E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

## E-NOTES OF DSS – 1

# **B.TECH 6th Sem**

- Lesson 1 : Structures and its kinds
- Lesson 2 : Rolled Structural steel sections
- **Lesson 3 : Loads on structures**
- Lesson 4 : Stresses on structures
- **Lesson 5 : Riveted connections**
- Lesson 6 : Design of Riveted connections
- Lesson 7 : Welded connection
- Lesson 8 : Tension members
- **Lesson 9 : Design of Tension member**
- Lesson 10 : Design of columns
- **Lesson 11 : Design of Compression members**
- Lesson 12 : Design of column base Slab base
- Lesson 13 : Steel Beams
- Lesson 14 : Design of steel beams

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **LESSON 1. Structures and its kinds**

### **1.1 INTRODUCTION**

The Structural engineering is a branch of engineering which deals with structural analysis and structural design. The structural engineering plays an important role in civil engineering, mechanical engineering, electrical engineering, naval engineering, aeronautical engineering and in all the specialized phases of engineering. The structural analysis deals with the development of suitable arrangement of structural elements for the structures to support the external loads or the various critical combinations of the loads which are likely to act on the structure. The analysis also deals with the determination of internal forces developed in the various members, nature of stresses or critical combination of the stresses at the various points and the external reactions due to the worst possible combination of the loads. The structural design deals with the selection of proper material, proper sizes, proportions and shape of each member and its connecting details. The selection is such that it is economical and safe. The structural design further deals with the preparation of final layout of the structure and the design drawings are necessary for fabrication and construction.

#### **1.2 DEFINITION**

Construction or framework of structural elements (members) which gives form and stability, and resists stresses and strains. Structures have defined boundaries within which each element is physically or functionally connected to the other elements, and the elements themselves and their interrelationships are taken to be either fixed (permanent) or changing only occasionally or slowly.

### **1.3 CLASSIFICATION OF STRUCTURES**

The structures may be classified as statically determinate structures and statically indeterminate structures. When the equation of statics ( $\Sigma$ H=0,  $\Sigma$ V=0 and  $\Sigma$ M=0) are enough to determine all the forces acting on the structure and in the structures are known as statically determinant structures. When the equation of equilibrium are not sufficient to determine all the forces acting on the structure, then the structures are known as statically indeterminate

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

structures. The equations of consistent deformations are added to the equations of equilibrium in order to analyze the statically indeterminate structures.

The structures are also classified as shell structures and framed structures. The shell roof covering of large buildings, air planes, rail road cars, ship wells, tanks etc are the examples of shell structures. The plates or sheets serve functional and structural purposes. The plates act as a load carrying elements. The plates are stiffened by frames which may or may not carry the principal loads. The framed structures are built by assemblies of elongated members. The truss frames, truss girders, rigid frames etc are the examples of framed structures. The main members are used for the transmission of loads.

The structures may be further classified depending on the materials used as plastic structures, aluminium structures, timber structures, R.C.C structures and steel structures.

#### **1.4 ADVANTAGES OF STEEL STRUCTURES**

- 1. Steel has a high strength and so steel components have smaller sections for the same strength compared to corresponding components of other material. The existing steel structures and structural component may be strengthened by connecting additional sections or plates.
- 2. Steel members are gas and watertight, because of high density of steel.
- **3**. Steel structures can be fabricated at site easily.
- 4. Steel structures have great durability and serve for many years.
- 5. Steel members can be readily disassembled or replaced.
- 6. The existing steel structures and structural component may be strengthened by connecting additional sections or plates.

#### **1.5 DISADVANTAGES OF STEEL STRUCTURES**

- 1. Steel structures are liable to corrosion and need painting frequently.
- 2. Steel structures have a low fire resistance and are liable to lose their strength and get deformed at high temperature.

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **1.6 STRUCTURAL STEEL**

The structural steel is the steel used for the manufacture of rolled structural steel sections, fastenings and other elements for use in structural steel works. Steel is an alloy of iron, carbon and other elements in varying percentages. The strength, hardness and brittleness of steel increases and ductility of steel decreases with the increase of percentage of carbon. Depending on the chemical composition, the different type of steel are classified as mild steel, medium carbon steel, high carbon steel, low alloy steel and high alloy steel. The mild steel, medium carbon steel and low alloy steel are generally used for steel structures. The copper bearing quality of steel contains small percentage of copper contents. The corrosive resistance of such steel is increased.

Mild steel is used for the manufacture of rolled structural steel sections, rivets and bolts. The following operations can be done easily on mild steel 1.Cutting, 2. Punching, 3.Drilling, 4. Machining, 5. Welding and 6. Forging when heated. All structural steels used in general construction, coming within the purview of IS:800-84 shall, before fabrication, comply with one of the following Indian Standard specifications

- 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)

### **1.6.1 IS : 226-1975 structural steel (standard quality).**

The mild steel is designated as St 44-S for use in structural work. This steel is also available in copper bearing quality in which case it designated as St 44-SC. The copper content is between 0.20 and 0.35 per cent. The physical properties of structural steel are given below:

- 1. Unit weight of steel 78.430 to 79.000  $kN/m^3$
- 2. Young's modulus of elasticity, E=2.04 to  $2.18 \times 10^5$  N/mm<sup>2</sup>

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

#### Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

- 3. Modulus of rigidity, G=0.84 to  $0.98 \times 10^5$  N/mm<sup>2</sup>
- 4. Coefficient of thermal expansion (or contraction)  $\alpha = 12 \times 10^{-6}$ °C or 6.7 x 10<sup>-6</sup>/°F.

The tensile strength, yield stress and percentage elongation for IS : 226-1975 structural steel standard quality, determined in accordance with IS : 1608-1960. The steel confirming to IS : 226 is suitable for all types of steel structures subjected to static, dynamic and repeated cycles of loadings. It is also suitable for welding up to 20 mm thickness. When the thickness of element is more than 20 mm, it needs special precautions while welding.

### **1.6.2 IS : 1977-1975 structural steel (ordinary quality).**

The steel which did not comply with IS : 226, was formerly called as steel of untested quality. The standards for such steel have been laid down in IS : 1977-75 (ordinary quality). There are two grades in this standard which are designated as St 44.0 and St 32.0. The steel St 44.0 is intended to be used for structures not subjected to dynamic loading other than wind loads e.g., platform roofs, office buildings, foot over bridge. The copper bearing quality is designated as St 44.0C.

The steel confirming to IS : 1977 is not suitable for welding and for the structures subjected to high seismic forces (earth quake forces). The steel structures using steel confirming to IS : 1977 must not be analyzed and designed by plastic theory.

#### 1.6.3 IS : 2062-1984 weldable structural steel.

This structural steel intended to be used for members in structures subjected to dynamic loading where welding is employed for fabrication and where fatigue and great restraint are involved e.g., crane gantry girder, road and rail bridges etc,. it is designated as St 42-W and copper bearing quality is designated as St 42-WC. It is suitable for welding the elements of thickness between 28 mm and 50 mm. when the thickness of elements is less than 28 mm; it may be welded provided the limiting maximum carbon content is 0.22 per cent.

#### **1.6.4 IS : 961-1975 structural steel (high tensile).**

The high tensile steel forms a specific class of steel in which enhanced mechanical properties and in most of the cases increased resistance to atmospheric corrosion are obtained by the incorporation of low proportions of one or more alloying elements, besides carbon. These steels

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

are generally intended for application where saving in weight can be effected by reason of their greater strength and atmospheric corrosion resistance. Standards of high tensile steel have been given in IS : 961-1975. It has been classified into two grades designated as St 58-HT and St 55-HTW. St 58-HT is intended for use in structures where fabrication is done by methods other than welding. St 55-HTW is intended for use in structures where welding is employed for fabrication. The high tensile steel is also available in copper bearing quality and two grades are designated as St 58-HTC and St 55-HTWC. The steel conforming to IS : 961 is suitable for bridges and general building construction.

### 1.6.5 IS : 8500-1977 weldable structural steel (medium and high strength qualities)

Various medium and high strength qualities of weldable structural steel are, Fe 440 (HT1 and HT2) Fe 540 (HT, HTA and HTB), Fe 570 HT, Fe 590 HT and Fe 640 HT.

### **1.7 PRODUCTION OF STEEL**

The steel is produced in the form of ingots and converted to different shapes. In our country, Tata Iron and Steel Company, Indian Iron and Steel Company, Mysore Iron and Steel Company and Hindustan Steel produce steel at their plants

### **1.8 RECENT DEVELOPMENTS IN MATERIAL**

A number of developments in material such as steel have been made recently. The weldable qualities of steel (IS : 2062) designated as St 42-W and IS : 961 designated as St-55-HTW are developed with the large scale use of welding. IS : 961 has been developed with high tensile strength and there is saving in weight due to enhanced mechanical properties. Its weldable quality is advantageous for composite construction.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **LESSON 2. Rolled Structural Steel Sections**

### 2.1 INTRODUCTION

The steel sections manufactured in rolling mills and used as structural members are known as rolled structural steel sections. The steel sections are named according to their cross sectional shapes. The shapes of sections selected depend on the types of members which are fabricated and to some extent on the process of erection. Many steel sections are readily available in the market and have frequent demand. Such steel sections are known as regular steel sections. Some steel sections are rarely used. Such sections are produced on special requisition and are known as special sections. 'ISI Handbook for Structural Engineers' gives nominal dimensions, weight and geometrical properties of various rolled structural steel sections.

### 2.2 TYPES OF ROLLED STRUCTURAL STEEL SECTIONS

The various types of rolled structural steel sections manufactured and used as structural members are as follows:

- 1. Rolled Steel I-sections (Beam sections).
- 2. Rolled Steel Channel Sections.
- 3. Rolled Steel Tee Sections.
- 4. Rolled Steel Angles Sections.
- 5. Rolled Steel Bars.
- 6. Rolled Steel Tubes.
- 7. Rolled Steel Flats.
- 8. Rolled Steel Sheets and Strips.
- 9. Rolled Steel Plates.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

### 2.3 ROLLED STEEL BEAM SECTIONS

The rolled steel beams are classified into following four series as per BIS : (IS : 808-1989)

1.	Indian Standard Joist/junior Beams	ISJB
2.	Indian Standard Light Beams	ISLB
3.	Indian Standard Medium Weight Beams	ISMB
4.	Indian Standard Wide Flange Beams	ISWB

The rolled steel columns/heavy weight beams are classified into the following two series as per BIS (IS : 808-1989)

1.	Indian Standard Column Sections	ISSC
2.	Indian Standard Heavy Weight Beams	ISHB

The cross section of a rolled steel beam is shown in Fig. 2.1. The beam section consists of web and two flanges. The junction between the flange and the web is known as fillet. These hot rolled steel beam sections have sloping flanges. The outer and inner faces are inclined to each other and they intersect at an angle varying from 1½ to 8° depending on the section and rolling mill practice. The angle of intersection of ISMB section is 8°. Abbreviated reference symbols (JB, LB, MB, WB, SC and HB) have been used in designating the Indian Standard Sections as per BIS (IS 808-1989)

The rolled steel beams are designated by the series to which beam sections belong (abbreviated reference symbols), followed by depth in mm of the section and weight in kN per metre length of the beam, e.g., MB 225 @ 0.312 kN/m. H beam sections of equal depths have different weights per metre length and also different properties e.g., WB 600 @ 1.340 kN/m, WB 600 @ 1.450 kN/m, HB 350 @0.674 kN/m, HB 350 @0.724 kN/m.

I-sections are used as beams and columns. It is best suited to resist bending moment and shearing force. In an I-section about 80 % of the bending moment is resisted by the flanges and the rest of the bending moment is resisted by the web. Similarly about 95% of the shear force is resisted by the web and the rest of the shear force is resisted by the flanges. Sometimes I-

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

sections with cover plates are used to resist a large bending moment. Two I-sections in combination may be used as a column.

### 2.4 ROLLED STEEL CHANNEL SECTIONS

The rolled steel Channel sections are classified into four categories as per ISI, namely,

1.	Indian Standard Joist/Junior Channels	ISJC
2.	Indian Standard Light Channels	ISLC
3.	Indian Standard Medium Weight Channels	ISMC
4.	Indian Standard Medium Weight Parallel Flange Channels	ISMCP

The cross section of rolled steel channel section is shown in Fig 2.2. The channel section consists of a web and two flanges. The junction between the flange and the web is known as fillet. The rolled steel channels are designated by the series to which channel section belong (abbreviated reference symbols), followed by depth in mm of the section and weight in kN per metre length of the channel, e.g., MC 225 @ 0.261 kN/m

Channels are used as beams and columns. Because of its shape a channel member affords connection of an angle to its web. Built up channels are very convenient for columns. Double channel members are often used in bridge truss. The channels are employed as elements to resist bending e.g., as purlins in industrial buildings. It is to note that they are subjected to twisting or torsion because of absence of symmetry of the section with regards to the axis parallel to the web, i.e., yy-axis. Therefore, it is subjected to additional stresses. The channel sections are commonly used as members subjected to axial compression in the shape of built-up sections of two channels connected by lattices or batten plates or perforated cover plates. The built-up channel sections are also used to resist axial tension in the form of chords of truss girders.

As per IS : 808-1989, following channel sections have also been additionally adopted as Indian Standard Channel Sections

1. Indian Standard Light Channels with parallel flanges ISLC(P)

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

2.	Medium weight channels	MC
3.	Medium weight channels with parallel flanges	МСР
4.	Indian Standard Gate Channels	ISPG

In MC and MCP channel sections, some heavier sections have been developed for their intended use in wagon building industry. The method of designating MC and MCP channels is also same as that for IS channels.

### 2.5 ROLLED STEEL TEE SECTIONS

The rolled steel tee sections are classified into the following five series as per ISI:

1.	Indian Standard Normal Tee Bars	ISNT
2.	Indian Standard Wide flange Tee Bars	ISHT
3.	Indian Standard Long Legged Tee Bars	ISST
4.	Indian Standard Light Tee Bars	ISLT
5.	Indian Standard Junior Tee Bars	ISJT

The cross section of a rolled steel tee section has been shown in Fig. 2.3. The tee section consists of a web and a flange. The junction between the flange and the web is known as fillet. The rolled steel tee sections are designated by the series to which the sections belong (abbreviated reference symbols) followed by depth in mm of the section and weight in kN per metre length of the Tee, e.g., HT 125 @ 0.274 kN/m. The tee sections are used to transmit bracket loads to the columns. These are also used with flat strips to connect plates in the steel rectangular tanks.

A per IS: 808-1984, following T-sections have also been additionally adopted as Indian Standard T-sections.

1.	Indian Standard deep legged Tee bars	ISDT
2.	Indian Standard Slit medium weight Tee bars	ISMT

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

3. Indian Standard Slit Tee bars from I-sections ISHT

It is to note that as per IS 808 (part II) 1978, H beam sections have been deleted.

#### 2.6 ROLLED STEEL ANGLE SECTIONS

The rolled steel angle sections are classified in to the following three series.

1.	Indian Standard Equal Angles	ISA
2.	Indian Standard Unequal Angles	ISA
3.	Indian Standard Bulb Angles	ISBA

Angles are available as equal angles and unequal angles. The legs of equal angle sections are equal and in case of unequal angle section, length of one leg is longer than the other. Thickness of legs of equal and unequal angle sections are equal. The cross section of rolled equal angle section, unequal angle section and that of bulb angle section is shown in Fig. 2.4. The bulb angle consists of a web a flange and a bulb projecting from end of web.

The rolled steel equal and unequal angle sections are designated by abbreviated reference symbols  $\bot$  followed by length of legs in mm and thickness of leg, e.g.,

∟ 130 x 130 x 8 mm (∟ 130 130 @ 0.159 kN/m)

∟200 x 100 x 10 mm (∟ 200 100 @ 0.228 kN/m)

The rolled steel bulb angles are designated by BA, followed by depth in mm of the section and weight in kN per metre length of bulb angle.

Angles have great applications in the fabrications. The angle sections are used as independent sections consisting of one or two or four angles designed for resisting axial forces (tension and compression) and transverse forces as purlins. Angles may be used as connecting elements to connect structural elements like sheets or plates or to form a built up section. The angle sections are also used as construction elements for connecting beams to the columns and purlins to the chords of trusses in the capacity of beam seats, stiffening ribs and cleat angles.

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

The bulb angles are used in the ship buildings. The bulb helps to stiffen the outstanding leg when the angle is under compression.

As per IS : 808-1984, some supplementary angle sections have also additionally adopted as Indian Standard angle sections. However prefix ISA has been dropped. These sections are designated by the size of legs followed by thickness e.g.,  $\lfloor 200 \ 150 \ x \ 15$ .

#### 2.7 ROLLED STEEL BARS

The rolled steel bars are classified in to the following two series:

1.	Indian Standard Round Bars	ISRO
2.	Indian Standard Square Bars	ISSQ

The rolled steel bars are used as ties and lateral bracing. The cross sections of rolled steel bars are shown in Fig. 2.5. The rolled steel bars are designated by abbreviated reference symbol RO followed by diameter in case of round bars and ISSQ followed by side width of bar sections. The bars threaded at the ends or looped at the ends are used as tension members.

#### 2.8 ROLLED STEEL TUBES

The rolled steel tubes are used as columns and compression members and tension members in tubular trusses. The rolled steel tubes are efficient structural sections to be used as compression members. The steel tube sections have equal radius of gyration in all directions. The cross section of rolled steel tube is shown in Fig. 2.6.

#### 2.9 ROLLED STEEL FLATS

The rolled steel flats are used for lacing of elements in built up members, such as columns and are also used as ties. The cross section of rolled steel flat is shown in Fig. 2.7. the rolled steel flats are designated by width in mm of the section followed by letters (abbreviated reference symbol) F and thickness in mm, e.g., 50 F 8. This means a flat of width 50 mm and thickness 8 mm. The rolled steel flats are used as lattice bars for lacing the elements of built up columns. The rolled steel flats are also used as tension members and stays.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### 2.10 ROLLED STEEL SHEETS AND STRIPS

The rolled steel sheet is designated by abbreviated reference symbol SH followed by length in mm x width in mm x thickness in mm of the sheet. The rolled steel strip is designated as ISST followed by width in mm x thickness in mm, e.g., SH 2000 x 600 x 8 and ISST 250 x 2.

### 2.11 ROLLED STEEL PLATES

The rolled steel plates are designated by abbreviated reference symbol PL followed be length in mm x width in mm x thickness in mm of the plates, e.g., PL 2000 x 1000 x 6.

The rolled steel sheets and plates are widely used in construction. Any sections of the required dimensions, thickness and configuration may be produced by riveting or welding the separate plates. The rolled plates are used in the web and flanges of plate girders, plated beams and chord members and web members of the truss bridge girders. The rolled steel plates are used in special plate structures, e.g., shells, rectangular and circular steel tanks and steel chimneys.

### 2.12 RECENT DEVELOPMENTS IN SECTIONS

The rolled steel beam sections with parallel faces of flanges are recently developed. These beam sections are called as parallel flange sections. These sections have increased moment of inertia, section modulus and radius of gyration about the weak axis. Such sections used as beams and columns have more stability. Theses sections possess ease of connections to other sections as no packing is needed as in beams of slopping flanges. The parallel flange beam sections are not yet rolled in our country.

New welded sections using plates and other steel sections are developed because of welding. The development of beams with tapered flanges and tapered depths is also due to welding. The open web sections and the castellated beams were also developed with the rapid use of welding.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

b=WIDTH - FLANGE FILLET h=DEPTH WEB TEE - SECTION Fig. 2.3 Tee section WEB h DEPTH FLANGE b **₩IDTH** L2 L = LENGTH OF LEG L = LENGTH OF LEG (B) UNEQUAL ANGLE (A) EQUAL ANGLE

ANGLE SECTIONS

Fig. 2.1 Beam section

Fig. 2.2 Channel section

(C) BULB ANGLE

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,









E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

( Prepared By: Ms Kajal, lecturer, CE)





E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **LESSON 3. Loads on structures**

### **3.1 INTRODUCTION**

The structures and structural members are designed to meet the functional and structural aspects. Both aspects are interrelated. The functional aspect takes in to consideration the purpose for which the building or the structure is designed. It includes the determination of location and arrangement of operating utilities, occupancy, fire safety and compliance with hygienic, sanitation, ventilation, special equipment, machinery or other features, incident to the proper functioning of the structures. In the structural aspect, it is ensured that the building or the structure is structurally safe, strong, durable and economical. The minimum requirements pertaining to the structural safety of buildings are being covered in codes dealing with loads by way of laying down minimum design loads which have to be assumed for dead loads, imposed loads, wind loads and other external loads, the structure would be required to bear. Unnecessarily, heavy loads without proper assessment should not be assumed. The structures are designed between two limits, namely, the structural safety and economy. The structures should be strong, stable and stiff.

Estimation of the loads for which a structure should be designed is one of the most difficult problems in structural design. The designer must be able to study the loads which are likely to be acting on the structure throughout its life time and the loads to which the structure may be subjected during a short period. It is also necessary to consider the combinations of loads for which the structure has to be designed.

### **3.2 TYPES OF LOADS**

The loads to which a structure, will be subjected to consist of the following

- 1. Dead loads,
- 2. Live loads or imposed loads,
- 3. Wind load,
- 4. Snow load
- 5. Seismic load

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

#### 6. Temperature effects

In addition o the above loads, following forces and effects are also considered while designing the structures.

- 1. Foundation movements
- 2. Elastic axial shortening
- 3. Soil and fluid pressures
- 4. Vibrations
- 5. Fatigue
- 6. Impact
- 7. Erection loads
- 8. Stress concentration effects

#### 3.3 DEAD LOADS

Dead load of a structure means the weight of the structure itself. The dead load in a building will consist of the weight of all wall partitions, floors and roofs. Loads due to partition shall be estimated on the basis of actual constructional details of the proposed partitions and their positioning in accordance with plans and the loads thus estimated shall be included in the dead load for the design of the floors and the supporting structures. If the loads due to partitions cannot be actually computed for want of data, the floors and the supporting structures shall be designed to carry in addition to other loads a uniformly distributed dead load per square metre of not less than  $33\frac{1}{3}$  per cent of the weight per metre run of finished partitions over the entire floor area subjected to minimum uniformly distributed load of 1000 N/m<sup>2</sup> in the case of floors used for office purposes. Dead loads can be estimated using the unit-weight of materials used in building construction as per IS : 875 (part I) -1987

### 3.4 LIVE LOADS OR IMPOSED LOADS

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

Live loads are the loads which vary in magnitude and in positions. Live loads are also known as imposed or transient loads. Imposed loads consist of all loads other than dead loads. Live loads are assumed to be produced by the intended use of occupancy in building including the weight of movable partitions, distributed loads, concentrated loads, loads due to impact and vibration and snow loads. Live loads are expressed as uniformly distributed static loads. Live loads include the weight of materials stored, furniture and movable equipments. Efforts have been made at the international level to decide live loads on floors and these have been specified in the International standards (2103 Imposed floor loads in residential and public building and 2633 Determination of imposed floor loads in production buildings and warehouses). These codes have been published in the International Organization.

Code IS : 875 (part 2) -1987 defines the principal occupancy for which a building or part of a building is used or intended to be used. The buildings are classified according to occupancy as per IS : 875 (part 2)-1987.

### 3.5 WIND LOAD

The wind loads are the transient loads. The wind usually blows horizontal to the ground at high wind speeds. The vertical components of atmospheric motion are relatively small, therefore, the term wind denotes almost exclusive the horizontal wind. The winds of very high speeds and very short duration are called Kal Baisaki or Norwesters occur fairly frequently during summer months over North East India.

