.

(*iv*) ***Depth of web*** (d) : It is the depth of web plate.

Economical Depth. The weight of a plate girder consists of weight of flanges, web, stiffeners, splices etc. If the S.F. is not large, it is desirable to keep the thickness of web at its minimum specified value, and this thickness is kept constant along the span length. If the depth of web is decreased, the area of flanges will increase, though the weight of stiffeners and splices etc will decrease. Similarly, if the depth of web is increased, the area of flange will decrease, though the weight of stiffeners, splices, etc. will increase.

Let M = maximum bending moment in the girder
d = depth of web
d_e = effective depth
σ_b = allowable stress in bending
t_w = thickness of web plate
l = length of plate girder
ρ = unit mass of steel
γ_s = unit weight of steel $= \rho g$
w_s = self weight of girder per unit length.

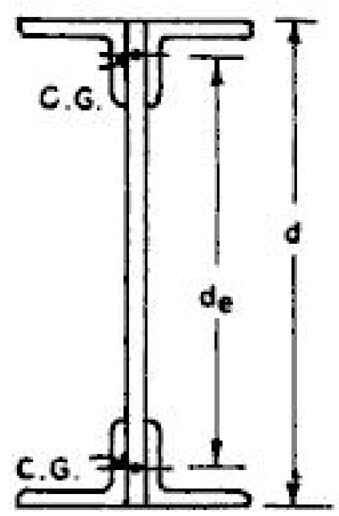

FIG. 14.5.

Area of flange (A_f) at section of max. B.M. $= \dfrac{M}{\sigma_b\, d}$ ...(14.1 *a*)

The maximum B.M. will occur only at a particular section, and B.M. will vary along the span. Hence flanges can be curtailed along the length. It can be safely assumed that the *average area* of the flange will be 80% of the flange area at the section of maximum B.M.

$\therefore$ Area of both the flanges $= 2\times 0.8\,\dfrac{M}{\sigma_b . d} = 1.6\,\dfrac{M}{\sigma_b\, d}$ ...(14.1 *b*)

This is based on the assumption that the whole of the B.M. is resisted by flanges only. If, however, contribution of web in resisting B.M. is also taken into account, we have

$$I = 2A_f\left(\frac{d_e}{2}\right)^2 + t_w\frac{d^3}{12} = A_f\frac{d_e^2}{2} + A_w\frac{d^2}{12}$$

or

$$I = A_f\frac{d^2}{2} + A_w\frac{d^2}{12} = \frac{d^2}{2}\left(A_f + \frac{A_w}{6}\right) \quad \text{(by taking } d_e \approx d\text{)} \qquad ...(14.2\ a)$$

The term $(A_f + \frac{A_w}{6})$ is known as *effective flange area* where A_w is the area of web. However, rivets are used to connect the flange angles to the web and hence net areas should be taken into account. Generally, the net area of web may be taken equal to $\frac{3}{4}$ of gross area. Hence net effective area will be $A_f' + \frac{3}{4}\times\frac{A_w}{6} = A_f' + \frac{A_w}{8}$ where A_f' is the net flange area.

Hence
$$I = \frac{d^2}{2}\left(A_f' + \frac{A_w}{8}\right) \qquad ...(14.2)$$

Now
$$\frac{M}{I} = \frac{\sigma_b}{y} \quad \text{where } y = \frac{d}{2}$$

$\therefore$
$$M = \frac{I}{y}\,\sigma_b = \left(A_f' + \frac{A_w}{8}\right) d\,\sigma_b$$

In plate girders, where fairly deep and thin web plates are used, the web may fail due to buckling. For a web plate having dimensions d and c (Fig. 14.7), the critical stress in shear (τ_{cr}) is given by

$$\tau_{cr} = k_s \frac{\pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} \qquad ...(14.16)$$

where μ = Poisson's ratio
d = smaller dimension of plate
t_w = thickness of plate
k_s = constant depending upon the *aspect ratio* c/d, where c is the larger dimension of the plate.

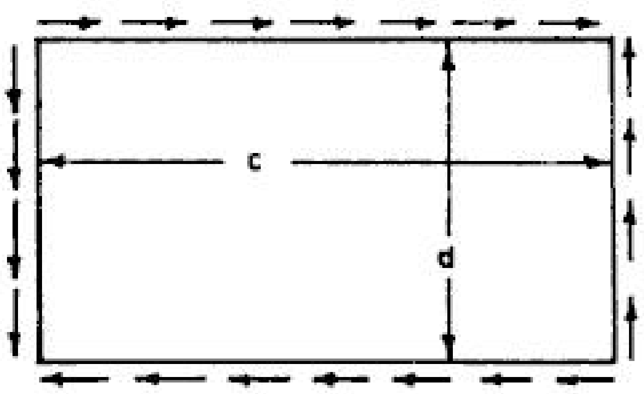

FIG. 14.7.

The value of k_s is given by the expressions :

$$k_s = 5.34 + \frac{4.0}{(c/d)^2} \quad \text{for } \frac{c}{d} > 1 \qquad ...(14.17\ a)$$

and

$$k_s = 4.0 + \frac{5.34}{(c/d)^2} \quad \text{for } \frac{c}{d} < 1 \qquad ...(14.17\ b)$$

For plate girders, the dimension c is always greater than d, and hence c/d is greater than 1. For such a case, the values of k_s for different aspect ratios are given in Table 14.1.

TABLE 14.1. VALUES OF k_s

c/d	1.0	1.2	1.4	1.6	1.8	2.0	3.0	5.0	∞
k_s	9.34	8.12	7.38	6.90	6.57	6.34	5.78	5.5	5.34

For *unstiffened web*, where c is very large in comparison to d, k_s may be taken equal to 5.34 for all practical purposes. From the above table, we find that k_s does not change materially when c/d ratio changes from 5 to ∞. For girders with unstiffened webs, taking $k_s \approx 5.34$, and for structural steel, adopting $E = 2 \times 10^5$ N/mm² and $\mu = 0.3$, Eq. 14.16 reduces to

$$\tau_{cr} \approx \frac{9.65 \times 10^5}{(d/t_w)^2} \text{ N/mm}^2 \qquad ...(14.18)$$

In the above expression, d is taken equal to the vertical distance between flange angles (for riveted plate girders), while for welded plate girders where there are no flange angles, d is taken equal to the clear distance between the flanges. Fig 14.8 shows a plot between τ_{cr} and d/t_w ratio. It is seen that τ_{cr} reaches value of yield stress in shear (= 150 N/mm²), when

$$\frac{d}{t_w} = \sqrt{\frac{9.65 \times 10^5}{150}} \approx 80 \qquad ...(14.19)$$

Hence it can be concluded that failure will occurs by buckling when $d/t_w > 80$. For rolled sections, d/t_w is always kept less than 60, and hence web buckling due to shear does

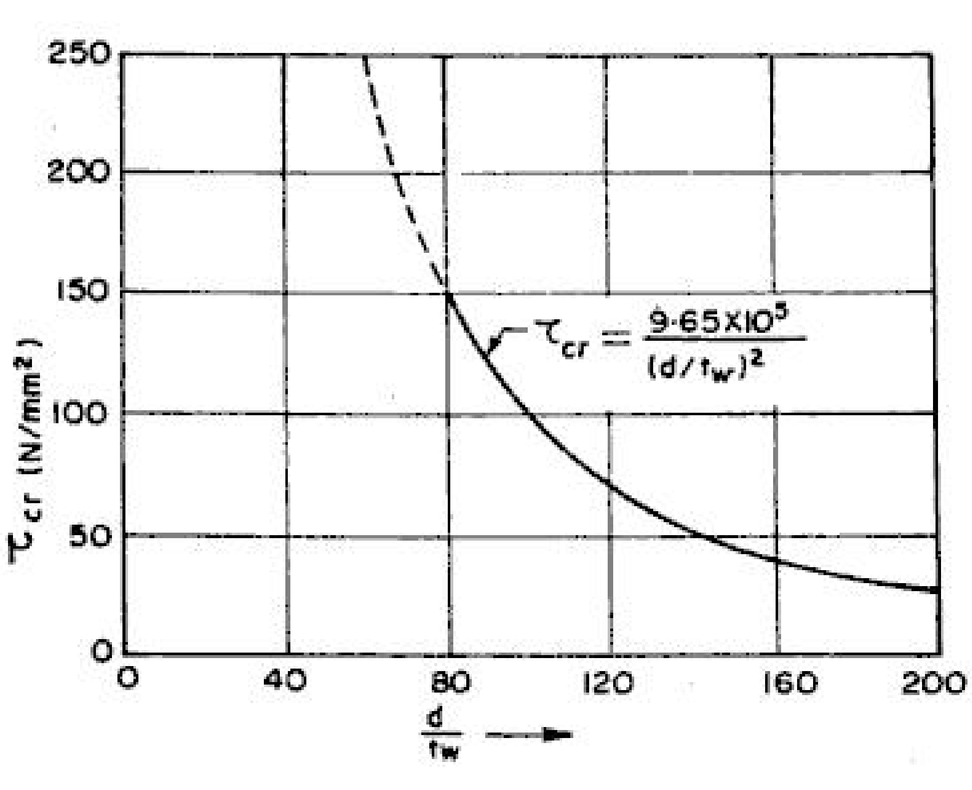

FIG. 14.8.

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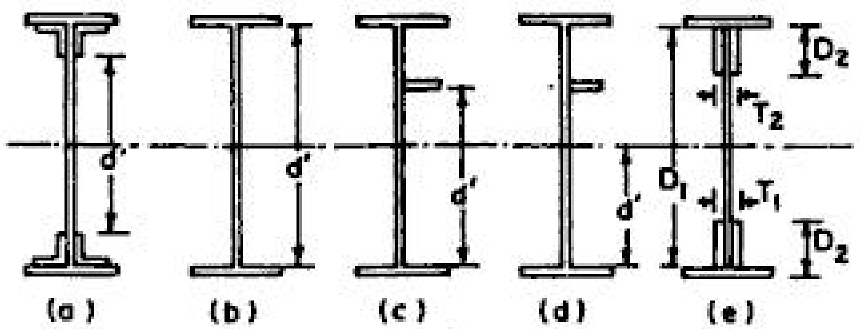
TABLE 14.4. PERMISSIBLE AVERAGE SHEAR STRESS τ_{va} IN STIFFENED WEBS OF STEEL WITH $f_y = 340\ N/mm^2$

d'/t ↓	Stress τ_{va} for different distances c between stiffeners												
	0.3 d'	0.4 d'	0.5 d'	0.6 d'	0.7 d'	0.8 d'	0.9 d'	1.0 d'	1.1 d'	1.2 d'	1.3 d'	1.4 d'	1.5 d'
75	136	136	136	136	136	136	136	136	136	136	136	136	136
80	136	136	136	136	136	136	136	136	136	136	136	136	136
85	136	136	136	136	136	136	136	136	136	136	136	134	133
90	136	136	136	136	136	136	136	136	136	135	133	132	131
95	136	136	136	136	136	136	136	136	135	133	131	129	128
100	136	136	136	136	136	136	136	135	132	130	128	127	126
105	136	136	136	136	136	136	135	133	130	128	126	124	123
110	136	136	136	136	136	135	133	131	128	126	124	122	120
115	136	136	136	136	136	133	131	129	126	123	121	119	118
120	136	136	136	136	135	131	129	127	124	121	119	117	115
125	136	136	136	136	133	129	127	125	121	119	116	114	113
130	136	136	136	135	131	127	125	122	119	116	114	112	110
135	136	136	136	134	129	126	123	120	117	114	111	109	108
140	136	136	136	132	127	124	121	118	115	112	109	107	105
150	136	136	135	129	124	120	117	114	110	107	104	102	100
160	136	136	132	126	120	116	113	110	106	102	99	97	95
170	136	136	129	123	117	112	109	106	101	98	95	92	90
180	136	135	127	119	113	108	105	102	97	93	90	87	84
190	136	133	124	116	110	105	100						
200	136	130	121	113	106	101	96						
210	136	128	118	110	103	97							
220	136	126	116	107	99	93							
230	135	123	113	103	96								
240	134	121	110	100	92								
250	132	119	107	97	89								
260	130	116	104	94									
270	128	114	102	91									



Note : Intermediate values may be obtained by linear interpolation.

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(c) Stiffeners shall be symmetrical about the web, where possible and at points of support shall project as nearly as practicable to the outer edges of the flanges.

(d) Load bearing stiffeners shall be provided with sufficient rivets or weld to transmit to the web the whole of the concentrated load.

(e) The end of load bearing stiffeners shall be fitted to provide a tight and uniform bearing upon the loaded flange unless welds or rivets designed to transmit the full reaction or load are provided between the flange and the stiffener. At points of support, this requirement shall apply at both flanges.

(f) Bearing stiffeners shall not be joggled and shall be solidly packed through.

(g) For plate girders, where load bearing stiffeners at supports are the sole means of providing restraint against torsion, the moment of inertia I, of the stiffener about the centre line of the web plate shall not be less than the following value :

$$I \not< \frac{D^3 T}{250} \times \frac{R}{W} \quad ...(14.45)$$

where : D = overall depth of the girder
T = maximum thickness of compression flange
R = reaction of the beam at the support and
W = total load on the girder between the supports.

In addition, the bases of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to the horizontal force acting at the bearing in a direction normal to the compression flange of the beam at the level of centroid of the flange and having a value equal to not less than 2.5 percent of the maximum force occurring in the flange.

Example 14.5. *A plate girder, simply supported over a span of 15 m carries a total uniformly distributed load of 4000 kN, inclusive of its own weight. The section of the plate girder is shown in Fig. 4.22. Design the intermediate stiffeners for the plate girder.*

Solution

Clear depth of web $= d_1 = 2600 - 2 \times 150 = 2300$ mm

d_2 = twice clear distance from N.A. to the toe of compression face flange
$= 2(1300-150) = 2300$ mm.

$\therefore \quad d_1 = d_2 = 2300$ mm.

Ratio $\frac{d_1}{t_w} = \frac{2300}{8} = 287.5 = \frac{d_2}{t_w}$

Since $\frac{d_1}{t_w} > 85$, vertical stiffeners are required.

Since $\frac{d_2}{t_w} > 250$, we will require *two* horizontal stiffeners,

one to be placed at $2d_c/5$ from comp. flange, where d_c = distance of comp. flange from N.A. Here d_c = 1150 mm. Hence provide the stiffener at $2 \times 1150/5 = 460$ mm, and the other to be placed at N.A.

Design of vertical stiffeners

With the horizontal stiffener placed at N.A.,
$d' = \frac{1}{2} \times 2300 = 1150$ mm.

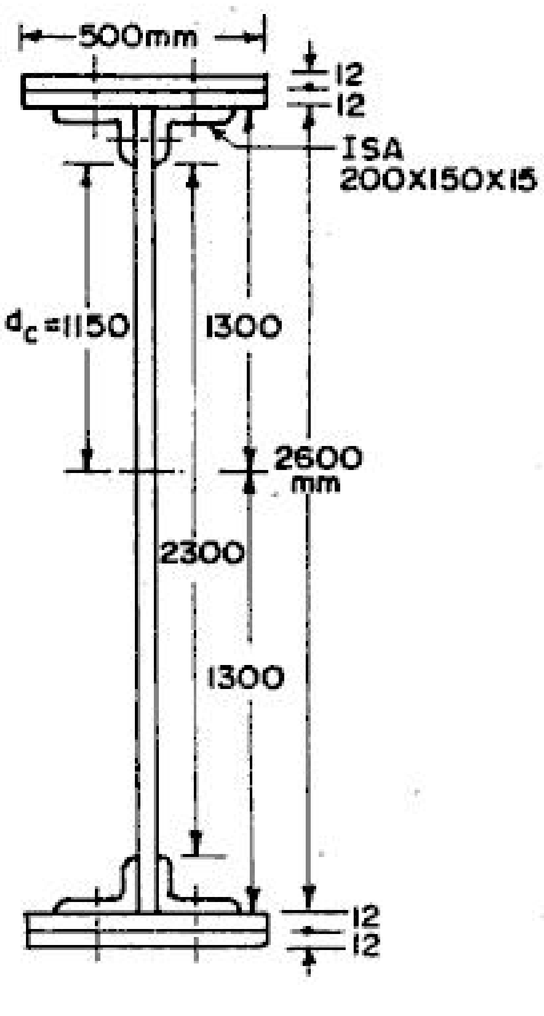

FIG. 14.22.

Splice plate A

Let p = pitch of rivets in vertical direction

n = number of rivets in one horizontal row on one side of the splice

R = rivet value

τ_v = shear stress in web $= \dfrac{V}{d \cdot t_w}$

V = shear force at the section.

f_b = bending stress in the web at the rivet line under consideration

f_{b_1} = bending stress at the level of rivets connecting flange angles with web.

y_1 = distance of level of rivets connecting flange angles with web.

Horizontal force on each rivet $= \dfrac{f_b \cdot p \cdot t_w}{n}$

Vertical force in each rivets $= \dfrac{\tau_v \cdot p \cdot t_w}{n}$

$\therefore$ Resultant force in each rivet $= \dfrac{p\, t_w}{n}\sqrt{f_b^2 + \tau_v^2} = R$

$$\therefore \quad \text{pitch } p = \frac{R \cdot n}{t_w\sqrt{f_b^2 + \tau_v^2}} \qquad \text{...(14.46 } a)$$

It is to be noted that while vertical shear stress τ_v is uniform over the whole depth, the bending stress f_b varies linearly with zero value at the N.A. to the maximum value at the top/bottom of the web plate. From Eq. 14.46 (*a*), it will be seen that p will increase towards the N.A. This will, however, be possible only if all the rivets develop equal strength (full strength). However, developed strength of a rivet is proportional to its *deformation*. Since the deformation of rivet at N.A. is minimum, the strength of rivet at N.A. is not fully developed, in comparison to the rivet at the level of connecting flange angles to the web. This suggests that pitch should be kept uniform over the depth. The pitch should be so fixed that no rivet is overstressed. Since the maximum stressed rivets are those at the level of flange angles connection with the web plate, the bending stress f_b to be substituted in Eq. 14.46 (*a*) should equal f_{b_1}. Hence

$$p = \frac{R \cdot n}{t_w\sqrt{f_{b_1}^2 + \tau_v^2}} \qquad \text{...(14.46)}$$

Splice plate B

The splice plate B is an *indirect splice* as it is not in contact with the web plate. The splice plate B provides the area of portion of web sandwiched between the flange angles. The rivets connecting the splice plate to flange angles have to perform dual function: (*i*) they transmit the horizontal force in that portion of web due to moment and (*ii*) they resist the horizontal shear between the flange angles and the web.

Let us consider the splice located to the left of the midspan of the plate girder. In such a case, since the B.M. is more towards the right of the splice, the flange stresses will also be greater on the right of the splice than on the left. Let the forces developed by web plate and flange angles at the splice point be P_2 and F_2 respectively, and those at at the left and right of the splice point be (P_1, F_1) and (P_3, F_3) respectively, as shown in Fig. 14.27.

The force P_2 developed in the web-plate, if it were not cut, is now being developed by the two splice plates B. Since the moment is increasing towards the right (*i.e.* towards the

This type of splice plates are designed to take full moment and shear carried by the web at that location.

As found in Example 14.6, $M_w = 251.4$ kN-m

Keeping 5 mm clearance, height of splice plate, $h_s = 1180 - 2 \times 5 = 1170$ mm.

The thickness of splice plate is given by Eq. 14.54:

$$t_s = \frac{3M_w}{h_s^2 . f_b}$$

where $$f_b = \frac{M}{I} . \frac{h_s}{2} = \frac{1600 \times 10^6}{1455567 \times 10^4} \times \frac{1170}{2}$$

$$= 64.3 \text{ N/mm}^2$$

$$\therefore \quad t_s = \frac{3 \times 251.4 \times 10^6}{(1170)^2 \times 64.3} = 8.57 \text{ mm}$$

Provide 10 mm thick plates on either side of the web. Let us provide 20 mm nominal dia. power driven rivets, at a pitch of 70 mm. Rivet value $R = 64500$ N, as found in the previous example. No. of rivets in height h_s (= 1170 mm) is

$$n = \frac{h_s}{p} = \frac{1170}{70} = 16.7 \text{ or } 16 \text{ rivets (say)}$$

Actual pitch available, keeping edge distance equal to half the pitch $= \frac{1170}{16} = 73.1$ mm. However, keep $p = 74$ mm, so that available edge distance $= \frac{1}{2}[1170 - 74 \times 15] = 30$ mm. Hence No. of vertical rows of rivets is given by Eq. 14.55 :

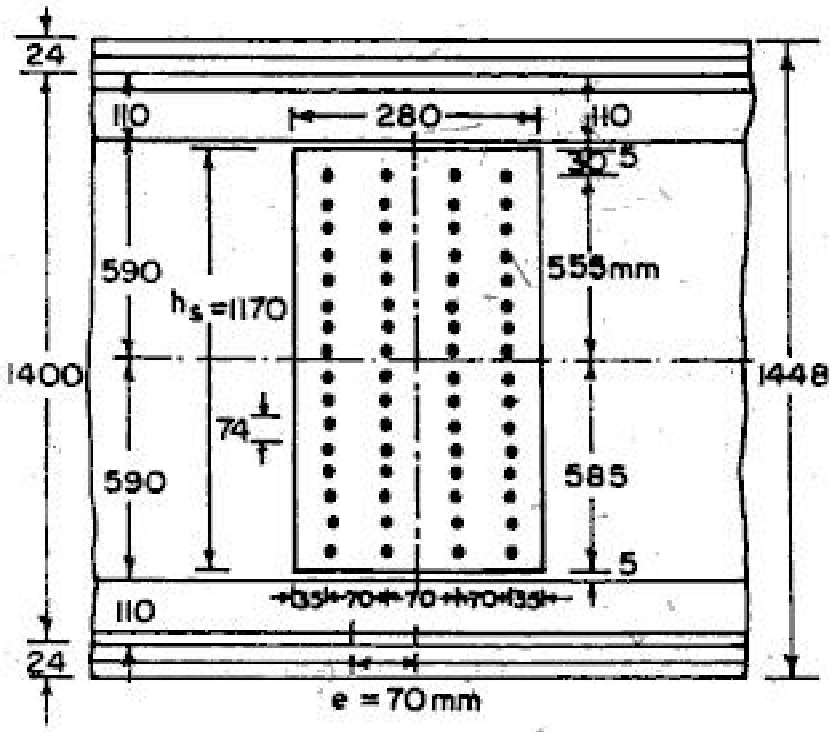

FIG. 14.33.

$$l = \frac{6 M_w}{n^2 p R} = \frac{6 \times 251.4 \times 10^6}{(16)^2 \times 74 \times 64500} = 1.23$$

Hence provide 2 rows of rivets. Total rivets to each side of splice line= 16 × 2= 32, as shown in Fig. 14.33. Keeping distance between vertical rows equal to 70 mm and edge distance equal to 35 mm, length of plate= (70 × 3) + (35 + 35) = 280 mm.

Let us now check the rivet group for resultant force in the highly stressed rivet A. Distance e of $c.g.$ of rivet group from the splice line= 70 mm.

$$\therefore \quad M_s = V . e + M_w = 200 \times 10^3 \times 70 + 251.4 \times 10^6$$

$$= 14 \times 10^6 + 251.4 \times 10^6 = 265.4 \times 10^6 \text{ N-mm}$$

$$\Sigma y^2 = 4[(37)^2 + (111)^2 + (185)^2 + (259)^2 + (333)^2 + (407)^2 + (481)^2 + (555)^2]$$

$$= 3723680 \text{ mm}^2$$

$$\Sigma x^2 = 32\,(35)^2 = 39200 \text{ mm}^2$$

$$\therefore \quad \Sigma r^2 = 39200 + 3723680 = 3762880 \text{ mm}^2$$

For farthest rivet A, $y_{max} = 555$ mm and $x_{max} = 35$ mm

$$\therefore \quad P_H = \frac{M_s}{\Sigma r^2} y_{max} = \frac{265.4 \times 10^6}{3762880} \times 555 = 39145 \text{ N}$$

$$P_V = \frac{M_s}{\Sigma r^2} . x_{max} + \frac{V}{N} = \frac{265.4 \times 10^6}{3762880} \times 35 + \frac{200 \times 10^3}{32} = 8719 \text{ N}$$

Maximum allowable pitch $= 32\,t = 32 \times 6 = 192$ mm.

Hence adopt a pitch of 170 mm. No filler plate will be used, and the stiffener will be joggled over the vertical legs of the flange angles.

Step 8. Bearing stiffeners under concentrated loads

There are two concentrated loads of 400 kN each. Hence bearing stiffeners will be provided under the concentrated loads, at both the locations.

$$\text{Allowable bearing stress} = 0.75 f_y = 0.75 \times 250 = 187.5 \text{ N/mm}^2$$

$$\therefore \quad \text{Bearing area required} = \frac{400 \times 1000}{187.5} = 2133.3 \text{ mm}^2$$

Try 2 Nos. ISA 130 × 130 × 10 mm @ 19.7 kg/m as shown in Fig. 14.43 (*a*). The connected legs of the bearing angles will be champhered to clear the fillets of flange angle and only the outstanding legs of the two angles will give bearing area (Fig. 14.43 *b*). Filler plates of 10 mm thickness equal to the thickness of flange angle will be provided. Radius at root of flange angle is $r_1 = 13.5$ mm. Hence available bearing length of the outstanding leg of the angle $= 130 - 13.5 = 116.5$ mm.

$$\therefore \quad \text{Bearing area provided} = 2 \times 116.5 \times 10 = 2330 \text{ mm}^2 > 2133.3 \text{ mm}^2$$

Let us now check the bearing angle as a column.

$$\text{Actual length} = 1700 - 2 \times 10 = 1680 \text{ mm}$$

$$\therefore \quad \text{Effective length } l = 0.7 \times 1680 = 1176 \text{ mm.}$$

For the ISA 130 × 130 × 10 mm, $I_{xx} = 402.7 \times 10^4 \text{ mm}^4$, $a = 2506 \text{ mm}^2$, $C_{xx} = 35.8$ mm

A length of web, equal to $20 \times t_w = 20 \times 10 = 200$ mm on each side of the c.g. of the stiffener will be considered with the area of the stiffener. Hence the moment of inertia (I_{xx}) about the centre line of web plate is given by

$$I_{xx} = 2\,[402.7 \times 10^4 + 2506\,(35.8 + 10 + 5)^2] + \frac{1}{12} \times 400\,(10)^3 = 2102 \times 10^4 \text{ mm}^4$$

$$\text{Cross-sectional area of column} = 2 \times 2506 + (200 + 200)\,10 = 9012 \text{ mm}^2$$

$$r = \sqrt{\frac{2102 \times 10^4}{9012}} = 48.3 \text{ mm}$$

$$\therefore \quad \frac{l}{r} = \frac{1176}{48.3} \approx 24.4$$

Corresponding $l/r = 24.4$, we get

$$\sigma_{ac} = 146.68 \text{ N/mm}^2$$

∴ Area required

$$= \frac{400 \times 10^3}{146.68} = 2727 \text{ mm}^2$$

actual *area provided* = 9012 mm².
Hence OK.

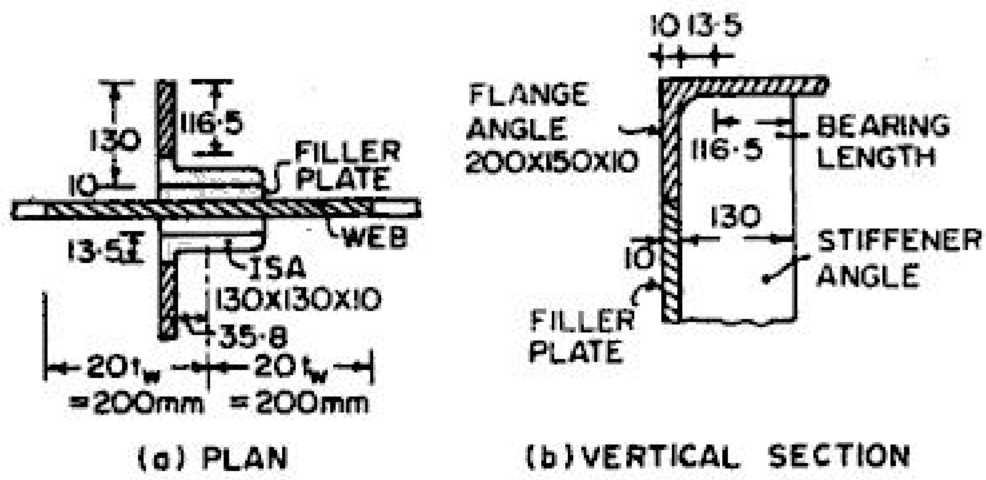

FIG. 14.43.

Design of rivets connection : Using 20 mm dia. power driven rivets, strength of each rivet in double shear $= 2 \times \frac{\pi}{4}(21.5^2 \times 100$ = 72610 N.

$$\text{Strength of rivet in bearing} = 21.5 \times 10 \times 300 = 64500 \text{ N}$$

$$\therefore \quad \text{Rivet value} = 64500 \text{ N}$$

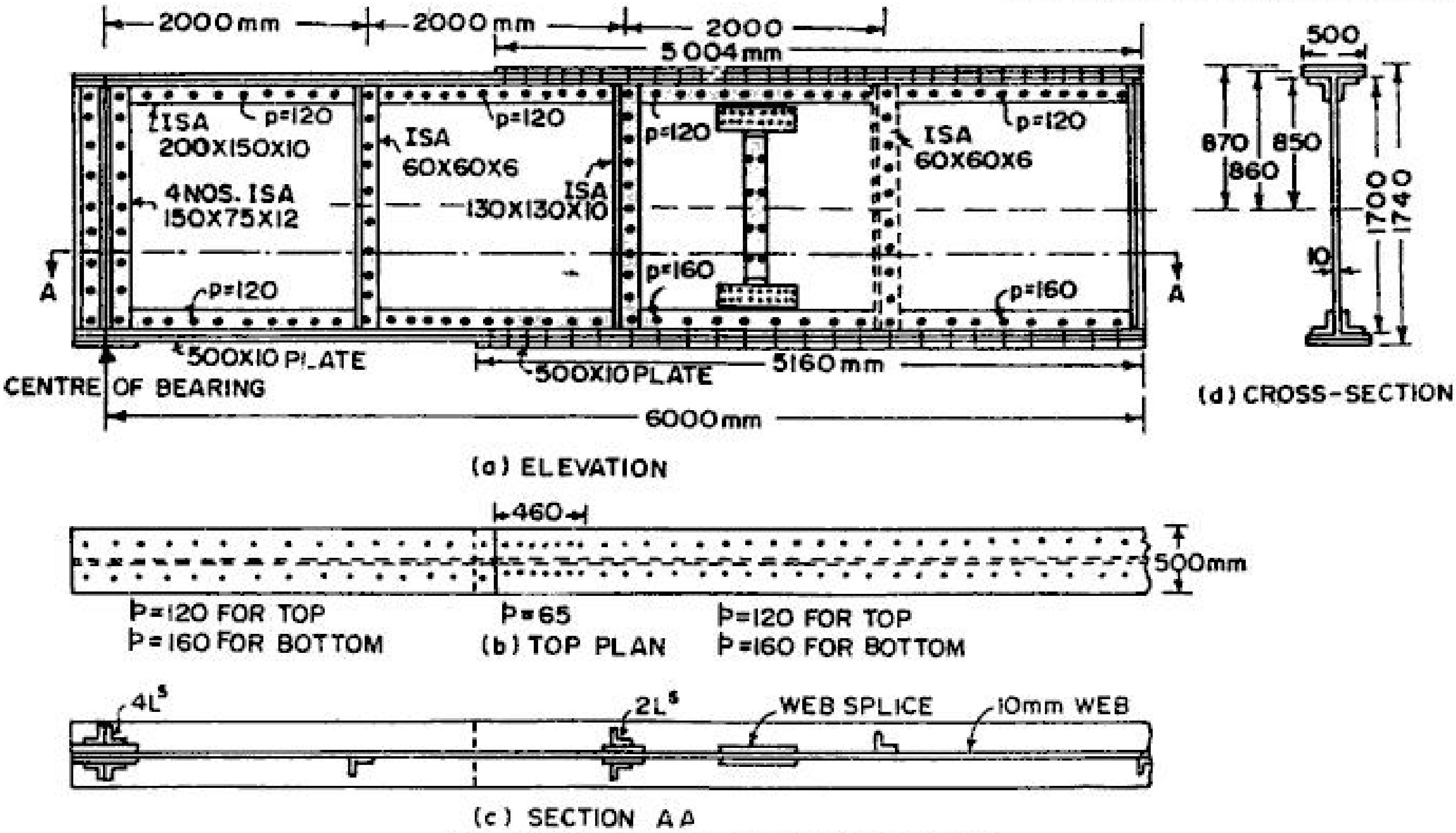

FIG. 14.47 DETAILS OF PLATE GIRDER.