The liability of a building or a structure to high wind pressure depends not only upon the geographical location and proximity of other obstructions to airflow but also upon the characteristics of the structure itself. In general, wind speed in the atmospheric boundary layer increases with height from zero at ground level to maximum at a height called the gradient height. The variation of wind with height depends primarily on the terrain conditions. However, the wind speed at any height never remains constant and it has been found convenient to resolve its instantaneous magnitude in to an average or mean value and a fluctuating component around this average value. The magnitude of fluctuating component of the wind speed is called gust, it depends upon averaging time. In general, smaller the averaging interval, greater is the magnitude of the gust speed. The wind load depends upon terrain, height of the structure and the shape and size of structure. It is essential to know the following terms to study the new concept of wind as described in IS : 875 (Part 3) – 1987

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### 3.6 SNOW LOAD

The snow load depends upon latitude of place and atmospheric humidity. The snow load acts vertically and it is expressed in  $kN/m^2$  of plan area. The actual load due to snow depends upon the shape of the roof and its capacity to retain the snow. When actual data for snow load is not available, snow load may be assumed to be 25 N/m<sup>2</sup> per mm depth of snow. It is usual practice to assume that snow load and maximum wind load will not be acting simultaneously on the structure.

### 3.7 SEISMIC LOAD (EARTHQUAKE LOAD)

It becomes essential to consider 'seismic load' in the design of structure, if the structure is situated in the seismic areas. The seismic areas are the regions which are geologically young and unstable parts and which have experienced earthquakes in the past and are likely to experience earthquakes in future. The Himalayan region, Indo Gangetic Plain, Western India, Cutch and Kathiawar are the places in our country which experience earthquakes frequently. Sometimes these earthquakes are violent also. Seismic load is caused by the shocks due to an earthquake. The earthquakes range from small tremors to severe shocks. The earthquake shocks cause movement of ground, as a result of which the structure vibrates. The vibrations caused because of earthquakes may be resolved in three perpendicular directions. The horizontal direction of vibration dominates over other directions. In some cases structures are designed for horizontal seismic forces only and in some case both horizontal seismic forces and vertical seismic forces are taken in to account. The seismic accelerations for the design may be arrived at from seismic coefficient, which is defined as the ratio of acceleration due to earthquakes and acceleration due to gravity. Our country has been divided in to seven zones for determining seismic coefficients. The seismic coefficients have also been recommended for different types of soils for the guidance of designers. IS: 1893-1962 Indian Standard Recommendations for Earthquake Resistant Design of Structure, may be referred to for actual design.

### 3.8 SOIL AND HYDROSTATIC PRESSURE

The pressure exerted by soil or water or both should be taken in to consideration for the design of structures or parts of structure which are below ground level. The soil pressure and hydrostatic pressure may be calculated from established theories.

#### 3.9 ERECTION EFFECTS

### E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

The erection effects include all effects to which a structure or part of structure is subjected during transportation of structural members and erection of structural member by equipments. Erection effects also take in to account the placing or storage of construction materials. The proper provisions shall be made, e.g., temporary bracings, to take care of all stresses caused during erection. The stress developed because of erection effects should not exceed allowable stresses.

### 3.10 DYNAMIC EFFECTS (IMPACTS AND VIBRATIONS)

The moving loads on a structure cause vibrations and have also impact effect. The dynamic effects resulting from moving loads are accounted for, by impact factor. The live load is increased by adding to it the impact load. The impact load is determined by the product of impact factor and live load.

#### 3.11 TEMPERATURE EFFECTS

The variation in temperature results in expansion and contraction of structural material. The range of variation in temperature varies from localities to localities, season to season and day to day. The temperature effects should be accounted for properly and adequately. The allowable stress should not be exceeded by stress developed because of design loads and temperature effects.

#### 3.12 LOAD COMBINATIONS

All the parts of the steel structure shall be capable of sustaining the most adverse combination of the dead loads, prescribed live loads, wind loads, earthquake loads where applicable and any other forces or loads to which the steel structure may reasonably be subjected without exceeding the stress specified. The load combinations for design purpose shall be the one that produces maximum forces and effects and consequently maximum stresses from the following combinations

- 1. Dead load + Imposed (live) load
- 2. Dead load + Imposed (live) load + wind or earthquake loads and
- **3**. Dead load + wind or earthquake loads

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **LESSON 4. Stresses on structures**

### 4.1 INTRODUCTION

When a structural member is loaded, deformation of the member takes place and resistance is set up against deformation. This resistance to deformation is known as stress. The stress is defined as force per unit cross sectional area. The nature of stress developed in the structural member depends upon nature of loading on the member.

### 4.2 TYPES OF LOADS

#### The following are the various types of stresses:

- 1. Axial stress (direct stress) : i. Tensile stress ii. Compressive stress
- 2. Bearing stress
- **3**. Bending stress
- 4. Shear stress

A member may be subjected to combined direct and bending stress. Such stress is known as combined stress. The tensile stresses are taken as positive and compressive stress as negative. This sign convention for stresses is convenient as a structural member elongates on application of tensile load and shortens on application of compressive load.

### 4.3 STRESS-STRAIN RELATIONSHIP FOR MILD STEEL

When a mild steel bar is subjected to a tensile load, it elongates. The elongation per unit length is known as strain. The stress is proportional to stain within limit of proportionality. The stress-strain relationship for mild steel can be studied by plotting stress-strain curve. The stress and load may be plotted on y-axis and strain may be plotted on x-axis as shown in Fig. 4.1.

### E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,



(Prepared By: Ms Kajal, lecturer, CE)

When the tensile load increases with increase in strain, stress-strain curve follows a straight line relationship up to 'Limit of proportionality'. The limit of proportionality is defined as stress beyond which straight line relationship ceases between stress and strain. Beyond the limit of proportionality stress approaches the elastic limit. The elastic limit is defined as the maximum stress up to which a specimen regains its original length on the removal of the applied load. There is hardly any distinct difference in the position of limit of proportionality and elastic limit. Practically, position of limit of proportionality coincides with the elastic limit. When the specimen is loaded beyond the elastic limit, the specimen does not resume its original length on the removal of applied load and a little strain is left in the specimen. This little strain is known as residual strain or permanent set.

When the tensile load further increases the stress reaches 'yield stress' and material starts yielding. The stress-strain curve suddenly falls showing a decrease in stress. The distinct position from where sudden fall of curve occurs marks the upper yield point and the position up to which fall of curve occurs is known as lower yield point. The material stretches suddenly at constant stress. The adjustment of stress takes place in the elements of material in between upper yield point and lower yield point. On further increase of load, stress increases with the increase of strain. However, strain increases more rapidly. Finally the load reaches the value of 'ultimate load'. The ultimate load is defined as maximum load, which can be placed prior to the breaking of specimen. The stress corresponding to the ultimate load is known as 'ultimate

Fig. 4.1 Stress-strain curve

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

stress'. The stress-strain curve suddenly falls with rapid increase in strain and specimen breaks. The load corresponding to breaking position is known as 'breaking load'. The cross-section of specimen decreases. If actual breaking stress is computed on the basis of decreased cross-sectional area, the breaking stress will be found to be more than the ultimate stress.

The boundaries of grains of mild steel are composed of brittle material. This forms a rigid skeleton. The rigid skeleton prevents plastic deformation of the grains at low stress and shows upper yield point in stress-strain curve. At upper yield point, this rigid skeleton breaks down. As a result of this, the stress in material drops down without elongation from upper yield point to lower yield point. This is followed by sudden stretching of the material at constant stress from lower yield point up to strain hardening.

#### 4.4 TENSILE STRESS

When a structural member is subjected to direct axial tensile load, the stress is known as tensile stress ( $\sigma_{at}$ ). The tensile stress is calculated on net cross-sectional area of the member:

$$\sigma_{at} = (P_t/A_n)$$

Where  $P_t$  is the direct axial tensile load and  $A_n$  is the net cross-sectional area of the member.

#### 4.5 COMPRESSIVE STRESS

When a structural member is subjected to direct axial compressive load, the stress is known as compressive stress ( $\sigma_{ac}$ ). The compressive stress is calculated on gross cross-sectional area of the member

$$\sigma_{ac} = (P_c/A_g)$$

Where  $P_c$  is the direct axial compressive load and  $A_g$  is the gross cross-sectional area of the member

#### 4.6 BEARING STRESS

When a load is exerted or transferred by the application of load through one surface for the another surface in contact, the stress is known as 'bearing stress'( $\sigma_b$ ). the bearing stress is calculated on net projected area of contact

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

#### $\sigma_b = (P/A)$

Where P is load placed on the bearing suface and A is the net projected area of contact.

### 4.7 WORKING STRESS

The working stress is also termed as allowable stress or permissible stress. The working stress is evaluated by dividing yield stress by factor of safety. For the purpose of computing safe load carrying capacity of a structural member, its strength is expressed in terms of working stress. The working stress is the stress which may be developed or set up in the member without causing structural damage to it. The actual stress resulting in a structural member from design loads should not exceed working stresses. This ensures the safety of structural member. The maximum working stresses are adopted from IS : 800-1984.

### 4.8 INCREASE IN PERMISSIBLE STRESS

A structure may be subjected to the different combinations of loads. These loads in combinations do not act for long period. Most of the national codes allow some increase in permissible stresses. Increase in permissible stresses as per IS : 800 is taken as follows:

- 1. When the effect of wind or seismic load is taken in to account, the permissible stress in steel are increased by  $33\frac{1}{3}$  percent.
- 2. For rivets, bolts and tension rods, the permissible stresses are increased by 25 per cent, when the effect of wind or seismic load is taken in to account.

The increased values of permissible stress must not exceed yield stress of the material.

### 4.9 FACTOR OF SAFETY

The factor of safety is defined as the factor by which the yield stress of the material is divided to give the working stress (permissible stress) in the material. A greater value of factor of safety results a larger cross-section of the member had to be adopted in design. If the factor of safety is comparatively small, results in appreciable saving in the material. The value of factor of safety is decided keeping in view of the following considerations.

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

- 1. The average strength of materials is determined after making test on number of specimens
- 2. The value of design loads remains uncertain
- 3. The values of internal forces in many structures depend upon methods of analysis
- 4. During fabrication, structural steel is subjected to different operations which causes the structural element are subjected to uncertain erection stress
- 5. The variations in temperatures and settlement of supports are uncertain
- 6. The failure of some small or some elements of a structure is less serious and less disastrous than the failure of large structure or main element of a structure

### 4.10 METHODS OF DESIGN

The following methods may be employed for the design of the steel frame work:

- 1. Simple design
- 2. Semi-rigid design
- 3. Fully rigid design and
- 4. Plastic design

### 4.10.1 Simple Design

This method is based on elastic theory and applies to structure in which the end connections between members are such that they will not develop restraint moments adversely affecting the members and the structures as a whole and in consequence the structure may be assumed to be pin jointed.

### 4.10.2 **Semi-rigid design**

This method permits a reduction in the maximum bending moment in beams suitably connected to their supports, so as to provide a degree of direction fixity. In the case of triangulated

### E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

frames, it permits rotation 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, it is ensured that the assumed partial fixity is available and 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 IS : 800-1984.

### 4.10.3 Fully rigid design

This method assumes that the end connections are fully rigid and are capable of transmitting moments and shears. It is also assumed that the angle between the members at the joint does not change, when it is subjected to loading. This method gives economy in the weight of steel used when applied in appropriate cases. The end connections of members of the frame shall have sufficient rigidity to hold virtually unchanged original angles between such members and the members they connect. The design should be based on accurate methods of elastic analysis and calculated stresses shall not exceed permissible stress.

#### 4.10.4 Plastic design

The method of plastic analysis and design is recently (1935) developed and all the problems related to this are not yet decided. In this method, the structural usefulness of the material is limited up to ultimate load. This method has its main application in the analysis and design of statically indeterminate framed structures. This method provides striking economy as regards the weight of the steel. This method provides the margin of safety in terms of load factor which one is not less than provided in elastic design. A load factor of 1.85 is adopted for dead load plus live load and 1.40 is adopted for dead load, live load and wind or earthquake forces. The deflection under working load should not exceed the limits prescribed in IS : 800-1984.

### 4.11 STABILITY OF STRUCTURE

According to the stability requirement, the stability of a structure as a whole against overturning is ensured so that the restoring moment is greater than the maximum overturning moment. The restoring moment shall be not less than the sum of 1.2 times the maximum overturning moment due to the characteristic dead load and 1.4 times the maximum overturning moment due to characteristic imposed loads.

### E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

#### Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

The structure should have adequate factor of safety against sliding due to the most adverse combination of the applied loads. The structure shall have a factor of safety against sliding not less than 1.4 under the most adverse combination of the applied characteristic forces. In case only dead loads are acting, only 0.9 times the characteristic dead load shall be taken in to account.

To ensure stability at all times, account shall be taken of probable variations in dead load during construction, repair or other temporary measures. The wind and seismic loading shall be treated as imposed loading. In designing the framework of a building, provisions shall be made by adequate moment connections or by a system of bracings to effectively transmit all the horizontal forces to the foundations.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

### **LESSON 5. Riveted Connections**

### **5.1 INTRODUCTION**

In engineering practice it is often required that two sheets or plates are joined together and carry the load in such ways that the joint is loaded. Many times such joints are required to be leak proof so that gas contained inside is not allowed to escape. A riveted joint is easily conceived between two plates overlapping at edges, making holes through thickness of both, passing the stem of rivet through holes and creating the head at the end of the stem on the other side. A number of rivets may pass through the row of holes, which are uniformly distributed along the edges of the plate. With such a joint having been created between two plates, they cannot be pulled apart. If force at each of the free edges is applied for pulling the plate apart the tensile stress in the plate along the row of rivet hole and shearing stress in rivets will create resisting force. Such joints have been used in structures, boilers and ships. The following are the usual applications for connection.

- 1. Screws,
- 2. Pins and bolts,
- 3. Cotters and Gibs,
- 4. Rivets,
- 5. Welds.

Of these screws, pins, bolts, cotters and gibs are used as temporary fastening i.e., the components connected can be separated easily. Rivets and welds are used as permanent fastenings i.e., the components connected are not likely to require separation.

### **5.2 RIVETS**

Rivet is a round rod which holds two metal pieces together permanently. Rivets are made from mild steel bars with yield strength ranges from 220 N/mm<sup>2</sup> to 250 N/mm<sup>2</sup>. A rivet consists of a head and a body as shown in Fig 5.1. The body of rivet is termed as shank. The head of rivet is formed by heating the rivet rod and upsetting one end of the rod by running it into the rivet

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

machine. The rivets are manufactured in different lengths to suit different purposes. The size of rivet is expressed by the diameter of the shank.



Holes are drilled in the plates to be connected at the appropriate places. For driving the rivets, they are heated till they become red hot and are then placed in the hole. Keeping the rivets pressed from one side, a number of blows are applied and a head at the other end is formed. When the hot rivet so fitted cools it shrinks and presses the plates together. These rivets are known as hot driven rivets. The hot driven rivets of 16 mm, 18 mm, 20 mm and 22 mm diameter are used for the structural steel works.

### E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

Some rivets are driven at atmospheric temperature. These rivets are known as cold driven rivets. The cold driven rivets need larger pressure to form the head and complete the driving. The small size rivets ranging from 12 mm to 22 mm in diameter may be cold driven rivets. The strength of rivet increases in the cold driving. The use of cold driven rivets is limited because of equipment necessary and inconvenience caused in the field.

The diameter of rivet to suit the thickness of plate may be determined from the following formulae:

1.	Unwins's formula	$d=6.05 t^{0.5}$
2.	The French formula	d=1.5 t + 4
3.	The German formula	$d=(50 t - 2)^{0.5}$

Where d= nominal diameter of rivet in mm and t= thickness of plate in mm.

### 5.3 RIVET HEADS

The various types of rivet heads employed for different works are shown in Fig. 5.2. The proportions of various shapes of rivet heads have been expressed in terms of diameter 'D' of the shank of rivet. The snap head is also termed as round head and button head. The snap heads are used for rivets connecting structural members. Sometimes it becomes necessary to flatten the rivet heads so as to provide sufficient clearance. A rivet head which has the form of a truncated cone is called a countersunk head. When a smooth flat surface is required, it is necessary to have rivets countersunk and chipped.

(Prepared By: Ms Kajal, lecturer, CE)

### E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,



### 5.4 RIVET HOLES

The rivet holes are made in the plates or structural members by punching or drilling. When the holes are made by punching, the holes are not perfect, but taper. A punch damages the material around the hole. The operation known as reaming is done in the hole made by punching. When the hole are made by drilling, the holes are perfect and provide good alignment for driving the rivets. The diameter of a rivet hole is made larger than the nominal diameter of the rivet by 1.5 mm of rivets less than or equal to 25 mm diameter and by 2 mm for diameter exceeding 25 mm.

### 5.5 DEFINITIONS OF TERMS USED IN RIVETING

### 5.5.1 Nominal diameter of rivet (d):

The nominal diameter of a rivet means the diameter of the cold shank before driving.

#### 5.5.2 Gross diameter of rivet (D):

### E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

The diameter of the hole is slightly greater than the diameter of the rivet shank. As the rivet is heated and driven, the rivet fills the hole fully. The gross or effective diameter of a rivet means the diameter of the hole or closed rivet. Strengths of rivet are based on gross diameter.

### 5.5.3 Pitch of rivet (p):

The pitch of rivet is the distance between two consecutive rivets measured parallel to the direction of the force in the structural member, lying on the same rivet line. Minimum pitch should not be less than 2.5 times the nominal diameter of the rivet. As a thumb rule pitch equal to 3 times the nominal diameter of the rivet is adopted. Maximum pitch shall not exceed 32 times the thickness of the thinner outside plate or 300 mm whichever is less.

#### 5.5.4 Gauge distance of rivets (g):

The gauge distance is the transverse distance between two consecutive rivets of adjacent chains (parallel adjacent lines of fasteners) and is measured at right angles to the direction of the force in the structural member.

### 5.5.5 Gross area of rivet:

The gross area of rivet is the cross sectional area of a rivet calculated from the gross diameter of the rivet.

### 5.5.6 Rivet line:

The rivet line is also known as scrieve line or back line or gauge line. The rivet line is the imaginary line along which rivets are placed. The rolled steel sections have been assigned standard positions of the rivet lines. The standard position of rivet lines for the various sections may be noted from ISI Handbook No.1 for the respective sections. These standard positions of rivet lines are conformed to whenever possible. The departure from standard position of the rivet lines may be done if necessary. The dimensions of rivet lines should be shown irrespective of whether the standard positions have been followed or not.

### 5.5.7 Staggered pitch:

The staggered pitch is also known as alternate pitch or reeled pitch. The staggered pitch is defined as the distance measured along one rivet line from the centre of a rivet on it to the

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

centre of the adjoining rivet on the adjacent parallel rivet line. One or both the legs of an angle section may have double rivet lines. The staggered pitch occurs between the double rivet lines.

### 5.6 TYPES OF JOINTS

Riveted joints are mainly of two types, namely, Lap joints and Butt joints.

5.6.1 Lap Joint: Two plates are said to be connected by a lap joint when the connected ends of the plates lie in parallel planes. Lap joints may be further classified according to number of rivets used and the arrangement of rivets adopted. Following are the different types of lap joints.

- 1. Single riveted lap joint (Fig.5.3),
- 2. Double riveted lap joint:

a. Chain riveted lap joint (Fig.5.4) b. Zig-zag riveted lap joint (Fi.5.5)

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

( Prepared By: Ms Kajal, lecturer, CE)


E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering , Sem-6<sup>th</sup>

( Prepared By: Ms Kajal, lecturer, CE)



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

## 5.6.1 Butt Joint:

In a butt joint the connected ends of the plates lie in the same plane. The abutting ends of the plates are covered by one or two cover plates or strap plates. Butt joints may also be classified into single cover but joint, double cover butt joints. In single cover butt joint, cover plate is provided on one side of main plate (Fig.5.6). In case of double cover butt joint, cover plates are provided on either side of the main plate (Fig.5.7). Butt joints are also further classified according to the number of rivets used and the arrangement of rivets adopted.

- 1. Double cover single riveted but joint
- 2. Double cover chain riveted butt joint
- 3. Double cover zig-zag riveted butt joint



(A) SINGLE COVER PLATE BUTT-JOINT



E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

## 5.7 FAILURE OF A RIVETED JOINT

Failure of a riveted joint may take place in any of the following ways

- 1. Shear failure of rivets
- 2. Bearing failure of rivets
- 3. Tearing failure of plates
- 4. Shear failure of plates
- 5. Bearing failure of plates
- 6. Splitting/cracking failure of plates at the edges

## 5.7.1 Shear failure of rivets :

Plates riveted together and subjected to tensile loads may result in the shear of the rivets. Rivets are sheared across their sectional areas. Single shear occurring in a lap joint and double shear occurring in but joint (Fig.5.8)



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

## 5.7.2 Bearing failure of rivets:

Bearing failure of a rivet occurs when the rivet is crushed by the plate (Fig.5.9)



## 5.7.3 Tearing failure of plates :

When plates riveted together are carrying tensile load, tearing failure of plate may occur. When strength of the plate is less than that of rivets, tearing failure occurs at the net sectional area of plate (Fig.5.10)



# 5.7.4 Shear failure of plates:

A plate may fail in shear along two lines as shown in Fig. 5.11. This may occur when minimum proper edge distance is not provided.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Co Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## 5.7.5 Bearing failure of plates:

**B**earing failure of a plate may occur because of insufficient edge distance in the riveted joint. Crushing of plate against the bearing of rivet take place in such failure (Fig. 5.12)



5.7.6 Splitting/cracking failure of plates at the edges:

This failure occurs because of insufficient edge distance in the riveted joint. Splitting (cracking) of plate as shown in Fig. 5.13 takes place in such failure.

Shearing, bearing and splitting failure of plates may be avoided by providing adequate proper edge distance. To safeguard a riveted joint against other modes of failure, the joint should be designed properly.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Cou Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



### 5.8 STRENGTH OF RIVETED JOINT

The strength of a riveted joint is determined by computing the following strengths:

1.	Strength of a riveted joint against shearing	- P <sub>s</sub>
2.	Strength of a riveted joint against bearing	- P <sub>b</sub>
3.	Strength of plate in tearing	- P <sub>t</sub>

The strength of a riveted joint is the least strength of the above three strength.

## 5.8.1 Strength of a riveted joint against shearing of the rivets:

The strength of a riveted joint against the shearing of rivets is equal to the product of strength of one rivet in shear and the number of rivets on each side of the joint. It is given by

 $P_s$  = strength of a rivet in shearing x number of rivets on each side of joint

When the rivets are subjected to single shear, then the strength of one rivet in single shear

$$=\frac{\pi}{4}D^2p_s$$

 $P_s = N \frac{\pi}{4} D^2 p_s$ 

Therefore, the strength of a riveted joint against shearing of rivets =

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

Where N=Number of rivets on each side of the joint; D=Gross diameter of the rivet;  $p_s$ =Maximum permissible shear stress in the rivet(1025 ksc).

When the rivets are subjected to double shear, then the strength of one rivet in double shear

 $=\frac{2\frac{\pi}{4}D^2p_s}{2\frac{\pi}{4}}$ . Therefore, the strength of a riveted joint against double shearing of rivets,

$$P_{\rm s} = N[2\frac{\pi}{4}D^2p_{\rm s}]$$

When the strength of riveted joint against the shearing of the rivets is determined per gauge width of the plate, then the number of rivets 'n' per gauge is taken in to consideration. Therefore,

For single shear of rivets, 
$$P_{s1} = n[\frac{\pi}{4}D^2p_s]$$
  
For double shear of rivets  $P_{s2} = n[2\frac{\pi}{4}D^2p_s]$ 

## 5.8.2 Strength of riveted joint against the bearing of the rivets:

The strength of a riveted joint against the bearing of the rivets is equal to the product of strength of one rivet in bearing and the number of rivets on each side of the joint. It is given by,

P<sub>b</sub>=Strength of a rivet in bearing x Number of rivets on each side of the joint

In case of lap joint, the strength of one rivet in bearing =  $D x t x p_b$ 

Where D= Gross diameter of the rivet; t=thickness of the thinnest plate;  $p_b$ = maximum permissible stress in the bearing for the rivet (2360 ksc). In case of butt joint, the total thickness of both cover plates or thickness of main plate whichever is less is considered for determining the strength of a rivet in the bearing.