14.18. DESIGN OF WELDED PLATE GIRDER

In the case of a welded plate girder, the whole of the section is effective in resisting the forces. Hence a welded plate girder is more efficient than a riveted plate girder. Beginning in 1950 when welding became more widely used (because of improved quality of welding and shop-fabricating economies resulting from increased use of automatic equipment), shop welded plate girder have gradually replaced the riveted girder.

Fig. 14.48 (*a*) shows the simplest form of plate girder, consisting of three plates welded together. Generally the flange consists of one plate of sufficient thickness. However, if plate of adequate thickness is not available, two plates can be used in each flange. Fig. 14.48 (*b*) shows another form of welded plate girder, commonly known as *hybrid girder*. The tendency now with *welded* girders is to *combine* materials of different strength. By changing the materials at different locations along the span, so that higher strength materials are available at location of higher B.M. and/or shear, a more *efficient* and *economical* girder can be obtained. Similarly, by using different materials for flanges than the web, such as in a *hybrid girder*, we get a more efficient and economical section. No flange angles are used in a welded plate girder. Other forms of plate girders are *box girder* (Fig. 14.48 *c*) and delta girder (Fig. 14.48 *d*). The box girder provides

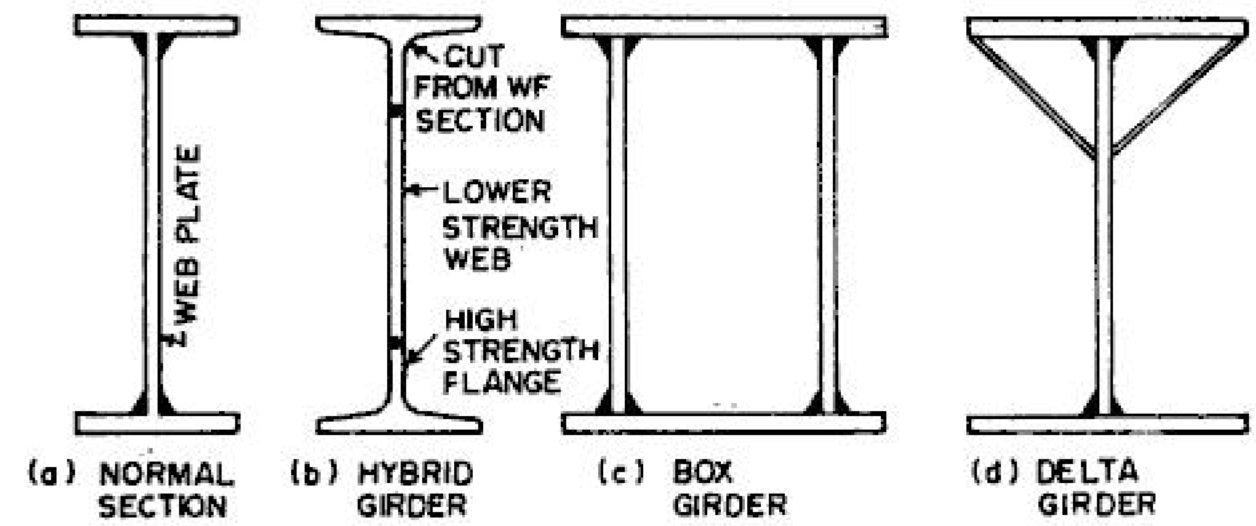

FIG. 14.48

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Step 8. Design of bearing stiffeners under point loads : Point load= 400 kN

Allowable bearing stress $= 0.75 f_y = 0.75 \times 250 = 187.5$ N/mm^2

$\therefore$ Bearing area required $= \dfrac{400 \times 1000}{187.5} = 2133.3$ mm^2

The bearing stiffener should extend, as far as possible, to the outer edges of the flanges. Let us provide the width of plate = 6 mm. Limiting b/t ratio to 16, we have

$$b / t = 16 \quad \text{...(1)}$$

Also, $$2b \times t = 2133.3 \quad \text{...(2)}$$

Hence from (1) and (2), we get $t = 8.16$ and $b = 130.6$ mm. Hence provide $t = 10$ mm. In that case $b_{max} = 16 \times t = 16 \times 10 = 160$ mm, while $b_{min} = \dfrac{2133.3}{2 \times 10} = 106.7$ mm.

Hence provide 120 × 10 mm size plate, giving b/t ratio = 12. The size of weld connecting web and flange is 6 mm. Allowing for the cutting of corners, of size 10 mm × 10 mm to clear the longi-tudial welds, bearing length = 120 − 10 = 110 mm. Hence bearing area = 2 × 110 × 10 = 2200 mm^2.

The stiffener will act as a column, and a length of web $= 20 t_w = 200$ mm will act with it on either side of the stiffener (Fig. 14.56).

Moment of inertia of the column, about the centre of the web is

$$I = 2\left[\frac{1}{12} \times 10\,(120)^3 + 10 \times 120\,(60+5)^2\right] + \frac{1}{12} \times 400\,(10)^3 = 1305.5 \times 10^4 \text{ mm}^4$$

Cross-sectional area of column = 2 × 120 × 10 + 400 × 10= 6400 mm^2

$\therefore$ $$r = \sqrt{\frac{1305.3 \times 10^4}{6400}} = 45.2 \text{ mm.}$$

Actual length of stiffener= d= 2000 mm

Effective length, l= 0.7 × 2000= 1400 mm

$\therefore$ $$\frac{l}{r} = \frac{1400}{45.2} = 31$$

Hence, $\sigma_{ac} = 144.6$ N/mm^2

$\therefore$ Area required

$$= \frac{400 \times 10^3}{144.6} = 2766.3 \text{ mm}^2 < 6400 \text{ mm}^2$$

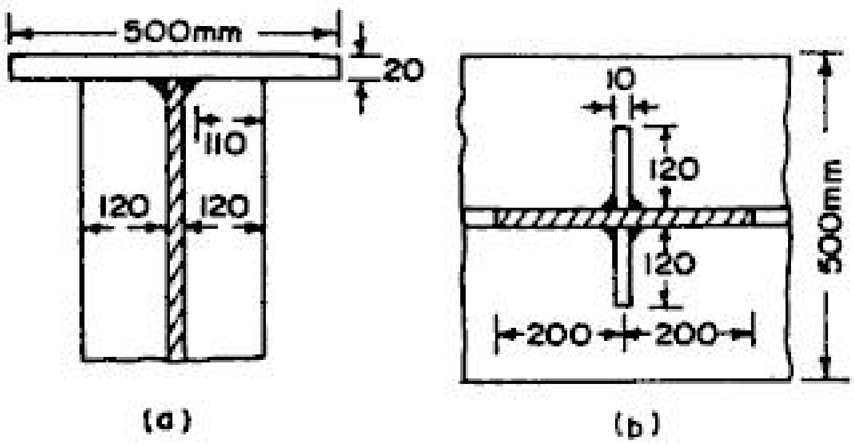

FIG. 14.56.

Available length of weld = 4 (2000 − 10)= 7960 mm

$\therefore$ Required strength of weld per mm $= \dfrac{400 \times 10^3}{7960} = 50.3$ N/mm

Size of weld $= \dfrac{50.3}{0.707 \times 110} = 0.65$ mm.

This is too small. Minimum size of weld for 10 mm plate = 3 mm. Hence use 3 mm size intermittent weld for 50% of the length, *i.e.* provide 75 mm effective lengths with clear spacing equal to 75 mm.

Step 9. Design of bearing stiffener at the ends

End shear= reaction = 1096 kN.

$\therefore$ Bearing area required $= \dfrac{1096 \times 10^3}{187.5} = 5845.3$ mm^2

Let us provide plates of size $b \times t$, on both the sides of the web.

From outstand consideration, $\dfrac{b}{t} = 16$ ...(*i*)

Let U-U, V-V = Principal centroidal axes
X-X, Y-Y = Any pair of centroidal rectangular axes
α = angle between U-U and X-X axes (Fig. 15.2).

If U-U, V-V are the principal axes, the product of inertia $\Sigma\, u\,.\,v\,.\,\delta a = 0$, where δa is an elementary area with co-ordinates u and v referred to the principal axes. If a plane area has an axis of symmetry, it is obviously a principal axis, since the axis of symmetry has to satisfy the condition $\Sigma\, u\, v\, \delta a = 0$ about it. In general, however, a plane area may not have any axis of symmetry. In that case the principal axes may be located provided its properties about any pair of rectangular axes X-X, Y-Y are known.

Let x, y be the co-ordinates of an elementary area δa, with respect to the X-Y axes, and u, v be the corresponding co-ordinates with respect to the principal axes U-V.

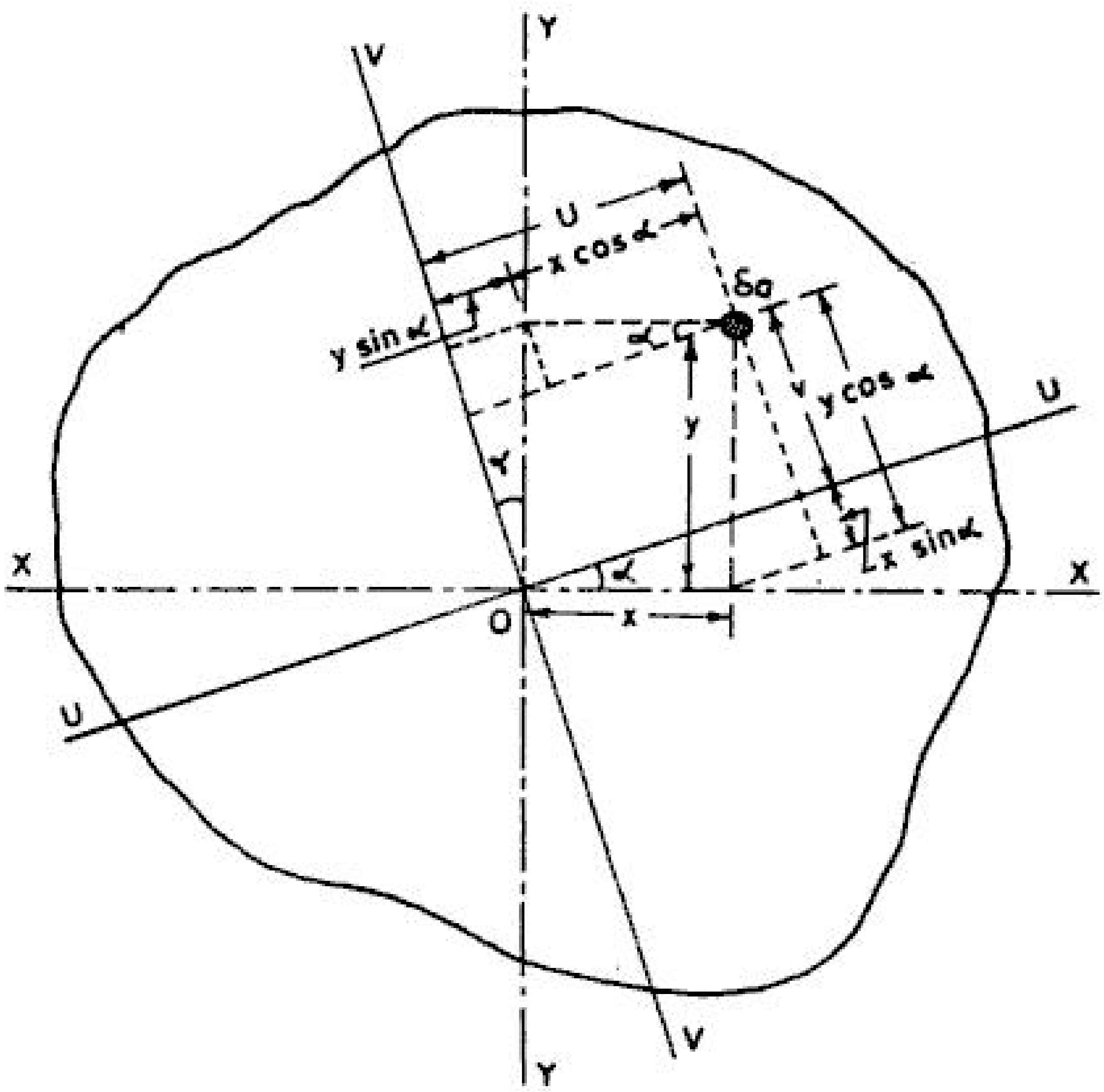

FIG. 15.2. PRINCIPAL AXES.

By definition, $I_{XX} = \Sigma y^2 \delta a;\ I_{YY} = \Sigma x^2 \delta a;\ I_{XY} = \Sigma x y \delta a$

Similarly, $I_{UU} = \Sigma v^2 \delta a;\ I_{VV} = \Sigma u^2 \delta a;\ I_{UV} = \Sigma uv\, \delta a$

The relationships between x, y and u, v co-ordinates are

$$u = x \cos\alpha + y \sin\alpha$$

and

$$v = y \cos\alpha - x \sin\alpha.$$

Hence

$$I_{UU} = \Sigma v^2 . \delta a = \Sigma\,(y\cos\alpha - x\sin\alpha)^2\,\delta a$$
$$= \cos^2\alpha\, \Sigma y^2 \delta a + \sin^2\alpha\, \Sigma x^2 \delta a - 2\sin\alpha\cos\alpha\, \Sigma xy\,\delta a$$
$$= I_{XX}\cos^2\alpha + I_{YY}\sin^2\alpha - I_{XY}\sin 2\alpha \qquad ...(15.1)$$

$$I_{VV} = \Sigma u^2 \delta a = \Sigma\,(x\cos\alpha + y\sin\alpha)^2\,\delta a$$
$$= \sin^2\alpha\, \Sigma y^2 \delta a + \cos^2\alpha\, \Sigma x^2 \delta a + 2\sin\alpha\cos\alpha\, \Sigma x y\,\delta a$$
$$= I_{XX}\sin^2\alpha + I_{YY}\cos^2\alpha + I_{XY}\sin 2\alpha \qquad ...(15.2)$$

and

$$I_{UV} = \Sigma uv\,\delta a = \Sigma\,(x\cos\alpha + y\sin\alpha)\,(y\cos\alpha - x\sin\alpha)\,\delta a$$
$$= \cos^2\alpha\, \Sigma x y\,\delta a - \sin^2\alpha\, \Sigma x y\,\delta a + \sin\alpha\cos\alpha\,(\Sigma y^2 \delta a - \Sigma x^2 \delta a)$$
$$= \cos^2\alpha\,.\,I_{XY} - \sin^2\alpha\,.\,I_{XY} + \sin\alpha\cos\alpha\,(I_{XX} - I_{YY})$$
$$= \left(\frac{I_{XX} - I_{YY}}{2}\right)\sin 2\alpha + I_{XY}\cos 2\alpha \qquad ...(15.3)$$

Example 15.1. *Determine the principal moments of inertia for an unequal angle section 60 × 40 × 6 mm shown in Fig. 15.6.*

Solution. Let O be the centroid of the section. Let the X-axis be at a distance C_X from face PQ, and Y-axis be at a distance C_Y from face PR.

Area $A = A_1 + A_2 = (40 \times 6) + (54 \times 6) = 240 + 324 = 564 \text{ mm}^2$

$$C_X = \frac{(40 \times 6 \times 30) + (54 \times 6 \times 33)}{564} = 20.2 \text{ mm}$$

$$C_Y = \frac{(40 \times 6 \times 20) + (54 \times 6 \times 3)}{564} = 10.2 \text{ mm}$$

$$I_{PQ} = (\tfrac{1}{3} \times 6 \times 60^3) + (\tfrac{1}{3} \times 34 \times 6^3) = 43.33 \times 10^4 \text{ mm}^4$$

$$I_{XX} = I_{PQ} - A \cdot C_X^2 = 43.44 \times 10^4 - 564\,(20.2)^2 = 20.34 \times 10^4 \text{ mm}^4$$

$$I_{PR} = (\tfrac{1}{3} \times 56 \times 6^3) + (\tfrac{1}{3} \times 6 \times 40^3) = 13.19 \times 10^4 \text{ mm}^4$$

$$I_{YY} = I_{PR} - A \cdot C_Y^2 = 13.19 \times 10^4 - 564\,(10.2)^2 = 7.33 \times 10^4 \text{ mm}^4$$

and $I_{XY} = A_1 . x_1 . y_1 + A_2 . x_2 . y_2$

where (x_1, y_1) are the co-ordinates of C.G. of area A_1

and (x_2, y_2) are the co-ordinates of C.G. of area A_2.

$$\therefore \quad I_{XY} = [240\,(20 - 10.2)\,(3 - 20.2)] + [324\,(33 - 20.2)\,(3 - 10.2)]$$

$$= -4.05 \times 10^4 - 2.99 \times 10^4 = -7.04 \times 10^4 \text{ mm}^4$$

From Fig. 15.4 the positions of principal axes are given by

$$\tan 2\alpha = \frac{2 I_{XY}}{I_{XX} - I_{YY}} = \frac{2 \times 7.04 \times 10^4}{(20.34 - 7.33)\,10^4} = 1.085$$

$$\therefore \quad 2\alpha = 47°20'$$

or $\alpha = 23°40'$ (anticlockwise)

$$\frac{I_{XX} + I_{YY}}{2} = \frac{(20.34 + 7.33)\,10^4}{2} = 13.84 \times 10^4$$

$$\frac{I_{XX} - I_{YY}}{2} = \frac{(20.34 - 7.33)\,10^4}{2} = 6.5 \times 10^4$$

$$\sqrt{\left(\frac{I_{XX} - I_{YY}}{2}\right)^2 - I_{XY}^2}$$

$$= \sqrt{(6.5 \times 10^4)^2 + (-7.04 \times 10^4)^2}$$

$$= 9.58 \times 10^4$$

Hence from Eqs. 15.5 and 15.6

$$I_{UU} = 13.84 \times 10^4 + 9.58 \times 10^4$$
$$= 23.42 \times 10^4 \text{ mm}^4$$
$$I_{VV} = 13.84 \times 10^4 - 9.58 \times 10^4$$
$$= 4.26 \times 10^4 \text{ mm}^2$$

Check $I_{XX} + I_{YY} = I_{UU} + I_{VV}$

$$(20.34 \times 10^4 + 7.33 \times 10^4)$$
$$= (23.42 \times 10^4 + 4.26 \times 10^4)$$
$$27.67 \times 10^4 \approx 27.68 \times 10^4.$$

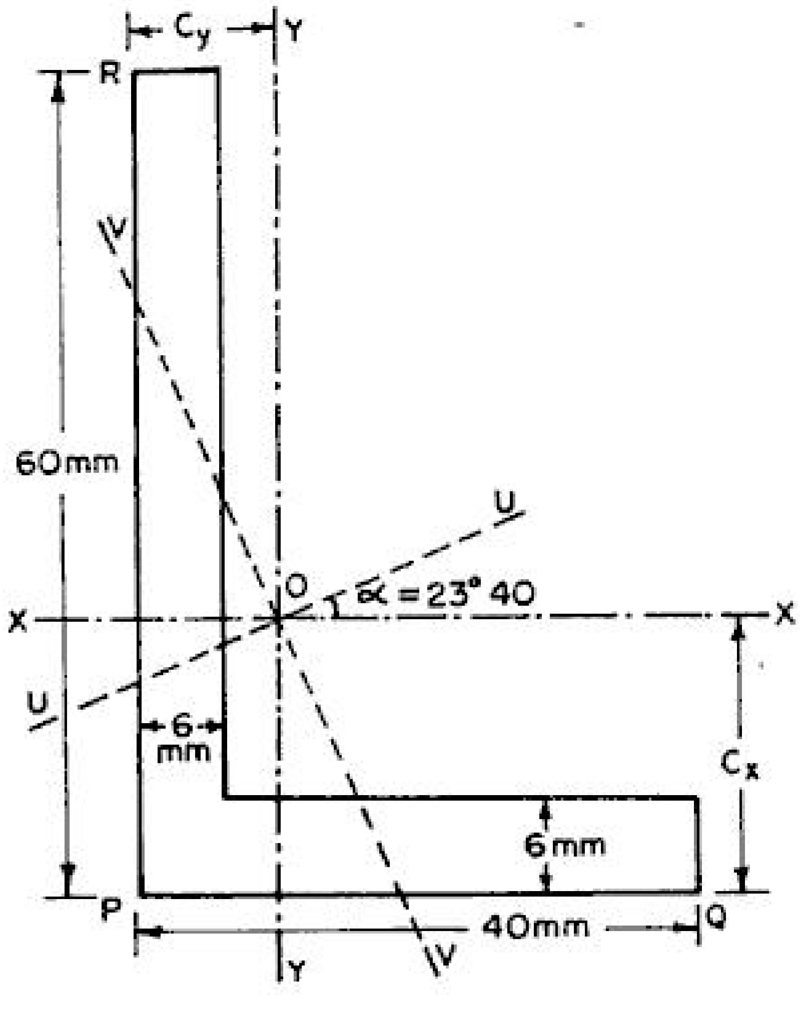

FIG. 15.6.

Example 15.5. *A 60 mm × 40 mm × 6 mm unequal angle is placed with the longer leg vertical, and is used as a beam. It is subjected to a bending moment of 12 kN-cm acting in the vertical plane through the centroid of the section. Determine the maximum bending stress induced in the section.*

Solution

We have found the properties of this section in example 15.1. For the position of the angle as shown in Fig. 15.14 (*a*), the various parameters are as follows :

$C_X = 20.2$ mm; $C_Y = 10.2$ mm

$A = 564\ \text{mm}^2$

$I_{XX} = 20.34 \times 10^4\ \text{mm}^4$;

$I_{YY} = 7.33 \times 10^4\ \text{mm}^4$;

$I_{XY} = +7.04 \times 10^4\ \text{mm}^4$

$I_{UU} = 23.42 \times 10^4\ \text{mm}^4$;

$I_{VV} = 4.26 \times 10^4\ \text{mm}^4$;

$\alpha = 23° 40'$

The plane of loading is vertical. Hence Y' axis and Y axis coincide.

$\therefore \quad \theta = \alpha = 23° 40'$.

(*a*) *Analytical Solution*

The inclination β of the neutral axis N-N with the U-U axis is given by

$$\tan\beta = \frac{I_{UU}}{I_{VV}} \tan\theta$$

$$= \frac{23.42 \times 10^6}{4.26 \times 10^6} \tan 23° 40' = 2.4$$

$$\therefore \quad \beta = 67° 24'$$

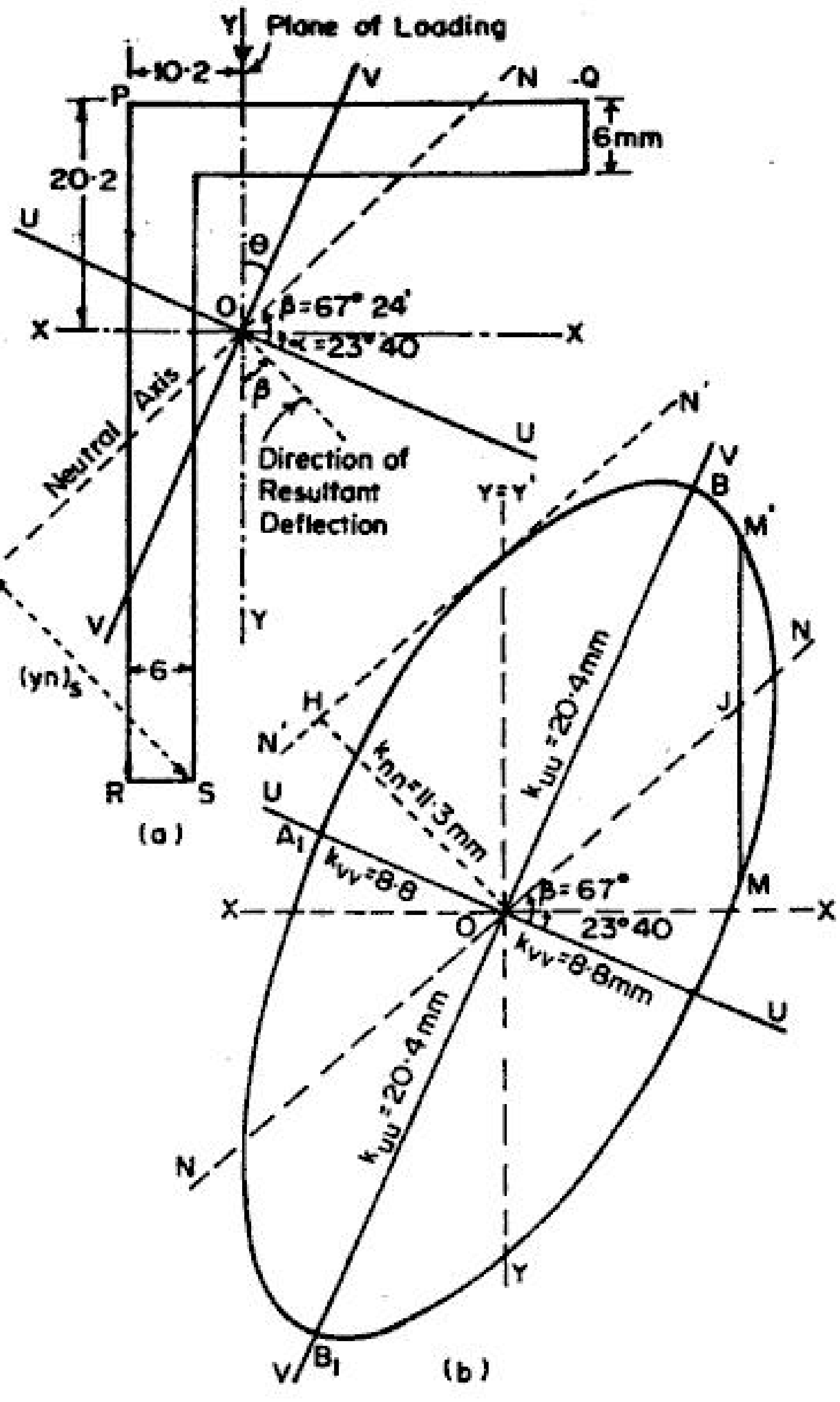

FIG. 15.14.

$$I_{NN} = I_{UU}\cos^2\beta + I_{VV}\sin^2\beta = 23.42\times10^4\cos^2 67° 24' + 4.26\times10^4 \sin^2 67° 24'$$

$$= 3.46 \times 10^4 + 3.64 \times 10^4 = 7.1 \times 10^4\ \text{mm}^4$$

Since point S is farthest from the N.A., it will be stressed maximum. The distance S from N.A. is given by

$$(y_N)_S = u \cdot \sin\beta + v\cos\beta$$

where u and v are the coordinates of point S referred to U-V axes. If (x, y) are the coordinates of S referred to x-y axes, we have

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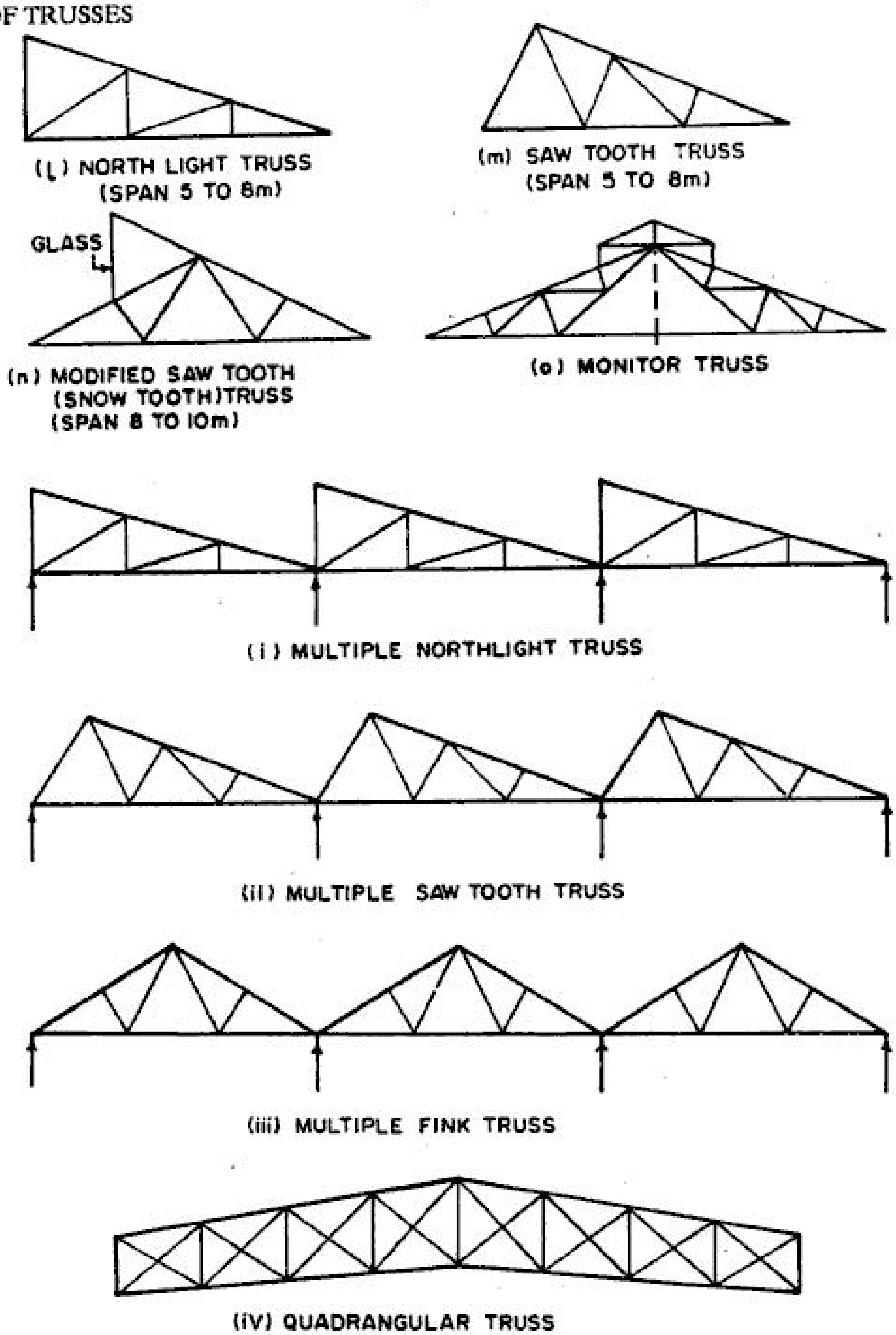

(p) TRUSSES FOR LARGE SPANS

FIG. 16.2. VARIOUS TYPES OF TRUSSES

16.3. COMPONENTS OF A ROOF TRUSS

The following are various components of a roof truss, shown in Fig. 16.3 :

(*i*) Principal rafter or Top chord

(*ii*) Bottom chord or main tie

(*iii*) Ties

(*iv*) Struts

(*v*) Sag tie

the weak axis caused by the tangential componens of the load. Generally two lines of sag rods are provided in each bay, which are connected to the *ridge purlins* (Fig. 16.4 *a,b*), and hence the ridge purlins must be designed for the reaction from the sag rods. The ridge purlins are made up of an *I*-beam or two channels, and must be stronger than other purlins.