The strength of a riveted joint against the bearing of rivets  $P_b = N \times D \times t \times p_b$ 

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

When the strength of riveted joint against the bearing of rivets per gauge widh of the plate is taken into consideration, then, the number of rivets 'n' is also adopted per gauge. Therefore,

 $P_{b1} = n \ge D \ge t \ge p_b$ 

## 5.8.3 Strength of plate in tearing

The strength of plate in tearing depends upon the resisting section of the plate. The strength of plate in tearing is given by  $Pt = Resisting section x p_t$ 

Where  $p_t$  is the maximum permissible stress in the tearing of plate (1500 ksc). When the strength of plate in tearing per pitch width of the plate is  $P_{t1} = (p-D) x t x p_t$ 

The strength of a riveted joint is the least of  $P_s$ ,  $P_b$ ,  $P_t$ . The strength of riveted joint per gauge width of plate is the least of  $P_{s1}$ ,  $P_{b1}$ ,  $P_{t1}$ .

# 5.9. STRENGTH OF LAP AND BUTT JOINT

The strength of riveted lap and butt joint given in the Fig. 5.14 is summarized as follows:



# 5.9.1 Strength of lap joint:

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

- 1. Strength of riveted joint against shearing  $P_s = 6\frac{\pi}{s}D^2p_s$
- 2. Strength of riveted joint against bearing  $P_b = 6 \times D \times t \times p_b$
- 3. Strength of riveted joint against tearing  $P_t = (b-3D) x t x p_t$
- 4. Strength of riveted joint against shearing per gauge width  $P_{s1} = 2\frac{\pi}{a}D^2p_s$
- 5. Strength of riveted joint against bearing per gauge width  $P_{b1} = 2 \times D \times t \times p_b$
- 6. Strength of riveted joint against tearing per gauge width  $P_{t1} = (p-D)x t x p_t$

## 5.9.2 Strength of butt joint:

- 1. Strength of riveted joint against shearing  $P_s = 9 x 2 x \frac{\pi}{4} x D^2 x p_s$
- 2. Strength of riveted joint against bearing  $P_b = 9 \times D \times t \times p_b$
- 3. Strength of riveted joint against tearing  $P_t = (b-3D) x t x p_t$
- 4. Strength of riveted joint against shearing per gauge width  $P_{s1} = 3 \times 2 \times \frac{\pi}{4} \times D^2 \times p_s$
- 5. Strength of riveted joint against bearing per gauge width  $P_{b1} = 3 \times D \times t \times p_b$
- 6. Strength of riveted joint against tearing per gauge width  $P_{t1} = (p-D)x t x p_t$

### 5.10 EFFICIENCY OR PERCENTAGE OF STRENGTH OF RIVETED JOINT

The efficiency of a joint is defined as the ratio of least strength of a riveted joint to the strength of solid plate. It is known as percentage strength of riveted joint as it is expressed in percentage.

Efficiency of riveted joint

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

$$\eta = \frac{\text{Least strength of riveted joint}}{\text{Strength of solid plate}} \times 100$$
$$\eta = \frac{\text{Least of } P_s, P_b \text{ or } P_t}{P} \times 100$$

Where P is the strength of solid plate =  $b x t x p_t$ 

Efficiency per pitch width

$$= \frac{(p-D)x t x p_t}{p x t x p_t} x 100$$
$$= \frac{(p-D)}{p} x 100$$

## 5.11 RIVET VALUE

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R).

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 6. Design of Riveted Connections**

## 6.1 INTRODUCTION

The perfect theoretical analysis for stress distribution in riveted connections cannot be established. Hence a large factor of safety is employed in the design of riveted connections. The riveted connections should be as strong as the structural members. No part in the riveted connections should be so overstressed. The riveted connections should be so designed that there is neither any permanent distortion nor any wear. These should be elastic. In general, the work of fabrication is completed in the workshops where the steel is fabricated.

## 6.2 ASSUMPTIONS FOR THE DESIGN OF RIVETED JOINT

Procedure for design of a riveted joint is simplified by making the following assumptions and by keeping in view the safety of the joint.

- 1. Load is assumed to be uniformly distributed among all the rivets
- 2. Stress in plate is assumed to be uniform
- 3. Shear stress is assumed to be uniformly distributed over the gross area of rivets
- 4. Bearing stress is assumed to be uniform between the contact surfaces of plate and rivet
- 5. Bending stress in rivet is neglected
- 6. Rivet hole is assumed to be completely filled by the rivet
- 7. Friction between plates is neglected

## **6.3 ARRANGEMENT OF RIVETS**

Rivets in a riveted joint are arranged in two forms, namely, 1. Chain riveting, 2. Diamond riveting.

6.3.1 **Chain Riveting:** In chain riveting the rivets are arranged as shown in Fig. 6.1 and in the figure 1-1, 2-2 and 3-3 shows sections on either side of the joint. Section 1-1 is the critical section as compared to the other section. At section 2-2 is equal to the strength of plate in

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

tearing at 2-2 plus strength of three rivets in bearing or shearing whichever is less at 1-1. At section 3-3 is equal to the strength of plate in tearing at 3-3 plus strength of rivets in bearing or shearing whichever is less (6 nos.).



Therefore,

Strength of plate in tearing at  $1-1 = (b-3D).t.p_t$ 

Where b= width of the plate; D=Gross diameter of the rivet and t=Thickness of the plate.

When safe load carried by the joint (P) is known, width of the plate can be found as follows;

$$b = \left(\frac{P}{t \times p_t} + 3D\right)$$

6.3.2 **Diamond Riveting:** In diamond riveting, rivets are arranged as shown in Fig.6.2. All the rivets are arranged symmetrically about the centre line of the plate. Section 1-1 is the critical section. Strength of the plate in tearing in diamond riveting section 1-1 can be computed as follows

## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)



$$P_t = (b - D).t.p_t$$

When the safe load carried by the joint (P) is known, width of the plate can be found as follows

$$b = (\frac{P}{t \times p_t} + D)$$

Where b=width of the plate, D=gross diameter of the rivet and t=thickness of the plate.

At section 2-2: All the rivets are stressed uniformly, hence strength of the plate at section 2-2 is

 $P_t = (b - 2D).t.p_t + strength of one rivet in shearing & bearing whichever is less$ 

At section 3-3,

 $P_t = (b - 3D)$ .t.p, + strength of three rivet in shearing & bearing whichever is less

In diamond riveting there is saving of material and efficiency is more. Diamond riveting is used in bridge trusses generally.

### 6.4 SPECIFICATION FOR DESIGN OF RIVETED JOINT

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

6.4.1 **Members meeting at Joint:** The centroidal axes of the members meeting at a joint should intersect at one point, and if there is any eccentricity, adequate resistance should be provided in the connection.

6.4.2 **Centre of Gravity:** The centre of gravity of group of rivets should be on the line of action of load whenever practicable.

## 6.4.3 **Pitch:**

a. Minimum pitch: The distance between centres of adjacent rivets should not be less than 2.5 times the gross diameter of the rivet.

b. Maximum pitch: Maximum pitch should not exceed 12t or 200 mm whichever is less in compression member and 16t or 200 mm whichever is less in case of tension members, when the line of rivets lies along the line of action of force. If the line of rivets does not lie along the line of action of force, its maximum pitch should not exceed 32t or 300 mm whichever is less, where t is the thickness of the outside plate.

6.4.4 Edge Distance: A minimum edge distance of approximately 1.5 times the gross diameter of the rivet measured from the centre of the rivet hole is provided in the rivet joint. Table 6.1 gives the minimum edge distance as per recommendations of BIS in IS : 800-1984.

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

	Edge distance of Hole			
Gross diameter of rivet (mm)	Distance to sheared or Hand flame cut edge (mm)	Distance to rolled machine flame cut or planed edge (mm)		
13.5 and below	19	17		
15.5	25	22		
17.5	29	25		
19.5	32	29		
21.5	32	29		
23.5	38	32		
25.5	44	38		
29.0	51	44		
32.0	57	51		
35.0	57	51		

# TABLE 6.1 EDGE DISTANCE OF HOLES

## 6.5 DESIGN PROCEDURE FOR RIVETED JOINT

For the design of a lap joint or butt joint, the thickness of plates to be joined is known and the joints are designed for the full strength of the plate. For the design of a structural steel work, force (pull or push) to be transmitted by the joint is known and riveted joints can be designed. Following are the usual steps for the design of the riveted joint:

## Step 1:

The size of the rivet is determined by the Unwin's formula

 $d = 6.04\sqrt{t}$ 

Where d= nominal diameter of rivet in mm and t= thickness of plate in mm.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

The diameter of the rivet computed is rounded off to available size of rivets. Rivets are manufactured in nominal diameters of 12, 14, 16, 18, 20, 22, 24, 27, 30, 33, 36, 39, 42 and 48 mm

### Step 2:

The strength of rivets in shearing and bearing are computed. Working stresses in rivets and plates are adopted as per ISI. Rivet value R is found. For designing lap joint or butt joint tearing strength of plate is determined as follows

$$P_t=(p-D).t.p_t$$

Where p=pitch of rivets adopted, t=thickness of plate and  $p_t$  = working stress in direct tension for plate. Tearing strength of plate should not exceed the rivet value R (P<sub>s</sub> or P<sub>b</sub> whichever is less) or

$$(p-D).t.p_t \leq R$$

From this relation pitch of the rivets is determined.

## Step 3:

In structural steel work, force to be transmitted by the riveted joint and the rivet value are known. Hence number of rivets required can be computed as follows

Number of rivets required in the joint = 
$$\frac{Force}{Rivet value}$$

The number of rivets thus obtained is provided on one side of the joint and an equal number of rivets is provided on the other side of joint also.

## Step 4:

For the design of joint in a tie member consisting of a flat, width/thickness of the flat is known. The section is assumed to be reduced by rivet holes depending upon the arrangements of the rivets to be provided, strength of flat at the weakest section is equated to the pull transmitted by

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

the joint. For example, assuming the section to be weakened by one rivet and also assuming that the thickness of the flat is known we have

$$(b-D)$$
.  $t. p_t = P$ 

Where b= width of flat, t=thickness of flat, pt=working stress in tension in plate and P=pull to be transmitted by the joint. From this equation, width of the flat can be determined.

**Example 6.1:** A single riveted lap joint is used to connect plate 10 mm thick. If 20 mm diameter rivets are used at 55 mm pitch, determine the strength of joint and its efficiency. Working stress in shear in rivets=80 N/mm<sup>2</sup> (MPa). Working stress in bearing in rivets=250 N/mm<sup>2</sup> (MPa). Working stress in axial tension in plates=156 N/mm<sup>2</sup>.

## Solution

Assume that power driven field rivets are used. Nominal diameter of rivet (D) is 20 mm and gross diameter of rivet is 21.5 mm.

Strength of rivet in single she	ear	$= (\pi/4) \ge 21.5^2 \ge 80/1000$			
	Ps	= 29.044 kN			
Strength of rivet in bearing		= 21.5 x 10 x 250/1000			
	$\mathbf{P}_{\mathbf{b}}$	= 53.750 kN			
Strength of plate in tension per gauge length = $P_t$ =(p-D).t.p <sub>t</sub>					

 $P_t = (55-21.5) \times 10 \times 156/1000$ = 52.260 kN

Strength of joint is minimum of Ps, Pb or Pt

Therefore, the strength of joint is = 29.044 kN

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Cour Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Efficiency of joint

$$\eta = \frac{Strength of joint per pitch length}{Strength of solid plate} \times 100$$
$$\eta = \frac{29.044 \times 10^3}{55 \times 10 \times 156} \times 100 = 33.85 \%$$

**Example 6.2:** A double riveted double cover butt joint is used to connect plates 12 mm thick. Using Unwin's formula, determine the diameter of rivet, rivet value, pitch and efficiency of joint. Adopt the following stresses;

Working stress in shear in power driven rivets=100 N/mm<sup>2</sup> (MPa).

Working stress in bearing in power driven rivets=300 N/mm<sup>2</sup> (MPa).

For plates working stress in axial tension  $=156 \text{ N/mm}^2$ .

### Solution

Nominal diameter of rivet from Unwin's formula

 $d = 6.04\sqrt{t} = 6.04\sqrt{12} = 20.923 \, mm$ 

Adopt nominal diameter of rivet = 22 mm; Gross diameter of rivet = 23.5 mm

Strength of rivet in double shear =  $2 \times \frac{\pi}{4} \times 23.5^2 \times \frac{100}{1000} = 86.75 \, kN$ 

Strength of rivet in bearing = D x t x  $p_b$  = 23.5 x 12 x 300/1000 = 84.6 kN

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R). Hence the Rivet value is 84.6 kN.

Let p be the pitch of the rivets.  $P_t = (p-D) x t x p_t = ((p-23.5) x 12 x 156/100) = 1.872 (p-23.5) kN$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

In double riveted joint,					
Strength of 2 rivets in s	shear	$P_s = 2 \ge 86.75 = 173.5 \text{ kN}$			
Strength of 2 rivets in l	bearing	$P_b = 2 \times 84.6 = 169.2 \text{ kN}$			
The pitch of the rivets can be computed by keeping $P_t = P_s$ or $P_b$ whichever is less					
Therefore	-23.5) = 169.2				
p-23.5 = (169.2/1.872) = 90.385					
	p= 90.38	5 + 23.5 = 113.885  mm			
Adopt pitch,	p= 100 p	mm			
Efficiency of joint	$\eta = \frac{(p-D)}{p} \times 10$	0			

**Example 6.3:** A double cover butt joint is used to connect plates 16 mm thick. Design the riveted joint and determine its efficiency.

 $=\frac{(100-23.5)}{100}\times100=76.5\%$ 

## Solution

Nominal diameter of rivet from Unwin's formula

 $d = 6.04\sqrt{t} = 6.04\sqrt{16} = 24.16 \, mm$ 

The hot driven rivets of 16 mm, 18 mm, 20 mm and 22 mm diameter are used for the structural steel works. Unwin's formula gives higher values. Hence, adopt nominal diameter of rivet = 22 mm; Gross diameter of rivet = 22 + 1.5 = 23.5 mm

In double cover butt joint, rivets are in double shear. As per IS : 800-84,

Shear stress for power driven rivets=100 N/mm<sup>2</sup> (MPa).

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Bearing stress for power driven rivets=300 N/mm<sup>2</sup> (MPa).

Strength of plate in tension = $156 \text{ N/mm}^2$ .

Strength of rivet in double shear =  $2 \times \frac{\pi}{4} \times 23.5^2 \times \frac{100}{1000} = 86.75 \, kN$ 

Strength of rivet in bearing = D x t x  $p_b$  = 23.5 x 16 x 300/1000 = 112.8 kN

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R). Hence the Rivet value is 86.75 kN.

Let p be the pitch of the rivets.  $P_t = (p-D) x t x p_t = ((p-23.5) x 16 x 156/100) = 2.496 (p-23.5) kN$ 

The pitch of the rivets can be computed by keeping  $P_t = P_s$  or  $P_b$  whichever is less

Therefore

2.496 (p-23.5) = 86.75

(p-23.5) = (86.75/2.496) = 34.756

p=55 mm

p = 34.756 + 23.5 = 58.256 mm

Adopt pitch,

Adopt thickness of each cover plate t  $\approx 5/8 \times 16 \approx 10 \text{ mm}$ 

Efficiency of joint 
$$\eta = \frac{(p-D)}{p} \times 100$$
  
=  $\frac{(55-23.5)}{55} \times 100 = 55.27\%$ 

**Example 6.4:** Determine the strength of a double cover butt joint used to connect two flats 200 F 12. The thickness of each cover plate is 8 mm. Flats have been joined by 9 rivets in chain

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

riveting at a gauge of 60 mm as shown in Fig. 6.3. What is the efficiency of the joint? Adopt working stresses in rivets and flats as per IS : 800-84.



## Solution

- Size of flat used = 200 F 12
- Width of flat = 200 mm
- Thickness of flat = 12 mm
- Use power driven rivets

Nominal diameter of rivet from Unwin's formula

 $d=6.04\sqrt{t}=6.04\sqrt{12}=20.923\,mm$ 

Adopt nominal diameter of rivet = 22 mm; Gross diameter of rivet D = 23.5 mm

Strength of rivet in double shear = 
$$2 \times \frac{\pi}{4} \times 23.5^2 \times \frac{100}{1000} = 86.75 \text{ kN}$$
Strength of rivet in bearing = D x t x p<sub>b</sub> = 23.5 x 12 x 300/1000 = 84.6 kN
Strength of joint in shear, P<sub>s</sub> = 9 x 86.75 = 780.75 kN
Strength of joint in bearing P<sub>b</sub> = 9 x 84.6 = 761.40 kN
Strength of plate in tearing P<sub>t</sub> = (b-3D) x t x p<sub>t</sub>

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

= ((200-3x23.5) x 12 x 156/1000)

= 242.42 kN

Strength of joint is minimum of P<sub>s</sub>, P<sub>b</sub> or P<sub>t</sub>

Therefore, the strength of joint is = 242.42 kN

Efficiency of joint

$$\eta = \frac{\text{Least of } P_{\text{s}}, P_{\text{b}} \text{ or } P_{\text{t}}}{\text{Strength of solid plate}} \times 100$$
$$\eta = \frac{242.42 \times 10^3}{200 \times 12 \times 156} \times 100 = 64.75 \%$$

**Example 6.5:** In a truss girder of a bridge, a diagonal consists of a 16 mm thick flat and carries a pull of 750 kN and is connected to a gusset plate by a double cover butt joint. The thickness of each cover plate is 8 mm. Determine the number of rivets necessary and the width of the flat required. What is the efficiency of the joint? Sketch the joint. Take

Working stress in shear in power driven rivets=100 N/mm<sup>2</sup> (MPa).

Working stress in bearing in power driven rivets=300 N/mm<sup>2</sup> (MPa).

For plates working stress in axial tension  $=156 \text{ N/mm}^2$ .



Solution

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Nominal diameter of rivet from Unwin's formula  $d = 6.04\sqrt{t} = 6.04\sqrt{16} = 24.16 mm$ 

The hot driven rivets of 16 mm, 18 mm, 20 mm and 22 mm diameter are used for the structural steel works. Unwin's formula gives higher values. Hence, adopt nominal diameter of rivet = 22 mm; Gross diameter of rivet = 22 + 1.5 = 23.5 mm

$$2 \times \frac{\pi}{4} \times 23.5^2 \times \frac{100}{1000} = 86.75 \, kN$$

Strength of rivet in double shear =

Strength of rivet in bearing = D x t x  $p_b$  = 23.5 x 16 x 300/1000 = 112.8 kN

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R). Hence the Rivet value is 86.75 kN.

Number of rivets required to transmit pull of 750 kN  $n = (750/86.75) = 8.67 \approx 9$  rivets.

Using diamond group of riveting, flat is weakened by one rivet hole. Strength of plate at section 1-1 in teaing

 $P_t = (b-d) x t x p_t = ((b-23.5) x 16 x 156/100) = 2.496 (b-23.5) kN$ 

Since P = 750 kN, 2.496 (b-23.5) = 750

b=(750/2.496)+23.5 = 323.98 mm

Hence provide 400 mm width of diagonal member. The design of joint is shown in Fig. 6.4.

Efficiency of the joint

$$\eta = \frac{(b-D) \times t \times p_t}{b \times t \times p_t} \times 100 = \frac{400 - 23.5}{400} \times 100 = 94.125\%$$

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

**Example 6.6:** A bridge truss diagonal carries an axial pull of 500 kN. It is to be connected to a gusset plate 22 mm thick by a double cover butt joint with 22 mm rivets. If the width of the tie bar is 250 mm, determine the thickness of flat. Design the economical joint. Determine the efficiency of the joint. Adopt working stresses in rivets and flats as per IS : 800-84.

### Solution

Nominal diameter of rivet = 22 mm; Gross diameter of rivet = 23.5 mm

Strength of power driven rivet in double shear =  $2 \times \frac{\pi}{4} \times 23.5^2 \times \frac{100}{1000} = 86.75 \, kN$ 

Strength of power driven rivet in bearing = D x t x  $p_b$  = 23.5 x 22 x 300/1000 = 155.1 kN

The strength of a rivet in shearing and in bearing is computed and the lesser is called the rivet value (R). Hence the Rivet value is 86.75 kN.

Number of rivets required to transmit pull of 500 kN  $n = (500/86.75) = 5.76 \approx 6$  rivets.

Provide six rivets in diamond group of riveting for efficient joint.

Let the thickness of flat be t mm

Strength of plate at weakest section  $P_t = (b-d) x t x p_t = ((250-23.5) x t x 156/100) = 500 kN$ 

Therefore t = 14.151 mm; Adopt 16 mm thickness of flat. Keep 40 mm edge distance from centre of rivet and 85 mm distance between centre to centre of rivet lines as shown in the Fig. 6.5.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



DIAMOND RIVETED DOUBLE COVER BUTT JOINT

# Efficiency of joint

$$\eta = \frac{(b-D) \times t \times p_t}{b \times t \times p_t} \times 100 = \frac{250 - 23.5}{250} \times 100 = 90.6\%$$

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

## **LESSON 7. Welded Connection**

## 7.1 INTRODUCTION

The development of welding technology in 1940s has considerably reduced the riveted joint applications. Welding is the method of locally melting the metals (sheets or plates – overlapping or butting) with intensive heating along with a filler metal or without it and allowing cooling them to form a coherent mass, thus creating a joint. A typical weld showing various zones of weld is shown in Fig. 7.1. Such joints can be created to make structures, boilers, pressure vessels, etc. and are more conveniently made in steel. The progress has been made in welding several types of steels, but large structure size may impede the use of automatic techniques and heat treatment which becomes necessary in some cases. Welded ships were made in large size and large number during Second World War and failures of many of them spurted research efforts to make welding a better technology.



### 7.2 ADVANTAGES OF WELDED CONNECTIONS

- 1. The gross sectional area of the welded members is effective since the welding process does not involve drilling holes.
- 2. Welded structures are comparatively lighter than corresponding riveted structures.
- **3**. A welded joint has a greater strength sometimes equal to the strength of the parent metal itself.
- 4. Repairs and further new connections can be done more easily than in riveting.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

- 5. Welded joints provide rigidity leads to smaller bending moments than corresponding riveted members.
- 6. Welded joints are economical to riveted joints due to low maintenance cost.
- 7. Members of such shapes that afford difficulty for riveting can be more easily welded.
- 8. A welded structure has a better finish and appearance than the corresponding riveted structure.
- 9. Connecting angles, gusset plates, splicing plates can be minimized.
- 10. Steel bars in reinforced concrete structure may be welded easily so that lapping of bars may be avoided.
- 11. It is possible to weld at any point at any part of a structure, but riveting will always require enough clearance.
- 12. The process of welding does not involve great noise compared to the noise produced in the riveting process.

## 7.3 DISADVANTAGES OF WELDED CONNECTIONS

- 1. Welding requires skilled labor and supervision.
- 2. Testing a welded joint is difficult. An X-ray examination alone can enable us to study the quality of the connection.
- **3**. Due to uneven heating and cooling, the welded members are likely to get warped at the welded surface.
- 4. Internal stresses in the welded zones are likely to be set up.

## 7.4 TYPES OF WELDED JOINTS

Welds may be classified into two main types namely butt-weld and fillet-weld.