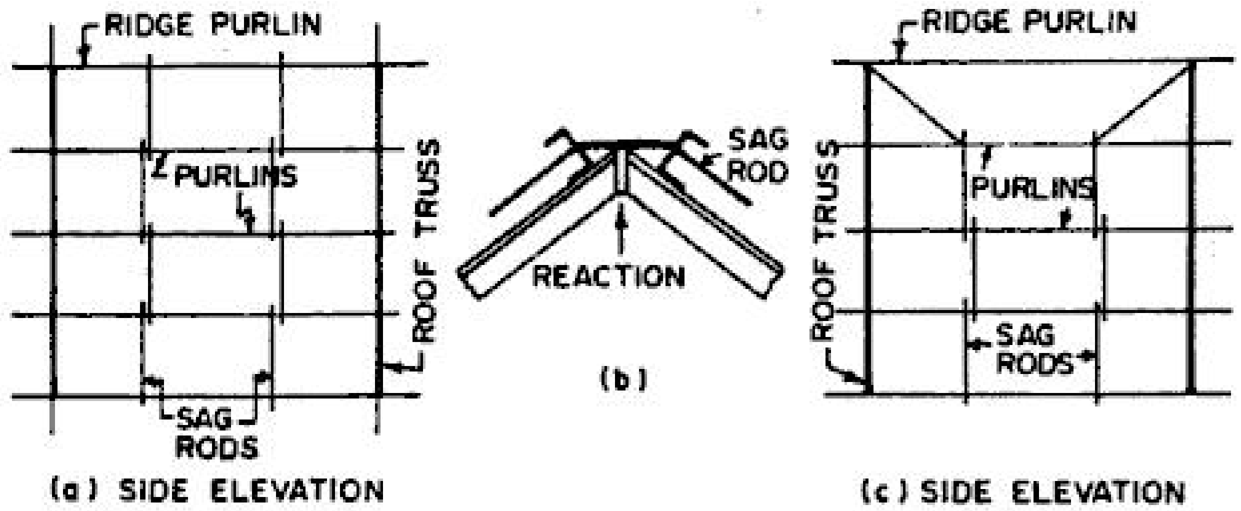

FIG. 16.4. PROVISION OF SAG RODS

Alternatively, the sag rods may be anchored on the roof trusses (Fig. 16.4 *c*). If corrugated steel covering is used, sag rods may be dispensed with, assuming that the steel covering sheets do the work of transmitting the tangential component of the load. Sag rods are made from 16 to 20 mm dia. bars with screwed ends. Nuts are tightened after the erection of purlins over the trusses but before installation of sheeting to induce some pre-tension in the sag rods. Depending upon the span of purlins, one, two or more lines of sag rods may be used. The purlins can then be analysed as *continuous beam*, for bending about the weak axis. If one line of sag rods is used, the tension T_s in the sag rods is given by

$$T_s = \left(\frac{L_1 L_2}{4\cos\theta}\right) W_g \sin\theta \qquad ...(16.2)$$

where L_1 is the span of the truss, L_2 is the spacing of the trusses, θ is the slope of trusses and W_g is the design gravity load.

16.7. LATERAL BRACING OF END TRUSSES

Bracing is required to resist horizontal loading (such as that due to wind etc.) in pin jointed buildings, including roof trusses. Bracing of roof trusses and supporting columns provide stiff rigid structure. When wind blows normal to the inclined surface of the trusses, it is efficiently resisted by all the members of the truss, and the wind forces are transferred to the supports at the ends of the truss. However, when the wind blows along the ridge, it causes large deformations in the roof if the truss is supported on columns and if strong gable walls are not provided at the ends of the shed. In such a case, it is essential to provide lateral bracings to the last two trusses, on either ends of the shed, both at top chord level as well as bottom chord levels (Fig. 16.5). Similarly, the last two

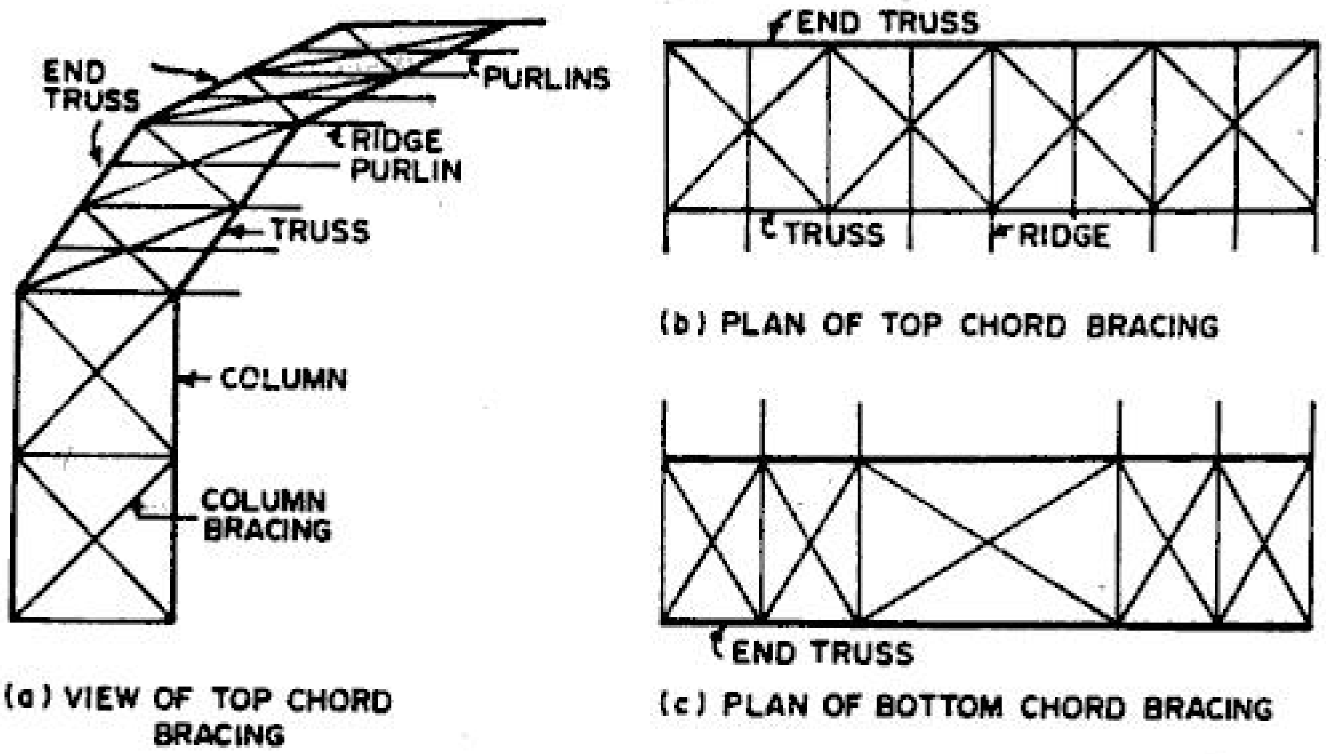

FIG. 16.5. BRACINGS OF ROOF TRUSSES

3. Snow Load. If the roof structure is situated in an area where show fall takes place, the roof will be subjected to snow load. IS 875 recommends a snow load of 2.5 N/m^2 per mm depth of snow. No snow load may be considered if the slopes are greater than 50°. The possibility of total or partial snow load should be considered, *i.e.* half the roof fully loaded with the design snow load and the other half loaded with half the design snow load.

4. Wind Load. The load due to wind is one of the most important loads to be considered in the design of roof trusses and other types of pitched roofs. A detailed discussion on wind pressure is given in chapter 2. The design wind pressure is p_z given by

$$p_z = 0.6\, V_z^2 = 0.6\,(k_1 k_2 k_3 V_b)^2$$

where V_b = basic wind speed in m/s at 10 m height (Fig. 2.1 and Table 2.6)
k_1 = Probability factor (or risk coeficient)
k_2 = Terrain, height and structure size factor
k_3 = Topography factor

The wind force F acting in a direction normal to the individual structural element or cladding unit is

$$F = (C_{pe} - C_{pi})\, A\, p_z \qquad ...(2.11)$$

where C_{pe} = external pressure coefficient (Table 2.15)
C_{pi} = internal pressure coefficient (Table 2.16)

Load combinations : The following load combinations should be tried:

(*i*) Dead load + Live load (*ii*) Dead load + Wind load

It should be clearly noted that if the main load on the purlin is due to wind, no increase in permissible stresses due to wind is allowed.

16.10. DESIGN OF PURLINS

The purlins, which support roof covering, batten or common-rafters, are supported on the principal rafters of the trusses. They may therefore, be considered as continuous beams. The purlins may be of rectangular section (timber), I-section, channel section or angle section, as shown in Fig. 16.8. Purlins are subjected to (*i*) dead load (W_d) due to self weight, roof covering, battens etc, acting in vertical direction and wind load (W_w) normal to the slope of roof. Because of the transverse component of dead load W_d, a purlin is subjected to unsymmetrical bending. If U-U and V-V are the principal centroidal axes, the maximum bending stress in the extreme fibre is given by Eq. 15.10

$$f_b = \frac{M\cos\theta}{I_{UU}}\cdot v + \frac{M\sin\theta}{I_{VV}}\, u.$$

or $$f_b = \frac{M_{UU}}{I_{UU}}\cdot v + \frac{M_{VV}}{I_{VV}}\cdot u \qquad ...(16.6)$$

For the components $M\cos\theta\ (= M_{UU})$, bending takes place about V-V for which U-U becomes the neutral axis. For the component $M\sin\theta\ (= M_{VV})$, bending takes place about U-U for which V-V becomes the neutral axis.

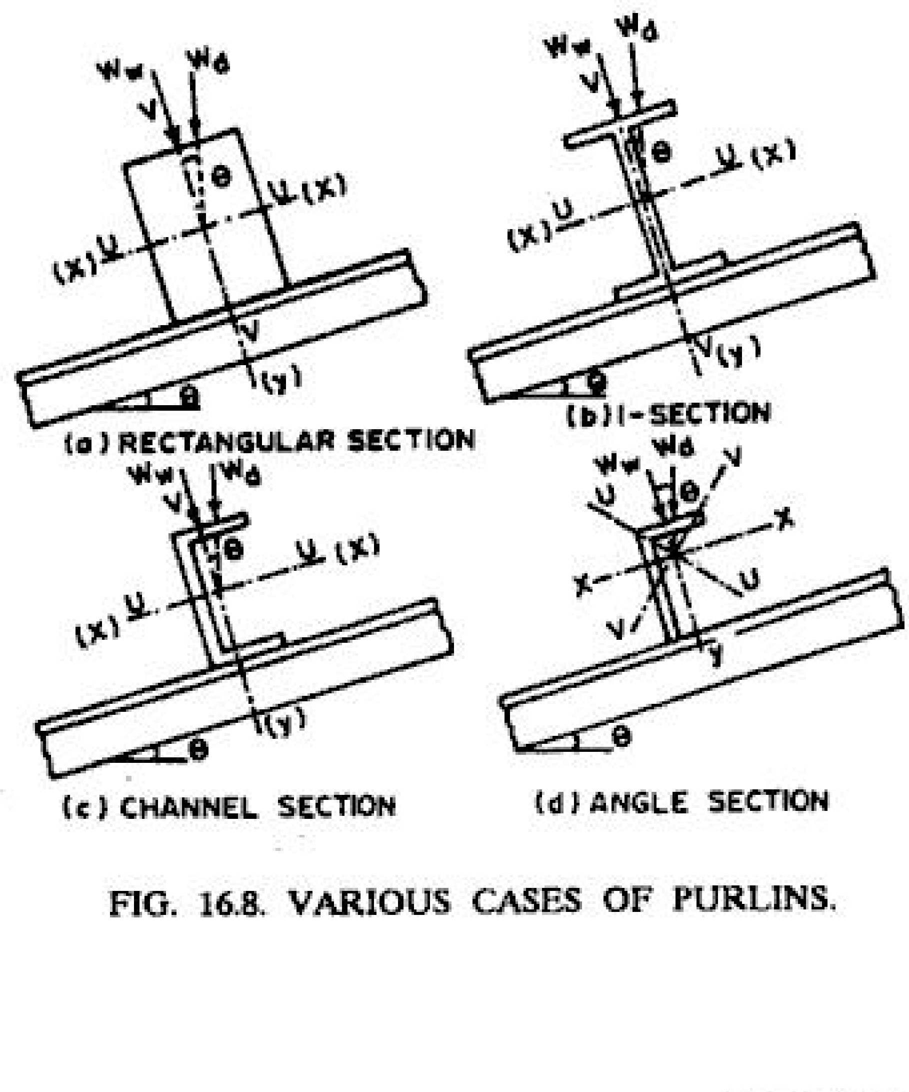

FIG. 16.8. VARIOUS CASES OF PURLINS.

depending upon the stress it carries. For convenience in fabrication, it is often economical to have same section continuous over several panels of a truss chord, even though the computed stresses may differ appreciably for the several panels. In the case of small span trusses, with light loads, calculations might indicate very small angle section. However, members should be sufficiently stiff to avoid damage during transportation and erection. Hence certain *minimum* section should be used. For example, for a compound Fink roof truss, the minimum angle section to be used for rafter and main tie should be ISA 75 × 50 × 6, while the minimum angle section for the central tie, main sling and main strut should be ISA 65 × 45 × 6, and minimum section for other members may be ISA 50 × 50 × 6.

The joints at each panel points should be properly designed. Eccentric condition should be avoided if possible, because they not only put additional stress on the rivets/welds, but also cause bending in the members connected. To avoid eccentricity in the important connections, the rivet lines of all members, together with the lines representing the external forces from the purlins and main axes of the supports, should intersect at points as drawn on the frame diagram. Theoretically, the lines representing the centres of gravity of the members – and not the rivet line – should intersect, but this is a refinement which is neglected in practice. Particular attention should be paid to the construction of the shoes where the forces in rafters and main tie are generally the greatest. Typical details of the joints are shown in Fig. 16.20 and 16.22. The following are the requirements of a good joint :

1. The line of thrust should pass through the C.G. of the rivet group and the rivets should be symmetrically arranged about this line
2. For a tension member, the rivets should be so arranged that the area of the member joined is not reduced more than necessary.
3. The number and the diameter of rivets should be sufficient to develop the maximum stresses induced in all the members at the connection.
4. Members should be straight and bolts used to draw them together before the rivets are driven.
5. Rivet holes should match. The rivets should be properly heated and well driven.

Example 16.2. *Design a roof truss for a factory building for a span of 20 m and a pitch of 1/5. The height of the truss at eves level is 10 m. The spacing of the trusses is 4.5 m. The factory building which is 36 m long, is situated at Delhi. Take $f_y = 250\ N/mm^2$ for the steel sections. Provide Fink truss.*

Solution.

Step 1. Arrangement of purlins

$$\text{Span} = 20 \text{ m};\quad \text{Pitch} = \frac{1}{5}\ ;\quad \text{Rise} = \frac{1}{5} \times 20 = 4 \text{ m}$$

Slope of roof truss, $\theta = \tan^{-1}\frac{1}{2.5} = 21.8°$; $\sin\theta = 0.3714$; $\cos\theta = 0.9245$

Length of top chord $= \sqrt{(10)^2 + (4)^2} = 10.77$ m

Dividing this length into four equal panels, length of each panel= 10.77/4= 2.693 m.

Let us use Standard Trafford asbestos sheets, along with Standard Trafford roof lights at certain interval. The maximum span for such asbestos sheets is 1.68 m which falls short of the panel length of 2.693 m, as shown in Fig 16.12 (*a*). Hence purlins will have to be placed at intermediate points and not at panel points. Let us provide 8 purlins (or 7 spaces) at spacing of 1530 mm, making a total of 1530 × 7 = 10710 mm leaving a gap of 60 mm at the apex, as shown in Fig. 16.12 (*b*).

Thus ISMC 100 is *inadequate* from deflection point of view. *Since deflection normal to the major axis is governing, there is no specific advantage in using sag rods, as far as deflection is concerned.* Hence use ISMC 125 @ 12.5 kg/m.

(c) ***Design of angle purlins***

As an alternative to channel purlins, angle purlins may be used. For angle purlins, IS 800-1984 provides an *alternative* to the above design procedure, if the certain requiremens given below are satisfied.

$$\text{Total } W = W_d + W_w = 1544 - 8950 = -7406 \text{ N}$$

$$M = \frac{Wl}{10} = \frac{7406 \times 4.5}{10} = 3332.7 \text{ N-m}$$

$$\text{Required } Z = \frac{3332.7 \times 10^3}{165} = 20.2 \times 10^3 \text{ mm}^3$$

Min depth of angle purlin $= l/45 = 4500/45 = 100$ mm.

Min. width of angle purlin $= l/60 = 4500/60 = 75$ mm.

Hence select ISA 100 × 75 × 10 @ 13.0 kg/m having :

$$Z_x = 23.6 \times 10^3 > 20.2 \times 10^3$$
$$\text{Depth} = 100 \geq 100 \text{ mm}$$
$$\text{Width} = 75 \geq 75 \text{ mm}$$

Note : This section will be found to be safe for load combination (*i*), i.e. D.L.+LL ($W_{dl} = 4830$ N) using procedure of unsymmetrical bending. It is left upon the reader to work out this . Since the weight of ISMC 125 @ 12.7 kg/m is slightly less than the weight of ISA 100 × 75 × 10 @ 13.0 kg/m, it is prefarable to use the channel section, which will be more adequate against deflection.

Step 5. Loads at panel points of truss

There are eight panels in all, with one side having four panels of 2.6925 m length each.

(*a*) *Dead loads*

Weight of asbestos sheet, per panel length $= 159 \times 2.6925 \times 4.5 \approx 1926$ N ...(*i*)

There are 14 purlins bearing directly on the truss. Hence load due to weight of purlin per panel $= \frac{1}{8}[14 \times 4.5 \times 12.7 \times 9.81] \approx 980$ N ...(*ii*)

$$\text{Weight of truss, } w = 10\left(\frac{L}{3} + 5\right) = 10\left(\frac{20}{3} + 5\right) \approx 116.7 \text{ N/m}^2$$

∴ Weight of truss per panel $= \frac{1}{8}(116.7 \times 20 \times 4.5) = 1314$ N ...(*iii*)

∴ Total dead weight per panel $= 1926 + 980 + 1314 = 4220$ N

Dead load at each end panel point $= \frac{1}{2} \times 4220 = 2110$ N

(*b*) *Imposed load.*

Imposed load (live load) on the truss $= \frac{2}{3} \times$ live load on purlins

$= \frac{2}{3} \times 514 = 342.67 \text{ N/m}^2$ of horizontal area.

∴ Live load per panel $= 342.67 \times 4.5 \times 2.6925 \cos 21.8° = 3854$ N

Live load on purlin= 730 N/m. But, for design of roof truss, only 2/3 live load is taken.

∴ Purlin reaction due to live load $= \frac{2}{3} \times 3286 = 2190$

∴ Total reaction from purlins $= 1654 + 2190 = 3844$ N

Reaction from end purlins $= \frac{1}{2} \times 3844 = 1922$ N.

Fig. 16.17 (*a*) shows the loads on the rafter, which can be assumed to be fixed at ends. The moment distribution is shown in Table 16.3. From the B.M.D. (and also from Table 16.3), max. B.M. due to (dead load + live load) combination occurs at *C*, its value being equal to 1755 N-m.

TABLE 16.3.

A	*B*		*C*		*D*		*E*	
	0.5	0.5	0.5	0.5	0.5	0.5		
− 1097	+ 1443 + 124	− 1691 + 124	+ 1674 + 19	− 1712 + 19	+ 1724 − 123	− 1479 − 123	+ 1124	F.E.M. Bal.
+ 62	− − 5	+ 10 − 5	+ 62 −	− 62 −	+ 10 − 5	− − 5	− 61	C.O. B.
− 3	− −	− −	− 3 + 3	− 3 + 3	− −	− −	− 3 −	C.O. B.
− 1038	+ 1562	− 1562	+ 1755	− 1755	+ 1606	− 1606	+ 1060	Final

(*b*) *Moments due to (dead load + wind load) combination*

$W_d = 1654$ N as before. Also, W_w (due to wind)= − 8950

Reaction from purlin due to (dead load + wind load) combinations= 8950 − 1654 = 7296 N

Reaction due to end purlins $= \frac{1}{2} \times 7296 = 3648$ N.

The B.M. in the rafter, due to (dead load + wind load) combination can be found by multiplying the values in Table 16.3 by the factor 7296/3844. Thus, $M_B = 2965$, $M_C = 3331$ and $M_D = 3048$ N-m.

(*c*) *Design of section.*

Assume the section of rafter as shown in Fig. 16.18, consisting of two angles, ISA 8050 × 8 mm, @ 7.7 kg/m, with distance back to back of 8 mm so as to accommodate a gusset plate of 8 mm thickness.

$A = 2 \times 978 = 1956 \text{ mm}^2$; $C_{xx} = 27.3$ cm ; $C_{yy} = 12.4$ cm

$I_{xx} = 61.9 \times 10^4 \text{ mm}^4$; $I_{yy} = 18.5 \times 10^4$; $r_{xx} = 25.2$ mm

$I_X = 2 \times 61.9 \times 10^4 = 123.8 \times 10^4 \text{ mm}^4$

$I_Y = 2\,[18.5 \times 10^4 + 978\,(4 + 12.4)^2] = 89.6 \times 10^4$

$r_Y = \sqrt{\frac{89.6 \times 10^4}{1956}} \approx 21.4$ mm

$r_X = r_{xx} = 25.2$ mm.

Effective length l_x about x-x axis

$= 0.7 \times$ panel length

$= 0.7 \times 2.6925 \times 1000 = 1885$ mm.

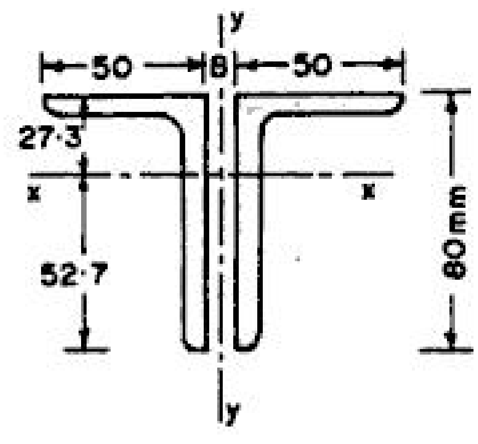

FIG. 16.18.

Step 13. Design of central hanger (member 7-8)

It carries no stress. However, provide angle section ISA 50 × 50 × 6 mm

Step 14. Design of joints

Let us use 16 mm dia. power driven shop rivets.

$$\text{Strength of rivet in single shear} = \frac{\pi}{4}(17.5)^2 \times 100 = 24050 \text{ N}$$

$$\text{Strength of rivet in double shear} = 2 \times 24050 = 48100 \text{ N}$$

$$\text{Strength in bearing on 6 mm plate} = 17.5 \times 6 \times 300 = 31500 \text{ N}$$

$$\text{Strength in bearing on 8 mm plate} = 17.5 \times 8 \times 300 = 42000 \text{ N.}$$

The number of rivets required to connect a member will depend upon the force carried by it. Table 16.4 gives the number of rivets required for different members.

TABLE 16.4.

Members	Design stress (N)	Rivet value (N)	No. of rivets required
1 Rafters (2 ISA 8050 × 8)	+ 76050 − 102600	42000	3
2 Main ties (2 ISA 80 × 80 × 6)	+ 87730 − 70430	42000	3
3 Main struts (ISA 65 × 65 × 6)	+ 15020 − 23650	24050	1*
4 Minor struts. (ISA 50 × 50 × 6)	+ 7500 − 11830	24050	1*
5 Main slings (2 ISA 70 × 70 × 6)	+ 47180 − 30280	42000	2
6 Minor slings. (ISA 65 × 65 × 6)	+ 15995 − 10090	24050	1*

* Minimum two rivets will be provided on each member at each joint.

Step 15. Design of end bearings

The end reaction at each end is transferred to wall/column through proper bearing which consists of a pair of shoe angles and a base plate. The load is first transferred from the truss to the shoe angle, through gusset plate. The shoe angles transfer the reaction to the base plate which rests directly on the masonry wall/concrete column. A pair of anchor bolts are also provided to take upward reaction due to wind forces.

The end supports so designed should satisfy the following considerations :

(1) The size of base plate should be sufficient so that the bearing pressure does not exceed the permissible value.

(2) Anchor bolts should be of sufficient length to take the uplift due to wind.

(3) Sliding joint should be provided at one end to accommodate the thermal expansion of the truss.

(4) The lines of forces in rafter, bottom tie and vertical end reaction should meet at a point.

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17.4. TUBULAR TENSION MEMBERS

Tubular tension members are designed on the basis of their net cross-sectional area. In this respect, steel tubes do not have any specific advantage over other rolled steel sections, since a tube or any rolled steel shapes with the same net cross-sectional area possess the same equivalent resistance to tensile force. However, tubular tension members are of common occurrence in steel tubular trusses, in the form of ties.

Permissible axial stress in tension. As per IS : 806-1957, the direct stress in axial tension on the net cross-sectional area of tubes shall not exceed the values of f_t given in Table 17.3.

TABLE 17.3. PERMISSIBLE AXIAL STRESS IN TENSION

Grade (IS : 1161-1979)	*Value of f_t*	
	kg/cm² (IS : 806)	*N/mm² (converted)*
Y_{st} 210	1250	122.6
Y_{st} 240	1500	147.1
Y_{st} 310	1900	186.3

17.5. TUBULAR COMPRESSION MEMBERS

Tubular compression members possess specific structural advantage since they possess equal lateral resistance in all the directions. The ratio of the diameter d to the thickness t should be small enough to ensure that failure due to local buckling does not occur.

Notwisthstanding the thickness requirements stated in § 1.2, thickness of tubes subject to compressive stress shall not be less than that given by the formula :

$$t = 0.865\, D^{1/3} \text{ mm} \qquad ...(17.1)$$

where t = thickness of tube in mm

D = outside diameter of tube in mm

However, the above requirement was deleted through 3rd Amendment (June 1965) to IS : 806-1957.

Permissible axial stress in compression (IS : 806-1957). The values of permissible axial stress on compression (f_c) are given in Table 17.4. For values of l/r less than 60, the values of f_c shall not exceed that obtained from linear interpolation between the value of f_c for $l/r = 60$ as found from Eq. 17.2 given below and a value of 0.6 f_y for $l/r = 0$. For values of l/r from 60 to 150, the average axial stress on the cross-sectional area of a strut or other compression member shall not exceed the value of f_c obtained by the following expression:

$$f_c = \frac{f_y/m}{1 + 0.15 \sec l/r \sqrt{m f_c/4E}} \qquad ...(17.2)$$

where f_c = Permissible average axial compressive stress

f_y = the guaranteed minimum compressive stress

m = factor of safety (taken as 1.67)

l/r = slenderness ratio, l being the effective length and r being the radius of gyration.

For values of l/r greater than 150, the average axial stress on the cross-sectional area of a strut or other compression member shall not exceed the value $f_c \left(1.2 - \frac{l}{750\,r}\right)$, where f_c is obtained from Eq. 17.2.

Effective length and maximum slenderness ratio. The effective length of a compression member, and maximum slenderness ratio are found as per provisions of IS : 800-1984. See Chapter 7 for details.

termediate stiffener or betweeen edge and intermediate stiffener.

4. Flat-width ratio. The flat-width ratio (w/t) of a single flat element is the ratio of flat-width w, exclusive of the end fillets, to the thickness. In case of sections, such as I, T, channel and Z-shaped sections, the width w is the width of the flat projection of the flange from web, exclusive of end fillets and of any stiffening lip that may be at the outer edge of the flange. In case of multiple web section (such as hat, U or box shape sections), the width w is the flat width of flange between adjacent webs, exclusive of fillets. See Fig. 18.6 for flat-width ratios.

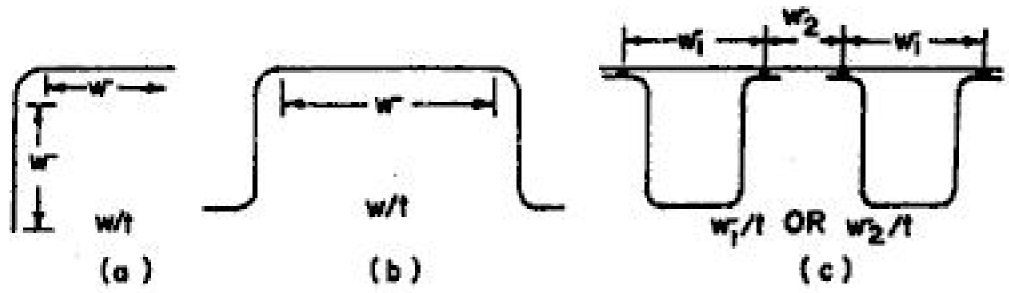


FIG. 18.6. FLAT-WIDTH RATIO

5. Effective design width. Where the flat-width (w) of an element is reduced for design purposes, the reduced design width (b) is termed as the *effective width* or *effective design width* (Fig. 18.7). It is determined as discussed in § 18.6.

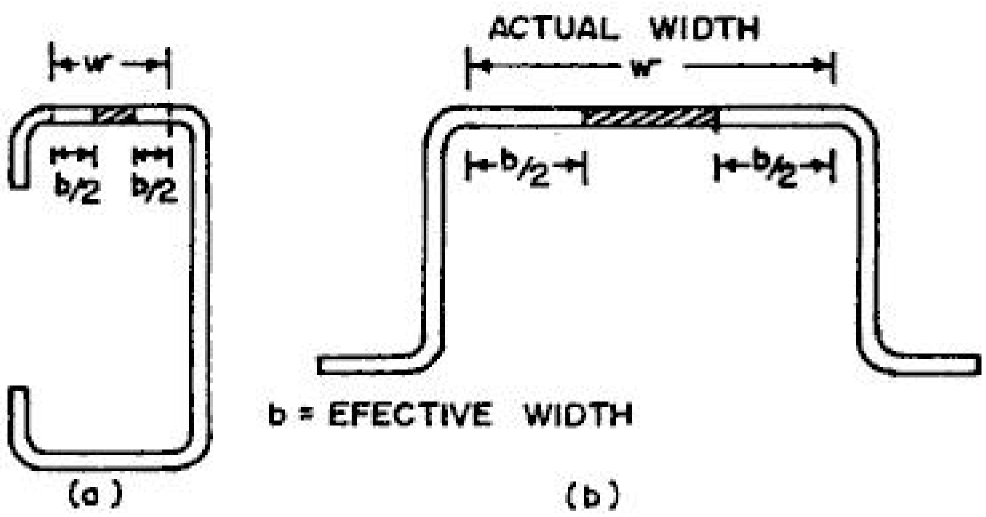


FIG. 18.7. EFFECTIVE DESIGN WIDTH

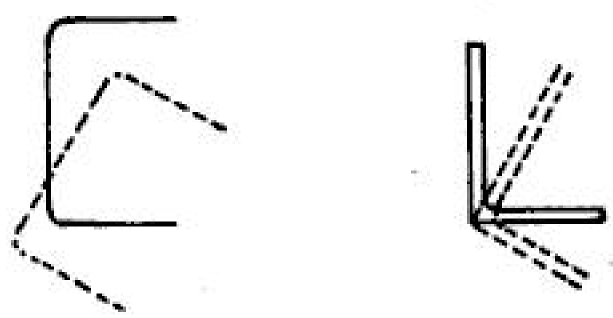


FIG. 18.8. TORSIONAL FLEXURAL BUCKLING

4. Thickness. The thickness of any element or section is taken as the base steel thickness, exclusive of coatings.

5. Torsional flexural buckling. It is a mode of buckling in which the compression members can bend and twist simultaneously (Fig. 18.8).

6. Point symmetric section. It is a section which is symmetrical about a point (centroid), such as a Z-section, having equal flanges (Fig. 18.9).

7. Yield stress (f_y). The cold-rolled sections are produced from *strip steel* conforming to IS : 1079-1973. The yield stresses of the steels are as given in Table 18.1.

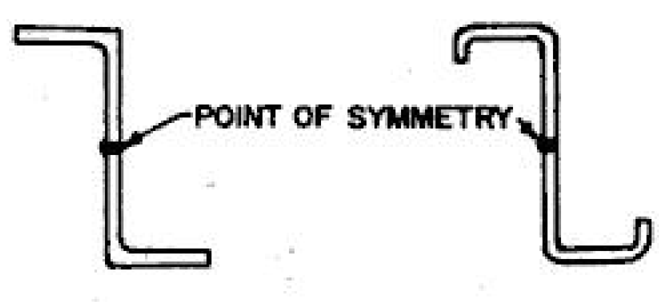


FIG. 18.9. POINT SYMMETRIC SECTION

TABLE 18.2. FORMULAE FOR EFFECTIVE WIDTH OF STIFFENED COMPRESSION MEMBERS WITHOUT INTERMEDIATE STIFFENER

MKS UNITS (f in kg/cm^2)	*SI UNITS* (f in N/mm^2)
1. For compression elements other than tubes	**1. For compression elements other than tubes**
(a) For load determination	*(a) For load determination*
For (w/t) upto $(w/t)_{lim} = 1435/\sqrt{f}$, $b = w$	For (w/t) upto $(w/t)_{lim} = 446/\sqrt{f}$, $b = w$
For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2120}{\sqrt{f}}\left[1 - \frac{465}{(w/t)\sqrt{f}}\right]$	For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{658}{\sqrt{f}}\left[1 - \frac{145}{(w/t)\sqrt{f}}\right]$
(b) For deflection determination	*(b) For deflection determination*
For (w/t) upto $(w/t)_{lim} = 1850/\sqrt{f}$, $b = w$	For (w/t) upto $(w/t)_{lim} = 574/\sqrt{f}$, $b = w$
For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2710}{\sqrt{f}}\left[1 - \frac{600}{(w/t)\sqrt{f}}\right]$	For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{842}{\sqrt{f}}\left[1 - \frac{186}{(w/t)\sqrt{f}}\right]$
2. For closed square and rectangular tubes	**2. For closed square and rectangular tubes**
(a) For load determination	*(a) For load determination*
For (w/t) upto $(w/t)_{lim} = 1540/\sqrt{f}$, $b = w$	For (w/t) upto $(w/t)_{lim} = 478/\sqrt{f}$, $b = w$
For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2120}{\sqrt{f}}\left[1 - \frac{420}{(w/t)\sqrt{f}}\right]$	For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{658}{\sqrt{f}}\left[1 - \frac{130}{(w/t)\sqrt{f}}\right]$
(b) For deflection determination	*(b) For deflection determination*
For (w/t) upto $(w/t)_{lim} = 1990/\sqrt{f}$; $b = w$	For (w/t) upto $(w/t)_{lim} = 618/\sqrt{f}$, $b = w$
For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{2710}{\sqrt{f}}\left[1 - \frac{545}{(w/t)\sqrt{f}}\right]$	For $(w/t) > (w/t)_{lim}$, $\frac{b}{t} = \frac{842}{\sqrt{f}}\left[1 - \frac{169}{(w/t)\sqrt{f}}\right]$

Note : The formulae in SI units can be obtained by dividing the constants of equations in MKS units (given in IS 801-1975) by a factor $\sqrt{2074000/200000} = 3.22$

Thus, total area of the section is given by

$$A = L_t \times t$$

where L_t = total length of all the elements.

Similarly, the moment of inertia of the section is given by

$$I = I' \times t$$

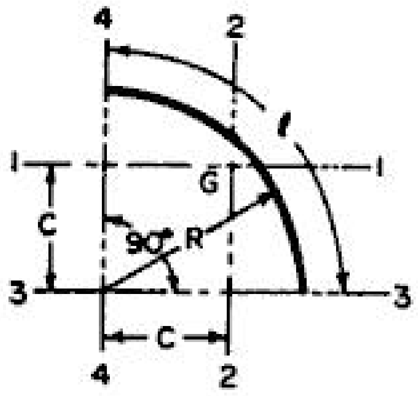
where I' = moment of inertia of the centre line of the steel sheet.

The section modulus is computed by dividing $I\,(= I' \times t)$ by the distance from the N.A. to the extreme and not to the central line of extreme element.

The first power dimensions such as x, y and r(radius of gyration) *are obtained directly by linear method and do not involve the thickness dimension.*

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. Fig. 18.17 and 18.18 gives the moment of inertia and location of centroid of these elements.

(a)



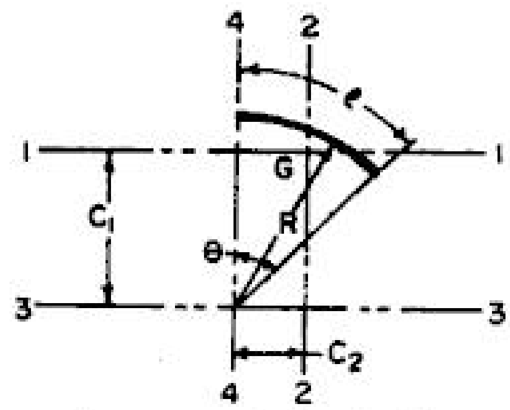
$l = 1.57\,R$; $C = 0.637\,R$

$I_1 = I_2 = 0.149\,R^3$

$I_3 = I_4 = 0.785\,R^3$

G = centre of gravity

(b)



θ (radians) = $0.1745 \times \theta$ (degrees)

$$l = \theta R$$

$$C_1 = \frac{R \sin\theta}{\theta}\ ;\ C_2 = \frac{R(1 - \cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2} - \frac{(\sin\theta)^2}{\theta}\right] R^3$$

$$I_2 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2} - \frac{(1 - \cos\theta)^2}{\theta}\right] R^3$$

$$I_3 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2}\right] R^3$$

$$I_4 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2}\right] R^3$$

FIG. 18.18. PROPERTIES OF CIRCULAR LINE ELEMENTS

18.13. AXIALLY LOADED COMPRESSION MEMEBERS

In hot-rolled sections, the width-thickness ratio of various elements of the section are so limited that no local buckling of these elements takes place. In contrast to this, the width-thickness ratios of the component elements in cold-formed light gauge sections are large due to which these elements undergo local buckling also. Hence effects of local buckling are incorperated in the allowable stress expression by incorporating a factor Q, known as *form factor*, in allowable stress expressions. This factor Q, which is less than 1 is associated with the yield stress f_y and if we substitute Qf_y for f_y in the well known axial stress expression, the expressions given in clause 6.6 of IS : 801-1975 can be obtained.

$$\therefore \quad \frac{b}{t} = \frac{658}{\sqrt{141}}\left[1 - \frac{130}{55\sqrt{141}}\right] = 44.38$$

$$\therefore \quad b = 44.38 \times 2 \approx = 88.8 \text{ mm}$$

The effective section is shown in Fig. 18.23.

3. ***Determination of factor Q***

$$A_{eff} = 1264 - 2 \times 2 \times 92 - 21.2 = 811.2 \text{ mm}^2$$

$$\therefore \quad Q = \frac{A_{eff}}{A} = \frac{811.2}{1264} = 0.642$$

4. ***Determination of permissible stress and load***

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} = \sqrt{\frac{2\pi^2 (2 \times 10^5)}{235}} = 129.61$$

$$(l/r)_{lim} = \frac{C_c}{\sqrt{Q}} = \frac{129.61}{\sqrt{0.642}} \approx 161.8 > 70.4$$

$$\therefore \quad f_a = \frac{12}{23} Q f_y - \frac{3}{23}\frac{(Q f_y)^2}{\pi^2 E}\left(\frac{l}{r}\right)^2 \qquad \text{...(18.35 } a)$$

$$= \frac{12}{23} \times 0.642 \times 235 - \frac{3}{23}\frac{(0.642 \times 235)^2}{\pi^2 \times 2 \times 10^5}(70.4)^2 = 71.26 \text{ N/mm}^2$$

$\therefore$ Permissible load $= f_a . A = 71.26 \times 1264 \approx 90080$ N $=$ **90.08 kN**

Example 18.3. *Two channels 200 mm × 80 mm with bent lips are connected with webs to act as column, as shown in Fig. 18.24 (a). The thickness of plate is 2.5 mm and the depth of lips is 25 mm. Determine the safe load carrying capacity if the effective length of column is (a) 4 m and (b) 6 m. Take* $f_y = 235 \text{ N/mm}^2$ *and* $E = 2 \times 10^5 \text{ N/mm}^2$.

Solution

1. ***Computation of sectional properties.*** By inspection I_{yy} will be smaller and hence will be computed. Fig. 18.24 (*b*) shows the simplified rectangular elements of the section.

$$I_{yy} = \frac{2 \times 2.5\,(160)^3}{12} + 4\,(25 - 2.5) \times 2.5\,(80 - 1.25)^2 + \frac{(200-5)\,(5)^3}{12} = 310.4 \times 10^4 \text{ mm}^4$$

$$A = (2 \times 2.5 \times 160) + 4\,(25 - 2.5) \times 2.5 + (200 - 5) \times 5 = 2000 \text{ mm}^2. \text{ Hence,}$$

$$r_{yy} = \sqrt{\frac{310.4 \times 10^4}{2000}} = 39.4 \text{ mm}$$

2. ***Computation of effective widths***

The section consists of both stiffened elements (*i.e.* web and flanges) as well as unstiffened elements (*i.e.* lips). Hence the effective width of plates will be computed for the allowable stress f_c in unstiffened elements.

For unstiffened element,

$$\frac{w}{t} = \frac{(25 - 8)}{2.5} = 6.8 < 12. \text{ Hence,}$$

$$f_c = f = 0.6 f_y = 0.6 f_y = 0.6 \times 235 = 141 \text{ N/mm}^2$$

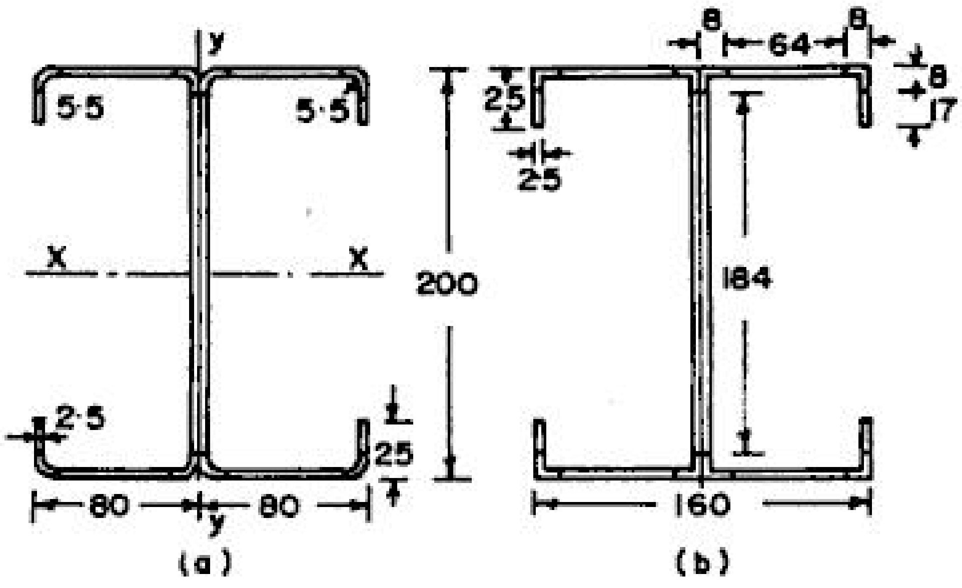


FIG. 18.24.

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$$b = 28.98 \times 2.5 = 72.4 \text{ mm} > 64$$

$$\therefore \quad b = w = 64 \text{ mm.}$$

2. *Determination of moment of inertia and section modulus.*

I and Z will be determined by taking $b = w = 64$ mm.

$$I_{xx} = 4(64 + 16) \times 2.5\,(100 - 1.25)^2 + \frac{1}{12} \times 5\,(195)^3 + 4 \times 2.5 \times 22.5\,(100 - 25 - 11.25)^2$$

$$= 12564570 \text{ mm}^4$$

$$\therefore \quad Z = \frac{12564570}{100} \approx 125.65 \times 10^3 \text{ mm}^3$$

3. *Determination of safe load*

$$M = fZ = 141 \times 125.65 \times 10^3 = 17.72 \times 10^6 \text{ N-mm} = 17.72 \text{ kN-m}$$

Let w be the load in kN/m

$$\therefore \quad \frac{w\,(4)^2}{8} = 17.72 \text{ kN/m.} \quad \text{From which } w = 8.858$$

4. *Check for web shear*

$$\text{Max. shear force} = \frac{8.858 \times 4}{2} = 17.716 \text{ kN.}$$

$$\text{Max. average shear strees} = \frac{17.716 \times 1000}{2 \times 2.5 \times 195} = 18.17 \text{ N/mm}^2$$

Now $$\frac{h}{t} = \frac{195}{2.5} = 78; \quad \frac{1425}{\sqrt{f_y}} = \frac{1425}{\sqrt{235}} = 92.96$$

Hence Eq. 18.39 (b) is applicable.

$$f_v = \frac{396\sqrt{f_y}}{h/t} = \frac{396\sqrt{235}}{78} = 77.83 \text{ N/mm}^2 \le 0.4 \times 236 = 94.4 \text{ N/mm}^2$$

Thus $f_v = 77.83$ N/mm^2 , which is much greater than the max. average shear stress of 18.17 N/mm^2. The beam is therefore safe in shear.

5. *Check for bending compression in web*

$$f_{bw}' = 141 \times \frac{100 - 2.5}{100} = 137.48 \text{ N/mm}^2$$

$$\text{Permissible } f_{bw} = \frac{3525000}{(h/t)^2} = \frac{3525000}{(78)^2} = 579.4 > 137.48. \text{ Hence safe.}$$

6. *Determination of deflection.*

For determination of deflection, one may use *effective section* at max. B.M. However, effective width for deflection determination is different from the one found for load determination, and is given by Eq. 18.9 (b).

$$\therefore \quad \frac{b}{t} = \frac{842}{\sqrt{141}}\left[1 - \frac{186}{64/2.5\sqrt{141}}\right] = 27.65$$

$$\therefore \quad b = 27.65 \times 2.5 = 69.1 > 64$$

$$\therefore \quad b = w = 64 \text{ mm.}$$

Hence $$I = 12564570 \text{ mm}^4, \text{ as computed earlier.}$$

is shown in Fig. 18.32, that is, in welding thin sheet, the weld shape generally obtained is that shown in the Figure, with the thickness of the weld actually exceeding that of the sheet. The intention is to disregard any material deposited beyond the dashed line in Fig. 18.32, and to calculate the throat thickness in the same manner as in heavy welded construction.

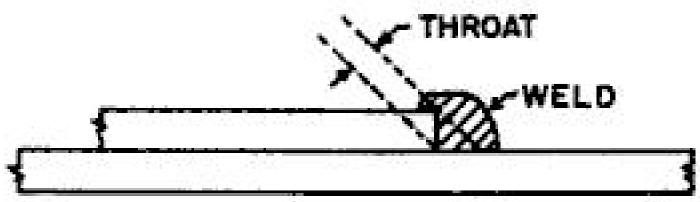

FIG. 18.32. THROAT OF A FILLET WELD

The allowable stress in tension or compression on butt welds shall be the same as prescribed for the lower grade of the base metals being jointed, provided the welds are of full penetration type and the yield strength of the filler metal is equal to or greater than the yield strength of the base metal. Stress due to eccentricity of loading, if any, shall be combined with the primary stresses, and the combined stresses shall not exceed the values given above.

Stresses in a fillet weld shall be considered as shear on the throat for any direction of the applied stress. Neither plug nor slot welds shall be assigned any value in resistance to any stresses other than shear.

When plug welds are made with pre-punched holes, the length of the fillet weld for computing weld strength is identical with the perimeter of the hole. When the hole is burned and weld made in the same operation, a frequent process (which is more aptly designated as *puddle welding*), a conservative procedure is to compute the perimeter for a hole of diameter 6 to 10 mm less than the visible diameter of the puddle.

(b) Resistance welding (spot welding)

In its normal form as well as by projection, welding is probably the most important means of shop connection in light gauge steel fabrication. As per IS 801-1975, in sheets joined by spot welding the allowable shear per spot shall be as given in Table 18.8.

TABLE 18.8. ALLOWABLE SHEAR STRENGTH PER SPOT

Thickness of Thinnest outside sheet (mm)	*Allowable shear strength per spot*		*Thickness of Thinnest outside sheet (mm)*	*Allowable shear strength per spot*	
	kg	*N*		*kg*	*N*
0.25	23	225	2.0	489	4796
0.50	57	559	2.50	625	6129
0.80	102	1000	2.80	750	7355
1.00	159	1559	3.15	909	8915
1.25	239	2344	5.00	1818	17829
1.60	330	3236			

2. Connecting two channels for forming an I-section

(a) ***For compression members.*** The maximum permissible longitudical spacing (s_{max}) of welds (or other connectors) joining two channels to form an I-section shall be:

$$s_{max} = \frac{L \,.\, r_{cy}}{2 r_1} \qquad ...(18.50)$$

where L = unbraced length of compression members

r_{cy} = radius of gyration of one channel about its centriodal axis parallel to the web,

and r_1 = radius of gyration of I-section about axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing, if any.

19.2. DIMENSIONS OF STEEL STACKS

Steel stacks are cylindrical in shape. The basic dimensions, *i.e.*, the height and clear diameter depends upon several factors such as the draft required, flue gas velocity, flue gas temeprature, natural or mechanical draft, turn-down ratio, site data (such as ambient temperature, barometric pressure etc.) and the type of fuel adopted. The chimney should be at least 5 m taller than the tallest building in the surrounding area of 150 m radius. Fig. 19.2 shows a self-supporting steel stack, which is cylindrical for the major portion of the height, except at the bottom, where the stack is given a conical flare for better stability and for easy entrance of flue gases. The diameter of the base of the conical flare may vary between 4/3 to 5/4 of the diameter of the cylindrical portion, while the height of conical portion may vary from $\frac{1}{3}$ to $\frac{1}{4}$ of the total height of the stack.

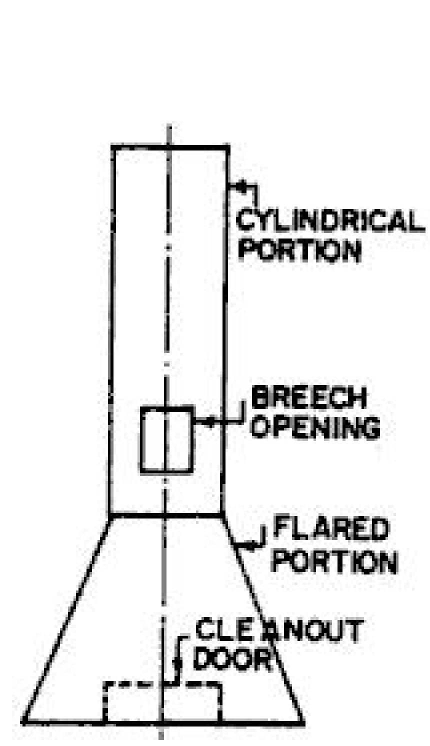

FIG. 19.2.

Though, due to the flare, the unit stresses are reduced, the thickness of plates in the flared portion should not be thinner than the thickness of the lowermost course in the cylindrical portion. As per IS : 6533 (part 2) : 1989, proportions of the basic dimensions of a self supporting chimney shall conform to the following.

(*a*) Minimum height of flare be equal to one third the height of chimney.

(*b*) Minimum outside diameter of unlined chimney shell at top be equal to one twentieth of the height of cylindrical portion of chimney and for lined chimney it shall be one-twenty fifth of the height of cylindrical portion.

(*c*) Minimum outside diameter of flared chimney shell at base be equal to 1.6 times the outside diameter of chimney shell at top.

The inside diameter of the chimney in m is given by

$$D = \sqrt{\frac{4Q}{\pi V}} \qquad ...(19.1)$$

where Q = quantity of the gas in m^3/sec.

V = velocity of the flue gas at exit point of chimney, in m/sec.

However, the diameter shall be so chosen that the velocity will not exceed, under any circumstance, 30 m/sec. The optimum range of velocity may be taken as 15 to 20 m/sec.

The height of chimney depends upon the dispersion requirement of the flue gases into the atmosphere.

The *Central Board for Prevention and Control of Water Pollution* (New Delhi), through its publications : (*i*) Emission Regulation (July 1984) Part I and (*ii*) *A method to determine minimum chimney height* has given the following recommendations for height of chimney :

(*a*) ***For chimney emitting particulate matter***

$$H = 74\,(Q_P)^{0.27} \qquad ...(19.2)$$

where H = height of chimney (*m*) and Q_P= particulate matter emission (tonnes/hour)

(*b*) ***For chimney emitting*** SO_2

$$H = 14\,(Q_s)^{1/3} \qquad ...(19.3)$$

where Q_s = SO_2 emission (kg/hour)

(*c*) ***Minimum values***

The board has further recommended that the height of the chimney calculated on the

$$1.6\left(\sigma_w + \sigma_m\right) - 0.9\,\sigma_d < 1.8\,\sigma_a \qquad ...(19.16)$$

where σ_w = stress produced due to wind

σ_m = stress produced due to any other load which may act to increase the combined stress

σ_d = stress produced by dead load and any other load which acts at all times and will reduce the combined stress.

σ_a = maximum permissible stresses at the operating temperature.

To ensure stability at all times, account shall be taken of probable variations in dead load during construction, repair or other similar work.

While computing the stability, it shall be ensured that the resulting pressure and shear forces to be transferred to the supporting soil foundation, will not cause failure of foundation.

(ii) Stability of structure and foundations

In the case of guyed or laterally supported chimneys, the stability of the structure and foundation as a whole or any part of it shall be investigated and weight or anchorage shall be provided so that, without exceeding the allowable material stresses and foundation bearing pressure, 0.9 times the least restoring moment including anchorage will not be less than the sum of 1.1 times the maximum overturning moment due to stress increasing dead loads, less 0.9 times that due to stress-reducing loads, plus 1.4 times that due to wind and imposed loads, that is :

$$1.4\,M_w + 1.1\,M_m - 0.9\,M_d < 0.9\,M_a \qquad ...(19.17)$$

where M_w = overturning moment produced by the wind and imposed loads

M_m = overturning moment produced by dead or other loads which may act to increase combined moment

M_d = overturning moment produced due to dead or other loads which act at all times to reduce combined moment, and

M_a = resisting moment produced by the foundation without exceeding the allowable material stress and the ground stress without exceeding the foundation bearing pressure.

(*b*) In the case of self supporting chimney, the stability of the structure as a whole shall be investigated, and weight or anchorages shall be so proportioned that the least resisting moment shall be not less than the sum of 1.5 times the maximum overturning moment due to dead load and wind load/earthquake load.

19.8. DESIGN OF BASE PLATE

The chimney is connected to a base plate, at its base, which in turn is connected to concrete foundation through foundation bolts. The base plate, of width m is annular is plan. Fig. 19.7 shows two alternative methods of joining the chimney to the base plate. Base plates are made of cast iron, cast steel or structural steel. It is preferable to use cast steel base plates as compared to cast iron base plates. The base plate is cast in section. For very high stacks, base plates are made of structural steel.

Let d_c be the diameter of the conical (or flared) portion of the chimney at its base.

The maximum compressive force per unit circumferential length of the plate is

$$F_c = \frac{W_s + W_l}{\pi\, d_c} + \frac{4\, W_w}{\pi\, d_c^2} \text{ kN/m} \qquad ...(19.18)$$

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$$\therefore \quad \text{Pitch of rivets} = \frac{\text{Rivet value}}{F_c} = \frac{86.748 \times 10^3}{638.49} = 135.86 \text{ mm}$$

Maximum permissible pitch $= 10\,t = 10 \times 16 = 160$ mm

Hence provide 22 mm dia. rivets @ 130 mm c/c.

$$\text{Projection } c = \frac{160}{2} - (10 + 8) = 62 \text{ mm.}$$

$$\text{Actual bearing pressure} = \frac{638.49}{160} = 3.99 \text{ N/mm}^2$$

$$\therefore \quad t_b = \sqrt{\frac{3\,\sigma_c}{\sigma_{bs}}} \times c = \sqrt{\frac{3 \times 3.99}{185}} \times 62 = 15.77 \text{ mm}$$

Provide 16 mm thick base plate.

10. Design of anchor bolts

Maximum uplift force per unit length of circumference

$$F_t = \frac{4\,M_w}{\pi\,d_c^2} - \frac{W_s}{\pi\,d_c} \qquad \text{...(19.20)}$$

As per IS 6533 (Part 2) : 1989, the overturning moment M_w should be increased to 1.5 times from stability considerations.

$$\therefore \quad F_t = \frac{4\,(8757.93 \times 1.5)}{\pi\,(4.8)^2} - \frac{771.806}{\pi \times 4.8} = 674.8 \text{ kN/m}$$

Let us provide 39 mm dia. ISO fine threaded bolts having effective area $= 1028 \text{ mm}^2$, at root of threads. Taking permissible tensile strength of 120 N/mm^2 at the root of thread.

Strength of each bolt $= 1028 \times 120 \times 10^{-3} = 123.36$ kN.

No increase in stress is recommended since wind is the major load in the case of chimneys.

$$\therefore \quad \text{Spacing of bolts} = \frac{123.36}{674.8} \times 1000 = 182.8 \text{ mm}$$

$$\therefore \quad \text{No. of bolts} \approx \frac{\pi \times 4.8 \times 1000}{182.8} = 82.5$$

However, provide 85 bolts of 39 mm nominal diameter on a bolt circle diameter $\approx 4.8 + 0.7 = 4.87$ m.

$$\therefore \quad \text{Actual spacing of bolts} = \frac{\pi \times 4.87 \times 1000}{85} = 180 \text{ mm.}$$

Alternatively use HTFG boits M 30 (10 K) having proof load of 392.7 kN (See Table 4.8).

$$\therefore \quad \text{Spacing of bolts} = \frac{392.7 \times 1000}{674.8} = 582 \text{ mm}$$

$$\text{No. of bolts} \approx \frac{\pi \times 4.8 \times 1000}{582} = 25.9$$

However, provide 30 HTFG bolts of 30 mm dia. of 10 K grade, on a bolt-circle diameter of 4.87 m.

$$\text{Actual spacing} = \frac{\pi \times 4.87 \times 1000}{30} \approx 510 \text{ mm.}$$

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upward is provided. The track is fastened with the stand pipe by short lug angles spaced about 0.6 m to 0.9 m centres, and a light traveller with two wheels is installed on this track. A similar trolly track is also provided inside the stand pipe. A properly reinforced manhole of appropriate size is provided in the first shell ring of the stand pipe for cleaning purpose.

1. Stresses in stand pipe due to water

At any section x-x, situated h below the water level,

$$p = w\,h \qquad \text{...(20.3 a)}$$

Taking the unit weight of water, w as 9.81 kN/m^3=9.81×10^{-6} N/mm^3, h as water depth in metre, and expressing hydrostatic pressure p in N/mm^2, we have

$$p = (9.81 \times 10^{-6})\,(h \times 1000) = 9.81 \times 10^{-3} h \text{ N/mm}^2 \qquad \text{...(20.3)}$$

Hence bursting force per linear vertical mm of stand pipe or cylinder is given by

$$F = p\,d \quad (\text{where } d \text{ is dia. of tank in m})$$

or

$$F = (9.81 \times 10^{-3} h)\,(1000\,d) = 9.81\,h\,d \text{ N/mm} \qquad \text{...(20.4)}$$

The hoop stress f_h (or circumferential stress) in the section is

$$f_h = \frac{F}{2\,(t \times 1)} = 9.81\,\frac{h\,d}{2\,t} \qquad \text{...(20.5)}$$

where t is the plate thickness in mm.

This should not exceed the maximum allowable tensile stress σ_{at} in axial tension, the value of which is given in Table 20.1. If η is the efficiency of riveted joint in the vertical direction,

we get

$$t = \frac{9.81\,h\,d}{2\,\eta\,\sigma_{at}} \qquad \text{...(20.6)}$$

Since the plate is in contact with water, the actual thickness should be 1.5 mm in excess of the one calculated above.

2. Stresses due to wind force. Let p be the design wind pressure in kN/m^2. Hence at any depth h m below the top,

$$P_h = k\,.\,p\,.\,d\,.\,h$$

Taking shape factor $k = 0.7$,

$$P_h = 0.7\,p\,d\,h \quad (\text{kN})$$

The wind moment is given by

$$M_w = 0.7\,p\,d\,h\left(\frac{h}{2}\right) = 0.35\,p\,d\,h^2 \text{ kN-m} \qquad \text{...(20.7 a)}$$

If H is the total height of stand pipe,

$$M_w = 0.35\,pd\,H^2 \text{ kN–m} = 0.35\,p\,d\,H^2 \times 10^6 \text{ N–mm} \qquad \text{...(20.7)}$$

Hence the maximum bending stress (f_{bw}) due to wind is

$$f_{bw} = \frac{M_w}{I}\cdot\frac{d}{2}$$

where I = moment of inertia of circular ring section $= \dfrac{\pi\,d\,t\,(d/2)^2}{2} = \dfrac{\pi\,d^3}{8}\,t$

Expressing f_{bw} in N/mm^2, we get

$$f_{bw} = \frac{(0.35\,p\,d\,H^2 \times 10^6)\,(1000\,d/2)}{\frac{\pi}{8}\,(1000\,d)^3\,t} = \frac{1.4\,p\,H^2}{\pi\,d\,t} \text{ N/mm}^2 \qquad \text{...(20.2)}$$

Maximum stresses in radial and circumferential joint : Maximum radial and circumferential stresses occur at the lower point of the bottom where $h = H + h_s$

$$(f_r)_{max.} = (f_c)_{max.} = \frac{9.81\, r\,(H + h_s)}{2\, t_s} \text{ N/mm}^2 \qquad ...(20.14\ a)$$

Hence thickness is given by $t_s = \dfrac{9.81\, r\,(H + h_s)}{2\,\eta\,\sigma_{at}}$ ...(20.14 *b*)

where η is the efficiency of the joint and σ_{at} is the permissible tensile stress. For hemispherical bottom, $r = d/2$ and $h_s = d/2$

$$\therefore \qquad t_s = \frac{9.81\,(H + h_s)\, d/2}{2\,\eta\,\sigma_{at}} = \frac{9.81\,(H + d/2)\, d/2}{2\,\eta\,\sigma_{st}} \qquad ...(20.14)$$

20.8. STRESSES IN CONICAL BOTTOM

Fig. 20.5 shows a cylinderical (or circular) tank with conical bottom. Let H the height of cylinderical portion and h_c be the height of conical portion. Let the conical walls subtend an angle θ with the horizontal. Let us consider a section X-X of the conical bottom. Let W be the total load above the section considered, transmitted by the water column of radius b.

$$\therefore \qquad W = w\,\pi\, b^2 h = 9.81\,\pi\, b^2 h \text{ kN} \qquad ...(i)$$

Weight of water per 1 mm of circumference of section X-X

$$= \frac{W}{2\pi b} = \frac{(9.81\,\pi\, b^2 h)\, 1000}{2\,\pi\, b \times 1000} = \frac{9.81}{2} bh \text{ N/mm} \qquad ...(ii)$$

If T is the meridional tension per mm of circumference, its vertical component will balance the weight of water per 1 mm of circumference.

$$\therefore \qquad T \sin\theta = \frac{9.81}{2} b\, h$$

or $\qquad T = \dfrac{9.81}{2} b\, h \operatorname{cosec}\theta$ ...(20.15 *a*)

Hence stress in the radial joint is

$$f_r = \frac{9.81\, b\, h \operatorname{cosec}\theta}{2\, t_c} \text{ N/mm}^2 \qquad ...(20.15)$$

where t_c is the thickness of the plate of conical bottom.

Stress in circumferential joint : Consider two horizontal sections so that they intercept 1 mm length of the cone. This 1 mm length of the cone surface will be subjected to hydrostatic water pressure $p = w\,h = 9.81 \times 10^{-3} h$ N/mm^2 . This pressure p is resolved into two components : (*i*) component p_1 along the length of the cone and (*ii*) component p_2 in the horizontal direction, such that

$$p_1 = p \cot\theta$$

and $\qquad p_2 = p \operatorname{cosec}\theta$

The component p_2 induces circumferential stress.