## 7.4.1. Butt weld

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

This type of weld is used when the members are in same plane. Butt weld is also termed as groove weld. The butt weld is used to join structural members carrying direct compression or tension. It is used to make tee-joint and butt-joint. The following types of butt welds are in practice. These are named depending upon shape of the grove made for welding.

### i. Square butt weld.

A square butt weld is a weld in the preparation of which the fusion faces lie approximately at right angles to the surfaces of the components to be joined and are substantially parallel to one another (Fig. 7.2 a & b).



### ii.Single V-butt weld

A single V-butt weld is a weld in the preparation of which the edges of both components are prepared so that in the cross-section, the fusion faces form a V as shown in Fig. 7.3.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## iii.Double V-butt weld

A double V-butt weld is a weld in the preparation of which the edges of both components are double beveled so that in cross-section, the fusion faces form two opposing V's as shown in Fig. 7.4.



## iv. Single U-butt weld

A single U-butt weld is a weld in the preparation of which the edges of both components are prepared so that in the cross section, the fusion faces form a U as shown in Fig. 7.5.



v. Double U-butt weld

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

A double U-butt weld is a weld in the preparation of which the edges of both components are prepared so that in the cross section, the fusion faces form two opposing U's as shown in Fig. 7.6.



## vi.Single J-butt weld

A single J-butt weld is a weld in the preparation of which the edges of one component are prepared so that in the cross section, the fusion faces is in the form a J and the fusion face of the other component is at right angles to the surface of the first component as shown in Fig. 7.7.



vii.Double J-butt weld

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

A double J-butt weld is a weld in the preparation of which the edges of one component are prepared so that in the cross section, the fusion faces is in the form of two opposing J's and the fusion face of the other component is at right angles to the surface of the first component as shown in Fig. 7.8.



viii.Single bevel butt weld

A single bevel butt weld is a weld in the preparation of which the edge of one component is beveled and the fusion face of the other component is at right angles to the surface of the first component as shown in Fig. 7.9.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## ix.Double bevel butt weld

A double bevel butt weld is a weld in the preparation of which the edges of one component are double beveled and the fusion face of the other component is at right angles to the surface of the first component as shown in Fig. 7.10.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,



(Prepared By: Ms Kajal, lecturer, CE)

## 7.4.2. Specifications of the butt weld

### i. Size of butt weld

The size of a butt weld is specified by the effective throat thickness. The effective throat thickness in case of complete penetration butt weld is taken as the thickness of thinner part joined. The double V, double U, double J and double bevel butt welds are the examples of complete penetration butt weld.

The effective throat thickness in case of incomplete penetration butt weld is taken as 7/8<sup>th</sup> of the thickness of the thinner part joined. But for the purpose of stress calculation, a required effective throat thickness not exceeding 5/8<sup>th</sup> of the thickness of thinner part joined should be used. An incomplete penetration butt weld is also termed as unsealed single butt weld. Single V, Single U, Single J, Single bevel butt joints are the examples of incomplete penetration butt weld, the weld metal is not deposited intentionally through the full thickness of the joint. The unwelded portion in incomplete penetration butt weld, welded from both sides shall not be greater than 1/4<sup>th</sup> of the thickness of thinner part joined and should be central in the depth of the weld.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

The unsealed butt welds V, U, J and bevel types and incomplete penetration butt welds should not be used for highly stressed joints and joints subjected to dynamic, repeated or alternating forces. The shall also not be subjected to a bending moment about the longitudinal axis of the weld other than that normally resulting from the eccentricity of the weld metal relative to the parts joined.

### ii. Effective length of butt weld

The effective length of butt weld is the length for which the specified size (throat thickness) of the weld exists.

### iii. Effective area of butt weld

The effective area of a butt weld is taken as the product of the effective throat thickness and the effective length of butt weld.

### iv. Reinforcement

The extra metal deposited above the surface of the parent metal as shown in Fig. 7.11 is called reinforcement. This reinforcement is provided to give sufficient surfaces convexity and to ensure full effectiveness at the joint. This requires a minimum practical surface convexity of 1.0 mm. This reinforcement should not exceed 3.0 mm. This is not considered as part of throat thickness. This reinforcement may also be removed if a flush surface is desired.



When the structural members of unequal thickness are butt welded and difference in thickness of members exceeds 25 per cent of the thinner part or 3.0 mm in metal arc welding and 6.0 mm or more in oxy-acetylene welding, the thicker part is beveled so that the slop of the surface

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

from one part to the other is not steeper than one in five as shown in Fig. 7.12.A. Where this arrangement is not practicable, the weld metal should be built-up at the junction with the thicker part to dimension at least 25 per cent greater than that of the thinner part in metal arc welding as shown in 7.12.B. alternatively, the weld metal should be built-up to the dimensions of thicker members as shown in 7.12.C. In case of complete penetration butt weld, generally, deign calculations are not necessary, as these will usually provide the strength at the joint equal to the strength of the member connected.



### 7.4.3 Fillet-weld

This type of weld is used when the members to be connected overlap each other. A fillet weld is a weld of approximately triangular cross section joining two surfaces approximately as right angles to each other in lap joint or tee joint. A fillet weld is shown in Fig. 7.13.A. When the cross section of fillet weld is 45°, isosceles triangle as shown in Fig. 7.13.B.I, it is known as a standard fillet weld. The standard 45° fillet weld is generally used. When the cross section of the fillet weld is 30° and 60° triangle as shown in Fig. 7.13.B.II, it is known as a special fillet weld.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)


(Prepared By: Ms Kajal, lecturer, CE)

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,



A fillet weld is termed as concave fillet weld or convex fillet weld or mitre fillet weld depending on the weld face in concave or convex or approximately flat as shown in Fig. 7.14, respectively. A fillet weld is termed as normal fillet weld or deep penetration fillet weld depending upon the depth of penetration beyond the root is less than 2.4 mm or more respectively.



The fillet welds are of three types as shown in Fig. 7.15.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## i.Side fillet weld

It is fillet weld stressed in longitudinal shear, i.e., a fillet weld, the axis of which is parallel to the direction of these applied loads. It is also termed as longitudinal fillet weld.

## ii.End fillet weld

It is a fillet weld stressed in transverse shear, i.e., a fillet weld, the axis of which is at right angles to the direction of the applied load. It is also termed as transverse fillet weld.

## iii.Diagonal fillet weld

It is a fillet weld the axis of which is inclined to the direction of the applied load

A fillet weld is placed on the sides or end of the base metal and it is subjected to shear along with tension or compression and usually bending.

## 7.4.4 Specification of fillet weld

## i. Size of fillet weld

The size of normal fillet weld is specified as minimum leg length of a convex or mitre fillet weld or 1.414 times the effective throat thickness of a concave fillet weld. The size of deep

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

penetration fillet weld is specified as minimum leg length plus 2.4 mm. the length of leg is the distance from the root to the toe of a fillet weld, measured along the fusion face.

The International Standard code has recommended the minimum size of the weld. If the thickness of thicker part is up to 10 mm, the minimum size of the welding is 3 mm. If the thickness of thicker part is in between 10 mm to 20 mm, the minimum size of the welding is 5 mm. If the thickness of thicker part is in between 20 mm to 32 mm, the minimum size of the welding is 6 mm. If the thickness of thicker part is above 32 mm, the minimum size of the welding is 10 mm. When the minimum size of the fillet weld is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of thinner part. Where the thicker part is more than 50 mm, special precaution like preheating will have to be taken.

### ii. Effective throat thickness

The effective throat thickness of a fillet weld is the perpendicular distance from the root to the hypotenuse of the largest isosceles right angled triangle that can be inscribed within the weld cross section. The effective throat thickness of a fillet weld shall not be less than 3 mm and shall generally not exceed 0.7 times the thickness of thinner part and equal to the thickness of thinner part under special circumstances.

$$Effective \ throat \ thickness = \frac{1}{\sqrt{2}} \times Size \ of \ weld \ = 0.7 \ \times Size \ of \ weld$$

In general, for the purpose of stress calculation,

Effective throat thickness =  $K \times Size$  of weld

Where K is a constant. The value of K for different angles between fusion faces is adopted as per Table 7.1 as recommended in IS:816-1969

## TABLE 7.1. VALUE OF K FOR DIFFERENT ANGLES BETWEEN FUSION FACES

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Angle between fusion faces	Value of constant K
60°-90°	0.70
Þ1°-100°	0.65
101°-106°	0.60
107°-113°	0.55
114°-120°	0.50

A fillet weld is not used for joining parts, if the angle between fusion faces is greater than  $120^{\circ}$  or less than  $60^{\circ}$ .

# iii.Effective length

The effective length of the weld is the length of the weld for which the specified size and throat thickness i.e., correctly proportioned cross section of the weld, exist. It is taken as the actual length minus twice the size of weld, since the specified size and throat thickness do not exist at the ends. The effective length of the weld is shown on the drawings. In practice the actual length of weld is made equal to the effective length shown on the drawing plus twice the weld size. The effective length of fillet weld should not be less than four times the size of the weld.

When the ends are returned as shown in Fig. 7.16, then the ends should be carried continuous around the corners for distance not less than twice the size of weld. This should be applied particularly to side and top fillet weld in tension.



## iv.Effective area

# E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### Course: B.tech.

## (Prepared By: Ms Kajal, lecturer, CE)

The effective area of a fillet weld is taken as the product of effective length and effective throat thickness.

# 7.5 WORKING STRESSES IN WELDS

Working stresses in welds, when welded joints are constructed with mild steel conforming to IS:226-1962 as parent metal and with electrodes conforming to IS:814-1974 are adopted as per Table 7.2 are recommended in IS:816-1969.

# **TABLE 7.2 WORKING STRESSES IN WELDS**

S1 .No.	Kind of stresses	Max. Permissible value (N/mm <sup>2</sup> )
1	Tension on section through throat of butt weld	142
2	Compression on section through throat of butt weld	142
3	Fiber stresses in bending (tension and compression)	157.5
4	Shear on section through throat of butt and fillet weld	110
5	Plug weld	110

The maximum permissible value of stresses of shear and tension are reduced to 80 per cent of those given in Table 7.2, in case, the welding is done at site. When the effects of wind or earthquake forces are considered, then, maximum permissible values of stresses are increased by 25 per cent. It is to note that maximum permissible stresses given in the Table 7.2 are same as for the parent metal (mild steel IS:226-1962).

# 7.6 DESIGN OF WELDED JOINTS SUBJECTED TO AXIAL LOAD

The complete penetration butt weld does not require design calculations. In case of incomplete penetration butt weld, effective throat thickness of the weld is computed and welding is done up to the required length. In case of fillet weld, size of the weld is fixed keeping in view the minimum size of the weld as per IS:816-1969 recommends that when filet weld is applied to the square edge of member, the maximum size of weld should be less than the edge thickness by at least 1.5 mm as shown in Fig. 7.17. This avoids the washing down of edges of weld.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

When fillet weld is applied to the round toe of rolled steel sections, the maximum size of the weld should not exceed <sup>3</sup>/<sub>4</sub> of the thickness of the section at the toe. When fillet weld is used for lap joint, then overlap of the members connected as shown in Fig. 7.17, should not be less than five times thickness of thinner part.



The strength of the fillet weld is determined per mm length for the size of the weld adopted. The effective length of the weld is then computed for the pull or thrust to be transmitted by the weld. In case, only side fillet welds are applied, the length of the each weld should not be less than perpendicular distance between them and spacing between them shall not be more than 16 times the thinner part.

**Example 7.1**. Two plates 16 mm thick are joined by i. a double U butt weld, ii. A single U butt weld. Determine the strength of the welded joint in tension in each case. Effective length of weld is 150 mm. Allowable stress in butt weld in tension is 142 N/mm<sup>2</sup>.

## Solution

i.In case of double U but weld, complete penetration of weld takes place

Effective throat thickness of weld = 16 mm

Effective length of weld = 150 mm

Strength of single U butt weld = throat thickness x length of weld x permissible shear stress

= (16 x 150 x 142/1000) = 340.8 kN

ii.In case of single U butt weld, incomplete penetration of butt weld takes place

Effective throat thickness  $= 5/8 \times 16 = 10 \text{ mm}$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Effective length of weld = 150 mm

Strength of single U butt weld  $= (10 \times 150 \times 142/1000) = 213.0 \text{ kN}$ 

**Example 7.2**. In a truss girder of a bridge, a tie as shown in Fig. 7.18 is connected to the gusset plate by fillet weld. Determine the strength of the weld. The size of the weld in the fillet weld is 6 mm.



## Solution

Size of weld	= 6 mm	
Effective throat thickness	= 0.7  x  6 = 4.2  mm	
Effective length of fillet weld	= 200 + 200 + 200	= 600 mm
Strength of fillet weld	= (4.2 x 600 x 110/10	(000) = 277.2  kN

**Example 7.3**. In Example 7.2, the pull to be transmitted by the tie is 300 kN. Determine the necessary overlap of the tie.

## Solution

Size of weld	= 6 mm
Effective throat thickness	= 4.2 mm

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Pull transmitted by the end fillet weld  $= (4.2 \times 200 \times 110/1000) = 92.4 \text{ kN}$ 

Let *l* be the necessary overlap required, the pull transmitted by the side fillet is

= (4.2 x 2 x l x 110/1000) = 0.924 l kN

Total pull transmitted = 92.4 + 0.924 l = 300 kN

Therefore, the necessary overlap of the tie l = 224.7 mm.

**Example 7.4**. The web plate of a built-up welded I-section is 200 mm x 12 mm and the flange plates are 100 mm x 12 mm. The size of fillet weld is 6 mm. Compute the maximum shear force that may be allowed at any section, if the average allowable shear in the web is 0.4  $f_y$  and maximum allowable shear in the weld is 110 N/mm<sup>2</sup>.

## Solution

Moment of inertia of the built-up section (about xx axis)

 $I_{xx} = 1/12[10 \text{ x } 22.4^3 - 8.8 \text{ x } 20^3] \text{ x } 10^4 = 3499.52 \text{ x } 10^4 \text{ mm}^4$ 

Intensity of shear stress (at weld section)

$$\tau_{z} = \left(\frac{F.A\bar{y}}{I_{xx},(2t)}\right)$$

Where,

 $A\overline{y}$  = Moment of the area above the section about xx axis

F = Shear force at the section

t = Effective throat thickness of one weld

$$\tau_{s} = \left(\frac{F \times 100 \times 12 \times (100 + 6)}{3499.52 \times 10^{4} \times 2 \times 0.7 \times 6}\right) = 0.11 \ kN/mm^{2}$$
$$F = 254.21 \ kN$$

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

The average shear stress in the web is 0.4 x 250 N/mm<sup>2</sup>. Allowable shear force in the web  $F_1=(200 \text{ x } 12 \text{ x } 0.4 \text{ x } 250/1000) = 240 \text{ kN}.$ 

The design drawing is Fig. 7.19.



**Example 7.5**. Design a suitable longitudinal fillet weld to connect the plates as shown in Fig. 7.20 and to transmit a pull equal to the full strength of thin plate. Allowable stress in the weld is  $110 \text{ N/mm}^2$  and tensile stress in the plate  $0.6 \text{ f}_y \text{ N/mm}^2$ . The plates are 10 mm thick.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## Solution

The minimum size of weld required for thickness up to 20 mm is 5 mm. The maximum size of fillet weld is limited by the thickness of the plate is (10-1.5)=8.5 mm. Provide 6 mm fillet weld.

Pull transmitted by 1 mm weld	$= (1 \times 0.7 \times 6 \times 110/1)$	1000)	= 0.462  kN
Tensile strength of thin plate	$= (120 \times 10 \times 0.6 \times 2)$	50/1000)	= 180 kN
Necessary length of the weld	= (180 / 0.462)	= 389.61 mm	

Provide 195 mm longitudinal weld on each side.

Check: Length of the weld 195 mm is greater than perpendicular distance 120 mm between welds

**Example 7.6**. Two plates 120 mm x 10 mm are overlapped and connected together by transverse fillet weld to transmit pull equal to full strength of the plate. Design the suitable welding. Allowable stress in the weld is 110 N/mm<sup>2</sup>. Allowable stress in tension in the plate is  $0.6 \text{ f}_{\text{y}} \text{ N/mm}^2$ .

### Solution

Minimum size of weld = 5mm

Maximum size of weld = (10-1.5) = 8.5 mm

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

Total length of two welds = 240 mm

Total load transmitted by 6 mm weld  $= (240 \times 0.7 \times 6 \times 110/1000) = 110.88 \text{ kN}$ 

Maximum pull that can be transmitted by the plate =  $(120 \times 10 \times 0.6 \times 250/1000) = 180 \text{ kN}$ 

To transmit the pull equal to the full strength of plate, provide additional weld by plug weld. Provide two rectangular plug welds 30 mm x 15 mm as shown in Fig. 7.21 which satisfies the specification.



Strength of two plug welds =  $(2 \times 30 \times 15 \times 110/1000)$  = 99 kN.

Total pull now transmitted = (110.88 + 99) = 209.88 kN > 180 kN. Hence satisfactory.

**Example 7.7**. A tie member consists of two MC 225, @ 0.250 kN/m. The channels are connected to either side of a gusset plate 12 mm thick. Design the welded joint to develop the full strength of the tie. The overlap limited to 400 mm.

## Solution

From ISI Handbook No. 1, for MC 225, @ 0.250 kN/m

Thickness of web = 6.4 mm

Thickness of flange = 12.4 mm

Sectional area  $= 3301 \text{ mm}^2$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Tensile strength of each channel section  $= (3301 \times 0.6 \times 250/1000) = 495.15 \text{ kN}$ Provide 4 mm weld Strength of weld per mm length  $= (1 \times 0.7 \times 4 \times 110/1000) = 0.308 \text{ kN}$ Total length of fillet weld necessary to connect one channel section = (495.15/0.308) = 1607.6 mm

The overlap of channel is limited to 400 mm. the width of slot should not be less than 3 times thickness ( $3 \times 6.4 = 19.2 \text{ mm}$ ). Provide two slots 20 mm side. The distance between edge of the slot and edge of channel or between adjacent slots also should not be less than twice the thickness (2x6.4=12.8 mm). Provide these distances as shown in Fig. 7.22. Let x be the length of the slot.

The total length of the weld = 800 + 225 + 4x - 2x20 = 1607.6 mm

Therefore, the length of the slot  $x = 155.65 \text{ mm} \approx 160 \text{ mm}$  long fillet welding is done as shown in Fig. 7.22.



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 8. Tension Member**

## 8.1 INTRODUCTION

A tension member is a member which carries mainly a tensile force in the direction parallel to its longitudinal axis. A tension member is also called as a tie member or simply a tie. In some cases tension member also subjected to bending either due to eccentricity of the longitudinal load or due to transverse loads acting in addition to the main longitudinal load. A tension member is one of the most commonly occurring types of structural members. Tension members may occur either as minor tension members such as bars, flats, rods etc. or as major tension members of roof and bridge trusses

## 8.2 MINOR TYPES OF TENSION MEMBERS



The minor types of tension members are shown in Fig. 8.1.

## i.Eye-bars

These members are used where flexible end connections are desired. They are used as the members of pin-connected truss bridges. Eye bars are made by first upsetting each end of a bar of rectangular section to a nearly round shape and then boring holes of the desired sizes on the enlarged ends. A pin is passed through the eye or the hole in the bar and also through

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

corresponding holes in the other members meeting at the joint. The pin provides means of transmission of load from the eye bar to the other members at the joint.

## ii.Loop bars

These are made by bending each end of a bar of square or round section, back upon the bar itself and then welding it so as to form a loop. Stress transmission is exactly similar to that in the eye-bar.

### iii.Threaded bars

These consist of round bars whose ends are threaded. Nuts are attached on the threaded ends after the bar has been placed in its proper position. The ends of the rod are first upset and then threaded so that the sectional area at the root of the threads is not less than the sectional area of the bar. After upsetting, usually the sectional area at the ends will be about 20 per cent greater than the sectional area of the bar. If a non-upset threaded bar is to be selected, the designer must select a bar in which the diameter at the root of the threads will be at least 1.5 mm greater than the normally required diameter.

## iv.Welded bars

These are flat bars carrying light tensile loads and welded at their ends.

# 8.3 MAJOR TYPES OF TENSION MEMBERS

Single angle tension members are commonly used in roof trusses carrying light loads. They are also used as bracings for members of composite section. A single angle member transfers its load eccentrically to the gusset plate and is hence also subjected to bending moment. This factor should also be taken into account in the design.

Double angle tension members are often used connected on either side of a gusset plate at the end. If provided in this manner eccentric load transfer to the gusset plate will be avoided and hence the member will be practically free from bending stresses. These are most commonly used in roof trusses and foot bridge trusses.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

Double channel tension members may also be used in a manner similar to double angle members. In view of considerably greater depth of web available two or even three rows of rivets can be provided. These members therefore require less length of gusset plate.

Besides the above two angle members, four angle members with or without a plate; two channel members may be used as tension members in more heavily loaded bridge trusses. The major types of tension members are shown in Fig. 8.2.



## 8.4 PERMISSIBLE TENSILE STRESS

For a tension member, it is necessary that the intensity of tensile stress on the net section of the member shall be less than the permissible limit. The I.S. specification has recommended the following permissible stresses.

The permissible stress in axial tension on the net effective area of the section shall not exceed  $0.6 \text{ f}_{y}$ , where  $f_{y}$  is the minimum yield stress of steel.

For example, if  $f_y=250 \text{ N/mm}^2$ , Safe stress in axial tension = 0.6 x 250 = 150 N/mm<sup>2</sup>.

## 8.5 NET SECTIONAL AREA

The maximum stress for a tension member occurs at the section where the area is a minimum. The net area for tension members should be determined as follows:

1. Threaded rods: The sectional area at the root of the threads is regarded of the net area.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

2. Riveted members: The net area at any section is equal to the gross area of the member at the section minus area of rivet holes at the section.

In making deduction for rivets and bolts less than 25 mm in diameter, the diameter of the hole shall be assumed to be 1.5 mm in excess of the nominal diameter of the rivet or bolt, unless specified otherwise. If the diameter of the rivet or bolt is greater than 25 mm the diameter of the hole shall be assumed to be 2 mm in excess of the nominal diameter of the rivet or bolt unless specified otherwise.

**Minimum net section:** When a number of holes are present in a tie member the minimum net section should be determined as follows:

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

( Prepared By: Ms Kajal, lecturer, CE)



E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,



(Prepared By: Ms Kajal, lecturer, CE)

Consider the plate shown in Fig 8.3 carrying a pull P and provided with three holes, B, C and D. If we consider the possibility of a failure along the section ABDE (Fig. 8.4), the net area at the section equals the gross area of the plate minus area of two rivet holes. There may be possibility of a failure along the section ABCDE (Fig. 8.5). This can occur when the staggered pitch p is within a certain limit. For a case like this the net effective width  $b_e$  can be determined by the following rule:

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

$$b_e = b - nd + n_z \left(\frac{p^2}{4g}\right)$$

Where, b = gross width of the plate

- n = number of rivet holes along any possible failure section
- n= number of zigzag lines along the line of failure
- p = staggered pitch, i.e. the distance between any two consecutive rivet measured parallel to the direction of stress in the member.
- g = the gauge, i.e. the distance between the same two consecutive rivets in a chain line measured at right angles to the direction of stress in the members.

For a failure along the section ABCDE, the effective net width

$$b_e = b - 3d + 2\left(\frac{p^2}{4g}\right)$$

Effective net area =  $b_{e.t}$ 

 $Maximum \ stress \ intensity \ in \ the \ plate = \frac{Load \ on \ the \ plate}{Minimum \ effective \ net \ area}$ 

## 8.6 NET EFFECTIVE SECTION FOR ANGLES AND TEES IN TENSION

The I.S 800 specification has stated the following in connection with the determination of the net effective section for angles and tees in tension.