$\therefore$ Stress in circumferential joint, f_c

$$= \frac{p_2\,(2\,b)}{2\, t_c} = \frac{p_2\, b}{t_c}$$

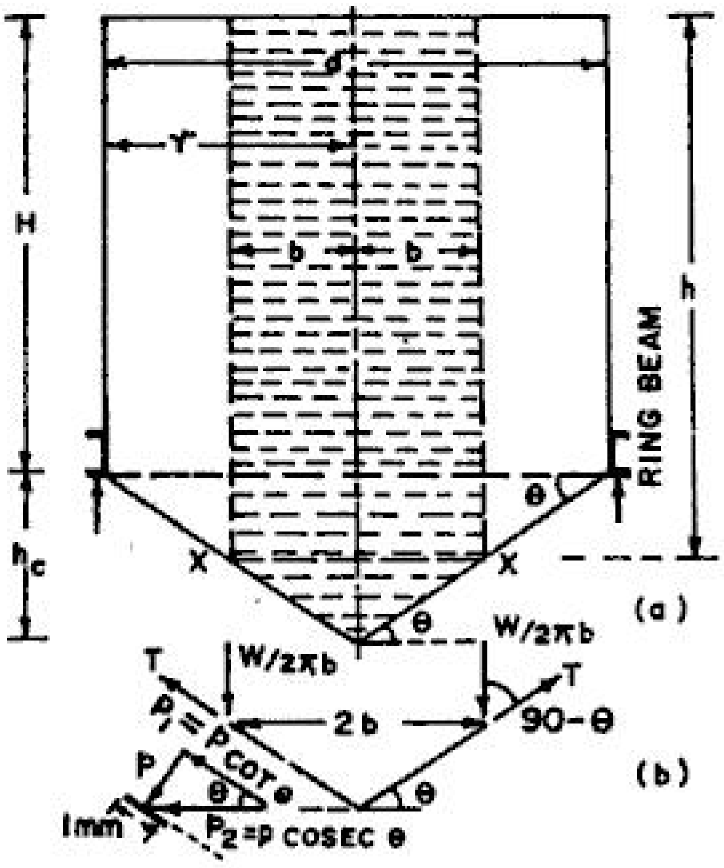

FIG. 20.5. CONICAL BOTTOM

(*b*). The maximum horizontal shear ($= 2P_H/n = P_H/3$) occurs for case (*b*). Since the braces support the columns laterally also, an additional shear equal to $2\frac{1}{2}$ % of the column load is taken as shear in the panel. Thus total horizontal force or transverse shear Q will be equal to $+2\frac{1}{2}$ % of $\left(\frac{W+P_w}{n}\right)$.

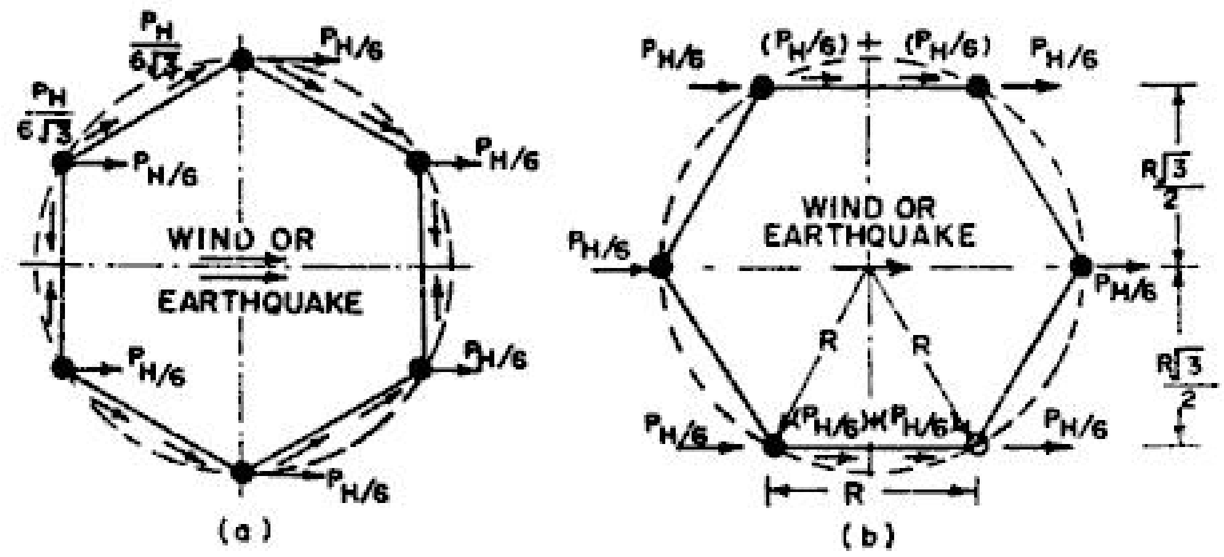

FIG. 20.10. WIND FORCES IN HORIZONTAL BRACE.

Fig. 20.11 shows the deflected shape of a vertical braced panel, consisting of (*i*) horizontal braces at top and bottom, (*ii*) two vertical columns, and (*iii*) one of the diagonal braces. The compressive force in horizontal brace AB is Q. The tensile force F_T in diagonal is given by

$$F_T = \frac{Ql}{a} \qquad ...(20.28)$$

where a = length of horizontal brace

l = length of diagonal

$= \sqrt{a^2 + h^2}$

h = height of panel.

The elongation $B''B' = \frac{F_T l}{AE} = \frac{Ql^2}{aAE}$

$\therefore$ Deflection $\Delta = B''B' \times \frac{l}{a} = \frac{Ql^3}{a^2 AE}$ ...(20.29 *a*)

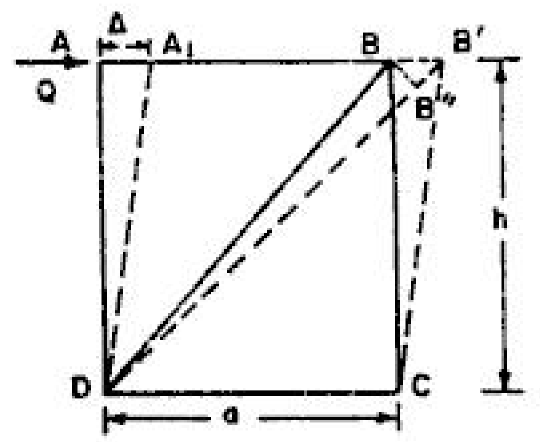

FIG. 20.11 VERTICAL PANEL

where A = area of cross-section of diagonal brace.

If there are several panels, situated one above the other in a vertical plane, the total deflection at the top is given by

$$\Delta = Q\Sigma \frac{l^3}{a^2 AE} \qquad ...(20.29)$$

20.12. RECTANGULAR ELEVATED TANKS

Rectangular tanks are made from plates of not less than 6 mm thickness. The width of the plates is generally 1.25 m. The vertical plates are designed for hydrostatic water pressure, while the bottom plates are designed on the basis of bending caused due to weight of water in the tank. The size of bending panel of bottom plates depends upon the size and shape of supporting frame. The bottom plates are turned up at the ends, to form a butt joint with the vertical side plates. The joints of the bottom plates are covered by T-sections which are turned up to form vertical stiffeners for the side plates. Flat straps are used to cover all the vertical and horizontal joints. Stays are provided at mid-height, both transversely as well as longitudinally, to stay the side plates, and are connected to vertical stiffeners. The bottom corners are curved in two directions and forged bosses are provided at these corners.

9. Design of columns. The columns will be designd both for the super-imposed loads transferred from the tank, as well as for the wind loads. Let us consider the wind pressure acting on the longer side. For the purposes of simplicity, let us assume that wind pressure on tank and beam acts over a depth of 3 m.

$\therefore$ Total $P_T = 8 \times 3 \times 1.5 = 36$ kN

The height of columns is 12 m, and this is divided into three panels of 4 m height each. Hence the resultant wind force P on the tank will act at a height of $1.5 + 0.3 + 12 = 13.8$ m above the base, as shown in Fig. 120.30.

The centre to centre spacing and placement of columns (6 Nos.) is shown in Fig. 20.31. Area of one face of column staging $= 7.2 \times 12 = 86.4\ \text{m}^2$

Let us assume the exposed area of columns and bracings as 20% of the area of one face.

Effective exposed area

$= 0.2 \times 86.4 = 17.28\ \text{m}^2$.

$\therefore$ Wind pressure on staging $= P_s = 17.28 \times 1.5 = 25.92$ kN, and this acts at a height of 6 m above the base.

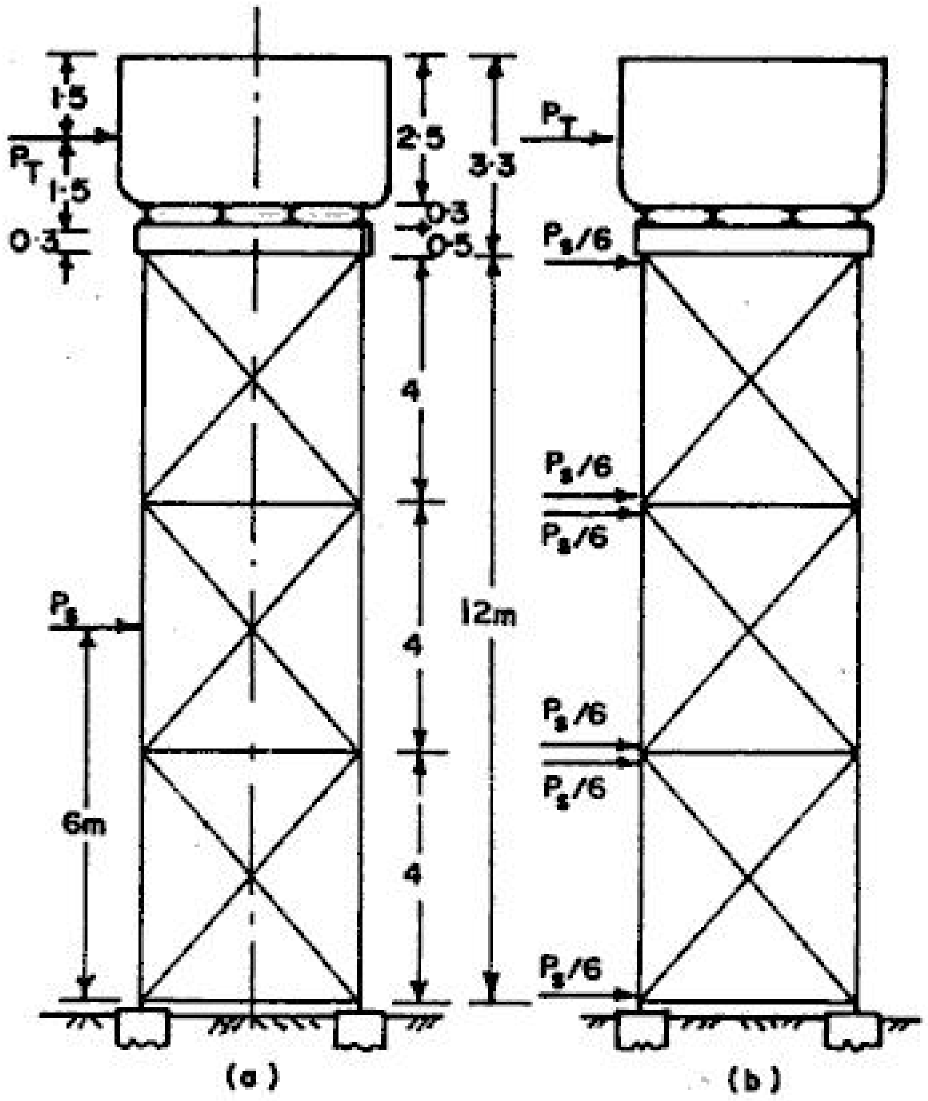

FIG. 20.30.

Moment of wind forces at the base is

$$M_w = 36 \times 13.8 + 25.92 \times 6 = 652.32 \text{ kN-m}.$$

This will produce thrust (R) in the three leeward columns and uplift in the three windward columns. The thrust and the uplift will be of equal magnitude (say R), and will provide the resisting couple.

$\therefore \quad 3R \times 3.6 = M_w$

$$\therefore \quad R = \frac{652.32}{3 \times 3.6} = 60.4 \text{ kN}$$

Alternatively, $\quad R = \dfrac{M_w . r}{\Sigma r^2}$

Where $\quad r = 1.8$ m ; $\Sigma r^2 = 6\,(1.8)^2 = 19.44$

$$\therefore \quad R = \frac{652.32 \times 1.8}{19.44} = 60.4 \text{ kN}$$

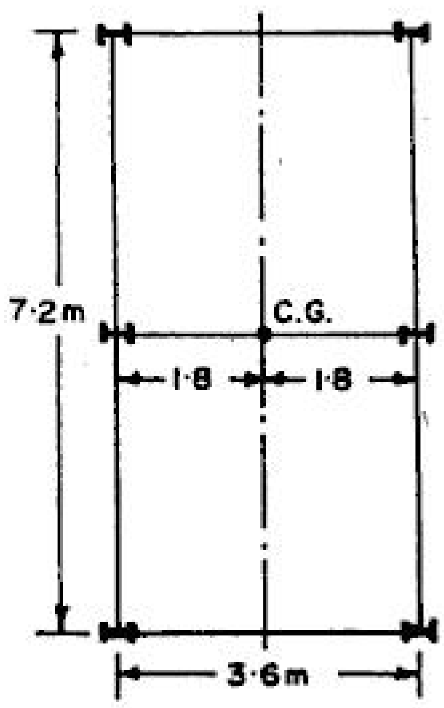

FIG. 20.31

Max. vertical load in column, when tank is full= 258.631 kN

When wind force is taken into account, the permissible stresses are increased by $33\frac{1}{3}$ %. Since the thrust due to wind load (*i.e.* 60.4 kN) is less than one-third of the super-imposed load, the design of the column will be done on the basis of super-imposed load of 258.631 kN. Let us assume the total load as 265 kN to account for the self weight also.

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21

Design of Towers and Masts

21.1. INTRODUCTION

A tower or mast is a tall skeleton structure with a relatively small cross-section, which has a large ratio between height and maximum width. A tower is a freely standing self supporting structure fixed to the base or foundation while a mast is tall structure, pinned to the base or foundation and braced with guys etc. Steel towers (short, medium and tall) are normally used for the following purposes.

(*i*) Electric power transmission
(*ii*) Microwave transmission for communication
(*iii*) Radio transmission (short and medium wave wireless)
(*iv*) Television transmission
(*v*) Satellite reception
(*vi*) Air traffic control
(*vii*) Flood light stand
(*viii*) Meteorological measurements
(*ix*) Derrick and crawler cranes
(*x*) Oil drilling masts.
(*xi*) Over head tanks.

The height of towers for electric power transmission may vary from 10 to 45 m while those for flood lights in stadiums and large flyover intersections may vary from 15 to 50 m. The height of television towers may vary from 100 m to 300 m while for those for radio transmission and communication networks the height may vary from 50 to 200 m. Though the height of a tower is fixed by the user, the structural designer has the task of designing (*i*) general configuration (*ii*) individual members, and (*iii*) joint details. The task of designing a stable tall tower *economically* is a challenging job for a structural engineer.

Depending upon the size and type of loading, towers are grouped into two heads.

(*a*) Towers with large vertical loads (*b*) Towers with mainly horizontal wind loads

Towers with large vertical loads (such as those of over head water tanks, oil tanks, meteorological instrumentation towers etc.) have their sides made up of vertical or inclined trusses. The towers, falling under the second category and subjected predominantly to wind loads, may be of two types :

Wind load

A detailed discussion on wind load on various structures has been given in Chapter 2. However, a brief is given here with special reference to towers.

The design wind speed V_z (m/s) is given by

$$V_z = V_b \, . \, k_1 \, . \, k_2 \, . \, k_3 \qquad ...(21.1)$$

where V_b = basic wind speed in m/s at 10 m height (Fig 2.1 and Table 2.6).

k_1 = probability factor (or risk coefficient). The value of which may be taken from Table 2.7

k_2 = terrain, height and structure size factor (Table 2.8)

k_3 = topography factor, the value of which varies from 1 to 1.4, depending upon the topography; for plain lands, $k_3 = 1$.

The design wind pressure is given by

$$p_z = 0.6\, V_z^2 \qquad ...(21.2)$$

where p_z = design wind pressure (N/m^2) and V_z is the design wind speed (m/s).

The wind force on any member is given by

$$F = A\, C_f\, p_z \qquad ...(21.3)$$

where A = effective frontal area.

C_f = net wind force coefficient, which depends upon solidity ratio φ of the tower (Table 21.1, 21.2 and 21.3)

φ = Solidity ratio

$$= \frac{\text{obstruction area of the front face}}{\text{gross area of the front face}}$$

For towers, φ varies from 0.15 to 0.3, and is to be *assumed* in the beginning of the design. After designing the members, the assumed solidity ratio is compared with the actual solidity ratio to test the adequecy of the structure.

TABLE 21.1. VALUES OF NET WIND FORCE COEFFICIENT C_f FOR TOWERS COMPOSED OF FLAT SIDED MEMBERS

Solidity ratio φ	*C_f for*	
	Square towers	*Equilateral triangular towers*
0.05	4.0	3.3
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

TABLE 21.2 FORCE COEFFICIENT C_f FOR SQUARE TOWERS COMPOSED OF ROUNDED MEMBERS

Solidity ratio φ	*C_f for*			
	$D.V_d \le 6\,m^2/s$		$D.V_d \ge 6\,m^2/s$	
	face	*corner*	*face*	*corner*
0.05	2.4	2.5	1.1	1.2
0.1	2.2	2.3	1.2	1.3
0.2	1.9	2.1	1.3	1.6
0.3	1.7	1.9	1.4	1.6
0.4	1.6	1.9	1.4	1.6
0.5	1.4	1.9	1.4	1.6

TABLE 21.3 FORCE COEFFICIENT C_f FOR EQUILATERAL-TRANGULAR TOWERS COMPOSED OF ROUNDED MEMBERS

Solidity ratio φ	*C_f for*	
	$D.V_d < 6\,m^2/s$ *(all wind directions)*	$D.V_d \ge 6\,m^2/s$ *(all wind directions)*
0.05	1.8	0.8
0.1	1.7	0.8
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
0.5	1.4	1.2

(3) Influence line for $P_{L_1 L_2}$

Pass a section *bb*.

$$P_{L_1 L_2} = \frac{M_{U_1}}{5} \text{ (tension)}$$

Hence the I.L. for $P_{L_1 L_2}$ will be a triangle, having a maximum ordinate of $\frac{4 \times 20}{24} \times \frac{1}{5} = \frac{2}{3}$ under L_1 as shown in Fig. 22.2 (*d*)

(4) Influence line for $P_{U_1 L_1}$

When the unit load is at L_0, $P_{U_1 L_1} = 0$

When the unit load is at L_1 $P_{U_1 L_1} = 1$ (tension)

When unit load is at L_2 or to the right of L_2, $P_{U_1 L_1} = 0$

The influence line for $P_{U_1 L_1}$ will therefore be a triangle having a maximum ordinate of unity under L_1 as shown in Fig. 22.2 (*e*).

(5) Influence line for $P_{L_0 U_1}$

The force in $L_0 U_1$ can be found by resolution of forces at A in the vertical direction. When the unit load is at A, $R_A = 1$, and hence $P_{L_0 U_1} = 0$. When the unit is at L_1, $R_A = \frac{20}{24} = \frac{5}{6}$, and

$$P_{L_0 U_1} = R_A \operatorname{cosec} \theta = \frac{5}{6} \times 1.28 = 1.07 \quad \text{(comp.)}$$

When the load is at B, $R_A = 0 \therefore P_{L_0 U_1} = 0$

The I.L. for $P_{L_0 U_1}$ is shown in Fig. 22.2 (*f*).

22.3. PRATT TRUSS WITH INCLINED CHORDS

Fig. 22.3 (*a*) shows Pratt truss with inclined chords, consisting of 6 panels each of a 4 m length.

(1) Influence line for $P_{L_1 L_2}$

Pass a section *aa* cutting three members

$$P_{L_1 L_2} = \frac{M_{U_1}}{U_1 L_1} = \frac{M_{U_1}}{3} \text{ (tension)}$$

The influence line diagram will therefore be a triangle having a maximum ordinate $= \frac{1}{3}\left(\frac{4 \times 20}{24}\right) = \frac{10}{9} = 1.111$ under L_1 as shown in Fig. 22.3 (*b*).

(2) Influence line for $P_{U_1 U_2}$

$$P_{U_1 U_2} = \frac{M_{L_2}}{x} \text{ (compression)}$$

where x is the perpendicular distance between point L_2 and $U_1 U_2$. In order to find x, prolong $U_2 U_1$ back to meet $L_2 L_1$ produced, in O.

Now $\tan \alpha = \frac{5-3}{5} = \frac{1}{2} \therefore \alpha = 26°34'$

$\therefore$ $\sin \alpha = 0.447$; $\cos \alpha = 0.894$

$$\therefore \qquad P_{U_2U_4} = \frac{1}{15}\left(\frac{5}{6} \times 24 - 1 \times 12\right) = \frac{8}{15} = 0.533 \text{ (comp.)}$$

When the unit load is at L_3 ,

$$R_A = \frac{1 \times 54}{72} = 0.75$$

$$\therefore \qquad P_{U_2U_4} = \frac{1}{15}(\ 0.75 \times 24\ ) = 1.2 \quad \text{(comp.)}$$

When the load is at B, $R_A = 0$

$$\therefore \qquad P_{U_2U_4} = 0$$

The influence line for $P_{U_2U_4}$ is shown in Fig. 22.7 (*c*).

(3) Influence line for $P_{U_2L_2}$

Pass a section *bb*. Consider equilibrium of the left portion

$$P_{U_2L_2} = \frac{M_A}{12} \text{ (tension)}$$

M_A and hence $P_{U_2L_2}$ are zero when the unit load is at A.

When the unit load is L_2

$$P_{U_2L_2} = \frac{1}{12}(1 \times 12) = 1 \text{ (tension)}$$

When the unit load is at L_3 or beyond L_3 on right side, there is no external force to the left of section *bb* except R_A . Hence $M_A = 0$. Therefore, $P_{U_2L_2}$ is zero. The I.L. for $P_{U_2L_2}$ is, therefore, a triangle having zero ordinates under A and L_3 and a maximum ordinate of unity under L_2, shown in Fig. 22.7 (*d*).

(4) Influence line for $P_{U_2M_3}$

Considering the equilibrium to the left of section *aa*,

$$P_{U_2M_3} \sin\theta = \text{shear in panel } L_2L_3 = F_{L_2L_3}$$

$$\sin\theta = \frac{15}{\sqrt{15^2 + 12^2}} = 0.781 \ ; \ \cos\theta = \frac{12}{\sqrt{15^2 + 12^2}} = 0.625$$

When the unit load is at L_2, $R_A = \dfrac{1 \times 60}{72} = 0.833$

$$F_{L_2L_3} = 1 - R_A = 1 - 0.833 = 0.167$$

$$P_{U_2M_3} = \frac{F_{L_2L_3}}{\sin\theta} = \frac{0.167}{0.781} = 0.214 \text{ (compression)}$$

When the unit load is at L_3 , $R_A = \dfrac{1 \times 54}{72} = 0.75 = F_{L_2L_3}$

$$\therefore \qquad P_{U_2M_3} = \frac{F_{L_2L_3}}{\sin\theta} = \frac{0.75}{0.781} = 0.96 \text{ (tension)}$$

When the unit load is at A or G, $F_{L_2L_3}$ and $P_{U_2M_3}$ are zero. The I.L. for $P_{U_2M_3}$ is shown in Fig. 22.7 (*e*).

(5) Influence line for $P_{M_3L_3}$

M_3L_3 is a secondary member.

When the unit load is at L_2 or to the left of L_2, $P_{M_3L_3}=0$

The influence line diagram for $P_{U_2U_4}$ is thus a triangle having a maximum ordinate of $\frac{24 \times 48}{72} \times \frac{1}{15} = 1.067$ (comp.) under L_4 as in Fig. 22.9 (*b*).

(2) Influence line for $P_{L_2L_4}$

$$P_{L_2L_4} = \frac{M_{U2}}{15} \text{ (tension)}$$

When the unit load is at L_2, $R_A = \frac{60}{72} = 0.833$

$$\therefore \quad P_{L_2L_4} = \frac{1}{15}(0.833 \times 12) = 0.667 \text{ (tension)}$$

When the unit load is at L_3, $R_A = \frac{54}{72} = 0.75$

$$\therefore \quad P_{L_2L_4} = \frac{1}{15}(0.75 \times 12 + 1 \times 6) = 1 \text{ (tension)}$$

When the unit load is at L_4, $R_A = \frac{48}{72} = 0.667$

$$\therefore \quad P_{L_2L_4} = \frac{1}{15}(0.667 \times 12) = 0.533 \text{ (tension)}$$

The I.L. diagram for $P_{L_2L_4}$ is shown in Fig. 22.9 (*c*).

(3) Influence line for $P_{M_3L_4}$

Resolving the forces vertically,

$$P_{M_3L_4} \sin\theta = \text{shear in panel } L_3L_4 = F_{L_3L_4}$$

or

$$P_{M_3L_4} = F_{L_3L_4} \operatorname{cosec}\theta$$

$$\sin\theta = \frac{15}{\sqrt{15^2 + 12^2}} = 0.781 \,;\, \operatorname{cosec}\theta = 1.28 \,;\, \cos\theta = 0.625$$

When the unit load is at L_3, $R_A = 0.75$

$$\therefore \quad P_{M_3L_4} = (1 - 0.75) \times 1.28 = 0.32 \text{ (comp.)}$$

When the unit load is at L_4, $R_A = 0.667$

$$\therefore \quad P_{M_3L_4} = 0.667 \times 1.28 = 0.853 \text{ (tension)}$$

The I.L. diagram for $P_{M_3L_4}$ is shown in Fig. 22.9 (*d*).

(4) Influence line for $P_{M_3L_3}$

M_3L_3 is a secondary member and hence it will be stressed only when the load is in panel L_2L_4 will be a Evidently, I.L. for $P_{M_3L_3}$ will be a triangle having zero ordinates under L_2 and L_4 and an ordinate of unity under L_3 as shown in Fig. 22.9 (*e*).

(5) Influence line for $P_{L_2M_3}$

Pass a horse shoe section *bb*, cutting five members U_2M_3, M_3L_1, L_2M_3, L_2L_3 and L_3L_4. Out of these, line of action of forces in four members pass through point L_4. Hence take the moment of the forces, about L_4 and consider the equilibrium of the portion enclosed by the horse shoe section. Note that L_2M_3 is a secondary member, and hence it will be stressed only when the load is in panel L_2L_4 .

When the unit load is at L_2, $M_{L_4} = 0$ and hence $P_{L_2M_3} = 0$

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When the spans of beams are not equal, substitute frames should be selected in which the largest span forms the centre span and also frames in which the smallest span forms the centre span. Several trial computations may be necessary to get the frame for which the bending moments are maximum. To get bending moment in the wall columns and wall beams, substitute frame shown in Fig. 24.12(*b*) should be used.

The bending moments due to dead loads are found separately. The bending moments for dead and live loads are then added, and the beam is designed.

(*b*) **Maximum bending moment in columns**

The bending moments in columns increase with increase in their rigidity. Hence they are largest in the lower storeys, and smallest in the upper storeys. The maximum compressive stresses in columns is found by combining maximum vertical loads with maximum bending moments. The maximum tensile stresses in columns is found by combining the maximum bending moment with minimum vertical loads. Though the bending moment is smallest in the upper floors, its effect is much larger since the dimensions of the columns are the smallest there and also the vertical loads are much smaller than in lower storeys. Also the possibility of tensile stresses in columns is much larger in upper storeys than in lower storeys.

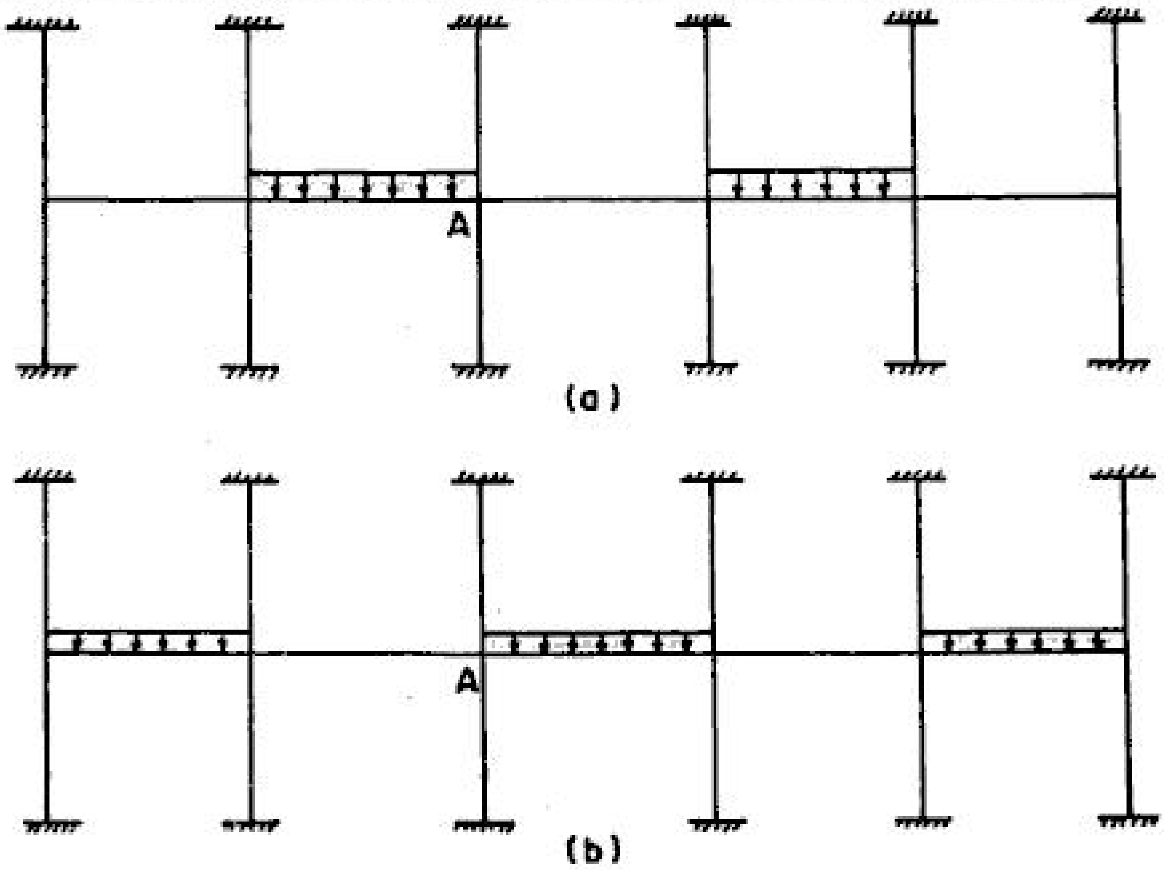

FIG. 24.14. LOADING FOR MAX. B.M. AT COLUMN A.

The maximum moments is columns occur when alternative spans are loaded as shown in Fig. 24.14(*a*),(*b*). The corresponding axial loads are found. The column is designed to resist the stresses provided by every combination of axial load and the corresponding moment.

24.7. METHODS OF COMPUTING B.M.

The bending moments in the beams and columns of a substitute frame may be computed by the following methods :

1. Slope-deflection method.
2. Moment distribution method.
3. Building frame formulae.
4. Kani's method.

The slope deflection method results in too many equations to be solved simultaneously. Hence moment distribution method, using two cycles is used. Taylor, Thomson and Smulski recommend the use of building frame formulae which they have developed using slope deflection equations.

Example 24.1. *Fig. 24.15 shows an intermediate frame of a multistoreyed building. The frames are spaced at 4 metres centre to centre. Analyse the frame taking live load of 4000 N/m² and dead load as 3000 N/m², 3250 N/m² and 2750 N/m² for the panels AB, BC and CD respectively. The self weight of the beams may be taken as under:*

Beams of 7 m span : 5000 N/m

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be anchored to the roof trusses (Fig. 25.6 *c*). If corrugated steel covering is used, sag rods may be dispensed with, assuming that the steel covering sheets do the work of transmitting the tangential component of the load. Sag rods are made from 16 to 20 mm dia. bars with screwed ends. Nuts are tighened after the erection of purlins over the trusses but before installation of sheeting, to induce some pretension in the sag rods.

If *one line of sag rods* is used, the tension T_s in the sag rods is given by

$$T_s = \left(\frac{L_1 L_2}{4\cos\theta}\right) W_g \sin\theta \qquad ...(25.1)$$

where L_1 is the span of the truss, L_2 is the spacing to trusses, θ is the slope of the truss and W_g is the design gravity load.

2. GANTRY GIRDER AND COLUMN BRACKET

Gantry girders or crane girders carry hand operated or electric overhead cranes in industrial buildings, to lift heavy materials, equipment etc. and to carry them from one location to the other within the building. The essential components of crane system are (Fig. 25.7 and 25.8):

1. Crane bridge or cross girder
2. Trolly or crab mounted on crane bridge
3. Gantry girder or crane grider
4. Crane runway (rail)
5. Column brackets.

The crane bridge spans the bay of the shop. The trolly or crab mounted on crane bridge, can travel transversely along the bridge. The bridge has wheels at the ends, and is capable of moving longitudinally on rails. The rails are mounted on *gantry girders*. The gantry girders span between brackets attached to columns which may either be of steel or of RCC. Thus the span of the gantry girder is equal to centre to centre spacing of columns. The complete procedure for the design of gantry girder, along with a design example is given in Chapter 9 of this book.

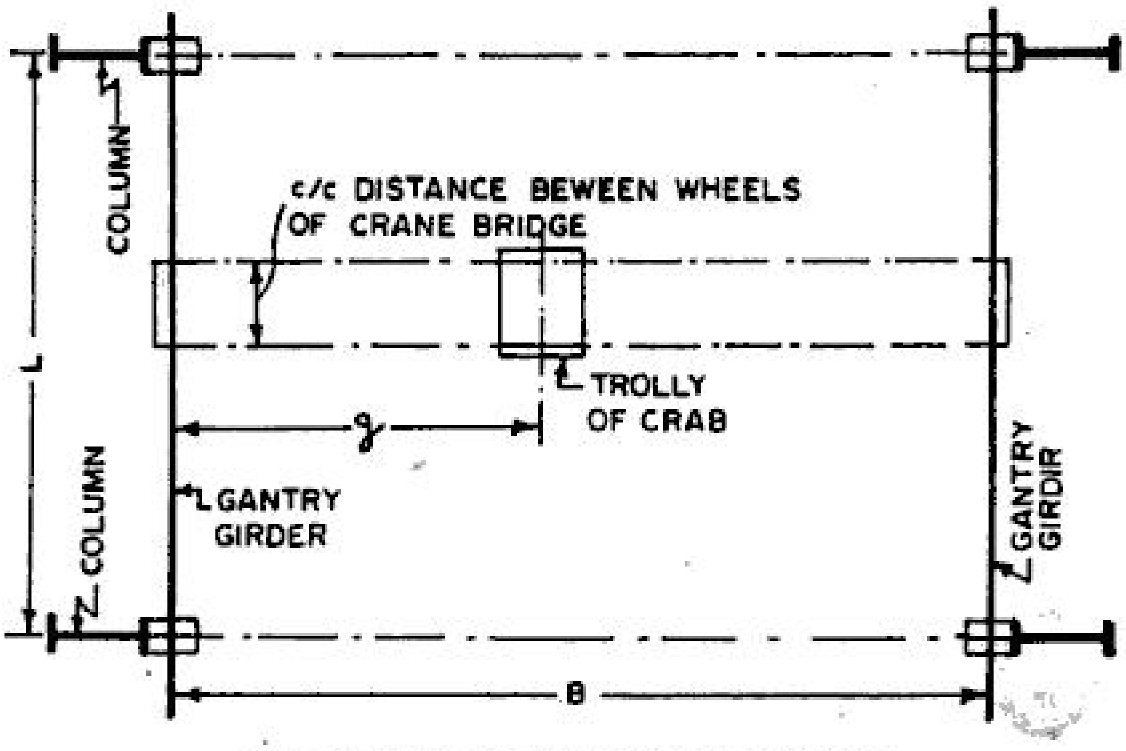

FIG. 25.7. PLAN OF CRANE SYSTEM.

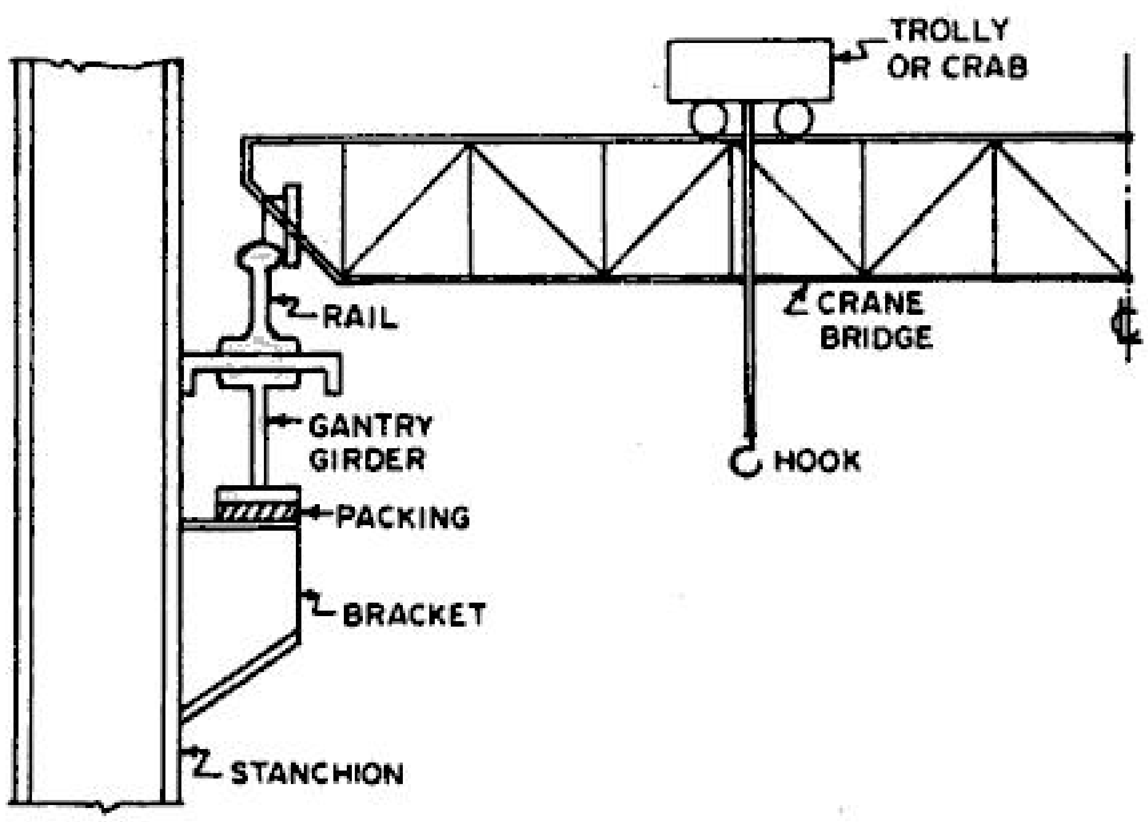

FIG. 25.8. COMPONENTS OF CRANE SYSTEM.

Solving (*i*) and (*ii*), we get

$$P_2 = \frac{P}{2k+1} \quad \text{and} \quad P_1 = \frac{kP}{2k+1} \qquad ...(iii)$$

It k is smaller than 1, $P_2 > P_1$. Hence as the load P is increased, smaller rod will yield first. The maximum value of load, which P_2 can attain is $\sigma_y . A$.

At the stage when yield occurs in the smaller bar, the total load P becomes equal to P_y which will be defined as the total load causing yield any where. The longer rods will be below their yield value. Putting $P_2 = \sigma_y A$ in Eq. (*i*) and noting

That $P_1 = k P_2 = k \sigma_y A$, we get from (*i*)

$$P_y = 2k\sigma_y A + \sigma_y A = \sigma_y A\,(2k+1) \qquad ...(iv)$$

The deflection at the yield load is given by

$$\Delta y = \frac{P_2 L_2}{AE} = \frac{\sigma_y A\,(kL)}{AE} = \frac{\sigma_y . kL}{E} \qquad ...(v)$$

As the load P is increased further after the central rod yields, unrestricted plastic flow does not occur, because the longer rods are still within elastic range and will continue to take more load. As the load is further increased, the two longer rods will also yield and the stress in each one of them will be the yield stress σ_y . When the outer two bars also start yielding, unobstructed plastic flow starts taking place. The load corresponding to this stage is then *ultimate load* (at the limit of usefulness) and its value is

$$P_u = 3 . \sigma_y A \qquad ...(26.3)$$

Thus the ultimate load is computed so easily. The basic reason for this simplicity is that the continuity condition need not be considered when the ultimate load in the plastic range is being computed.

The deflection when the ultimate load is first reached is given by

$$\Delta_u = \frac{\sigma_y . L}{E}$$

The three essential features of the above plastic analysis are :

(*i*) Each portion of the structure (*i.e.* each bar) reached a plastic yield condition

(*ii*) The equilibrium condition was satisfied at ultimate load, and

(*iii*) There was unrestricted plastic flow at the ultimate load.

(3) Frame consisting of three bars

Consider three bars *OA*, *OB* and *OC*, meeting at a common point *O* and hinged at the other ends *A*, *B* and *C* respectively (Fig. 26.4). Let a vertical load P be applied at the point *O*. We shall first solve the problem by elastic method.

Let P_1 be the tensile force in *OB* and P_2 be the force in *OA* and *OC*. Point *O* moves vertically to *O'* after the application of the load. Let Δ_1 and Δ_2 be the axial deformations of rods *BO* and *AO* (or *CO*) respectively. The dotted lines show the deformed portion of the structure.

From Statics, $P_1 + 2P_2 \cos\theta = P$ ...(1)

The structure is statically indeterminate to single degree. The second equation is obtained from the compatibility of deformations :

$$\Delta_2 = \Delta_1 \cos\theta$$

4, 3 and 2 may become plastic and failure may take place by rotation about A. The free body diagrams for both these possibilities are shown in Fig. 26.8 (*a*), (*b*).

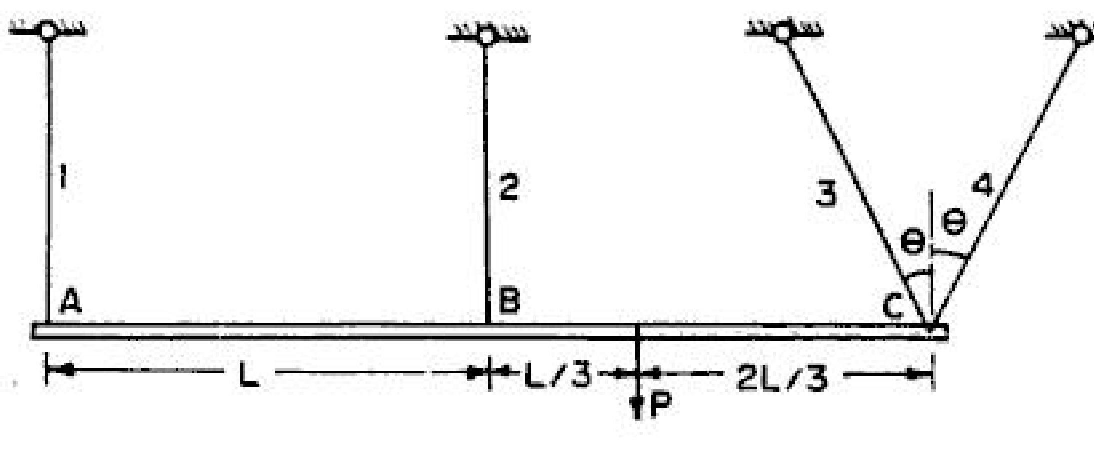

FIG. 26.7

The first possibility of collapse is shown in Fig. 26.8 (*a*). By inspection, rod 1 will yield first, when the force in it is $P_1 = \sigma_y . A$. With the further increase in the external load, P_1 will remain constant at $\sigma_y . A$ while P_2 will increase till it also becomes equal to $\sigma_y . A$. At this stage, the structure will turn into mechanism, and collapse will take place by rotation about C. Just before such collapse, we get, by taking moments about C.

$$\sigma_y . A . 2L + \sigma_y . A . L = P_{L1} \left(\frac{2L}{3} \right)$$

or

$$P_{L1} = \frac{9}{2} \sigma_y . A \qquad ...(1)$$

Let us now consider the second possibility, when the rod 4, 3 and 2 yield, and collapse takes place by rotation about A. At the yield stage, force carried by each of these rods is equal to $\sigma_y . A$. Hence we get by taking moments about A,

$$\sigma_y . A . L + 2\sigma_y . A \cos\theta . 2L = P_{L2} . \frac{4}{3} L$$

or

$$P_{L2} = \frac{3}{4} \sigma_y . A (1 + 4\cos\theta) \qquad ...(2)$$

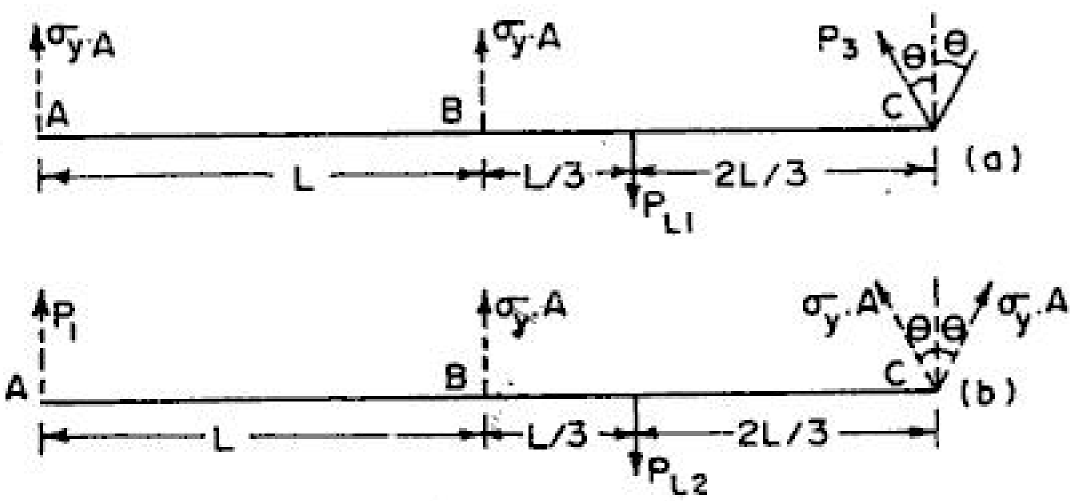

FIG. 26.8

It will be seen that P_{L2} is less than P_{L1} for all values of θ. Hence the collpase load is given by Eq. (2).

Thus,

$$P_u = \frac{3}{4} \sigma_y . A (1 + 4\cos\theta)$$

26.4. PLASTIC BENDING OF BEAMS

Let a beam be subjected to an increasing bending moment M (pure bending). The beam has atleast one axis of symmetry so that bending is symmetrical about that axis. When the bending stresses are within the elastic range, the bending stress distribution will be as shown in Fig. 26.9 (b – 1). The neutral axis will pass through the centroid of the section. As the moment is increased, yield stress will appear either in the top most or in the bottom most fibre as the case may be with the neutral axis still passing through the centroid of the section [Fig. 26.9 (b – 2)]. The moment at which the first yield has occurred is called the *yield moment* (M_y). With further increase of M, the yield will also occur in the bottom fibre and it will spread inwards in the top portion [Fig. 26.9 (b – 3)]. The neutral axis no longer passes through the centroid, its location being determined by the fact that the total compressive force is equal to the total tensile force over the cross-section.

In case of wind, earthquake, and other forces, specification normally provide a *one-third increase* in stresses. Hence load factor for combined dead, live and wind loading would be: $Q = \frac{3}{4} \times 1.85 = 1.40$. The *load factors* recommended by SP : 6(6) : 1972 are as follows :

(*i*) Dead plus live load : $Q = 1.85$

(*ii*) Dead plus live load, plus wind, earthquake etc : $Q = 1.40$

In general, the load factor is selected in such a way that the real factor of safety for any structure is at least as great as that afforded in the conventional design of simple beam.

IS 800-1984 recommends a min. load factor values of 1.7 and 1.3 respectively. See§ 26.14.

MARGIN OF SAFETY

The load factor Q accounts for the *margin of safety* in the plastic design. The Collapse load or ultimate load is found by multiplying the working load by the load factor.

Thus $W_C = F\,W$

In the case of dead loads, live loads plus wind or earthquake

$$W_{CW} = F_W\,W_W$$

In the case of elastic design, the margin of safety provided is 1.65. For mild steel used as structural steel, there is an additional reserve strength of 12% of yield load, due to ductility of steel. Hence in the plastic design also, there is a margin of safety of $1.12 \times 1.65 = 1.85$. *Thus, in plastic design, the margin of safety provided is in no way less than that provided in the elastic design.*

26.11. CONDITIONS AND BASIC THEOREMS OF PLASTIC ANALYSIS

1. Conditions of plastic analysis

In the elastic mehtod of analysis, three conditions must be satisfied : (1) continuity condition, (2) equilibrium condition, and (3) limiting stress condition. Thus, an elastic analysis requires that the deformations must be compatible, the structure should be in equilibrium and the bending moments anywhere in the structure should be less than M_y (or the stress should be less than σ_y).

Compared to this, an analysis according to the plastic method must satisfy the following *fundamental conditions* :

1. Mechanism condition. The ultimate load or collapse load is reached when a mechanism is formed. There must, however, be just enough plastic hinges that a mechanism is formed.

2. Equilibrium condition. The summation of the forces and moments acting on a structure must be equal to zero.

3. Plastic moment condition. The bending moment anywhere must not exceed the fully plastic moment.

Actually, these conditions are similar to those in elastic analysis which requires a consideration of (*i*) the *continuity*, (*ii*) the *equlibrium*, and (*iii*) the *limiting stress* condition. The similarity is demonstrated in Fig. 26.19.

With regard to continuity, in plastic analysis, the situation is just the reverse. Theoretically, plastic hinges interrupt continuity, so the requirement is that sufficient plastic hinges form to allow structure (or past of it) to deform as a mechanism. This could be termed as a *mechanism condition*. The *equilibrium condition* is the same. However, instead of *initial yield*, the limit of

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Hinge method of computation of deflection at ultimate load

The hinge method is based on the idealized M-ϕ relationship which means that each span retains its elastic flexural rigidity (EI) for the whole segment between the sections at which plastic hinges are located. Further, although 'kinks' form at the other hinge sections, just as the structure attains the computed ultimate load, there is still continuity at that section at which the last plastic hinge forms. As a consequence, the slope deflection equations may be used to solve for relative deflection of *segments* of the structure. With reference to the segment shown in Fig. 26.55 (*a*), we have the following slope deflection equations :

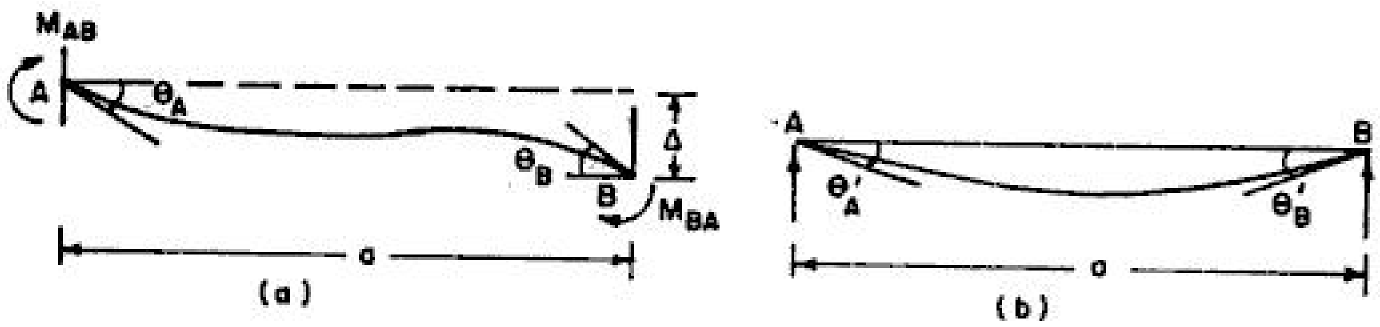

FIG. 26.55

$$\theta_A = \theta_A' + \frac{\Delta}{a} + \frac{a}{3EI}\left(M_{AB} - \frac{M_{BA}}{2}\right) \quad \text{...26.33 (a)}$$

and

$$\theta_B = \theta_B' + \frac{\Delta}{a} + \frac{a}{3EI}\left(M_{BA} - \frac{M_{AB}}{2}\right) \quad \text{...26.33 (b)}$$

where θ_A' and θ_B' are the slopes due to applied loading on a simply supported span of length a (Fig. 26.55 *b*) taken positive if clockwise. In Eqs. 26.33, clockwise moment and clockwise angle change are taken positive.

However, the basic question is : which hinge is last to form? A simple procedure consists of calculation of the deflection on the assumption that each hinge, in turn, is the last to form. The correct deflection at ultimate load is the maximum value obtained from the various trials. SP : 6(6) - 1972 recommends the following steps for computation of deflection at ultimate load :

Step 1 : Obtain the ultimate load, from plastic analysis.

Step 2 : Compute the deflection of the various *frame segments* assuming, in turn, that each hinge is last to form :

(*i*) Draw the free-body diagram of segment, and

(*ii*) Solve slope-deflection equation for assumed condition of continuity.

Step 3 : Correct deflection is the largest value (corresponds to last plastic hinge).

Step 4 : As a check : From a deflection calculation based on an arbitrary assumption, compute the 'kinks' formed due to the incorrect assumption. Remove the 'kinks' by mechanism motion and obtain correction deflection.

As an illustration, let us compute ultimate deflection δ_u of a fixed beam subjected to U.D.L. $W_u(=w_u \cdot L)$, shown in Fig. 26.56.

To start with, let us assume that hinge at 2 is formed last. The corresponding free body diagram is shown in Fig. 26.56 (*d*). Using this condition, the slope deflection equation for member $2-1$ is

The plastic moment of resistance of beam $M_P = 274.6$ kN-m

The significant values are as under :

$$0.85\, M_P = 0.85 \times 274.6 = 233.41 \text{ kN-m}$$

$$\frac{640\, v \,.\, r_y}{\sqrt{f_y}} = \frac{640 \times 1 \times 31.5}{\sqrt{250}} = 1276 \text{ mm}$$

$$\frac{960\, v \,.\, r_y}{\sqrt{f_y}} = \frac{960 \times 1 \times 31.5}{\sqrt{250}} = 1912 \text{ mm}$$

Referring to the B.M. diagram shown in Fig. 26.54, the critical flange is the top flange at $0.414 \times 7 \approx 2.9$ m from D and the bottom flange at C, where plastic hinges are formed. Let us now calculate the value of a in which moment $\geq 0.85\, M_P$.

In span DC $$V_D = \frac{60 \times 7}{2} - \frac{274.6}{7} = 170.77 \text{ kN}$$

$$M_x = 170.77x - \frac{60x^2}{2} = 0.85\, M_P = 233.41$$

or $x^2 - 5.692x + 7.78 = 0$. From which $x = 2.28$ m and 3.41 m

$\therefore$ $$a = 3.41 - 2.28 = 1.13 \text{ m} = 1130 \text{ mm}$$

$\therefore$ $$a < 640\, v\, r_y / \sqrt{f_y}$$

Hence one lateral brace at M_P location in adequate.

Note : If a were greater than 640 $v \,.\, r_y/\sqrt{f_y}$, two lateral bracings would be required, equidistant from M_P location at 1276/2 m on either side. For example, if a were equal to 1800 mm, then provide two lateral bracings equidistant from M_P location at 1276/2 = 638 mm on either side. Then the remaining length = 1800/2 − 638 = 262 mm, which would be smaller than a, and hence no other brace would be required.

At support C. In span CD, a is given by

$$M_P \frac{(L-a)}{L} - \left(\frac{w_u \times L}{2} + \frac{M_P}{L}\right) a = 0.85\, M_P$$

or $$274.6 \frac{(7-a)}{7} - \left(\frac{60 \times 7}{2} + \frac{274.6}{7}\right) a = 233.41$$

From which $a = 0.143$ m

In span CB, a is given by

$$\frac{100 \times 6}{2} a = M_P - 0.85\, M_P = 274.6 - 233.41$$

From which $a = 0.137$ m

$\therefore$ Total length $a = 0.143 + 0.137 = 0.28$ m $= 280$ mm. Hence $a < 640\ v \,.\, r_y/\sqrt{f_y}$

Therefore one lateral brace at support point is adequate. The check for *elastic buckling* in span CB can be carried out as usual.

Proceeding on similar lines, it can be shown that no lateral brace is required in span AB. However, provide lateral brace at support B.

Example 26.23. Design of continuous beam of variable section

Design the continuous beam ABCD, analysed in example 26.16 (a), providing the most economical section. Take $\sigma_y = 250\ N/mm^2$.

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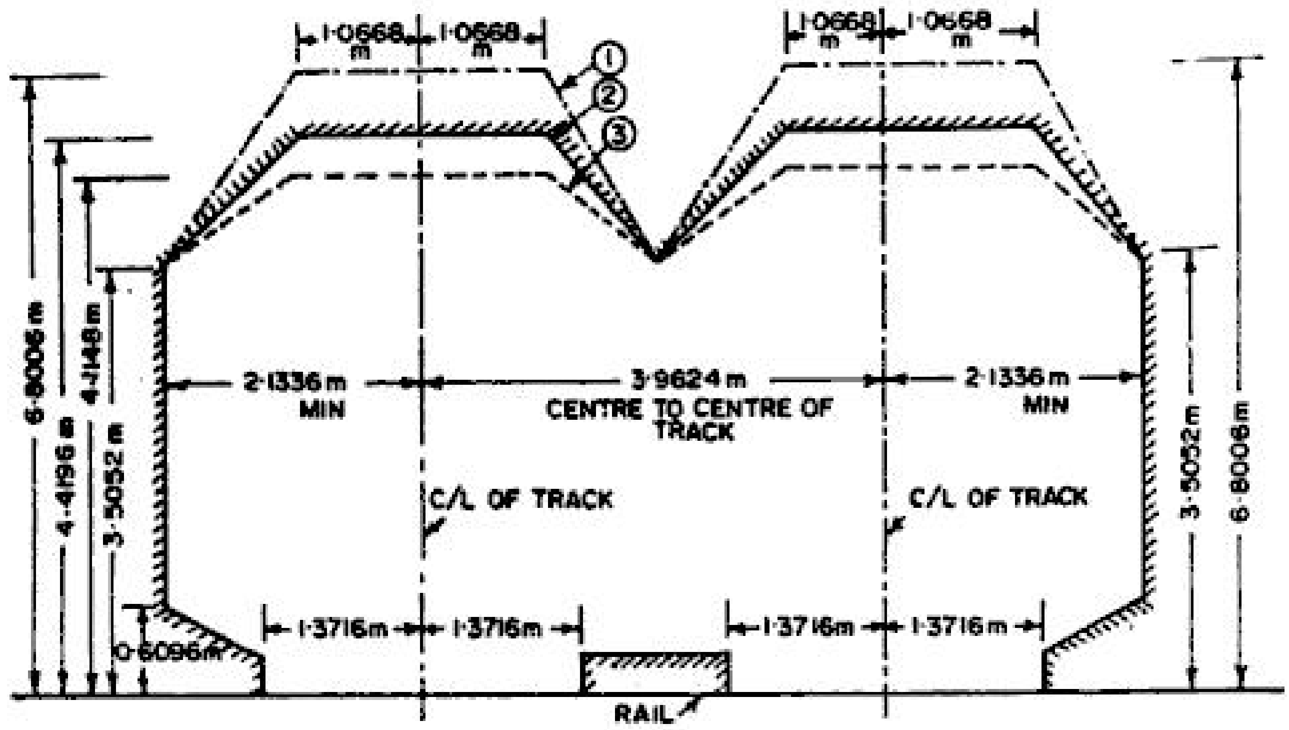
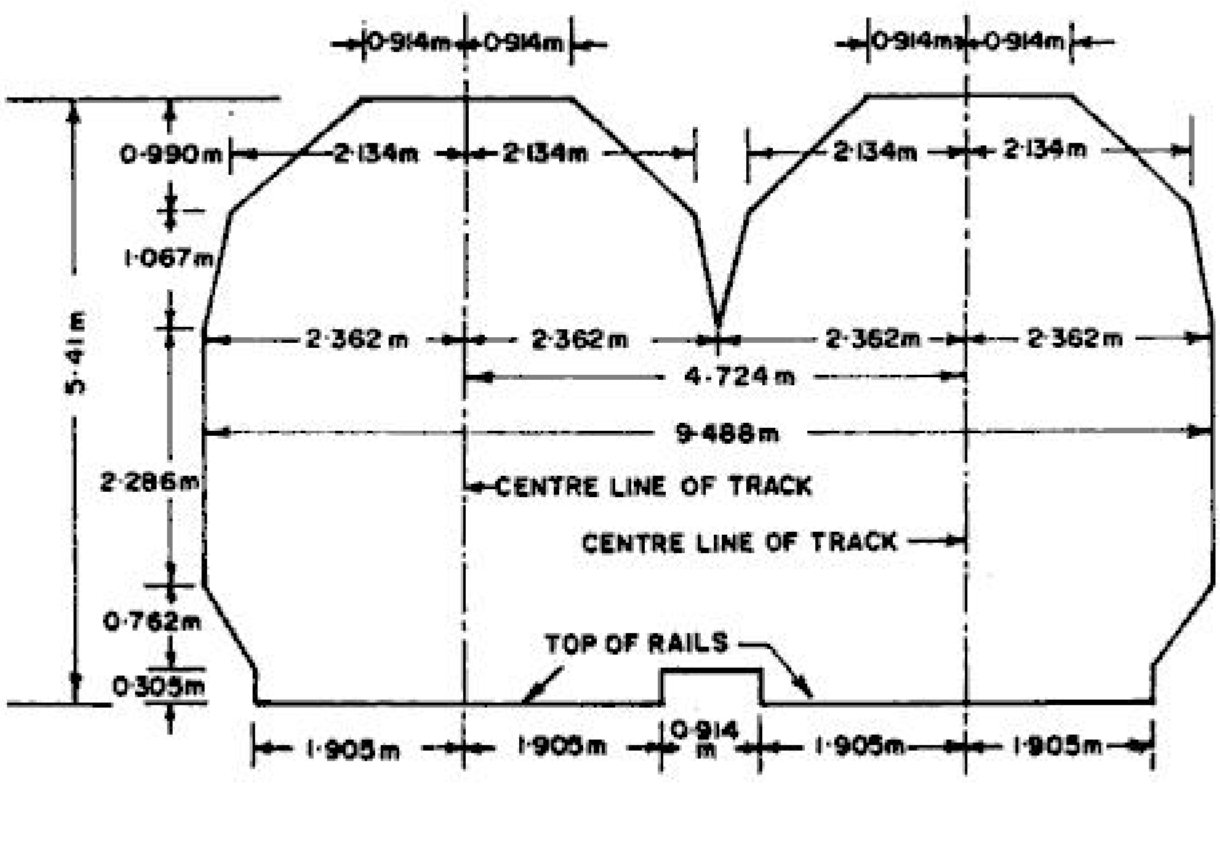


1. CHAIN DOTTED LINE INDICATES THE MINIMUM OUTLINE FOR TUNNEL AND OVER BRIDGE WHERE ELECTRIC TRACTION IS IN USE OR LIKELY TO BE USED.
2. FULL LINE INDICATES THE MINIMUM OUTLINE FOR TUNNEL AND OVER BRIDGE WHERE ELECTRIC TRACTION IS NOT LIKELY TO BE USED.
3. MINIMUM OUTLINE FOR THROUGH AND SEMI THROUGH BRIDGES (EVEN ELECTRIC TRACTION IS IN USE OR LIKELY TO BE USED).

(A) CLEARANCE DIAGRAM FOR METRE GAUGE



(B) CLEARANCE DIAGRAM FOR BROAD GAUGE.
FIG. 27.9 CLEARANCE DIAGRAM FOR RAILWAY BRIDGE.