## a. Singles angles:

In the case of single angles in tension connected by one leg only (Fig. 8.6),

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Cours Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



The net effective section of the angle =  $A_1 + A_2K$ 

Where,  $A_1$  = Effective cross sectional area of the connected leg,

 $A_2$  = The gross-sectional area of the unconnected leg, and

$$K = \frac{3A_1}{3A_1 + A_2}$$

## **b.** Double angles back to back (and single tees)

When only one leg of each angle is connected to the same side of a gusset (or when the flange of the tee is connected to the gusset) as shown in Fig. 8.7,

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



The net effective section  $= = A_1 + A_2 K$ 

Where,  $A_1 =$  Effective cross sectional area of the connected leg,

 $A_2$  = The gross-sectional area of the unconnected leg, and

$$K = \frac{5A_1}{5A_1 + A_2}$$

The angles shall be connected together along their lengths with tacking or stitch rivets at a spacing not exceeding 32 times the thickness or 300 mm whichever is less.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

- 1. When the double angles are connected to each side of a gusset, the area to be taken in computing the mean tensile stress shall be the full gross area minus the area of rivet holes. The angles shall also be connected together along their lengths with tacking rivets.
- 2. When the double angles are not tack riveted each angle shall be designed as a single angle connected through one leg only.

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 the area of the leg of a tee as the product of the thickness and the depth minus the thickness of the flange.

## 8.7 LUG ANGLES

To accommodate the necessary number of rivets for the connection of a member to a gusset plate a certain length of the member is utilized. When the load in the member is large many rivets may be required for the connection and the length of the member utilized for the connection itself may be large and may involve a considerable length of gusset plate. By using lug angles the length of the connection can be decreased.



# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

Fig 8.8 shows a lug angle used for the connection of an angle member to the gusset plate. Similarly for the connection of a channel member two lug angles may be used as shown in Fig. 8.9.



In the 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 per cent in excess of the force in the outstanding leg of the angle and the attachment of the lug angles to the angle member shall be capable of developing 40 per cent in excess of that force.

Lug angles connecting a channel shaped member shall, as for as possible, be deposed symmetrically with respect to the section of the member. 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 per cent 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 per cent in excess of that force.

In no case shall less than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 9. Design of Tension Member**

# 9.1 INTRODUCTION

When a tension member is subjected to axial tensile force, then the distribution of stress over the cross-section is uniform. The complete net area of a member is effectively used at the maximum permissible uniform stress. Therefore, a tensile member subjected to axial tensile force is used to be efficient and economical member. The procedure of the design of a tension member is explained below with help of example problems.

## 9.2 STEPS TO BE FOLLOWED IN THE DESIGN OF A TENSION MEMBER

The following steps may be followed in the design of axially loaded tension members.

- 1. Corresponding to the loading on the structure of which the tension member is a part, the tensile force in the member is first computed.
- 2. The net area required for the member is determined by dividing the tensile force in the member by the permissible tensile stress.
- **3**. Now, a suitable section having gross area about 20 per cent to 25 per cent greater than the estimated area is selected. For the member selected deductions are made for the area of rivet holes and the net effective area of the section is determined. If the net area of the section of the member so determined is greater than the net area requirement estimated in step i, the design is considered safe.
- 4. The slenderness ratio of a tension member shall not exceed 400. In the case of a tension member liable to reversal of stress due to the action of wind or earthquake, slenderness ratio shall not exceed 350. If the reversal of stress is due to loads others than wind or earthquake, the slenderness ratio shall not exceed 180.

**Example 9.1:** Determine the tensile strength of the 12 mm thick plate shown in Fig 9.1. Rivets used for the connection are 20 mm diameter. Allowable tensile stress is 150 N/mm<sup>2</sup>.

## Solution

Diameter of the rivet hole = 20 + 1.5 = 21.5 mm

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

The critical section to be considered is a section like ABCDE.

Effective width at critical section	$= b - nd = 180 - (3 \times 21.5) = 115.5 mm$		m
Effective net area	$= 115.5 \text{ x } 12 \text{ mm}^2$	$= 1386 \text{ mm}^2$	
Strength of plate	= 1386 x 150	= 207900 N	= 207.9 kN

**Example 9.2:** Find the strength of the 12 mm thick plate shown in Fig. 9.2. All the holes are 21.5 mm as gross diameter. Take  $f_t=150 \text{ N/mm}^2$ .



## Solution

Gross diameter of rivet hole = 21.5 mm

The effective net width will be computed along the various chain lines

Staggered pitch p = 40 mm

Gauge distance g = 50 mm

Net width corresponding to the chain ABCD  $= 210 - (2 \times 21.5) = 167 \text{ mm}$ 

Net width corresponding to the chain ABECFG

Course: B.tech,

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

$$= b - nd + n_z \frac{p^2}{4g} = 210 - (4 \times 21.5) + \left(3 \times \frac{40^2}{4 \times 50}\right) = 148 \text{ mm}$$

Net width corresponding to the chain ABEFG

$$= b - nd + n_x \frac{p^2}{4g} = 210 - (3 \times 21.5) + \left(1 \times \frac{40^2}{4 \times 50}\right) = 153.5 \text{ mm}$$

Net width corresponding to the chain ABECD

$$= b - nd + n_x \frac{p^2}{4g} = 210 - (3 \times 21.5) + \left(2 \times \frac{40^2}{4 \times 50}\right) = 161.5 \text{ mm}$$

Therefore minimum net width

= 148 mm

Safe load

= Safe stress x Area of minimum net effective section =  $150 \times 148 \times 12$  = 2,66,400 N = 266.4 kN.

**Example 9.3:** The tension member of a roof truss consist of two unequal angles  $70 \ge 45 \ge 8$  with the longer legs connected by 16 mm diameter rivets. Find the safe tension for the member, the angles being one on either side of the gusset plate.

## Solution

Gross area of 2 angles,	70 x 45 x 8	$= 1,712 \text{ mm}^2$
Area of 2 rivet holes $2 \times (16 + 1.5) \times 8$		$= 280 \text{ mm}^2$
Net area of the member	1712 - 280	$= 1,432 \text{ mm}^2$
Therefore safe tension for the	e member	= 150  x  1432 = 2,14,800  N = 214.8  kN.

**Example 9.4:** The tension member of a roof truss carries a maximum axial tension of 250 kN. Design the section. Diameter of connecting rivets = 20 mm. Safe stress in tension =  $150 \text{ N/mm}^2$ .

## Solution

Net area required  $A_{net} = \frac{250 \times 1000}{150} = 1667 \ mm^2$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Referring to steel tables, let us choose two angles of I.S.A. 75 x 50 x 10 mm thick.

Gross area =  $75 \text{ mm x } 50 \text{ mm x } 10 \text{ mm} = 2,300 \text{ mm}^2$ .

Let us assume that the angles are placed back to back on either side of the gusset plate (Fig. 9.3) with two rivets.



Diameter of rivet hole = 20 + 1.5 = 21.5 mm

Net area provided  $= 2,300 - (2 \times 21.5 \times 10) = 1,870 \text{ mm}^2$ .

But the area required is only 1,667 mm<sup>2</sup>. Hence the section is safe.

**Example 9.5:** The tie of a truss carries an axial tension of 225 kN. Design the section of the member and also the connection of the member to 10 mm thick gusset plate. Use 20 mm diameter rivets.

## Solution

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

Net area required  $A_{net} = \frac{225 \times 1000}{150} = 1500 \text{ mm}^2$ Referring to steel tables, let us choose two angles of I.S.A. 75 x 50 x 8 mm thick. Gross area = 75 mm x 50 mm x 8 mm = 1,872 mm<sup>2</sup>. Let us assume that the angles are placed back to back on either side of the gusset plate (Fig. 9.4).



Diameter of rivet hole = 20 + 1.5 = 21.5 mmNet area provided =  $1,872 - (2 \times 21.5 \times 8) = 1,528 \text{ mm}^2$ . But the area required is only  $1,500 \text{ mm}^2$ . Hence the section is safe. Strength of a rivet in double shear =  $\frac{2f_s\pi d^2}{4} = \frac{2 \times 100 \times \pi \times 21.5^2}{4} = 72,610 \text{ N}$ Strength of a rivet in bearing =  $f_b dt = 300 \times 21.5 \times 10 = 64,500 \text{ N}$ Number of rivets required for the connection =  $\frac{225 \times 10^3}{64500} \approx 4 \text{ rivets}$ 

**Example 9.6:** The tie in a bridge truss carries an axial tension of 350 kN. The member is to consist of two channels connected back to back on either side of a gusset plate. The diameter of rivets used for the connection is 16 mm. Two rivets are likely to appear in section. Design the member. Safe stress in tension is 150 N/mm<sup>2</sup>.

## Solution

Net area required

Referring to steel tables, let us select two channels ISLC, 125 (Fig. 9.5)

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

**Example 9.7:** The tension member of a roof truss consist of a single ISA 100 x 75 x 10 mm thick, connected at the end to a gusset plate with the longer leg vertical with 20 mm diameter rivet. Find the safe tension the member can withstand. Permissible tensile stress may be taken as  $150 \text{ N/mm}^2$ .

## Solution

Diameter of rivet hole = 20 + 1.5 = 21.5 mm

Net effective area provided

Where,  $A_1$  = Net sectional area of the connected leg,

 $A_2 = Area of the unconnected leg, and$ 

$$K = \frac{3A_1}{3A_1 + A_2}$$

In our case,

$$A_1 = \left(100 - \frac{10}{2}\right)10 - 21.5 \times 10 = 735 \ mm^2$$

$$A_2 = \left(75 - \frac{10}{2}\right)10 = 700 \ mm^2$$
$$K = \frac{3 \times 735}{3 \times 735 + 700} = 0.759$$

$$A_{eff} = 735 + 700 \times 0.759 = 1,266 \, mm^2$$

Safe axial tension  $= A_{eff} x$  Safe stress

The tension member is shown in Fig. 9.6.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



**Example 9.8:** Design a single angle tension member to sustain a tension of 1,30,000 N. Use 18 mm diameter rivets.

## Solution

The single angle tension member is shown in Fig. 9.7.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



Effective area required  $A_{eff} = \frac{1,30,000}{150} = 866.7 \ mm^2$ Approximate gross area required  $= A_{eff} + 30 \ \%$  more  $= 866.7 + (0.3 \ x \ 866.7) = 1,127 \ mm^2$ Let us try single angle 90 mm x 60 mm x 8 mm,

Let us ity single angle 90 min x 00 min x 8 min,

 Net area of the connected leg
  $A_1 = (90-4) \ge 8 - (19.5 \ge 8)$   $= 532 \ \text{mm}^2$  

 Area of the outstanding leg
  $A_2 = (60-4) \ge 8$   $= 448 \ \text{mm}^2$ 

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 532}{(3 \times 532) + 448} = 0.78$$

Therefore, available effective area =  $A_{eff} = A_1 + A_2K = 532 + 0.78 \times 448 = 881.44 mm^2$ Safe tension for the member = 150 x 881.44 = 1,32,216 N.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

But actual tension in the member is only 1,30,000 N. Hence the design is safe.

**Example 9.9:** A tension member consist of two angles  $60 \ge 60 \ge 8$  the angles being placed back to back on the same side of the gusset plate. One leg of each angle is connected to the gusset plate. The outstanding legs are also connected by tack rivets. Find the safe tension for the member. Rivets are 16 mm in diameter.

## Solution

The tension member is shown in Fig. 9.8.



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Net area of the legs connected to the gusset plate  $A_1 = 2\left[\left(60 - \frac{8}{2}\right)8 - 17.5 \times 8\right] = 616 \ mm^2$  $A_2 = 2\left[\left(60 - \frac{8}{2}\right)8\right] = 896 \ mm^2$ 

Net area of the outstanding legs,

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \times 616}{(5 \times 616) + 896} = 0.775$$
$$A_{eff} = A_1 + A_2 K = 616 + 0.775 \times 896 = 1310.4 \ mm^2$$

Therefore, safe tension for the member  $= 150 \times 1310.4$  = 1.96,560 N = 196.56 kN.

Example 9.10: A tension member of a truss consists of a single ISA 125 x 75 x 10 mm carrying a load of 200 kN. If 20 mm diameter rivets be used, design the connection to the gusset plate using a lug angle. Take f<sub>s</sub>=100 N/mm<sup>2</sup> and f<sub>b</sub>=300 N/mm<sup>2</sup>.

#### Solution

Finished diameter of rivet = 20 + 1.5 = 21.5 mmStrength of one rivet in single shear  $= f_s \frac{\pi d^2}{4} = 100 \times \frac{\pi 21.5^2}{4} = 36,305 N$ Strength one rivet in bearing =  $f_b dt$  = 300 x 21.5 x 10 = 64,500 N Rivet value = 36,305 N Therefore number of rivets required  $=\frac{200 \times 1,000}{36,305} \approx 6 \ rivets$ Pitch of rivets =  $3 \times \text{diameter of rivets}$  =  $3 \times 20 = 60 \text{ mm}$ 

This being large the length of the connection can be decreased by using a lug angle,

Let the longer leg be the connected leg,

Area of the connected leg  $= (125-5)10 = 1,200 \text{ mm}^2$ 

Area of the outstanding leg  $= (75-5)10 = 700 \text{ mm}^2$ 

Therefore load taken by the connected leg

Therefore load taken by the outstanding leg = 200 - 126.3 = 73.7 kN

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

Connection to the gusset plate using a lug angle is shown in Fig. 9.9.



The lug angle should have a strength equal to 20 % greater than the strength of the outstanding leg.

Therefore strength required for the lug angle =  $1.2 \times 73.7 \text{ kN}$  = 88.44 kN

Ne area required for the lug angle  $= (88.44 \times 1000/150) = 589.6 \text{ mm}^2$ 

Let us try 1 lug angle 125 x 75 x 6 mm

Gross area  $= 1,166 \text{ mm}^2$ 

Net area = gross area – area of one rivet hole =  $1,166 - (21.5 \text{ x } 6) = 1,037 \text{ mm}^2$ 

Number of rivets required for connecting the 125 mm leg of the lug angle with the gusset plate

$$= \frac{Load \text{ in } lug \text{ angle}}{Least \text{ rivet } value}$$
  
Rivet value in bearing on 6 mm plate = 300 x 21.5 x 6 = 38,700 N  
Rivet value in single shear = 36,305 N  
Number of rivets for connecting the lug angle to the gusset plate =  $\frac{88.44 \times 1000}{36,305} \approx 3$   
We will therefore provide 3 rivets for connecting the 125 mm leg of the lug angle to the gusset plate  
plate. Number of rivets required to connect the 125 mm leg of the main angle to the gusset plate

$$=\frac{126.3\times1000}{36,305}\approx4\,rivets$$

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

Strength required for connection between lug angle and the main angle is equal to 1.4 times the strength of the outstanding leg.

Therefore, strength required for this connection  $= 1.4 \times 73.7 = 103.18 \text{ kN}$ 

Number of rivets required =  $(103.18 \times 1000/36, 305) \approx 3$ 

## **LESSON 10. Design of Columns**

## **10.1 INTRODUCTION**

A column is defined as a structural member subjected to compressive force in a direction parallel to its longitudinal axis. The term stanchions and posts are also used for columns. In truss bridge girders, end compression members are termed as end posts. Columns are commonly classified as short and long columns. This classification is arbitrary and there is no absolute way to determine the exact limits for each classification.

## **10.2 AXIALLY LOADED COLUMNS**

In an axially loaded column, the load is applied at the centroid of the section and in a direction parallel to the longitudinal axis of the column. The terms centrally loaded and concentrically loaded are also used for axially loaded columns. An axially loaded column as defined by the structural engineers transmits a compressive force without an explicit design requirement to carry lateral loads or end moments.

An ideal column is assumed initially to be perfectly straight and is centrally loaded. Consider a case of a slender ideal column. The column is vertically fixed at the base and free at the upper end and subjected o an axial load P as shown in Fig. 10.1. The column is assumed to be perfectly elastic. When the value of load P is less than critical load and stress is within the limit of proportionality, the column remains straight. The column is in stable equilibrium. If a small lateral load is applied at the free end, the column defect. On withdrawal of the lateral load, the
## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

column resumes its vertical position and deflection vanishes. When the axial load P gradually increased, a stage will be reached when the vertical position of the column is in the unstable equilibrium. If a small lateral load is applied, a deflection will be produced, which will not vanish on withdrawal of lateral load.

The axial load which is sufficient to keep the column in such a slight deflected shape is called critical load. Critical load is also called a buckling load or crippling load. The buckling load is defined as the load at which a member or a structure as a whole collapses in service (or buckles in load test). The buckling is defined as the sudden bending, warping, curling or crumbling of the elements or members under compressive stresses. The direction of buckling of a column depends upon flexural rigidity EI, of the column. It buckles in a direction perpendicular to the axis about which the moment of inertia of the section is minimum.



E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

In about 1759, a Swiss mathematician Prof. Leonhard Euler derived the most popular column formula. The critical load for the column as shown in Fig 10.1 was determined as follows.

The differential equation of the deflected shape of the column is

$$EI\frac{d^2y}{dx^2} = +M \ (Hogging \ moment + ve)$$

The bending moment at any point on the deflected shape  $M = +P(\delta - y)$ 

Therefore

$$EI\frac{d^2y}{dx^2} = P(\delta - y)$$
$$\frac{d^2y}{dx^2} - \frac{P}{EI}(\delta - y) = 0$$

Let  $n = (\frac{p}{r})^{1/2}$ , Then the differential equation becomes,

$$\frac{d^2y}{dx^2} + n^2y - n^2\delta = 0$$

The general solution of this equation is  $y = A \cdot \sin(nx) + B \cdot \cos(nx) + \delta$ 

At 
$$\left(x = 0, y = 0, \frac{dy}{dx} = 0\right)$$
,  $B = -\delta$ ,  $A = 0$ ,  
 $y = \delta(1 - \cos nx)$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

At x = l,  $y = \delta$  This condition is satisfied when  $\delta \cos(nl) = 0$ 

From this, either  $\delta = 0$  or nl = 0

If  $\delta$  is zero, buckling of column does not occur, if cos nl=0, then

$$nl = \frac{\pi}{2}(2\pi - 1)$$
 where  $n = 1, 2, 3, ...$ 

For n=1, the values of nl is smallest. It is equal to  $\pi/2$ 

Therefore

$$P_{cr} = \left(\frac{\pi^2 \text{EI}}{4l^2}\right)$$

The values of critical loads for other end conditions can be determined from this case. For a column, hinged at both ends,

$$P_{cr} = \left(\frac{\pi^2 \mathrm{EI}}{l^2}\right)$$

Where Pa=Critical load (or Buckling load).

Since each half of the column is in the same position as the whole of the column. This is called fundamental case of buckling of a bar.

## **10.3 EFFECTIVE LENGTH OF COMPRESSION MEMBER**

The effective length of a compression member depends upon end restraint conditions. The end restraint conditions are of two types as given below:

- 1. Position restraint
- 2. Direction restraint

10.3.1 **Position restraint** 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

In position restraint, end of the column is not free to change its position but rotation about the end of the column can take place e.g., hinged end of column as shown in Fig. 10.2.A

## **10.3.2 Direction restraint**

In direction restraint, end of the column is free to change its position but rotation about the end of the column cannot take place.

When an end of a column is having restraint in position and direction both, then end is not free to change its position and the rotation about the end of the column also cannot take place as shown in Fig. 10.2.B



There are various possible combinations of restraints about either or both axes. The restraint conditions at the two ends of a column may be different or may be same. Following are the ideal cases of the end conditions:

1. Both ends of column hinged

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

- 2. Both ends of column fixed
- 3. One end of column fixed and the other end hinged
- 4. One end of column fixed and the other end free.

The deflected shapes of columns under critical loads have been shown in Fig. 10.3. A, B, C and D respectively. The actual lengths have been indicated by 'L'. Case one, both ends of the column hinged have been considered as standard case. The effective length (l) of a column is expressed in terms of equivalent length of compression members, hinged at both ends. It is the length of column between two adjacent points of zero moments and is represented by 'l'. It is also called as unsupported length.



The ideal end conditions cannot be achieved in actual practice. The effective length of a compression member is adopted as per Table 10.1 as recommended by BIS in IS:800-1984 for different types of compression members. The effective length as given in this table will be adequate in most of the cases and the same may also be adopted where the column directly form part of the frame structures.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

## Table 10.1 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

S1. No.	Туре	Effective length of member (1)
1	Effectively held in position and restrained against rotation at both ends [Fig. 10.3.B]	0.65 L
2	Effectively held in position at both ends and restrained against rotation at one ends [Fig. 10.3.C]	0.80 L
3	Effectively held in position at both end but not restrained against rotation [Fig. 10.3.A]	1.00 L
4	Effectively held in position and restrained against rotation at one end and at the other end restrained against rotation but not held in position [Fig. 10.4.A]	1.00 L
5	Effectively held in position and restrained against rotation at one end and at the other end partially restrained against rotation but not held in position. [Fig. 10.4.B]	1.50 L
6	Effectively held in position at one end but not restrained against rotation, at the other end restrained against rotation but not held in position. [Fig. 10.4.C]	2.00 L
7	Effectively held in position and restrained against rotation at one end but not held in position [Fig. 10.3.D] or restrained against rotation at the other end.	2.00 L

Note: For battened struts, the effective length l is increased by 10 per cent.

Where the exact frame analysis is not done, the effective length of columns in the frame structures may be found from the ratio of effective length to the unsupported length (l/L) from Fig. 10.5 when the relative displacements of the column is prevented (i.e. when there is no sway) and from Fig. 10.6 when the relative lateral displacement of the ends is not prevented (i.e. without restraint against sway viz., the sway occurs), when sway occurs, IS:800-1984 recommends that the effective length ratio, (l/L) may not be taken to be less than 1.2.

In Fig. 10.5 and Fig. 10.6,

# E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

$$\beta_{1} = \left(\frac{\sum k_{c}}{\sum k_{c} + \sum k_{b}}\right) \qquad \text{So also} \quad \beta_{2} = \left(\frac{\sum k_{c}}{\sum k_{c} + \sum k_{b}}\right)$$

Where, the summation is to be done for the members framing into a joint at top and bottom respectively, and

 $k_c$  = flexural stiffness of the column and

 $k_b$  = flexural stiffness of the beam

Fig. 10.5 and Fig. 10.6 are from the paper titled as Effective lengths of columns in multistory buildings by Professor R.H.Wood, published in the structural Engineer Vol. 52, No. 7 July 1974. It is worthwhile to note that IS;800-1984 'code of practice for general construction in steel' and IS:456-1978 'code of practice for plain and reinforced concrete' have recommended the same effective lengths for the columns with similar support conditions.

## **10.4 EFFECTIVE SECTIONAL AREA**

The gross cross-sectional area is the area as calculated from the specified size of the member or part thereof. The effective sectional area of a compression member is the gross cross-sectional area of the member. The deduction is not made for members connected by rivets, bolts and pins. If the holes are not filled by the fastening material, then deduction is made for unfilled holes. The effective area is however modified when the ratio of the outstand to thickness exceeds the limits specified by BIS. The deduction is also made from the gross cross-sectional area for excessive effective plate width (if any) to determine the effective sectional area.

## **10.5 RADIUS OF GYRATION**

The radius of gyration of a section is a geometrical property of the section and it is denoted by r

 $r = (I/A)^{1/2}$ 

where I=moment of inertia of the section about the axis.

r=radius of gyration of the section about the axis

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

A=effective sectional area of the section.

## 10.6 SLENDERNESS RATIO OF COMPRESSION MEMBER

The slenderness ratio of a compression member is defined as ratio of effective length of compression member (l) to appropriate radius of gyration (r)

## Slenderness ratio, $\lambda = 1/r$

The radii of gyration about various axes of rolled steel sections can be obtained from structural steel section tables (ISI Handbook No.1). The minimum radius of gyration is used for computing the maximum slenderness ratio. For built-up compression members, value of radius of gyration is calculated. The slenderness ratio for a compression member should be as small as possible so that the material may be stressed to its greatest possible limit. The maximum slenderness ratio of compression members should not exceed the value given in Table 10.2. These limits have been laid down by BIS in IS:800-1984.

The end restraints of columns are often different in the two principal planes. The different moments of inertia of the column cross-section in these planes are sometimes desirable to achieve approximately equal slenderness ratios. The intermediate supports are provided to the columns for this purpose.

Table 10.2 MAXIMUM SLENDERNESS RATIO OF COMPRESSION MEMBER ( $\lambda = 1/r$ )

S1. No.	Type of member	λ=1/r
1	A member carrying comprehensive loads resulting from dead loads and imposed loads	
2	A member subjected to compression forces resulting from wind/earthquake forces provided that the deformation of such members 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 subjected to possible reversal of stress resulting from the action of wind or earthquake forces	350

The intermediate supports reduce the unsupported length of the columns. When the unsupported lengths of columns are reduced then the smaller sections may be used at a higher average stress. Sometimes, the intermediate supports are furnished only in one direction, for

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

example, a rolled steel I-section column is having its continuous length up to two storeys. At the level of one storey, intermediate support is provided by connecting beams with the web. The radius of gyration,  $r_{yy}$  of the section, about yy-axis (axis parallel to the web) is much smaller than the radius of gyration  $r_{xx}$  of the section, about xx-axis.

By providing the intermediate support, the effective length of the column become different in two different directions. The effective length of column,  $l_{yy}$  for bending about yy-axis is found by considering the length of column between one-storey only. The effective length of column  $l_{xx}$  for bending about xx-axis is found by considering the length of column between two storeys. It is seen that the effective length of column  $l_{yy}$  is much smaller than that of  $l_{xx}$ . The values of slenderness ratio ( $l_{yy}/r_{yy}$ ) and ( $l_{xx}/r_{xx}$ ) about two directions may be made approximately equal. As such the use of sections with different values of radii of gyration in two directions may be made economical. When a column is subjected to different bending moments in two directions then, the greater value of r may be kept in the direction of greater moment. The intermediate supports in the weak direction make the use of I-section and channel section economical.

## 10.7 COLUMN FORMULAE FOR AXIAL STRESS IN COMPRESSION

The strength of a column depends upon large number of variables. The efforts are made to obtain a design formula by fitting a curve to experimentally found buckling loads for the intermediate range of the slenderness ratio. It is tried to draw a curve which may merge with the Euler hyperbola in the very slender column range on one side and with the material yield strength for the zero length on the other side.

A perfectly straight column of perfectly homogeneous material (i.e. an ideal column) is subjected to an axial load. The primary object is to find the average axial stress in compression, which corresponds to the allowable load. The average axial stress is uniform across the section. It is given by

$$\sigma_{ac} = (P_a/A)$$

Where  $P_a$  = allowable load and A = cross-sectional area of the column

The required cross-sectional area for a given design load may be found conveniently in case  $\sigma_c$  is known

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

 $A_{reqd} = (P/\sigma_c)$ 

Where P is the design load

## 10.8 DESIGN OF AXIALLY LOADED COMPRESSION MEMBER

When a column or compression member is designed, for given load, actual length of the member and its support conditions, the cross-sectional shape of the member is determined. The cross-sectional shape of axially loaded compression member depends largely on whether the compression member is long or short and whether it carries a small load or a large load. It is difficult to decide, whether a column is short or long. It is arbitrarily decided.

When the slenderness ratio of a column is less than 60, it may be considered as a short column. When the slenderness ratio is between 60 and 180, the column may be considered as long column. Following are the length and load categories arbitrarily made for design of compression members:

- 1. Short compression members with small loads
- 2. Short compression members with large loads
- 3. Long compression members with small loads
- 4. Long compression members with intermediate load.

The strength of axially loaded compression member depends upon slenderness ratio  $(l/r_{min})$ . For the design of axially loaded compression member load to be carried, the length of compression member and end conditions are known. The effective length of the compression member for the given end conditions is computed. The radius of gyration of compression member is not known as the cross-sectional shape of the compression member is not known. The allowable working stress in compression can be found when the slenderness ratio is known. There is no direct method of designing a compression member. The compression member is designed by trial and error method. The design of compression member is also done by using safe load tables, if available.

ISI handbook No.1 provides tables for safe concentric loads on rolled steel column sections (HB-sections) for bending about xx-axis and yy-axis. The effective length of column is determined knowing the end conditions. The values of safe concentric loads corresponding to respective effective lengths are given for various sizes of HB-sections. A column section

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

having safe axial load equal to or slightly greater than the required load on the column is selected.

**Design procedure**: Following are the usual steps in design of compression members.

**Step 1**. The slenderness ratio for the compression member and the value of yield stress for the steel are assumed. For the rolled steel beam section compression members, the slenderness ratio varies from 70 to 90. For struts, the slenderness ratio varies from 110 to 130. For compression members carrying large loads, the slenderness ratio is about 40.

Step 2. The effective sectional area (A) required for compression member is determined.

 $A = (P/\sigma_c)$ , Where P is the design load to be carried by the member

**Step 3.** From the steel section tables, section for the compression member of the required area is selected. The section for the compression member is selected such that it has the largest possible radius of gyration for the required sectional area. It should also be most economical section.

**Step 4.** Knowing the geometrical properties of the section slenderness ratio is computed and allowable axial stress in compression is found from IS:800-1984 for the quality of steel assumed.

Step 5. The safe load carrying capacity of the compression member is determined.

The section selected for the compression member is revised in case the safe load carrying of the compression member is less than or much larger than the load to be carried by it.

**Example 10.1** A rolled steel beam section HB 350 @0.674 kN/m is used as a stanchion. If the unsupported length of the stanchion is 4 m, determine safe load carrying capacity of the section.

## Solution:

Step 1: Properties of I-section

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

HB 350 @ 0.674 kN/m section is used as a stanchion. From the steel tables, the geometrical properties of the section are as follows:

Sectional area  $A = 8591 \text{ mm}^2$ 

Radius of gyration  $r_{xx} = 149.3 \text{ mm}$ 

Radius of gyration  $r_{yy} = 53.4 \text{ mm}$ 

Step 2: Slenderness ratio

Minimum radius of gyration  $r_{min}$ = 53.4 mm

Unsupported length l=4 m

Slenderness ratio of the stanchion  $\frac{l}{r_{min}} = \left(\frac{4 \times 1000}{53.4}\right) \approx 75$ 

Step 3: Safe load

From IS:800-1984 for 1/r=75 and the steel having yield stress,  $f_{N}=260 \text{ N/mm}^2$ , allowable working stress in compression  $\sigma_{ac}=109 \text{ N/mm}^2$  (MPa)

The safe load carrying capacity of the stanchion  $P = (\sigma_{ac}A) = \left(\frac{109 \times 8591}{1000}\right) = 936.42 \ kN$ 

Example 10.2 In Example 10.1, in case standard column section SC 25°, @85.6 kg/m is used as a column, determine the safe load carrying capacity of the section. Solution:

Step 1: Properties of I-section

From IS:808-1964, the geometrical properties of the section are as follows:

Sectional area  $A = 109 \times 10^2 \text{ mm}^2$ 

Radius of gyration  $r_{xx} = 107 \text{ mm}$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Co Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Radius of gyration  $r_{yy} = 54.6 \text{ mm}$ 

Step 2: Slenderness ratio

Minimum radius of gyration  $r_{min}$  = 54.6 mm

Unsupported length l=4 m = 4000 mm

Sienderness ratio of the stanchion  $\frac{l}{r_{min}} = \left(\frac{4 \times 1000}{54.6}\right) = 73.26$ 

Step 3: Safe load

From IS:800-1984 for 1/r=73.26 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression

 $\sigma_{ac} = (115 - (12/10 \text{ x } 3.26)) = 111.088 \text{ N/mm}^2 (\text{MPa})$ 

The safe load carrying capacity of the stanchion  $P = (\sigma_{ac}A) = \left(\frac{111.088 \times 109 \times 100}{1000}\right) = 1210.86 \ kN$ 

**Example 10.3** A built-up column consist of three rolled steel beam sections WB 450 @0.794 kN/m, connected effectively to act as one column as shown in Fig. 10.7. Determine the safe load carrying capacity of built-up section, if unsupport length of column is 4.25 m.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



## Solution:

Step 1: Properties of built-up section

The built-up column consist of 3 WB 450 @0.794 kN/m

From steel section tables, area of section  $=(3 \times 101.15 \times 100) = 30345 \text{ mm}^2$ 

Moment of inertia of built-up section about xx-axis

 $I_{xx} = [2 x 35057.6 + 1706.7] x 10^4 = 71821 x 10^4 mm^4$ 

Moment of inertia of built-up section about yy-axis

 $I_{yy} = [35057.6 \text{ x } 10^4 + 2 \text{ x } 1706.7 \text{ x } 10^4 + 2\text{Ah}^2] = 38471 \text{ x } 10^4 + 2\text{Ah}^2 \text{ mm}^4$ 

Where A=sectional area of one I-section

 $I_{yy} = [38471.0 + 2 \ x \ 101.15(22.5 + \frac{1}{2} \ x 0.92)^2 \ ] \ x \ 10^4 \ mm^4$ 

 $= 145115.79 \text{ x } 10^4 \text{ mm}^4$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Step 2: Slenderness ratio

 $I_{xx}$  is less than  $I_{yy}$ . Therefore  $r_{xx}$  is Minimum.

Minimum radius of gyration  $r_{min} = \left(\frac{71821.9 \times 10^4}{30345}\right)^{1/2} = 154 \ mm$ Unsupported length  $l = 4.25 \ m = 4250 \ mm$ Slenderness ratio of the column  $\frac{l}{r_{min}} = \left(\frac{4.25 \times 1000}{154.0}\right) = 27.6$ 

Step 3: Safe load From IS:<u>800-1984</u> for 1/r=27.6 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression

 $g_{ac} = 151.72 \text{ N/mm}^2 \text{ (MPa)}$ 

Safe load carrying capacity of built-up section  $P = (\sigma_{ac}A) = \left(\frac{151.72 \times 303.45}{1000}\right) = 46.04 \ kN$ 

**Example 10.4** Design a rolled steel beam section column to carry an axial load 1100 kN. The column is 4 m long and adequately restrained in position but not in direction at both ends:

**Design:** The slenderness ratio for the column and the value of yield stress for the steel to be used may be assumed as 80 and 260 N/mm<sup>2</sup> respectively.

Step 1. Selection of trial section

Allowable stress as per IS:800-1984,  $\sigma_{ac} = 103 \text{ N/mm}^2$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Effective sectional area required =  $\left(\frac{1100 \times 1000}{103}\right) = 10679.61 \, mm^2$ 

Effective length of the column is 4 m

Step 2: Properties of trial section From steel section tables, try HB 450,@0.872 kN/m section Sectional area A=11114 mm<sup>2</sup>, r<sub>xx</sub>=187.8 mm, r<sub>yy</sub>=51.8 mm

Step 3. Slenderness ratio

Therefore rmin=51.8 mm

Slenderness ratio  $\frac{l}{r_{min}} = \left(\frac{4.0 \times 1000}{51.8}\right) = 77.2$ 

Step 4. Check for safe load From IS:800-1984 for 1/r=77.2 and the steel having yield stress,  $f_y=260 \text{ N/mm}^2$ , allowable working stress in compression  $\sigma_{ac}=106.36 \text{ N/mm}^2$  (MPa) Safe load carrying capacity of the column  $P = (\sigma_{ac}A) = \left(\frac{106.36 \times 11114}{1000}\right) = 1182.08 \text{ kN}$ 

The column section lighter in weight than this is not suitable. Hence the design is satisfactory.

### Alternatively:

Step 2: Properties of trial section From IS:808-1984, try SC 250,@85.6 kg/m (standard column section) A= 109 x 100 mm<sup>2</sup>, r<sub>xx</sub>=107.0 mm, r<sub>vv</sub>=54.6 mm

Step 3: Slenderness ratio

## E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Effective length of column is 4000 mm. Minimum radius of gyration,  $r_{min}=54.6$  mm. Therefore, Slenderness ratio  $\frac{l}{r_{min}} = \left(\frac{4000}{54.6}\right) = 73.26$ 

Step 4. Check for safe load

From IS:800-1984 for 1/r=73.26 and the steel having yield stress, fy=260 N/mm<sup>2</sup>, allowable working stress in compression  $\sigma_{ac} = \left(115 - \frac{12}{10} \times 3.26\right) = 111.088 \text{ N/mm}^2$  (MPa) Safe load carrying capacity of the column  $P = (\sigma_{ac}A) = \left(\frac{111.088 \times 109 \times 100}{1000}\right) = 1210.86 \text{ kN}$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Example 10.5 A column 5 m long is to support a load 4500 kN. The ends of the column are effectively held in position and direction. Design the column if rolled steel beams and 18 mm plates are only available.

Design:

Step 1: Selection of trial section

Length of column L=5m

Effective length of column is (0.65 x 5) = 3.25 m

In order to support large load, the slenderness ratio for the built-up column and the value of yield stress for the steel may be assumed as 40 and 260 N/mm<sup>2</sup>, respectively.

Allowable working stress in compression  $\sigma_{ac} = 145 \text{ N/mm}^2 \text{ (MPa)}$ 

Effective sectional area required =  $\left(\frac{4500\times1000}{145}\right) = 31034.48 \ mm^2$ 

Step 2: Properties of trial section

From steel section tables, try HB 450, @0.925 kN/m section,  $r_{xx}=185.0$  mm,  $r_{yy}=50.8$  mm

For the columns carrying large loads,  $r_{xx} = r_{yy}$ 

Step 3. Slenderness ratio Therefore  $r_{min}$  for the plated built-up column may be estimated as 185 mm Slenderness ratio  $\frac{l}{r_{min}} = \left(\frac{3.25 \times 1000}{185}\right) = 17.567$ 

## Step 4. Area of plates

From IS:800-1984 for l/r=17.567 and the steel having yield stress,  $f_y=260 \text{ N/mm}^2$ , allowable working stress in compression  $\sigma_{ac}=154.486 \text{ N/mm}^2$  (MPa)

Sectional area required =  $\left(\frac{4500 \times 1000}{154.486}\right) = 29128.85 \ mm^2$ 

Area of HB 450,@0.925 kN/m is 11789 mm<sup>2</sup>

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Area to be provided by two cover plates	$= 17339.74 \text{ mm}^2$
Area to be provided by one plate	$= 8669.87 \text{ mm}^2$
Plates available	= 18 mm
Width required	= (8669.87/18) =481.66 mm

Provide 700 mm width of cover plate

Step 5: Check for outstanding width; thickness ratio for cover plate

Outstanding width  $= \frac{1}{2} (700-140) = \frac{560}{2} = 280 \text{ mm}$ 

Thickness = 18 mm

$$\left(\frac{Outstanding \ width}{Thickness}\right) = \frac{280}{18} = 15.55 < 16 \ which is satisfactory$$

Step 6: Check for load carrying capacity

1. Properties of section		
I <sub>yy</sub> of plates	$= 2 \text{ x} (1/12) \text{ x } 1.8 \text{ x } 70^3 \text{ x } 10^4 \text{ mm}^4$	$= 102900 \text{ x } 10^4 \text{ mm}^4$
Iyy of H-section	$= 3045 \text{ x } 10^4 \text{ mm}^4$	
Iyy of compound section	$= 105945 \text{ x } 10^4 \text{ mm}^4$	
Area of section	$= (117.89 + 2 x 1.8 x 100 mm^2 = 36)$	59.89 x 100 mm <sup>2</sup>
	2212	

$$r_{yy} = \left(\frac{105945 \times 10^4}{369.9 \times 100}\right)^{1/2} = 169.2 \, mm$$

rxx=185 mm (for I-section alone)

2. Slenderness ratio of column section

$$\frac{l}{r_{min}} = \left(\frac{3.25 \times 1000}{169.2}\right) = 19.21$$

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

From IS:800-1984 for l/r=19.21 and the steel having yield stress,  $f_y=260 \text{ N/mm}^2$ , allowable working stress in compression  $\sigma_{ac}=154.158 \text{ N/mm}^2$  (MPa)

Safe load carrying capacity  $P = (\sigma_{ac}A) = \left(\frac{154.158 \times 36989}{1000}\right) = 5702.15 \ kN$ 

Hence, the design is satisfactory. Provide HB 450,@0.925 kN/m with two plates 700 mm x 18 mm. One plate is connected with each flange of I-section. The design drawing is given in Fig. 10.8



Step 4. Area of plates

From IS:800-1984 for l/r=17.567 and the steel having yield stress,  $f_y=260 \text{ N/mm}^2$ , allowable working stress in compression  $\sigma_{ac}=154.486 \text{ N/mm}^2$  (MPa)

Sectional area required =  $\left(\frac{4500 \times 1000}{154.486}\right) = 29128.85 \ mm^2$ 

Area of HB 450,@0.925 kN/m is 11789 mm<sup>2</sup>

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Area to be provided by two cover plates	$= 17339.74 \text{ mm}^2$
Area to be provided by one plate	$= 8669.87 \text{ mm}^2$
Plates available	= 18 mm
Width required	= (8669.87/18) =481.66 mm

Provide 700 mm width of cover plate

Step 5: Check for outstanding width; thickness ratio for cover plate

Outstanding width  $= \frac{1}{2} (700-140) = \frac{560}{2} = 280 \text{ mm}$ 

Thickness = 18 mm

$$\left(\frac{Outstanding \ width}{Thickness}\right) = \frac{280}{18} = 15.55 < 16 \ which is satisfactory$$

Step 6: Check for load carrying capacity

1. Properties of sec	tion		
Iyy of plates	= 2 x (2	1/12) x 1.8 x 70 <sup>3</sup> x 10 <sup>4</sup> mm <sup>4</sup>	$= 102900 \text{ x } 10^4 \text{ mm}^4$
Iyy of H-section	= 3045	x 10 <sup>4</sup> mm <sup>4</sup>	
Iyy of compound sect	ion	$= 105945 \text{ x } 10^4 \text{ mm}^4$	
Area of section		=(117.89 + 2 x 1.8 x 100 mi	$m^2 = 369.89 \text{ x} 100 \text{ mm}^2$

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 11. Design of Compression Members**

## **11.1 INTRODUCTION**

A strut is defined as a structural member subjected to compression in a direction parallel to its longitudinal axis. The term strut is commonly used for compression members in roof trusses. A strut may be used in a vertical position or in an inclined position in roof trusses. The compression members may be subjected to both axial compression and bending.

When compression members are overloaded then their failure may take place because of one of the following:

- 1. Direct compression
- 2. Excessive bending
- **3**. Bending combined with twisting

The failure of column depends upon its slenderness ratio. The load required to cause above mentioned failures decreases as the length of compression member increases, the cross-sectional area of the member being constant.

## 11.2 COMMON SECTIONS OF COMPRESSION MEMBERS

The common sections used for compression members are shown in Fig. 11.1 with their approximate radii of gyration. A column or a compression member may be made

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,



(Prepared By: Ms Kajal, lecturer, CE)

of many different sections to support a given

load. Few sections satisfy practical requirement in a given case. A tubular section is most efficient and economical for the column free to buckle in any direction. The radius of gyration r for the tubular section in all the directions remains same. The tubular section has high local buckling strength. The tubular sections are suitable for medium loads. However, it is difficult to have their end connections. A solid round bar having a cross-sectional area equal to that of a tubular section has radius of gyration, r much smaller than that of tube. The solid round bar is less economical than the tubular section. The solid round bar is better than the thin rectangular section or a flat strip. The radius of gyration of flat strip about its narrow direction is very small. Theoretically, the rods and bars do resist some compression. When the length of structural member is about 3 m, then the compressive strengths of the rods and bars are very small.

Single angle sections are rarely used except in light roof trusses, because of eccentricity at the end connections. Tee-sections are often used in roof trusses. The single rolled steel I-section and single rolled steel channel section are seldom used as column. The value of radius of gyration r, about the axis parallel to the web is small. The intermediate additional supports in

## E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

the weak direction make the use of these sections economical. Sometimes the use of I-sections and channel sections are preferred because of the method of rolling at the mills, since, the outto-out dimensions remain same for a given depth. This failure is not there with other rolled steel sections. The costs of single rolled steel sections per unit weight are less than those of built-up sections. Therefore the single rolled steel sections are preferred so long as their use is feasible.

## 11.3 STRENGTH OF COMPRESSION MEMBERS

The strength of a compression member is defined as its safe load carrying capacity. The strength of a centrally loaded straight steel column depends on the effective cross-sectional area, radius of gyration (viz., shape of the cross-section), the effective length, the magnitude and distribution of residual stresses, annealing, out of straightness and cold straightening. The effective cross-sectional area and the slenderness ratio of the compression members are the main features, which influence its strength. In case, the allowable stress is assumed to vary parabolically with the slenderness ratio, it may be proved that the efficiency of a shape of a compression member is related to  $A/r^2$ . The efficiency of a shape is defined as the ratio of the allowable load for a given slenderness ratio to that for slenderness ratio equal to zero. The safe load carrying capacity of compression member of known sectional area may be determined as follows:

Step 1. From the actual length of the compression member and the support conditions of the member, which are known, the effective length of the member is computed.

Step 2. From the radius of gyration about various axes of the section given in section tables, the minimum radius of gyration  $(r_{min})$  is taken.  $r_{min}$  for a built up section is calculated.

Step 3. The maximum slenderness ratio  $(l/r_{min})$  is determined for the compression member.

Step 4. The allowable working stress ( $\sigma_{ac}$ ) in the direction of compression is found corresponding to the maximum slenderness ratio of the column from IS:800-1984.

Step 5. The effective sectional area (A) of the member is noted from structural steel section tables. For the built up members it can be calculated.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

Step 6. The safe load carrying capacity of the member is determined as  $P=(\sigma_{ac}.A)$ , where P=safe load

## **11.4 ANGLE STRUTS**

The compression members consisting of single sections are of two types:

- 1. Discontinuous members
- 2. Continuous members

## 11.4.1 **Continuous members**

The compression members (consisting of single or double angles) which are continuous over a number of joints are known as continuous members. The top chord members of truss girders and principal rafters of roof trusses are continuous members. The effective length of such compression members is adopted between 0.7 and 1.0 times the distance between the centres of intersections, depending upon degree of restraint provided. When the members of trusses buckle in the plane perpendicular to the plane of the truss, the effective length shall be taken as 1.0 times the distance between the points of restraint. The working stresses for such compression members is adopted from IS:800-1984 corresponding to the slenderness ratio of the member and yield stress for steel.

## 11.4.2 **Discontinuous members**

The compression members which are not continuous over a number of joints, i.e., which extend between two adjacent joints only are known as discontinuous members. The discontinuous members may consist of single angle strut or double angle strut. When an angle strut is connected to a gusset plate or to any structural member by one leg, the load transmitted through the strut, is eccentric on the section of the strut. As a result of this, bending stress is developed along with direct stress. While designing or determining strength of an angle strut, the bending stress developed because of eccentricity of loading is accounted for as follows:

## i.Single angle strut

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



- 1. When single angle discontinuous strut is connected to a gusset plate with one rivet as shown in Fig. 11.2.A, its effective length is adopted as centre to centre of intersection at each end and the allowable working stress corresponding to the slenderness ratio of the member is reduced to 80 per cent. However, the slenderness ratio of such single angle strut should not exceed 180.
- 2. When a single angle discontinuous strut is connected with two or more number of rivets or welding as shown in Fig. 11.2.B, its effective length is adopted as 0.85 times the length of strut centre to centre of intersection of each end and allowable working stress corresponding to the slenderness ratio of the member is not reduced.