TABLE 28.1 (A) : EQUIVALENT UNIFORMLY DISTRIBUTED LOAD ON EACH TRACK AND IMPACT FACTOR FOR BG BRIDGES (REVISED 1975) (EUDL)

L (m)	*Total load for B.M.*		*Total Load for shear*		CDA $0.15 + \frac{8}{6+L}$
	tonnes	*kN*	*tonnes*	*kN*	
1	45.8	449.1	45.8	449.1	1.000
2	45.8	449.1	51.7	507.0	1.000
3	46.2	453.1	64.5	632.5	1.000
4	55.8	547.2	73.0	715.9	0.950
5	66.5	652.2	85.4	837.5	0.877
6	77.9	763.9	93.7	918.9	0.817
7	86.1	844.4	100.4	984.6	0.765
8	92.2	904.2	110.7	1086	0.721
9	96.9	950.3	118.8	1165	0.683
10	103.6	1016	125.2	1228	0.650
12	125.2	1228	138.6	1359	0.594
14	136.0	1334	151.1	1482	0.550
15	142.1	1394	157.8	1548	0.531
16	147.5	1446	165.1	1619	0.514
18	162.3	1592	181.2	1777	0.483
20	177.1	1737	199.1	1953	0.458
22	192.9	1892	214.3	2102	0.436
24	207.4	2034	227.5	2231	0.417
25	213.7	2096	235.6	2311	0.408
26	221.8	2175	243.1	2384	0.400
28	237.6	2330	257.5	2525	0.385
30	252.0	2471	273.3	2680	0.372
35	286.6	2811	312.5	3065	0.345
40	325.2	3189	349.8	3430	0.324
45	360.9	3539	389.7	3822	0.307
50	397.2	3895	426.7	4184	0.293
60	470.9	4618	503.7	4940	0.271
70	546.9	5363	581.2	5700	0.255
80	621.8	6098	657.9	6452	0.243
90	698.0	6845	734.1	7199	0.233
100	775.1	7601	810.6	7949	0.225
110	851.1	8346	887.5	8703	0.219
120	926.7	9088	964.4	9458	0.213
130	1002.8	9834	1041.5	10214	0.209

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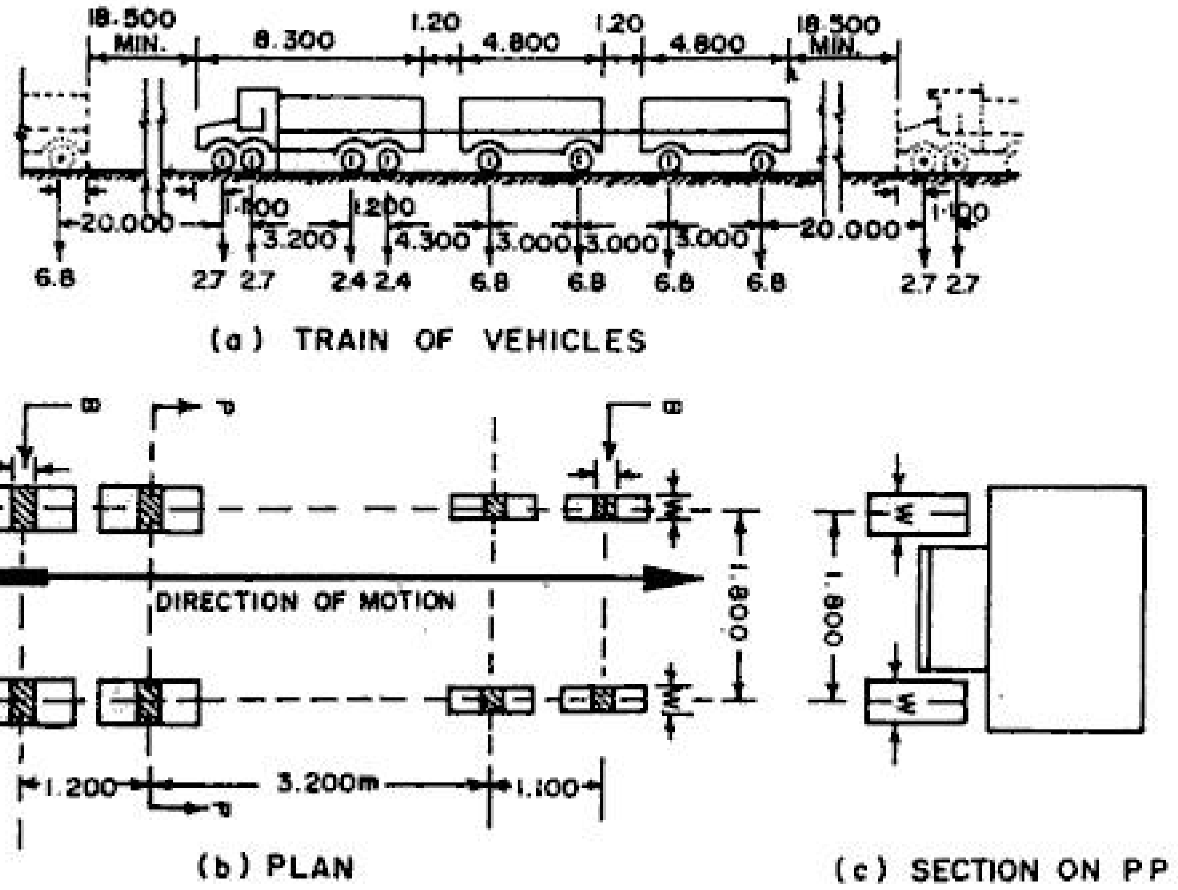

FIG 28.7 IRC CLASS A LOADING

Note 3: The ground contact area of the wheels shall be as under (Table 28.5).

TABLE 28.5 GROUND CONTACT AREA OF CLASS A VEHICLE WHEELS

Axle load (tonnes)	*Ground contact area*	
	B (mm)	*W (mm)*
11.4	250	500
6.8	200	380
2.7	150	200

Note 4: The minimum clearance, *f*, between outer edge of the wheel and the roadway face of the kerb, and the minimum clearance '*g*' between the outer edges of passing or crossing vehicles (Fig 28.8) on multi-lane bridges shall be as given in Table 28.6.

TABLE 28.6 VALUES OF *g* AND *f*

Clear carriage width	*g*	*f*
(*i*) 5.5 m to 7.5 m	Uniformly increasing from 0.4 m to 1.2 m	150 mm for all carriage way widths
(*ii*) Above 7.5 m	1.2 m	

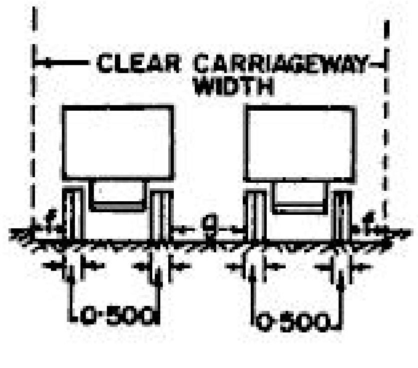

FIG 28.8

3. IRC CLASS B LOADING

This loading is to be normally adopted for temporary structures and for bridges in specified areas. The loading consists of one driving unit with two trailers which are not detachable from the driving unit. The details of the loading are shown in Fig. 28.9.

Note 1: The nose to tail distance between successive trains shall not be less than 18.4 m.

TABLE 28.10. (Contd.)

5.0	22.5	220.7	13.7	134.4	14.4	141.2	10.3	101.1
5.5	22.5	220.7	15.2	149.1	15.1	148.1	11.1	108.8
6.0	22.5	220.7	15.2	149.1	15.9	155.9	11.8	115.7
6.5	22.5	220.7	18.3	179.5	16.5	161.8	12.4	121.6
7.0	22.5	220.7	18.3	179.5	16.9	165.7	12.9	126.5
7.5	22.5	220.7	18.3	179.5	17.4	170.5	13.4	131.4
8.0	22.5	220.7	18.3	179.5	17.9	175.5	14.0	137.3
8.5	25.0	245.2	18.3	179.5	18.4	180.4	14.6	143.2
9.0	27.5	269.7	18.3	179.5	18.7	183.4	15.0	147.1
9.5	27.5	269.7	20.4	200.1	19.0	186.3	15.4	151.0
10.0	27.5	269.7	22.7	222.6	19.2	188.3	15.8	154.9
11.0	32.5	318.7	22.7	222.6	19.8	194.2	16.6	162.8
12.0	32.5	318.7	22.9	224.6	20.6	202.0	17.6	172.6
13.0	37.5	367.8	24.8	243.2	21.2	207.9	18.5	181.4
14.0	37.5	367.8	27.5	269.7	21.8	213.8	19.3	189.3
15.0	45.0	441.3	27.5	269.7	22.3	218.9	20.1	197.1
16.0	45.0	441.3	28.3	277.5	22.8	223.9	21.0	205.9
17.0	45.0	441.3	32.1	314.8	23.3	228.5	21.7	212.8
18.0	45.0	441.3	32.1	314.8	23.6	231.4	22.3	218.9
19.0	45.0	441.3	36.6	358.9	23.8	233.4	22.9	224.6
20.0	50.0	490.3	36.6	358.9	24.0	235.4	23.4	229.5
21.0	50.0	490.3	36.6	358.9	24.2	237.3	23.9	234.4
22.0	55.0	539.4	41.0	402.1	24 4	239.3	24.4	239.3
23.0	55.0	539.4	41.0	402.1	24.6	241.2	25.0	245.2
24.0	60.0	588.4	41.2	404.0	24.8	243.2	25.5	250.1
25.0	60.0	588.4	41.2	404.0	24.8	243.2	26.0	254.0
26.0	60.0	588.4	45.8	449.2	24.8	243.2	26.6	260.9
27.0	60.0	588.4	45.8	449.2	24.8	243.2	27.3	267.7
28.0	60.0	588.4	46.7	457.9	24.8	243.2	27.9	273.5
29.0	65.0	637.4	50.4	494.3	24.8	243.2	28.4	273.5
30.0	65.0	637.4	50.4	494.3	24.8	243.2	29.0	284.4
32.0	70.0	686.5	55.0	539.4	24.8	243.2	30.0	294.2
34.0	75.0	735.5	59.3	581.5	24.8	243.2	30.9	303.0
36.0	75.0	735.5	59.5	583.5	24.8	243.2	32.0	313.8
38.0	75.0	735.5	64.1	628.6	24.8	243.2	32.9	322.6
40.0	75.0	735.5	65.1	638.4	24.8	243.2	33.9	332.5
42.0	75.0	735.5	68.7	673.7	24.8	243.2	34.5	338.3
44.0	75.0	735.5	73.7	718.8	24.8	243.2	35.2	345.2
46.0	75.0	735.5	77.6	761.0	24.8	243.2	35.8	351.1

$$\therefore \quad M_{R_1} = \frac{3.5244 \times 10^{10}}{1012} \times 138 \times \frac{15564}{18112} = 4130 \times 10^6 \text{ N-mm} = 4130 \text{ kN-m}$$

From Eq. (*i*), M_{x_1} becomes equal to M_{R_1} at distance x_1 given by

$5127.6 - 35.61 x_1^2 = 4130$

This gives $x_1 = 5.29$ m from mid-span or at $12 - 5.29 = 6.71$ m from ends.

Similarly, with both plates curtailed from each of the flanges

$$I_{xx0} = 0.8 \times 10^{10} + 1.51 \times 10^{10} = 2.31 \times 10^{10} \text{ mm}^4$$

$$A_{ft0} = 24112 - 12000 = 12112 \text{ mm}^2 \; ; \; A_{ft0}' = 21048 - 10968 = 10080 \text{ mm}^2$$

$$\therefore \quad M_{R_0} = \frac{2.31 \times 10^{10}}{1000} \times 138 \times \frac{10080}{12112} = 2653 \times 10^6 \text{ N-mm} = 2653 \text{ kN-m}$$

This occurs at a section distant x_0 from mid-span, given by

$5127.6 - 35.61 x_0^2 = 2653$ which gives $x_0 = 8.34$ m

The theoretical points of cut off (T.P.O.C.) are shown in Fig. 29.13 (*a*). Plates should however, be extended beyond the T.P.O.C. by a length sufficient to a accommodate average compressive stress in the outer plate, at the point of cut off $= \frac{M_{x_1}}{I_{xx}} . y_{mean}$

$$\therefore \quad \sigma_{bc} = \frac{4130 \times 10^6}{4.7688 \times 10^{10}} \times (1024 - 6) = 88.16 \text{ N/mm}^2$$

$$\therefore \quad \sigma_{bt} = \sigma_{bc} \times \frac{A_{ft}}{A_{ft}'} = 88.16 \times \frac{24112}{21048} \approx 101 \text{ N/mm}^2$$

$\therefore$ Tensile force in the curtailed plate $= 101\,(10968/2) \times 10^{-3} = 554$ kN

Rivet value of 20 mm dia rivets in single shear $= \frac{\pi}{4}(21.5)^2 \times 100 \times 10^{-3} = 36.3$ kN

$\therefore$ No of rivets required $= 554/36.3 \approx 16$.

These rivets are to be provided in two rows. Hence provide 8 rivets in each row. Keeping pitch equal to 3 times the rivet diameter and edge distance equal to 1.5 times the dia., the distance of actual point of cutoff $= 5.29 + 8 \times 3 \times 21.5 \times 10^{-3} \approx 5.29 + 0.52 = 5.81$ m from mid span.

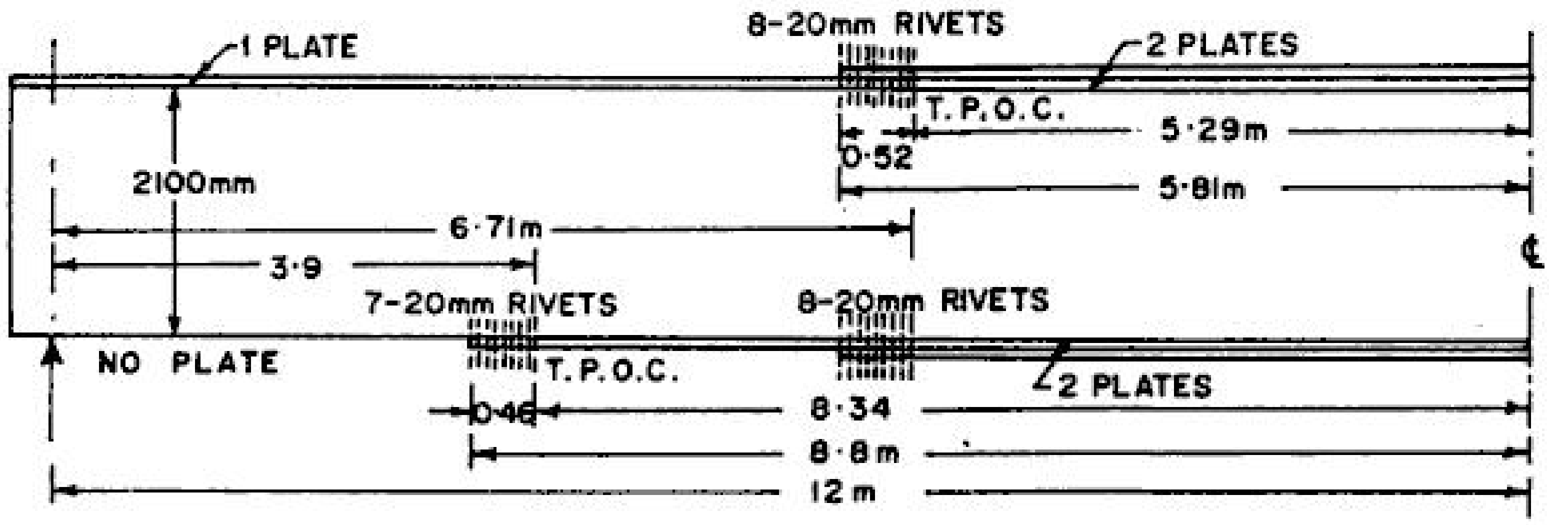

FIG. 29.15

Similarly, average compressive stress in the inner plate at the point of cutoff is,

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Design of end strut : $P = 207.36$ kN

Let us provide two angles connected to the same side of gusset plate. Hence effective length $= 1.0 \times 5 = 5$ m. Let us assume $\lambda = 120$. Hence from Table 28.15, $\sigma_{ac} = 62 \text{ N/mm}^2$.

$\therefore$ Required area $= 207.36 \times 10^3/62 = 3345 \text{ mm}^2$. Using 2 ISA 130 × 130 × 10 mm placed on the same side of a 10 mm thick gusset plate, we get :

Area $a = 2506 \text{ mm}^2$; $r_{xx} = 40.1$ mm

$\therefore$ $l/r = 5000/40.1 \approx 125$. Hence $\sigma_{ac} = 58.5 \text{ N/mm}^2$

$\therefore$ Load capacity $= 2 \times 2506 \times 58.5 \times 10^{-3} = 293.2 \text{ kN} > 207.36$ kN. Hence OK.

No. of 20 mm dia. rivets $= 207.36/36.3 \approx 6$.

Hence provide 3 rivets on each angle for connecting these to 10 mm thick gusset plate.

Design of end diagonal member : $P = 221.3$ kN (tension)

Permissible tensile stress $= 138 \text{ N/mm}^2$ (Table 28.13)

$\therefore$ $A_{net} = 221.3 \times 10^3/138 = 1604 \text{ mm}^2$ and Gross area $\approx 1.2 \times 1604 = 1924 \text{ mm}^2$

Try 2 ISA 80 × 80 × 8, each having $a = 1221 \text{ mm}^2$. Using 20 mm dia. rivets, deduction for rivet holes $= 21.5 \times 8 = 172 \text{ mm}^2$ for each angle.

$\therefore$ Total net area provided $= 2\,[1221 - 172] = 2098 \text{ mm}^2$. Hence OK.

No. of 20 mm dia rivets $= 221.3/36.3 \approx 6$.

Hence provide 3 rivets on each angle for connecting these to the 10 mm thick gusset plate. Similarly, the struts and diagonals for intermediate panels can be designed.

Step 11 Design of internal gusset plates

In order to resist the wind pressure on the plate girder, let us provide internal gusset plates @ 3 m c/c, i.e. at the same interval at which cross-girders are provided.

The gusset plate will be provided between the base of top flange angles and top of cross-beam. As shown in Fig. 29.40, available height of the gusset plate will be $= 2.048 - (0.18 + 0.57 + 0.174) = 1.124$ m

Total wind force on plate girder $= P_{w1} + P_{w2}$ $= 147.5 + 126 = 273.5$ kN

Since gusset plates are being provided @ 3m c/c, there will be 24/3=8 panels. Hence wind force on each panel point $P = 273.5/8 \approx 34.18$ kN.

It is assumed that half of this load $P(=34.15 \text{ kN})$ acts at top and half at bottom, as shown in Fig. 29.42.

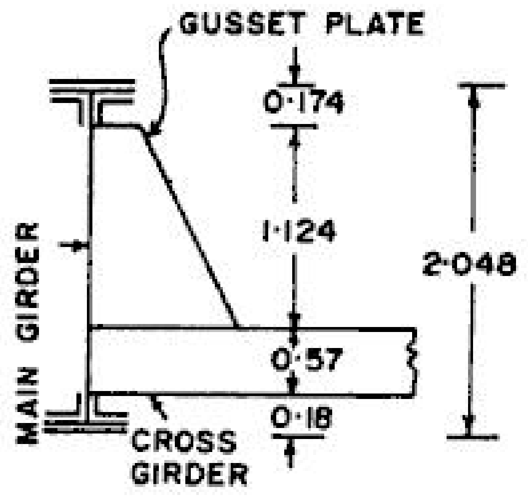

FIG. 29.40

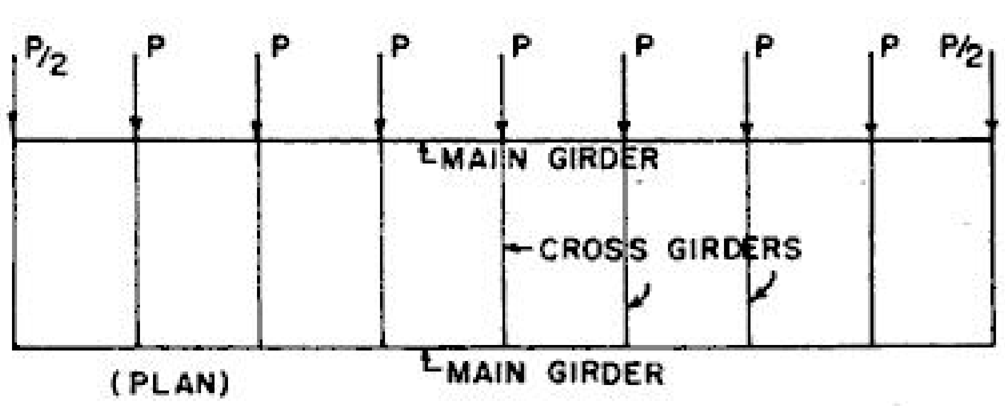

FIG. 29.41

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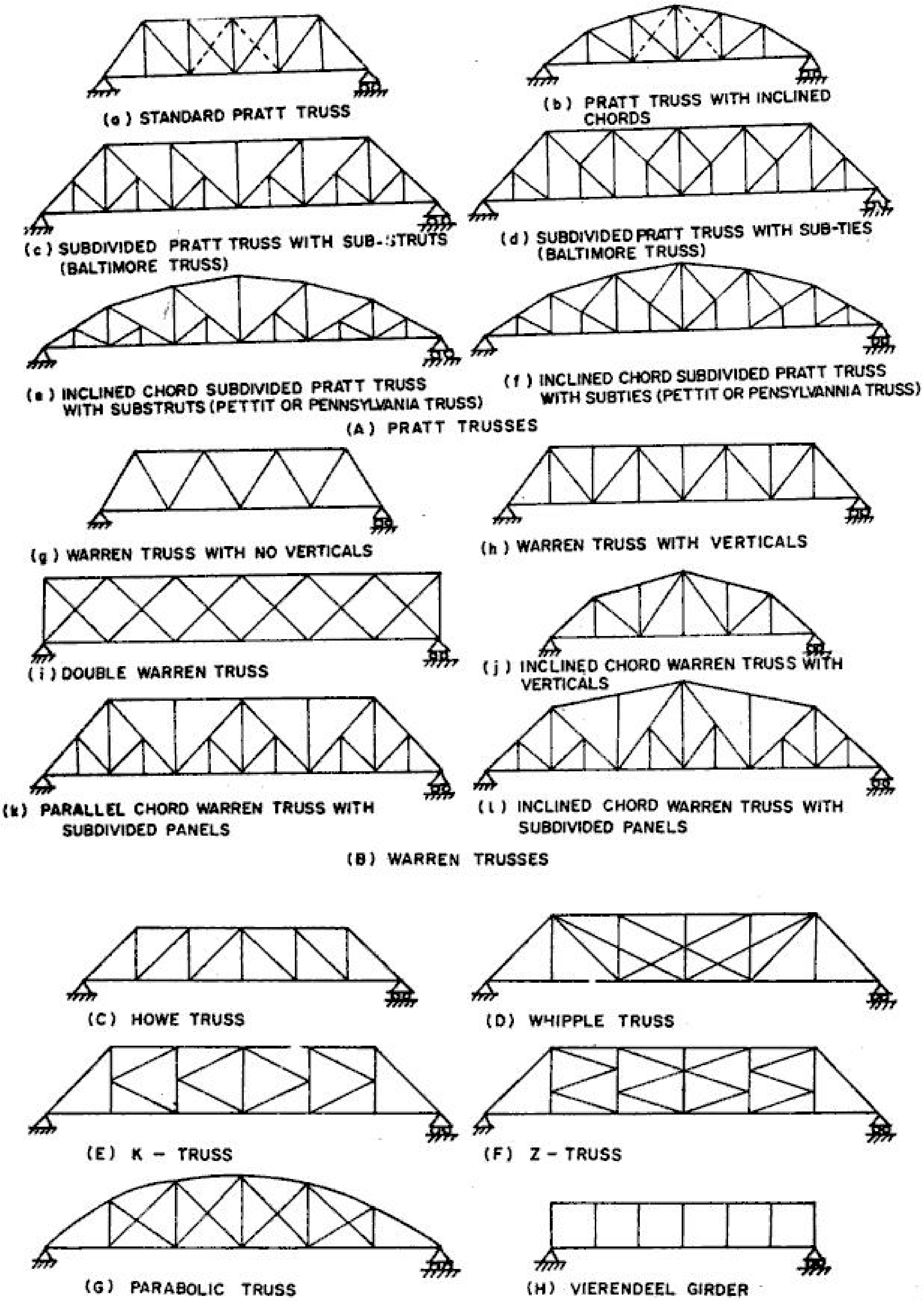

FIG. 30.4. TYPES OF TRUSS GIRDERS FOR BRIDGES

TABLE 30.12

Element	*Gross area* (mm^2)	*L.A. from top fibre (mm)*	*Moment of area about top fibre* (mm^3)
(*i*) Cover plate 300 × 12	3600	6	21600
(*ii*) Two web plates 300 × 12	7200	162	1166400
(*iii*) 4-ISA 100 × 75 × 12	4 × 1956 = 7824	162	1267488
Total	18624		2455488

$\therefore\ \bar{y} = 2455488/18624 = 131.85$ mm

$$I_{xx} = 3600\,(131.85-6)^2 + \frac{2\times 12\,(300)^3}{12} + 7200\,(162-131.85)^2$$
$$+\,4\times 187.5\times 10^4 + 2\times 1956\,(131.85-44.7)^2 + 2\times 1956\,(312-131.85-32.7)^2$$

$$I_{yy} = \frac{1}{12}\times 12\,(300)^3 + 2\times 300\times 12\,(150-6)^2 + 4\times 89.5\times 10^4 + 4\times 1956\,(150-12-20.3)^2$$
$$= 28827\times 10^4\ \text{mm}^4$$

$\therefore\quad r_x = \sqrt{23983\times 10^4/18624} = 113.5$ mm and $r_y = \sqrt{28827\times 10^4/18624} = 124.4$ mm

Length of member = 9552 mm (Fig. 30.33)

$\therefore\ l_x = 0.7\times 9552 = 6686$ mm and $l_y = 0.85\times 9552 = 8119$ mm (See Table 30.1)

$$\therefore\quad \lambda_x = \frac{l_x}{r_x} = \frac{6686}{113.5} = 58.9 \text{ and } \lambda_y = \frac{l_y}{r_y} = \frac{8119}{124.4} = 65.3$$

For normal loads :

From Table 28.15, $\sigma_{ac} = 111.5\ \text{N/mm}^2$ (corresponding to $\lambda_{max} = \lambda_y = 65.3$)

Also, from Table 28.13, $\sigma_{bc} = 147\ \text{N/mm}^2$

Now $$\sigma_{ac,\,cal} = \frac{1414.6\times 10^3}{18624} = 75.96\ \text{N/mm}^2$$

and $$\sigma_{bc,\,cal} = \frac{73.46\times 10^6}{28827\times 10^4}\times 150 = 38.22\ \text{N/mm}^2$$

Hence from Eq. 30.9, $\dfrac{\sigma_{ac,\,cal}}{\sigma_{ac}} + \dfrac{\sigma_{bc,\,cal}}{\sigma_{bc}} = \dfrac{75.96}{111.5} + \dfrac{38.22}{147} = 0.681 + 0.26 = 0.941 < 1.$ OK.

For occasional loads (i.e. wind load consideration)

$$\sigma_{ac,\,cal} = \frac{1562.1\times 10^3}{18624} = 83.88\ \text{N/mm}^2\ ;\ \sigma_{bc,\,cal} = \frac{111.93\times 10^6}{28827\times 10^4}\times 150 = 58.24\ \text{N/mm}^2$$

$$\therefore\quad \frac{\sigma_{ac,\,cal}}{\sigma_{ac}} + \frac{\sigma_{bc,\,cal}}{\sigma_{bc}} = \frac{83.88}{111.5} + \frac{58.24}{147} = 0.752 + 0.396 = 1.148 < 1.167$$

Hence the section is safe.

(*v*) ***Diagonal*** U_1L_2 (tension)

The diagonal U_1L_2 carries tensile force. Let us choose a section comprising of four angles and two web plates, with their outer distance equal to 300 mm, so as to fit in the gusset plates. (Fig. 30.40). Force with normal loads = 980.6 kN

Force with occasional loads = 1056.6 kN

Permissible stress in axial tension, under normal loads = 138 N/mm²

steel laminates by the process of vulcanisation. The bearing caters for translation and/or rotation of the superstructure by elastic deformation. Usually, laminated elastomeric bearings are used. Such bearings are composed of alternate layers of *elastomer* and *laminates*, internally bonded during vulcanisation. A *laminate* is a reinforcing material(i.e. thin sheets of mild steel) integrally bonded to elastomer during vulcanisation process, to restrain the lateral expansion of the elastomer. Elastomeric bearings provide horizontal movement upto about 70 mm and rotation about horizontal axis upto about 0.02 radians.

Combined mechanical and elastomeric bearings : When the required horizontal movement exceeds the practical limit of 70 mm provided for in elastomeric bearings, combined mechanical and elastomeric bearings are used. In such a type of combined bearing, the mechanical bearings provide for horizontal movement while the elastomeric bearings provide for rotational movements.

31.4. SLIDING BEARINGS OR PLATE BEARINGS

A *sliding bearing* is the one which permit *sliding movement* between two surfaces. Since the two surfaces consists of two plates, such a bearing is also known as a *plate bearing.* Fig. 31.1 shows typical sliding bearing consisting of two plates. Fig. 31.2 shows the elements of a sliding bearing (IRC : 83-1982).

The simplest type of sliding bearing consists of two plates : (*i*) sole plate or shoe plate and (*ii*) bed plate or wall plate. The sole plate, some times also known as shoe plate is attached to the bottom flange of the bridge girder while the bed plate(also known as the wall plate) is fixed or anchored to the supporting masonry. Two anchor bolts, fixed in the masonry pass through both the plates. In the case of fixed bearing, the hole in the sole plate, for the anchor bolt to pass, is perfectly circular, while in the expansion bearing, this hole in the sole plate is kept slotted (or elliptical) to allow for longitudinal movement of the lower chord or flange. The size of the two plates is decided on the basis of end reaction and allowable bearing pressure on the masonry. The thickness of the plates should be sufficient to provide adequate rigidity to distribute the end reaction as uniformly as possible. However, because of flexibility of plates, this type of bearings cannot distribute heavy loads evenly on the masonry.

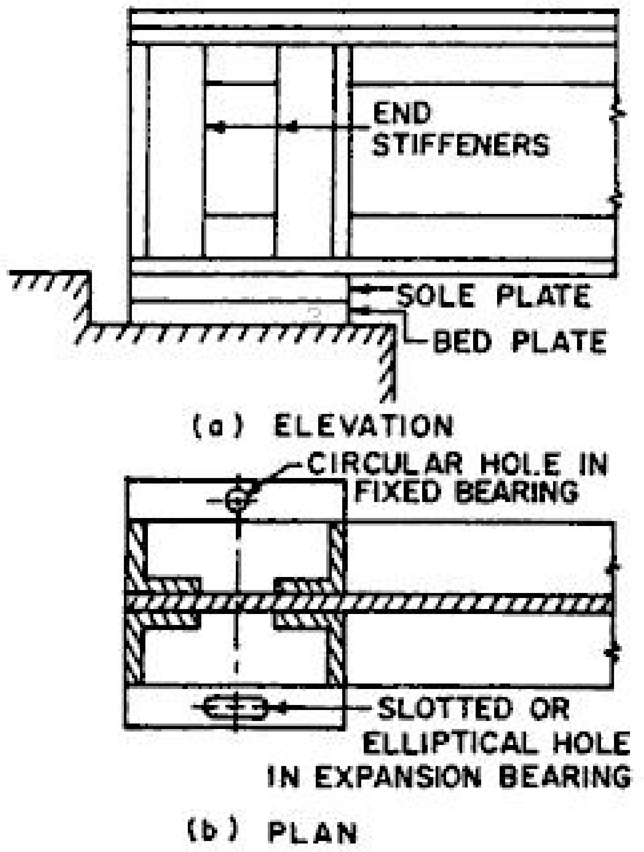

FIG. 31.1. SLIDING BEARING

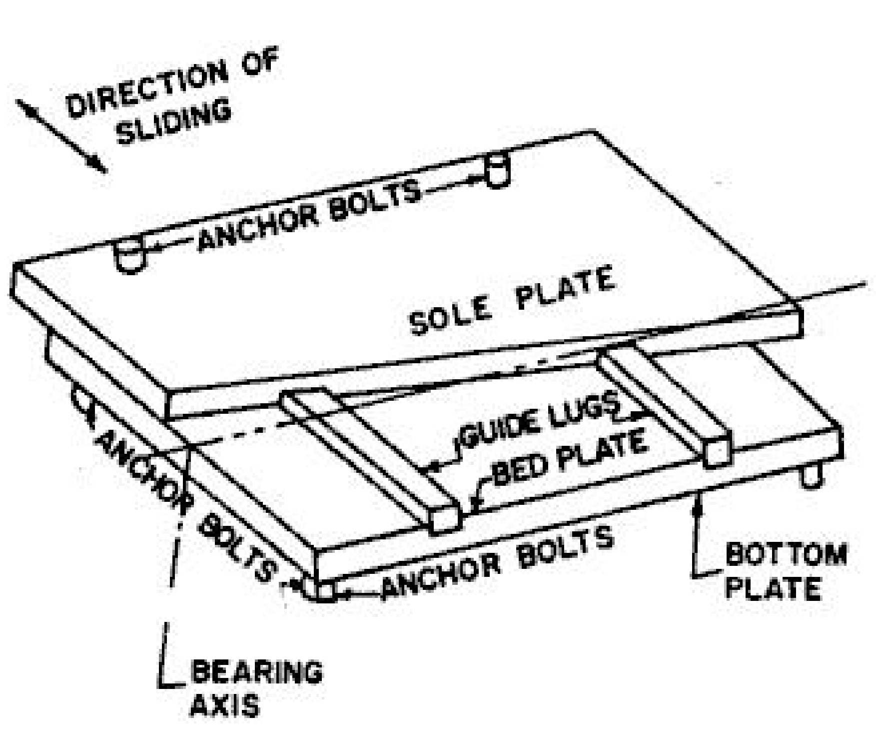

FIG. 31.2. ELEMENTS OF SLIDING BEARING

4. Design of bottom base plate

Allowable bearing pressure on concrete : 4 N/mm^2 = 4000 kN/m^2

∴ Area of base plate required = 924.4/4000 = 0.2311 m^2

Let us provide the base plate of size 0.6 × 0.6 m, giving A = 0.36 m^2.