### ii.Double angle strut

 A double angle discontinuous strut with angles placed back to back and connected to both sides of a gusset or any rolled steel section by not less than two rivets or bolts or in line along the angles at each end or by equivalent in welding as shown in Fig. 11.3.A, can be regarded as an axially loaded strut. Its effective length is adopted as 0.85 times the distance between intersections, depending on the degree of restraint provided and in the plane perpendicular to that of the gusset, the effective length 'l'

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

shall be taken as equal to the distance between centres of the intersections. The tacking rivets should be provided at appropriate pitch.

2. The double angles, back to back connected to one side of a gusset plate or a section by one or more rivets or bolts or welds as show in Fig. 11.3.B, these are designed as single angle discontinuous strut connected by single rivet or bolt.

If the struts carry in addition to axial loads, loads which cause transverse bending, the combined bending and axial stress shall be checked as described for the columns subjected to eccentric loading. The tacking rivets should be provided at appropriate pitch.

The tacking rivets are also termed as stitching rivets. In case of compression members, when the maximum distance between centres of two adjacent rivets exceeds 12 t to 200 mm whichever is less, then tacking rivets are used. The tacking rivets are not subjected to calculated stress. The tacking rivets are provided throughout the length of a compression member composed of two components back to back. The two components of a member act together as one piece by providing tacking rivets at a pitch in line not exceeding 600 mm and such that minimum slenderness ratio of each member between the connections is not greater than 40 or 0.6 times the maximum slenderness ratio of the strut as a whole, whichever is less.

In case where plates are used, the tacking rivets are provided at 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 weather the pitch in line shall not exceed 16 times the thickness of the 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.

The single angle sections are used for the compression members for small trusses and bracing. The equal angle sections are more desirable usually. The unequal angle sections are also used. The minimum radius of gyration about one of the principal axis is adopted for calculating the slenderness ratios. The minimum radius of gyration of the single angle section is much less than the other sections of same cross-sectional area. Therefore, the single angle sections are not suitable for the compression member of long lengths. The single angle sections are commonly used in the single plane trusses (i.e., the trusses having gusset plates in one plane). The angle sections simplify the end connections.

## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

The tee-sections are suitable for the compression members for small trusses. The tee-sections are more suitable for welding.

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

### Solution:

Step 1: Properties of angle section

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

Sectional area	$A = 3459 \text{ mm}^2$	
Radius of gyration	$r_{xx} = r_{yy} = 149.3 \text{ mm}$	
Radius of gyration	$r_{uu}$ = 58.3 mm, $r_{vv}$ =29.3 mm	
Step 2: Slenderness ratio,		
Minimum radius of gyration r <sub>min</sub> = 29.3 mm		
Effective length of strut l= 3.5 m		
Slenderness ratio of the strut		

Step 3: Safe load

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

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

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

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

The safe load carrying capacity

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

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

## Solution:

## **Step 1: Properties of angle section**

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

Sectional area  $A = 4300 \text{ mm}^2$ 

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

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

## Step 2: Slenderness ratio,

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

Effective length of strut l= 3.5 m

 $\frac{l}{r_{min}} = \left(\frac{3.5 \times 1000}{29.3}\right) = 119.5$ 

Slenderness ratio of the strut

## Step 3: Safe load

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

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

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

$$P = (\sigma_{ac}A) = \left(\frac{51.56 \times 4300}{1000}\right) = 221.708 \, kN$$

The safe load carrying capacity

## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

**Example 11.3** In Example 11.1, if single angle discontinuous strut is connected with more than two rivets in line along the angle at each end, calculate the safe load carrying capacity of the section.

## Solution:

## **Step 1: Properties of angle section**

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

Sectional area	$A = 3459 \text{ mm}^{2}$

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

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

Length of strut between centre to centre of intersection L=3.50 m

## Step 2: Slenderness ratio,

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

Effective length of discontinuous strut double riveted

0.85 x L= 0.85 x 3.5 = 2.975 m

$$\frac{l}{r_{min}} = \left(\frac{2.975 \times 1000}{29.3}\right) = 101.5$$

Slenderness ratio of the strut

## Step 3: Safe load

From IS:800-1984 for l/r=101.5 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression  $\sigma_{ac}=71.65$  N/mm<sup>2</sup> (MPa)

Allowable working stress for discontinuous strut double riveted is not reduced.

$$P = (\sigma_{ac}A) = \left(\frac{71.65 \times 3459}{1000}\right) = 247.84 \, kN$$

The safe load carrying capacity

## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

**Example 11.4** A double angle discontinuous strut ISA 125 mm x 95 mm x 10 mm (ISA 125 95,@0.165 kN/m) long legs back to back is connected to both the sides of a gusset plate 10 mm thick with 2 rivets. The length of strut between centre to centre of intersections is 4 m. Determine the safe load carrying capacity of the section.

### Solution:

### **Step 1: Properties of angle section**

The double angle discontinuous strut 2 ISA 125 mm x 95 mm x 10 mm (ISA 125 95,@0.165 kN/m) is shown in Fig. 11.4. Assume the tacking rivets are used along the length. From the steel tables, the geometrical properties of (two angle back to back) the sections are as follows:

Radius of gyration  $r_{xx} = 39.4 \text{ mm}$ 

Angles are 10 mm apart

Radius of gyration  $r_{yy} = 40.1 \text{ mm}$ 

Length of strut between centre to centre of intersection L=4 m

### Step 2: Slenderness ratio,

Minimum radius of gyration  $r_{min}$ = 39.4 mm

Effective length of discontinuous strut 0.85 x L = 0.85 x 4.0 = 3.40 m

$$\frac{l}{r_{min}} = \left(\frac{3.4 \times 1000}{39.4}\right) = 86.3$$

Slenderness ratio of the strut

### Step 3: Safe load

From IS:800-1984 for l/r=86.3 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression  $\sigma_{ac}=95.96$  N/mm<sup>2</sup> (MPa)

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

 $P = (\sigma_{ac}A) = \left(\frac{95.96 \times 4204}{1000}\right) = 403.416 \, kN$ 

The safe load carrying capacity

**Example 11.5** In Example 11.4, if double discontinuous strut is connected to one side of a gusset, determine safe load carrying capacity of the strut.

## Solution:

### **Step 1: Properties of angle section**

The double angle discontinuous strut 2 ISA 125 mm x 95 mm x 10 mm (ISA 125 95,@0.165 kN/m) connected to one side of a gusset is shown in Fig. 11.5. Assume the tacking rivets are used along the length. From the steel tables, the geometrical properties of (two angle back to back) the sections are as follows:



Sectional area  $A = 4204 \text{ mm}^2$ 

Radius of gyration  $r_{xx}= 39.4 \text{ mm}$ 

Distance back to back of angles is zero

Radius of gyration  $r_{yy}= 36.7 \text{ mm}$ 

Effective length of strut whether single riveted or double riveted L=4 m

Step 2: Slenderness ratio,

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Minimum radius of gyration  $r_{min}$  = 36.7 mm

$$\frac{l}{r_{min}} = \left(\frac{4 \times 1000}{36.7}\right) = 109$$

Slenderness ratio of the strut

## Step 3: Safe load

From IS:800-1984 for l/r=109 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression  $\sigma_{ac}=73.9$  N/mm<sup>2</sup> (MPa)

For above strut, allowable working stress  $0.80 \sigma_{ac} = (0.80 \text{ x } 73.9) = 59.12 \text{ N/mm}^2$ .

$$P = (\sigma_{ac}A) = \left(\frac{59.12 \times 4204}{1000}\right) = 248.54 \ kN$$

The safe load carrying capacity

**Example 11.6** In Example 11.4, double angle strut is continuous and connected with a gusset plate with single rivet; determine safe load carrying capacity of the strut.

### Solution:

### **Step 1: Properties of angle section**

The double angle discontinuous strut 2 ISA 125 mm x 95 mm x 10 mm (ISA 125 95,@0.165 kN/m) is singly riveted as shown in Fig. 11.4. Assume the tacking rivets are used along the length. From the steel tables, the geometrical properties of (two angle back to back) the sections are as follows:

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



Sectional area  $A = 4204 \text{ mm}^2$ 

Radius of gyration  $r_{xx} = 39.4 \text{ mm}$ 

Angles are 10 mm apart

Radius of gyration  $r_{yy} = 40.1 \text{ mm}$ 

Length of strut between centre to centre of intersection L=4 m

# Step 2: Slenderness ratio,

Minimum radius of gyration  $r_{min}$ = 39.4 mm

Effective length L=4 m

$$\frac{l}{r_{min}} = \left(\frac{4 \times 1000}{39.4}\right) = 101.5$$

Slenderness ratio of the strut

**Step 3: Safe load** 

## E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

From IS:800-1984 for l/r=101.5 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression  $\sigma_{ac}=71.65$  N/mm<sup>2</sup> (MPa)

$$P = (\sigma_{ac}A) = \left(\frac{71.65 \times 4204}{1000}\right) = 301.22 \ kN$$

The safe load carrying capacity

**Example 11.7** Design a single angle discontinuous strut to carry 110 kN load. The length of the strut between centre to centre of intersections is 3.25 m.

Design:

## **Step 1: Selection of trial section**

Assuming that the angle strut is connected to the gusset plate with two or more than two rivets.

Effective length of strut  $l=0.85L=(0.85 \times 3.25 \times 1000) = 2762.5 \text{ mm}.$ 

The slenderness ratio for the single angle discontinuous strut and value of yield stress for the steel may be assumed as 130 and 260  $N/mm^2$ , respectively.

Therefore, allowable stress in compression for strut  $\sigma_{ac}$  =57 N/mm<sup>2</sup> (MPa)

$$=\left(\frac{110\times1000}{57}\right)=1929.82\ mm^2$$

Effective sectional area required

The equal angle section is suitable for single angle strut. It has maximum value for minimum radius of gyration.

## **Step 2: Properties of trial section**

From steel section tables, try ISA 110 mm x 110 mm x 10 mm (ISA 110 110@0.165 kN/m)

Sectional area A=2106 mm<sup>2</sup>,  $r_{xx}=r_{yy}=33.6$  mm,  $r_{uu}=42.5$  mm,  $r_{vv}=21.4$  mm

Therefore  $r_{min}=21.4$  mm

**Step 3: Slenderness ratio**
E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

$$\frac{l}{r_{min}} = \left(\frac{2762.5}{21.4}\right) = 129.09$$

Slenderness ratio

# Step 4: Safe load

From IS:800-1984, allowable working stress in compression for the steel having yield stress as  $260 \text{ N/mm}^2$   $\sigma_{ac} = 57.56 \text{ N/mm}^2$  (MPa)

$$P = (\sigma_{ac}A) = \left(\frac{57.56 \times 2106}{1000}\right) = 121.22 \ kN$$

The safe load carrying capacity

The angle section lighter in weight than this is not suitable. Hence the design is satisfactory.

# Step 5: Check for width of outstanding leg

$$P = (\sigma_{ac}A) = \left(\frac{57.56 \times 2106}{1000}\right) = 121.22 \ kN$$

Width of outstanding leg to thickness ratio

Hence, satisfactory. Provide ISA 110 mm x 110 mm x 10 mm (ISA 110 110@0.165 kN/m) for discontinuous strut.

# Alternatively:

## **Step 2: Properties of trial section**

From IS:808-1984, try angle section 120 x 120 x 10 (@ 18.2 kg/m)

Sectional area, A=2320 mm<sup>2</sup>,  $r_{xx}=r_{yy}=36.7$  mm,  $r_{uu}=46.3$  mm,  $r_{vv}=23.6$  mm

# Step3: Slenderness ratio

Effective length of strut is 2762.5 mm

Minimum radius of gyration  $r_{min}=23.6 \text{ mm}$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

Slenderness ratio  $\frac{l}{r_{min}} = \left(\frac{2762.5}{23.6}\right) = 117.055$ 

## **Step 4: Safe load carrying capacity**

From IS:800-1984 for l/r=117.055 and the steel having yield stress,  $f_y=260$  N/mm<sup>2</sup>, allowable working stress in compression

$$\sigma_{ac} = \left(73 - \frac{9}{10} \times 7.55\right) = 66.205 \text{ N/mm}^2 \text{ (MPa)}$$

$$P = (\sigma_{ac}A) = \left(\frac{66.205 \times 2320}{1000}\right) = 153.596 \, kN$$

Safe load carrying capacity

The angle section lighter in weight than this is not suitable. Hence the design is satisfactory.

**Example 11.8** Design a double angle discontinuous strut to carry 150 kN load. The length of strut between centre to centre of intersections is 4 m

Design:

# **Step 1: Selection of trial section**

Assuming that the strut is connected to both sides of gusset 10 mm thick by two or more than two rivets.

Length of strut L=4.00 m

Effective length of strut l=0.85L=(0.85 x 4) = 3.40 m.

The slenderness ratio of a double angle discontinuous strut and the value of yield stress for the steel may be assumed as 120 and 260 N/mm<sup>2</sup>, respectively.

Therefore, allowable stress in compression  $\sigma_{ac} = 64 \text{ N/mm}^2$  (MPa)

$$=\left(\frac{150\times1000}{64}\right)=2343.75\ mm^2$$

Effective sectional area required

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

## **Step 2: Properties of trial section**

From steel section tables (properties of two angles back to back), try 2 ISA 100 mm x 65 mm x 8 mm (2 ISA 100 65,@0.099 kN/m)

Sectional area A=2514 mm<sup>2</sup>,  $r_{xx}$ =31.6 mm,

For angles having 10 mm distance back to back and long legs vertical  $r_{yy}=27.5$  mm

Therefore  $r_{min}=27.5 \text{ mm}$ 

# **Step 3: Slenderness ratio**

$$\frac{l}{r_{min}} = \left(\frac{3.40 \times 1000}{27.5}\right) = 123.6$$

Slenderness ratio

## Step 4: Safe load

From IS:800-1984, allowable working stress in compression for the steel having yield stress as  $260 \text{ N/mm}^2$   $\sigma_{ac} = 61.48 \text{ N/mm}^2$  (MPa)

The safe load carrying capacity

$$P = (\sigma_{ac}A) = \left(\frac{61.48 \times 2541}{1000}\right) = 156.22 \ kN$$

The angle section lighter in weight than this is not suitable. Hence the design is satisfactory. Provide 2 ISA 100 mm x 65 mm x 8 mm for the strut. Provide tacking rivets 18 mm in diameter at 500 mm spacing.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 12. Design of Colum Bases-Slab Base**

# **12.1 INTRODUCTION**

The columns are supported on the column bases. The column bases transmit the column load to the concrete or masonry foundation blocks. The column load is spread over large area on concrete or masonry blocks. The intensity of bearing pressure on concrete or masonry is kept within the maximum permissible bearing pressure. The safety of the structure depends upon stability of foundation. The column bases should be designed with utmost care and skill. In the column bases, intensity of pressure on concrete block is assumed to be uniform. The column bases shall be of adequate strength, stiffness and area to spread the load upon the concrete, masonry, other foundation or other supports without exceeding the allowable stress on such foundation under any combination of the load and bending moments. The column bases are of two types;

- 1. Slab base, and
- 2. Gusseted bases

The column footings are designed to sustain the applied loads, moments and forces and the induced reactions. The column load is spread over large area, so that the intensity of bearing pressure between the column footing and soil does not exceed the safe bearing capacity of the soil. it is ensured that any settlement which may occur shall be as nearly uniform as possible and limited to an accepted small amount. The column load is first transmitted to the column footing through the column base. It is then spread over the soil through the column footing. The column footings are of two types;

- 1. Independent footings, and
- 2. Combined footings.

# 12.2 SLAB BASE

The slab base as shown in Fig. 12.1 consists of cleat angles and base plate. The column end is faced for bearing over the whole area. The gussets (gusset plates and gusset angles) are not provided with the column with slab bases. The sufficient fastenings are used to retain the parts securely in plate and to resist all moments and forces, other than the direct compression. The forces and moments arising during transit,

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Con Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

unloading and erection are also considered. When the slab alone distributes the load uniformly the minimum thickness of a rectangular slab is derived as below;



The column is carrying an axial load P. Consider the load distributed over area h x w and under the slab over the area L x D as shown in Fig. 12.2.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### (Prepared By: Ms Kajal, lecturer, CE)

Let t=Thickness of the slab

W=Pressure or loading on the underside of the base

a=Greater projection beyond column

b=Lesser projection beyond column

 $\sigma_{bs}$ = Allowable bending stress in the slab bases for all steels, it shall be assumed as 185 N/mm<sup>2</sup> Consider a strip of unit width.

Along the xx-axis

$$M_{xx} = \left(\frac{wa^2}{2}\right)$$

 $M_{yy} = \left(\frac{wb^2}{2}\right)$ 

Along the yy-axis

If Poison ratio is adopted as <sup>1</sup>/<sub>4</sub> the effective moment for width D

$$=\frac{w}{2}\left(a^2-\frac{b^2}{4}\right)$$

 $=\frac{w}{2}\left(b^2-\frac{a^2}{4}\right)$ 

Effective moment for width L

Since a is greater projection from the column, the effective moment for width D is more. Moment of resistance of the slab base of unit width

 $M.R = \left(\frac{1}{6} \times 1 \times t^2 \times \sigma_{bs}\right)$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

Therefore  $\left(\frac{1}{6} \times 1 \times t^2 \times \sigma_{bs}\right) = \frac{w}{2} \left(a^2 - \frac{b^2}{4}\right)$ 

Hence the thickness of the slab base  $t = \left[\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)\right]^{1/2}$  (as per IS: 800-1984)

For solid round steel column, where the load is distributed over the whole area, the minimum thickness of square base as per IS:800-1984 is given by

$$t = 10 \left[ \frac{90w}{16\sigma_{bs}} \left( \frac{B}{B - d_0} \right) \right]^{1/2}$$

Where t= Thickness of plate in mm

W=Total axial load in kN

B=Length of the side of base of cap in mm

d<sub>0</sub>=Diameter of the reduced end (if any) of the column in mm

 $\sigma_{bs}$ = Allowable bending stress in steel (is adopted as 185 N/mm<sup>2</sup>)

The allowable intensity of pressure on concrete may be assumed as  $4 \text{ N/mm}^2$ . When the slab does not distribute the load uniformly or where the slab is not rectangular, separate calculation shall be made to show that stresses are within the specified limits.

When the load on the cap or under the base is not uniformly distributed or where end of the column shaft is not machined with the cap or base, or where the cap or base is not square in plan, the calculations are made on the allowable stress of 185 N/mm<sup>2</sup> (MPa). The cap or base plate shall not be less than 1.50 ( $d_0$ +75) mm in length or diameter.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Cour Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

The area of the shoulder (the annular bearing area) shall be sufficient to limit the stress in bearing, for the whole of the load communicated to the slab to the maximum value 0.75  $f_y$  and resistance to any bending communicated to the shaft by the slab shall be taken as assisted by bearing pressures developed against the reduced and of the shaft in conjunction with the shoulder.

The bases foe bearing upon concrete or masonry need not be machined on the underside provided the reduced end of the shaft terminate short of the surface of the slab and in all cases the area of the reduced end shall be neglected in calculating the bearing pressure from the base.

In cases where the cap or base is fillet welded direct to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to resist transmitting the forces specified above. Where the full length T butt welds are provided no machining of contact surfaces shall be required.

**Example 12.1** A column section HB 250, @ 0.510 kN/m carries an axial load of 600 kN. Design a slab for the column. The allowable bearing pressure on concrete is  $4 \text{ N/mm}^2$ . The allowable bending stress in the slab base is  $185 \text{ N/mm}^2$ (MPa).

Design:

# Step 1: Area of slab base required

Axial load of column = 600 kN

It is assumed uniformly distributed under the slab

$$= \left(\frac{600\times1000}{4}\right) = 15\times10^4 mm^2$$

Area of the slab base required

The length and width of slab base are proportioned so that projections on either side beyond the column are approximately equal.

Size of column section HB 250,@ 0.510 kN/mm is 250 mm x 25 mm

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

Area of slab base =(250+2a)(250+2b) mm<sup>2</sup>

# **Step 2: Projections of base plate**

Let projections a and b are equal

Area of slab  $(250+2a)^2 = 15 \times 10^4$ . Therefore a=68.45 mm

Provide projections a=b=70 mm

Provide slab base (250 + 2 x 70) (250 + 2 x 70) = 390 mm x 390 mm

Area of slab provided =  $390 \times 390 = 1,52,100 \text{ mm}^2$ 

Intensity of pressure from concrete under slab

$$w = \left(\frac{600 \times 1000}{152100}\right) = 3.945 \, N/mm^2$$

## **Step 3: Thickness of slab base:**

Thickness of slab  $= \left[\frac{3 \times 3.945}{195} \left(70^2 - \frac{70^2}{4}\right)\right]^{1/2} = 15.33 \, mm$ 

Provide 16 mm thick slab base. The fastenings are provided to keep the column in position.

**Example 12.2** A column section SC 250, @ 85.6 carries an axial load of 600 kN. Design a slab base for the column. The allowable bearing pressure on concrete is 4 N/mm<sup>2</sup>. The allowable bending stress in the slab base is 185 N/mm<sup>2</sup>(MPa).

Design:

# Step 1: Area of slab base required

Axial load of column is 600 kN. It is assumed uniformly distributed under the slab.

Area of slab base required  $=\left(\frac{600\times1000}{4}\right) = 15\times10^4 mm^2$ 

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

The length and width of slab base are proportioned so that the projections on either side beyond the column are approximately equal.

Size of column section SC 250, @ 85.6 kg/m = 250 mm x 250 mm

Area of slab base =  $(250 + 2a)(250+2b) \text{ mm}^2$ 

# **Step 2: Projections of base plate**

Let the projections a and b be equal.

Area of slab  $(250+2a)^2 = 15 \times 10^4$ . Therefore a=68.45 mm

Provide projections a=b=70 mm

Provide slab base  $(250 + 2 \times 70) (250 + 2 \times 70) = 390 \text{ mm} \times 390 \text{ mm}$ 

Area of slab provided =  $390 \times 390 = 1,52,100 \text{ mm}^2$ 

Intensity of pressure from concrete under slab

# Step 3: Thickness of slab base:

$$= \left[\frac{3 \times 3.945}{185} \left(70^2 - \frac{70^2}{4}\right)\right]^{1/2} = 15.33 \ mm$$

Thickness of slab

Provide 16 mm thick slab base. The fastenings are provided to keep the column in position.

# **LESSON 12. Design of Colum Bases-Slab Base**

## 12.1 INTRODUCTION

The columns are supported on the column bases. The column bases transmit the column load to the concrete or masonry foundation blocks. The column load is spread over large area on concrete or masonry blocks. The intensity of bearing pressure on concrete or masonry is kept within the maximum permissible bearing pressure. The safety of the structure depends upon stability of foundation. The column bases should be designed with utmost care and skill. In the column bases, intensity of pressure on concrete block is assumed to be uniform. The column

# E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

bases shall be of adequate strength, stiffness and area to spread the load upon the concrete, masonry, other foundation or other supports without exceeding the allowable stress on such foundation under any combination of the load and bending moments. The column bases are of two types;

- 1. Slab base, and
- 2. Gusseted bases

The column footings are designed to sustain the applied loads, moments and forces and the induced reactions. The column load is spread over large area, so that the intensity of bearing pressure between the column footing and soil does not exceed the safe bearing capacity of the soil. it is ensured that any settlement which may occur shall be as nearly uniform as possible and limited to an accepted small amount. The column load is first transmitted to the column footing through the column base. It is then spread over the soil through the column footing. The column footings are of two types;

- 1. Independent footings, and
- 2. Combined footings.