Larger size is being used because the stress will increase due to wind load.

Reaction due to DL+LL+IL+wind = 924.4+197.8 = 1122.2 kN

Total lateral load due to wind = 238.5 kN

∴ Lateral load due to wind at the pin of the bearing $= \frac{1}{4} \times 238.5 = 59.625$ kN

It is to be noted that lateral load acts in the direction of movement of rollers, and hence it will not have any moment at the base.

Let us assume the centre line of the pin at 400 mm above the base.

∴ Moment due to wind at the base = 59.625 × 0.4 = 23.85 kN-m

$$\therefore \quad p_{max} = \frac{1122.2}{0.6 \times 0.6} + \frac{23.85 \times 6}{0.6\,(0.6)^2}$$

$$= 3117.2 + 662.5 = 3779.7 \text{ kN/m}^2 < 1.33 \times 4000 \text{ . OK.}$$

Since an increase of $16\frac{2}{3}\%$ in steel stress is permitted when occasional loads are considered, let us design the base to withstand a pressure of 3779.7/1.167 = 3239 kN/m^2.

Maximum cantilever projection beyond the centre line of last roller $= \frac{1}{2}(600 - 4 \times 70) = 160$ mm

$$\therefore \quad M_{max} = \frac{3239\,(0.16)^2}{2} = 41.46 \text{ kN-m/m}$$

Let t be the thickness of the base plate. Taking permissible bending stress (σ_{bs}) equal to 185 N/mm^2 for the base plate (for all steels), we have

$$\frac{100\,t^2}{6} \times 185 = 41.46$$

From which we get t =36.7 mm.

Hence provide the base plate of 40 mm thickness.

31.10. ELASTOMERIC BEARINGS

Elastomeric bearings are made from natural rubber and synthetic materials having rubber like characteristics. Such a material is known as an *elastomer*, which is a member of a class of polymeric substance obtained after vulcanisation and possessing rubber like properties, specially the ability to regain shape almost completely after large deformation. *Elastomeric* bearing consists of one or more internal layers of elastomer bonded to internal steel laminates by the process of vulcanisation. The bearing caters for translation and/or rotation of the superstructure by elastic deformation. Usually laminated elastomeric bearings are used. Such bearings are composed of alternate layers of elastomer and laminates, integrally bonded during vulcanisation (Fig. 31.20). A laminate is a layer of reinforcing material integrally bonded to elastomer during vulcanisation process, to restrain the lateral expansion of the elastomer. Laminates of mild steel conforming to IS : 226 shall only be permitted to be used.

*The values of horizontal shears to be used only for beams.

**In all other cases, shear along grain to be used.

Note : The original values are in kg/cm^2 which have been converted in N/mm^2 units, using approximate relation of 1 $kg/cm^2 \approx 0.1\ N/mm^2$.

For other grades, the permissible stresses given in Table 32.4 shall be multiplied by the following factors to obtain the permissible stresses, assuming that the conditions (*a*) and (*b*) laid above are satisfied :

(*i*) For *Select Grade* timber : 1.6

(*ii*) For *Grade II* timber : 0.84

When *low durability* timbers are to be used on outside locations, the permissible stresses for all grades of timber, arrived at by the above considerations shall be multiplied by 0.8.

Modification factors for permissible stresses

(1) ***For change in slope of grain***

When the timber has not been graded and has major defects like slope of grain, knots and checks or shakes (but not beyond permissible value) the permissible stresses given in Table 32.4 shall be multiplied by the modification factor K_1 for different slopes of grain, as given in Table 32.5.

TABLE 32.5. MODIFICATION FACTOR K_1 TO ALLOW FOR CHANGE IN SLOPE OF GRAIN

Slope of grain	*Values of K_1*	
	Strength of beams, joists, and ties	*Strength of posts or columns.*
1 in 10	0.80	0.74
1 in 12	0.90	0.82
1 in 14	0.98	0.87
1 in 15 and flatter	1.00	1.00

2. ***For change in duration of load***

For duration of design load other than continuous, the permissible stresses given in Table 32.4 shall be multiplied by the modification factor K_2 given in Table 32.6.

TABLE 32.6. MODIFICATION FACTOR K_2 FOR CHANGE IN DURATION OF LOADING

	Duration of Loading	*Modification factor K_2*
1.	Continuous	1.0
2.	Two months	1.15
3.	Seven days	1.25
4.	Wind and Earthquake	1.33
5.	Instantaneous or impact	2.00

32.7. NET SECTION

IS : 883-1970 (Code of Practice for Design of Structural Timber in Building) gives the following recommendations for obtaining the net section :

1. The net section is obtained by deducting from the gross section the projected area of all material removed by boring, grooving or other means.

Note : In case of nailing, the area of prebored hole shall not be taken into account for this purpose.

2. The net section used in calculating the load-carrying capacity of a member shall be the least net section determined as above by passing a plane or a series of connected planes transversely through the members.

the edge of the hole to the edge of the member shall not be less than one quarter of the width of the face.

Example 32.6. *A column 200×200 mm is made of sal wood. Determine the safe load for the column, if its unsupported length is 2 m. Take $f_{cp} = 10.6\ N/mm^2$ and $E = 12700\ N/mm^2$ for sal wood.*

Solution

$$\frac{S}{d} = \frac{2000}{200} = 10 < 11$$

Hence the column is short for which safe load is given by.

$$P = A\,.f_c = A f_{cp} \qquad \text{(Eq. 32.17)}$$

$$= 200 \times 200 \times 10.6 \times 10^{-3} = \mathbf{424\ kN}$$

Example 32.7 . *Determine the safe load for the column of example 32.6 if its unsupported length 3.5 m.*

Solution

Here $$\frac{S}{d} = \frac{3.5 \times 1000}{200} = 17.5 > 11$$

Also, $$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}} = 0.702\sqrt{\frac{12700}{10.6}} = 24.3$$

Hence the column is intermediate column for which

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8\,.d}\right)^4\right] = 10.6\left[1 - \frac{1}{3}\left(\frac{3.5}{24.3 \times 0.2}\right)^4\right] = 9.65\ N/mm^2$$

$\therefore$ $$P = A\,.f_c = 200 \times 200 \times 9.65 \times 10^{-3} = \mathbf{386\ kN}$$

Example 32.8. *Determine the safe load for the column of example 32.6 if its unsupported length is 5 m long.*

Solution

Here $$\frac{S}{d} = \frac{5 \times 1000}{200} = 25$$

Since S/d ratio is greater than K_8 (= 24.3), it is a long column, for which

$$f_c = \frac{0.329\,E}{(S/d)^2} = \frac{0.329 \times 12700}{(25)^2} = 6.685\ N/mm^2$$

$\therefore$ $$P = A\,.f_c = 200 \times 200 \times 6.685 \times 10^{-3} = \mathbf{267.4\ kN}$$

Example 32.9. *A column of 200 mm dia. is made of sal wood. Determine the safe axial load, if its effective length is 2 m. Take $f_{cp} = 10.6\ N/mm^2$ and $E = 12700\ N/mm^2$.*

Solution

For a column of circular section, the least lateral dimension d is taken equal to the size of equivalent square of the same area. Hence from Eq. 32.16

$$d = 0.886\,D = 0.886\,(200) = 177.2\ \text{mm}$$

$\therefore$ $$\frac{S}{d} = \frac{2 \times 1000}{177.2} = 11.29 > 11$$

Also, $$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}} = 0.702\sqrt{\frac{12700}{10.6}} = 24.3$$

Hence it is an intermediate column for which

32.15. NAILED JOINTS IN TIMBER

Nailed joints are suitable for timber frames and structural members carrying light loads. They are also suitable for light roof trusses upto span of 15 m. The strength data and design specifications for nailed joint are given in IS : 2366-1963. The nail dimensions are specified in IS 723-1961. Nailed joints can safely transmit loads upto 25 kN in soft timber and 50 kN in hard timber.

A nail consists of a head and a slender body. Nails are classified as (*i*) cut nails, (*ii*) wrought iron nails, and (*iii*) wire nails. *Cut nails*, cut from a metal strip, are of rectangular cross-section which is constant over the whole length except the pointed end. Wrought iron nails, forged from wrought iron, are conical. *Wire nails*, machine made from mild steel wire, are the one which are most commonly used. Wire nails with plain head, commonly used in timber joints are shown in Fig. 32.22.

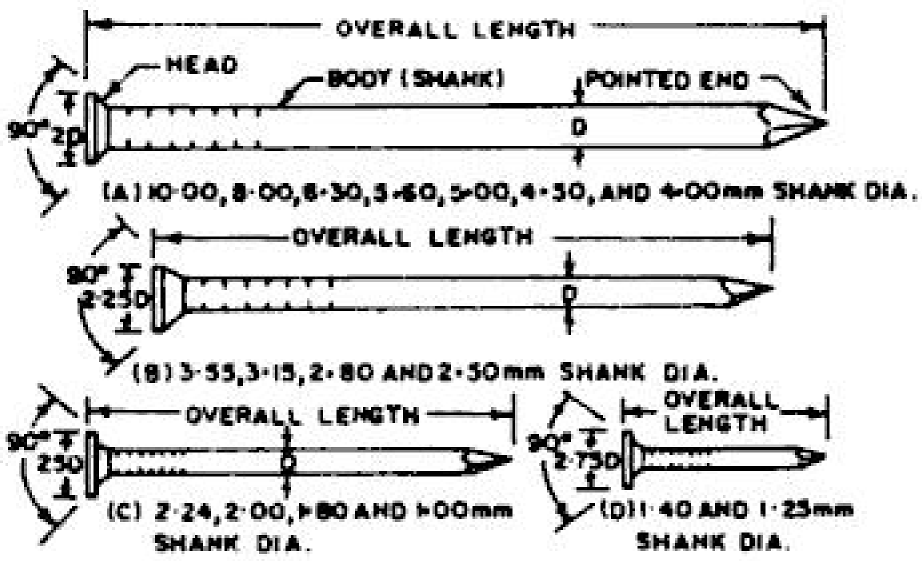

FIG. 32.22 PLAIN HEAD NAILS (IS 723-1961)

The dimensions of plain head round mild steel wire nails with pre-bore holes are given in Table 32.12.

TABLE 32.12. DIMENSIONS OF ROUND MILD STEEL WIRE NAILS

Dia. of shank (mm)	*Dia .of head (mm)*	*Length (mm)*	*Prebore in hard wood (mm)*	*Prebore in soft wood (mm)*
10.00	20.00	250	–	
8.00	16.00	225, 200	–	–
6.30	12.50	175, 150	–	–
5.60	12.00	150, 125	–	–
5.00	10.00	125, 100	4.00	3.55
4.50	9.00	200, 90	3.55	3.15
4.00	8.00	90, 80	3.15	2.80
3.55	8.00	90, 80, 70, 60	2.80	2.50
3.15	7.10	70, 60, 50	2.50	2.24
2.80	6.20	70, 60, 50	2.24	2.00
2.50	5.60	70, 60 , 50, 45, 40	2.00	1.80
2.24	5.60	70, 60, 50, 45, 40, 35	1.80	1.60
2.00	5.00	50, 45, 40, 35, 30, 25	1.60	1.40
1.80	4.50	30, 25	–	–
1.60	4.00	25, 20, 15	–	–
1.40	3.80	20	–	–
1.24	3.40	20	–	–

Clenching of nails : When nails are driven through the jointed members, their pointed ends protude from the other face. This projecting portion of the nail can either be cut so

G = specific gravity of the timber
d = shank dia. of nail, in cm.
R = withdrawal force, in N per cm of penetration of nail

The values of safe unit withdrawal resistance ($= \bar{k}\, G^{2.5}$) for different species of timber, are given in Table 32.13.

TABLE 32.13. PERMISSIBLE WITHDRAWAL RESISTANCE FOR NAIL PER CM PENETRATION

Species of timber		Hard (H) or soft (S)	Specific gravity (G)	Unit withdrawal Resistance ($= 954\, G^{2.5}$)
1.	Fir	S	0.465	140
2.	Babul	H	0.835	607
3.	Kala siris	H	0.735	442
4.	Deodar	S	0.560	224
5.	Eucalyptus	H	0.850	635
6.	Dhaman	H	0.755	473
7.	Mango	S	0.655	331
8.	Chir	S	0.575	239
9.	Sandan	H	0.865	664
10.	Oak	H	0.865	664
11.	Sal	H	0.800	546
12.	Jamun	H	0.850	635
13.	Teak	H	0.625	295
14.	White chugalam	H	0.690	377
15.	Black chuglam	H	0.835	608
16.	Sain	H	0.880	693

Safe lateral resistance

The permissible lateral resistance (in double shear) for a nail that has its point cut flush in the surface are as follows :

1. For lengthening joint, for permanent construction :

$$R_1 = \bar{k}_1\, d \qquad \text{...(32.35 } a\text{)}$$

2. For node joint for permanent construction :

$$R_2 = \bar{k}_2 \,.\, d \qquad \text{...(32.35 } b\text{)}$$

3. For lengthening joint and node joint, for temporary construction

$$R_3 = \bar{k}_3 \,.\, d \qquad \text{...(32.35 } c\text{)}$$

where

R_1, R_2, R_3 = permissible lateral strength (in N) in double shear, per nail
$\bar{k}_1, \bar{k}_2, \bar{k}_3$ = constants, depending upon the type of timber, the values of which are given in Table 8.14, when R_1, R_2, R_3 are expressed in Newtons.
d = diameter of nail, in cm.

If the nails are clenched across the grain, the values of the constants may be increased by 20% over the one given in Table 32.14.

In case the nails are driven in unseasoned timber, the permissible withdrawal resistance given by Eq. 32.34 is reduced to 75 percent and the permissible lateral resistances given by Eqs. 32.35 are to be reduced to 25 percent.

teak = *8.8 N/mm*^{2}. *(ii) Safe stress in compression perpendicular to grain in teak wood* = *4 N/mm*^{2} *and (iii) Safe working stress in shear along the grains for babul* = *2.22 N/mm*^{2}.

Solution

Fig. 32.34 shows a butt joint with dowels, to splice the member.

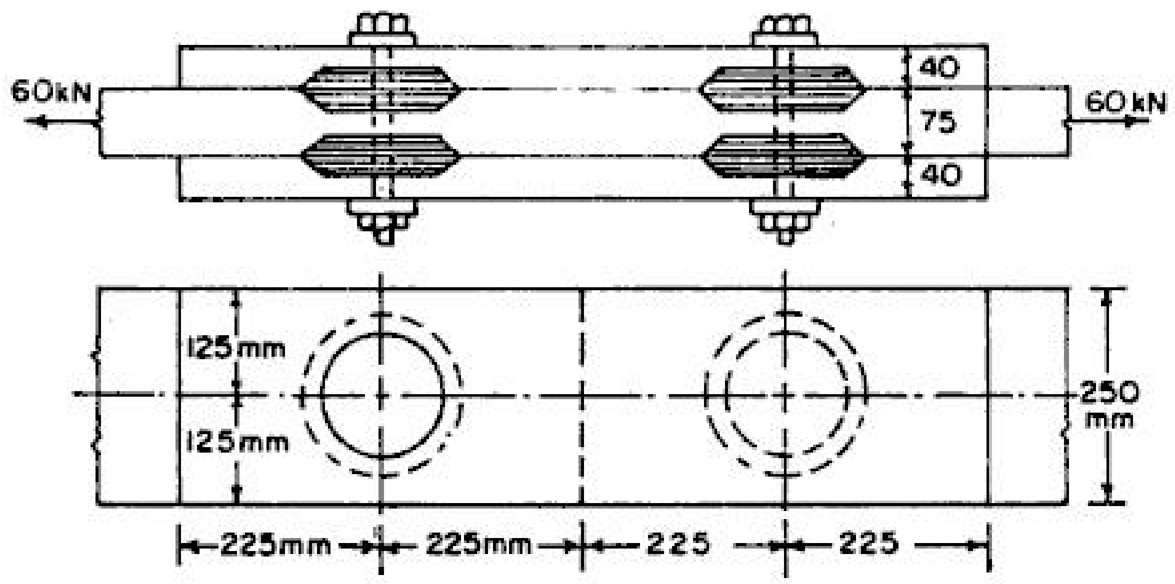

FIG. 32.34.

Thickness of member = 75 mm.

Max. thickness of dowel = 75 − 12 = 63 mm.

Let us keep dowel of 50 mm thickness (*i.e.* $t = 50$ mm)

Diameter D of dowel $= 3t = 3 \times 50 = 150$ mm. Make the dowel of babul.

Strength of dowel, in shear along the grain

$= \frac{\pi}{4}(150)^2 \times 2.22 \times 10^{-3}$

$= 39.23$ kN ...(*i*)

Safe working stress in bearing in teak wood = 8.8 N/mm^2

$$\text{Strength of dowel in bearing} = D\frac{t}{2}f_b = 150 \times \frac{50}{2} \times 8.8 \times 10^{-3} = 33 \text{ kN} \quad ...(ii)$$

$$\therefore \quad \text{Dowel value} = 33 \text{ kN}$$

$$\therefore \quad \text{No. of dowels required} = \frac{60}{33} = 1.82$$

Hence provide two dowels on each side, as shown in Fig. 32.34.

$$\text{Hence Force } F_h = P/2 = \frac{60}{2} = 30 \text{ kN.}$$

Max. bearing stress in the dowel is given by Eq. 32.37 :

$$f_{b,max} = \frac{16}{\pi} F_h \cdot \frac{t}{D^3} = \frac{16}{\pi} \times 30 \times 1000 \times \frac{50}{(150)^3} = 2.264 \text{ N/mm}^2$$

Safe bearing stress = permissible compressive stress perpendicular to grain in teak wood = 4 N/mm^2. Hence safe.

$$\text{Tensile force in bolt} = F_v = 2F_h \cdot \frac{t}{D}$$

or

$$F_v = 2 \times 30 \times \frac{50}{150} = 20 \text{ kN}$$

$$\therefore \quad \text{Net area required} = \frac{20 \times 1000}{0.6 \times 250} = 133.3 \text{ mm}^2$$

Provide 16 mm dia. bolt for which net area = 156 mm^2. Hence OK.

Provide two fish plates, each of size 40 mm × 250, one on each side.

$$\text{Effective edge distance required} = \left(\frac{D}{2} + 20\right) = \left(\frac{150}{2} + 20\right) = 95 \text{ mm}$$

Available edge distance = 125 mm.

Effective end distance required for tension member = $1.5D = 1.5 \times 150 = 225$ mm

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2. For no tension to develop, $e < b/6$, or $\bar{x} \leq 2b/3$
3. For no sliding to occur, $R_h < R_v \cdot \mu$

where $\mu = \tan\delta' =$ coefficient of friction between the base of the wall and the soil.
The factor of safety F against sliding is given by

$$F = \frac{R_v \cdot \mu}{R_h} \qquad ...(33.32)$$

The minimum value of F should be 1.5

4. For the wall to be stable against overturning, R must pass within the base width. However if the requirement of no tension is full-filled, complete safety against overturning is automatically assured.

Example 33.3. *Compute the intensities of active and passive earth pressure at depth of 8 metres in dry cohesionless sand with an angle of internal friction of 30° and the unit weight of 18 kN/m³. What will be the intensities of active and passive earth pressure if the water level rises to the ground level ? Take saturated unit weight of sand as 22 kN/m³.*

Solution (*a*) ***Dry soil*** :

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30°}{1 + \sin 30°} = \frac{\frac{1}{2}}{\frac{2}{3}} = \frac{1}{3}.$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1}{K_a} = 3.$$

$$p_a = K_a \gamma H = \frac{1}{3} \times 18 \times 8 = 48 \text{ kN/m}^2$$

$$p_p = K_p \cdot \gamma H = 3 \times 18 \times 8 = 432 \text{ kN/m}^2$$

(*b*) ***Submerged backfill***

$$\gamma' = \gamma_{sat} - \gamma_w = 22 - 9.81 = 12.19 \text{ kN/mm}^3$$

$$p_a = K_a \gamma' H + \gamma_w H = \frac{1}{3} \times 12.19 \times 8 + 9.81 \times 8 \approx 111 \text{ kN/m}^2$$

$$p_p = K_p \gamma' H + \gamma_w H = 3 \times 12.19 \times 8 + 9.81 \times 8 = 371 \text{ kN/m}^2$$

Example 33.4. : *A retaining wall, 4 m high, has a smooth vertical back. The backfill has a horizontal surface in level with the top of the wall. There is uniformly distributed surcharged load of 36 kN/m² intensity over the backfill. The unit weight of the backfill is 18 kN/m³ its angle of shearing resistance is 30° and cohesion is zero. Determine the magnitude and the point of application of active pressure per metre length of the wall.*

Solution :

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1}{3}$$

The lateral pressure intensity due to surcharge is given by

$$p_1 = K_a \cdot q = \frac{1}{3} \times 36 = 12 \text{ kN/m}^2$$

The pressure intensity due to backfill at depth $H = 4$ m is given by :

$$p_2 = K_a \gamma H = \frac{1}{3} \times 18 \times 4 = 24 \text{ kN/m}^2$$

against sliding may be due to friction alone, or due to friction and shear strength of the joint. Shear strength develops at the base if benched foundations are provided and at other joints if the joints are carefully laid so that a good bond develops. Shear strength also comes into play because of the interlocking of stones in masonry dams.

If the shear strength is not taken into account, the factor of safety is known as *factor of safety against sliding*. The factor of safety against sliding is defined as the ratio of *actual coefficient of static friction* (μ) on the horizontal joint to the *sliding friction*. The *sliding factor* (S.F.) is the minimum coefficient of friction required to prevent sliding. If ΣH is the horizontal forces causing the sliding and $\Sigma (V-U)$ is the *net vertical forces*, the sliding factor ($\tan\theta$) is given by :

$$S.F. = \tan\theta = \frac{\Sigma H}{\Sigma (V-U)} \quad ...(33.35)$$

and the factor of safety against sliding is

$$F.S. = \frac{\mu}{\tan\theta} = \frac{\mu\,\Sigma (V-U)}{\Sigma H} \quad ...(33.36)$$

The coefficient of friction μ varies from 0.65 to 0.75. The factor of safety against sliding should be greater than 1.

It is considered that a low gravity dam should be safe against sliding, considering friction alone. However, in large dams, shear strength of the joint should also be considered for an economical design. The factor of safety in that case is commonly known as the *shear friction factor* (S.F.F.) and is defined by the equation :

$$\text{S.F.F.} = \frac{\mu\,\Sigma (V-U) + b\,q}{\Sigma H} \quad ...(33.37)$$

where q = shear strength of the joint (usually 1.4 N/mm^2)
b = width of the joint or section

*3. **Compression or crushing*** : In order to calculate the normal stress distribution at the base, or at any section, let ΣH be the total horizontal force and ΣW be the total vertical force ($= \Sigma (V-U)$) and let R be the resultant force, cutting the base at an eccentricity e from the centre of base width b (Fig. 33.18).

Thus, direct stress $= \dfrac{\Sigma W}{b \times 1}$

Bending stress

$$= \pm \frac{M}{Z} = \pm \frac{\Sigma W . e}{\frac{1}{6} b^2} = \pm \frac{6\,\Sigma W . e}{b^2}$$

Hence the *total normal stress* p_n is given by

$$p_n = \frac{\Sigma W}{b}\left(1 \pm \frac{6e}{b}\right) \quad ...(33.38)$$

Thus, at toe, $$p_n = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) \quad ...(33.38a)$$

and, at heel, $$p_n = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) \quad ...(33.38b)$$

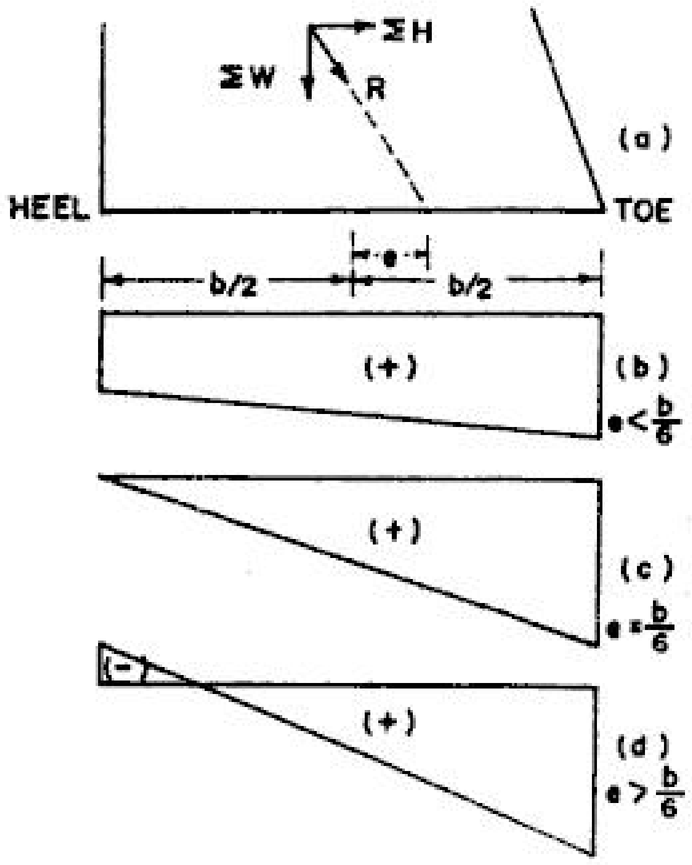

FIG. 33.18. NORMAL STRESS DISTRIBUTION AT THE BASE

Hence height of dam = 1482 − 1450 = 32 m

The practical profile of the dam can be designed with respect to the recommendations marked in Fig. 33.22.

Depth of water, h = 1480.5 − 1450 = 30.5 m

Top width a can be determined on the following criteria :

(*i*) a = 14% of H, for economy

$= 0.14 \times 32 \approx 4.5$ m

(*ii*) a = width of roadway, if any.

However, keep a = 4.5 m

Base width of elementary profile

$$= \frac{h}{\sqrt{\rho}} = \frac{30.5}{\sqrt{2.4}} = 19.7 \text{ m}$$

$$\text{Upstream offset} = \frac{a}{16} = \frac{4.5}{16} \approx 0.3 \text{ m}$$

∴ Total base width = 19.7+0.3 = 20 m

Distance upto which the u/s slope is vertical, from the u/s water level

$$= 2a\sqrt{\rho} = 2 \times 4.5\sqrt{2.4} = 14 \text{ m}$$

Distance upto which the u/s slope is inclined, from the u/s water level

$$= 3.1\,a\sqrt{\rho} = 3.1 \times 4.5\sqrt{2.4} = 21.6 \text{ m}$$

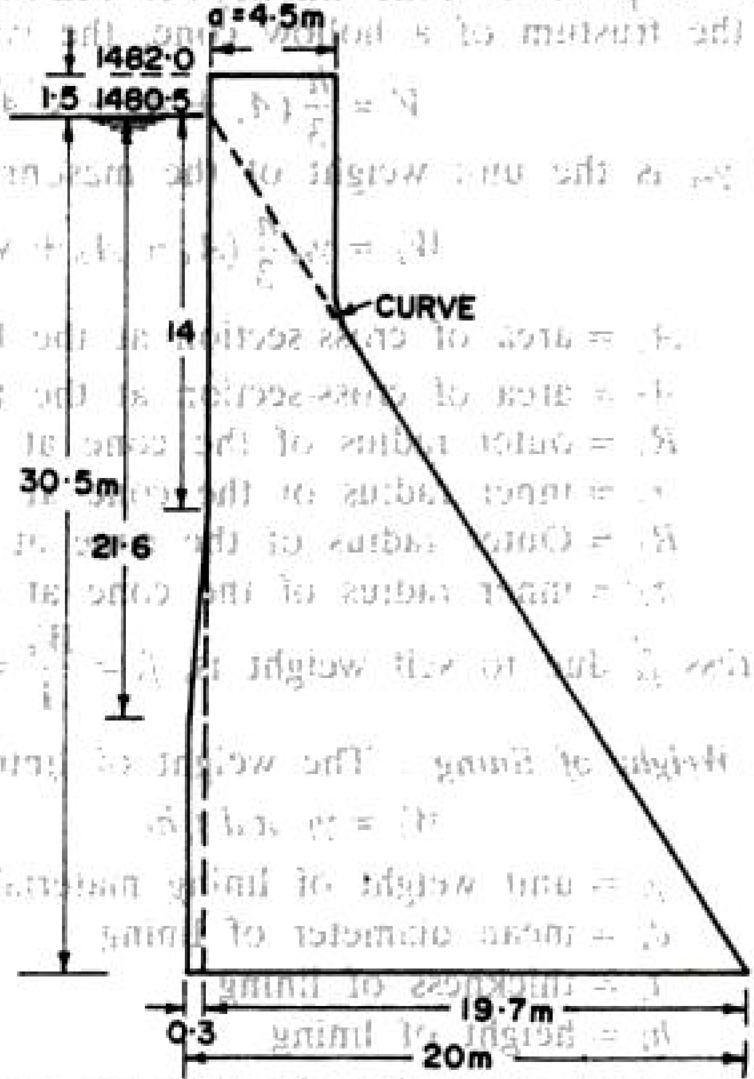

FIG. 33.28

Fig. 33.28 shows the practical profile of the masonry dam. The stability of this section can now be tested as usual.

33.6. MASONRY CHIMNEYS

Chimneys are used in almost all industries for the escape of flue gases to such a height that diffusion of gases that place so that they do not contaminate the surroundings. The height of chimney is fixed on the basis of required draft. However, masonry chimneys are restricted to small heights only, since the section of the chimney becomes very heavy for large height, requiring heavy foundations. Masonry chimneys are generally built in bricks, with either the common bricks or radial bricks. Masonry chimneys are constructed in various shapes, such as circular, square, hexagonal or octagonal; circular shape is preferred over others. Chimney are lined from inside with fire bricks, atleast upto 20 to 25% of its height from the bottom.

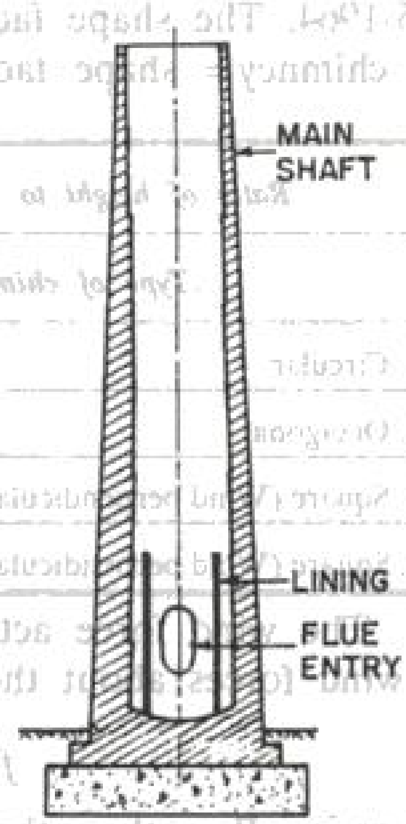

FIG. 33.29. MASONRY CHIMNEYS

Forces acting on a chimney

Various forces that act on a chimney are :

1. Self weight of masonry
2. Weight of brick lining
3. Wind pressure.
4. Seismic force.

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