# 12.2 SLAB BASE

The slab base as shown in Fig. 12.1 consists of cleat angles and base plate. The column end is faced for bearing over the whole area. The gussets (gusset plates and gusset angles) are not provided with the column with slab bases. The sufficient fastenings are used to retain the parts securely in plate and to resist all moments and forces, other than the direct compression. The forces and moments arising during transit, unloading and erection are also considered. When the slab alone distributes the load uniformly the minimum thickness of a rectangular slab is derived as below;

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



The column is carrying an axial load P. Consider the load distributed over area h x w and under the slab over the area L x D as shown in Fig. 12.2.

Let t=Thickness of the slab

W=Pressure or loading on the underside of the base

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

a=Greater projection beyond column

b=Lesser projection beyond column

 $\sigma_{bs}$ = Allowable bending stress in the slab bases for all steels, it shall be assumed as 185 N/mm<sup>2</sup>

Consider a strip of unit width.

Along the xx-axis

$$M_{xx} = \left(\frac{wa^2}{2}\right)$$

 $M_{yy} = \left(\frac{wb^2}{2}\right)$ 

Along the yy-axis

If Poison ratio is adopted as <sup>1</sup>/<sub>4</sub> the effective moment for width D  $= \frac{w}{2} \left( a^2 - \frac{b^2}{4} \right)$ 

Effective moment for width L

Since a is greater projection from the column, the effective moment for width D is more. Moment of resistance of the slab base of unit width

$$M.R = \left(\frac{1}{6} \times 1 \times t^2 \times \sigma_{bs}\right)$$

Therefore  $\left(\frac{1}{6} \times 1 \times t^2 \times \sigma_{bs}\right) = \frac{w}{2} \left(a^2 - \frac{b^2}{4}\right)$ 

Hence the thickness of the slab base  $t = \left[\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)\right]^{1/2}$  (as per IS: 800-1984)

For solid round steel column, where the load is distributed over the whole area, the minimum thickness of square base as per IS:800-1984 is given by

$$t = 10 \left[ \frac{90w}{16\sigma_{bs}} \left( \frac{B}{B - d_0} \right) \right]^{1/2}$$

$$=\frac{w}{2}\left(b^2-\frac{a^2}{4}\right)$$

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

Where t= Thickness of plate in mm

W=Total axial load in kN

B=Length of the side of base of cap in mm

d<sub>0</sub>=Diameter of the reduced end (if any) of the column in mm

 $\sigma_{bs}$ = Allowable bending stress in steel (is adopted as 185 N/mm<sup>2</sup>)

The allowable intensity of pressure on concrete may be assumed as 4 N/mm<sup>2</sup>. When the slab does not distribute the load uniformly or where the slab is not rectangular, separate calculation shall be made to show that stresses are within the specified limits.

When the load on the cap or under the base is not uniformly distributed or where end of the column shaft is not machined with the cap or base, or where the cap or base is not square in plan, the calculations are made on the allowable stress of 185 N/mm<sup>2</sup> (MPa). The cap or base plate shall not be less than 1.50 ( $d_0$ +75) mm in length or diameter.

The area of the shoulder (the annular bearing area) shall be sufficient to limit the stress in bearing, for the whole of the load communicated to the slab to the maximum value 0.75  $f_y$  and resistance to any bending communicated to the shaft by the slab shall be taken as assisted by bearing pressures developed against the reduced and of the shaft in conjunction with the shoulder.

The bases foe bearing upon concrete or masonry need not be machined on the underside provided the reduced end of the shaft terminate short of the surface of the slab and in all cases the area of the reduced end shall be neglected in calculating the bearing pressure from the base.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

(Prepared By: Ms Kajal, lecturer, CE)

In cases where the cap or base is fillet welded direct to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to resist transmitting the forces specified above. Where the full length T butt welds are provided no machining of contact surfaces shall be required.

**Example 12.1** A column section HB 250, @ 0.510 kN/m carries an axial load of 600 kN. Design a slab for the column. The allowable bearing pressure on concrete is  $4 \text{ N/mm}^2$ . The allowable bending stress in the slab base is  $185 \text{ N/mm}^2$ (MPa).

Design:

# Step 1: Area of slab base required

Axial load of column = 600 kN

Area of the slab base required

It is assumed uniformly distributed under the slab

$$= \left(\frac{600\times1000}{4}\right) = 15\times10^4 mm^2$$

The length and width of slab base are proportioned so that projections on either side beyond the column are approximately equal.

Size of column section HB 250,@ 0.510 kN/mm is 250 mm x 25 mm

Area of slab base =(250+2a)(250+2b) mm<sup>2</sup>

# **Step 2: Projections of base plate**

Let projections a and b are equal

Area of slab  $(250+2a)^2 = 15 \times 10^4$ . Therefore a=68.45 mm

Provide projections a=b=70 mm

Provide slab base  $(250 + 2 \times 70) (250 + 2 \times 70) = 390 \text{ mm} \times 390 \text{ mm}$ 

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

Area of slab provided =  $390 \times 390 = 1,52,100 \text{ mm}^2$ 

Intensity of pressure from concrete under slab

$$w = \left(\frac{600 \times 1000}{152100}\right) = 3.945 \, N/mm^2$$

## Step 3: Thickness of slab base:

Thickness of slab

$$= \left[\frac{3 \times 3.945}{185} \left(70^2 - \frac{70^2}{4}\right)\right]^{1/2} = 15.33 \, mm$$

Provide 16 mm thick slab base. The fastenings are provided to keep the column in position.

**Example 12.2** A column section SC 250, @ 85.6 carries an axial load of 600 kN. Design a slab base for the column. The allowable bearing pressure on concrete is 4 N/mm<sup>2</sup>. The allowable bending stress in the slab base is 185 N/mm<sup>2</sup>(MPa).

Design:

## Step 1: Area of slab base required

Axial load of column is 600 kN. It is assumed uniformly distributed under the slab.

Area of slab base required

$$=\left(\frac{600\times1000}{4}\right)=15\times10^4 mm^2$$

The length and width of slab base are proportioned so that the projections on either side beyond the column are approximately equal.

Size of column section SC 250, @ 85.6 kg/m = 250 mm x 250 mm

Area of slab base =  $(250 + 2a)(250+2b) \text{ mm}^2$ 

# **Step 2: Projections of base plate**

Let the projections a and b be equal.

Area of slab  $(250+2a)^2 = 15 \times 10^4$ . Therefore a=68.45 mm

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

Provide projections a=b=70 mm

Provide slab base  $(250 + 2 \times 70) (250 + 2 \times 70) = 390 \text{ mm} \times 390 \text{ mm}$ 

Area of slab provided =  $390 \times 390 = 1,52,100 \text{ mm}^2$ 

Intensity of pressure from concrete under slab

# Step 3: Thickness of slab base:

$$= \left[\frac{3\times3.945}{185} \left(70^2 - \frac{70^2}{4}\right)\right]^{1/2} = 15.33 \, mm$$

Thickness of slab

Provide 16 mm thick slab base. The fastenings are provided to keep the column in position.

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 13. Steel Beams**

# **13.1 INTRODUCTION**

A beam is defined as a structural member subjected to transverse loads. The plane of transverse load is parallel to the plane of symmetry of the cross-section of the beam and it passes through the shear centre, so that the simple bending occurs. The transverse loads produce bending moments and shear forces in the beams at all the section of the beam.

The term joist is use for beams of light sections. Joist support floor construction; they do not support other beams. The term subsidiary beam or secondary beam is also used for the beams supporting floor construction. Main beams are the supporting joists for subsidiary beams. These are called floor beams in buildings. The term girder is most commonly used in buildings. Any major beam in a structure is known as a girder.

In the roof trusses, horizontal beams spanning between the two adjacent trusses are known as purlins. The beams resting on the purlins are known as common rafter or simply rafters. In the buildings the beams spanning over the doors, windows and other openings in the walls are known as lintels. The beams at the outside wall of a building, supporting its share of the floor and also wall upto the floor above it are known as spandrel beams. The beams framed to two beams at right angles to it and usually supporting joists on one side of it; used at openings such as stair wells are known as headers. The beams supporting the headers are termed as trimmers. The beams supporting the stair steps are called as stringers.

In the brigde floors, the longitudinal beams supported by the floor beams are also called as stringers. In the mill buildings, the horizontal beams spanning between the wall columns and supporting wall covering are called as girts. The beams are also called simply supported, overhanging cantilever, fixed and continuous depending upon nature of supports and conditions.

# 13.2 ROLLED STEEL SECTIONS USED AS BEAMS

The rolled steel I-sections, channel sections, angle sections, tee-sections, flat sections and bars as shown in Fig. 13.1 are the regular sections, which are used as beams. The rolled steel I-sections as shown in Fig. 13.1.A are most commonly used as the beams and as such these sections are also termed as beam sections. The rolled steel I-sections are symmetrical sections.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

#### (Prepared By: Ms Kajal, lecturer, CE)

In these sections more material is placed near top and bottom faces, i.e., in the flanges as compared to the web portion. The rolled steel I-sections provide large moment of inertia about xx-axis with less cross sectional area. The rolled steel I-sections provide large moment of resistance as compared to the other sections and as such these are most efficient and economical beam sections. The rolled steel wide flange beams as shown in Fig. 13.1.B provide additional desirable features. As the name indicates, the flanges of the sections are wide. These sections provide greater lateral stability and facilitate the connections of flanges to other members. I-sections and wide flange beam sections have excellent strength.



The rolled steel channel sections as shown in Fig. 13.1.C are used as purlins and other small structural member. The channel sections have reasonably good lateral strength and poor lateral stability. The channel sections are unsymmetrical sections about yy-axis. When the channel sections are loaded and supported by vertical forces passing through the centroid of the channel, then the channel sections bend and twist if these are laterally unsupported, except for the special case, wherein the loads act normal to the plane of web, causing bending in the weakest direction. The rolled steel angle sections as shown in Fig. 13.1.D are also used as

# E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup>

Course: B.tech,

# (Prepared By: Ms Kajal, lecturer, CE)

purlins and so other small structural members. The angle sections act as unsymmetrical sections about both xx-axis and yy-axis.

The rolled steel tee-sections as shown in Fig. 13.1.E are used as beams in the rectangular water tanks. The angles and tee-sections are used for light loads. The rolled steel flats and bars as shown in 13.1.F, G and H are very rarely used. These sections are weak in resisting bending. Most commonly the beams are loaded in the direction perpendicular to xx-axis, so that the bending of beams occurs about strong and xx-axis becomes neutral axis. The beams are very rarely loaded in the direction perpendicular to yy-axis. In such cases, yy-axis becomes neutral axis.

In cases of bending of the beams about one axis, the load is considered to be applied through the shear centre of the beam sections. In case, the loading passes through the shear centre, the section may be analyzed for simple bending and shear. The shear centre for the beam section is at the centre of area and this load position produces simple bending about either axis. When the load does not pass through the shear centre as in channels, angles and some built-up sections, a torsional moment is produced along with the bending moment and both are considered to avoid over stressing of the member. For such sections, a special load device may be used so that the load passes through shear centre of the section and the torsional moment may be avoided.

In addition to the above, expanded or castellated beams as shown in Fig. 13.2.B are used. The castellated beams are light beams and light beams and these are economically used for the light construction. The castellated beams are made by splitting the web of rolled steel I-sections in a predetermined pattern as shown in Fig. 13.2.A. The splitted portions are rejoined in such a manner as to produce a regular pattern of opening in the web.

# **13.3 BENDING STRESS**

The bending stress is also termed as flexural stress. When the beams are loaded, they bend and bending stresses are setup at all the sections. The established theory of bending is expressed in the following formula:

$$\frac{M}{I} = \frac{\sigma_b}{y} = \frac{E}{R}$$

Where, M = Bending moment

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

I = Moment of inertia

= Bending stress at any point

y = Distance from the neutral axis to the point under consideration

R = Radius of curvature of the beam

The above equation holds good when the plane of bending coincides with the plane of symmetry of the beam section. The bending of beam occurs in the principal plane of the beam section. The simple bending of beam occurs, i.e., the bending is produced by the application of pure couples at the ends of the beam. In such bending the deflection of beam does not occur due to shear. In the above equation, it is assumed that the vertical sections of the beam plane before bending remain plane after bending. Then stress in any given fibre is proportional to its strain, i.e., Hooke's law holds good. For the material of beam, the value of E is same for the complete beam.

When the load is acting downward in a simply supported beam, then the distribution of bending stress for any section of beam is as shown in Fig. 13.3. The bending stress varies linearly. The bending stress is zero at the neutral axis. When the load is acting downward, the bending stress is compressive above the neutral axis of section and tensile below it and these are denoted by  $\sigma_{bc,cal}$  and  $\sigma_{bt,cal}$  respectively. The bending stress is maximum at the extreme fibre.

$$\sigma_{bmax} = \left(\frac{M}{I} \times y_{max}\right) = \left(\frac{M}{I/y_{max}}\right) = \left(\frac{M}{Z}\right)$$

Where, Z is the section modulus (Z=I/ $y_{max}$ ),  $y_{max}$  is the distance from the neutral axis to the extreme fibre and  $\sigma_{b.max}$  is the maximum bending stress.

The maximum bending stress in the beam section (if compressive) should be less than the allowable bending compressive stress and (if tensile); should be less than the allowable bending tensile stress. When the section of beam is symmetrical about the neutral axis then the value of  $y_{max}$  is equal to half the depth of section and the maximum bending stress in compression and in the tension at the extreme fibres are equal. When the beam section is not symmetrical about the neutral axis, then there are two distance  $y_1$  and  $y_2$  to the two extreme fibres from the nutral axis. The bending stresses at the extreme top and bottom are not equal. Then, the values of  $Z_1=(I/y_1)$ 

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

and  $Z_2=(I/y_2)$  both should be calculated and compared with the section modulus, Z of the beam section provided.

The total compressive force 'C' above the neutral axis is equal to the tensile force 'T', for the beam in equilibrium. These two forces act in opposite directions and form a couple. This couple resists the bending moment and this moment is known as moment of resistance ' $M_r$ '. the moment of resistance of a beam section is the moment of the couple which is set up at the section by the longitudinal forces C and T created in the beam due to bending.

M<sub>r</sub>=(C x Lever arm)=(T x Lever arm)

For the beam in equilibrium, the moment of resistance ' $M_r$ ' would be equal to the maximum bending moment 'M' at any section ( $M_r=M$ ).

## 13.4. ALLOWABLE STRESS IN BENDING

The allowable bending stress,  $\sigma_{bc}$  in the design of rolled steel beam section considerably depends on the geometrical properties of the section and the lateral support. In case of flange width/flange thickness (½bf/tf) and the depth of section/thickness of web (h/tw) ratios not adequate, the elements of beam section will tend to buckle at low compressive stresses (which will be due to bending combined with axial loads). If the compression flange is not laterally supported (i.e., supports at intervals or uniformly) along the compression zone, it will either buckle in plane or cut-of plane coupled with twisting.

The rolled sections are produced with adequate  $(\frac{1}{2}b_{f}/t_{f})$  and  $(h/t_{w})$  ratios such that the buckling of flange or web does not occur. The designers may provide supports at intervals or uniformly along he compression flange such that its buckling is avoided. The calculated bending compressive stress  $\sigma_{bc,cal}$  and bending tensile stress  $\sigma_{bt,cal}$  in the extreme fibres should not exceed the maximum permissible bending stress in compression ( $\sigma_{bc}$ ) or in tension  $\sigma_{bt}$  as below.

 $\sigma_{bc} \text{ or } \sigma_{bt} = 0.66 \ f_y$ 

The structural steel used in general construction may have yield stress as 220, 230, 240, 250, 260, 280, 300, 320, 340, 360, 380, 400, 420, 450, 480, 510 or 540 N/mm<sup>2</sup> (M Pa). The structural steels having these values of yield stress are also used in flexural members. The maximum permissible bending compressive stress in beams and channels with equal flanges

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

have been given separately in IS:800-1984. For an I-beam or channel with equal flanges bending about the axis of maximum strength (xx-axis), the maximum bending compressive stress on the extreme fibre, calculated on the effective section shall not exceed the values of maximum permissible bending compressive stress,  $\sigma_{bc}$ .

The safe compressive stress for a given grade of steel depends on a number of parameters as given below.

Let D=Overall depth of the beam

d<sub>1</sub>=Clear distance between the flanges

l=Effective length of the compression flange (Table 13.1)

r<sub>y</sub>=Radius of gyration of the section about its axis of minimum strength (yy-axis)

T=Mean thickness of the compression flange

t=Web thickness.

For the rolled steel sections, the mean thickness is that which one is given in ISI Handbook No.1. In the case of compound girders, with curtailed flanges, D shall be taken as the overall depth of the girder at the point of maximum bending moment and T shall be taken as the effective thickness of the compression flange and shall be calculated as

 $T = K_1 x$  mean thickness of the horizontal portion of the compression flange at the point of maximum bending moment where  $K_1 = a$  co-efficient given in Table 13.2

Table 13.1 EFFECTIVE LENGTH OF COMPRESS	SION FLANGE (1).
---	------------------

S1. No.	End conditions	Effective length (1)
i	Unrestrained against lateral bending (i.e., free to rotate in plan at the bearing)	1=span
ii	Partially restrained against lateral bending (i.e., not free to rotate in plan at the bearings)	1=0.85 x span
iii	Fully restrained against lateral bending (i.e., not free to rotate in plan at the bearings)	1=0.70 x span

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

# (Prepared By: Ms Kajal, lecturer, CE)

Restraint against torson can be provided by

i) Web or flange cleats, or

ii) Bearing stiffeners acting in conjection 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 wall.

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 the flange are free to move laterally, the above values of the effective length shall be increased by 20 percent.

The end constraint 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 to not less than 2.5 per cent of the maximum force occurring in the flange.

For cantilever beams of projecting length L, the effective length l to be used shall be taken as follows:

a)	Built in at the support, free at the end	1=0.85 L
b)	Built in at the support restrained against torsion at the end by continuous construction	1=0.75 L
c)	Built in at the support, restrained against lateral deflection and torsion at the free end	1=0.5 L
d)	Continuous at the support unrestrained against torsion at the support and free at the end	1=3 L
e)	Continuous at the support with partial restraint against torsion of the support and free at the end	1=2 L
f)	Continuous at the support, restrained against torsion at the support and free at the end	1= L

If there is a degree of fixity at the end, the effective length shall be multiplied by 0.5/0.85 in (b) and (c) above, and by 0.75/0.85 in (d), (e) and (f) above.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# 13.5 MAXIMUM PERMISSIBLE BENDING COMPRESSIVE STRESS IN BEAMS AND PLATE GIRDERS

For beams and plate girders bent about the 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  $\sigma_{bc}$  calculated from the following formula.

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

Where,

 $f_{cb}$  = elastic critical stress in bending  $f_y$  = yield stress of the steel n = a factor assumed as 1.4

The elastic critical stress  $f_{cb}$  for beams and plate girders with  $I_y$  smaller than  $I_x$  shall be calculated using the following formula.

$$f_{bc} = K_1 (X + K_2 Y) \frac{C_2}{C_1}$$
  
Where,  $X = Y \sqrt{1 + \frac{1}{20} \left(\frac{lT}{r_y D}\right)^2} N/mm^2$  and  $Y = \frac{26.5 \times 10^5}{\left(\frac{l}{r_y}\right)^2} N/mm^2$ 

 $K_1$ = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral resistant and depends on  $\psi_1$  the ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of resistant. Values of  $K_1$  for different values of  $\psi$  are given in the Table 13.2.

 $K_2$ = a coefficient to allow for the inequality of flanges, and depends on  $\omega$ , the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment. Values of  $K_2$  for different values of  $\omega$  are given in the Table 13.3.

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

 $C_1, C_2$ =respectively the lesser and the greater distances from the section neutral axis to the extreme fibres.

 $I_x$ =moment of inertia of the whole section about the axis normal to the plane of bending (x-x axis) and

 $I_{y}$ = moment of inertia of the whole section about the axis lying in the plane of bending (y-y axis)

	Tab	le 13.2	VALUES	S OF K1	FOR BE	EAMS W	VITH CU	<b>JRTAILI</b>	ED FLA	NGES	
Ψ	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K <sub>1</sub>	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

Table 13.3 VALUES OF K2 FOR BEAMS WITH UNEQUAL FLANGES											
Ω	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K <sub>2</sub>	0.5	0.4	0.3	0.2	0.1	0	-0.2	-0.4	-0.6	-0.8	-1.0

Values of X and Y for appropriate values of D/T and l/ry can be taken from standard text books

on design of steel structures. Values of  $f_{cb}$  shall be increased by 20 per cent when  $\frac{T}{t} \neq 2$  $\frac{d_1}{d_1} \neq \frac{1.344}{d_1}$ 

and  $t = \sqrt{f_y}$ . The maximum permissible bending stress in tension  $\sigma_{bt}$  or in compression  $\sigma_{bc}$  in beams bent about the axis of minimum strength shall not exceed 0.66 f<sub>y</sub> where f<sub>y</sub> is the yield stress of steel.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)



E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

# **LESSON 14. Design of Steel Beams**

# 14.1 INTRODUCTION

The following points should be considered in the design of a beam.

- 1. Bending moment consideration: The section of the beam must be able to resist the maximum bending moment to which it is subjected.
- 2. Shear force consideration: The section of the beam must be able to resist the maximum shear force to which it is subjected.
- 3. Deflection consideration: The maximum deflection of a loaded beam should be within a certain limit so that the strength and efficiency of the beam should not be affected. Limiting the deflection within a safe limit will also prevent any possible damage to finishing. As per the I.S. code, generally the maximum deflection should not exceed 1/325 of the span.
- 4. Bearing stress consideration: The beam should have enough bearing area at the supports to avoid excessive bearing stress which may lead to crushing of the beam or the support itself.
- 5. Buckling consideration: The compression flange should be prevented from buckling. Similarly the web, the beam should also be prevented from crippling. Usually these failures do not take place under normal loading due to proportioning of thickness of flange and web. But under considerably heavy loads, such failures are possible and hence in such cases the member must be designed to remain safe against such failures

# 14.2 SHEAR AND BEARING STRESSES

When the beams are subjected to loads, then, these are also required to transmit large shear forces either at supports or at concentrated loads. For simply supported beams, the shear force is maximum at the supports. The values of shear force at the concentrated loads also remain large. Due to shear force, the shear stresses are setup along with the bending stresses at all sections of the beams. The shear stress at any point of the cross-section is given by

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

$$\tau_{\nu} = \left(\frac{F.Q}{I.t}\right)$$

Where is the shear stress,

F = the shear force at cross-section,

Q = Static moment about the neutral axis of the portion of cross-sectional area beyond the location at which the stress is being determined.

I = Moment of inertia of the section about the neutral axis

t = Thickness of web (width of section at which the stress is being determined)



The distribution of shear stresses for rectangular section of beam and I-beam section are shown in Fig. 14.1. The maximum shear stress occurs at the neutral axis of the section. The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behavior of the member in flexure (bending) should not exceed the value of maximum permissible shear stress,  $\tau_{vm}$  found as follows.

$$\tau_{vm}=0.45f_y$$

Where  $f_y$  is the yield stress of structural steel to be used. It is to note that in the case of rolled beams and channels, the design shear is to be found as the average shear. The average shear stress for rolled beams or channels calculated by dividing the shear force at the cross-section by the gross-section of the web. The gross-cross-section of the web is defined as the depth of the beam or channel multiplied by its web thickness.

Course: B.tech,

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

( Prepared By: Ms Kajal, lecturer , CE)

Average shear stress for rectangular beam is given by  $\tau_{v} = \left(\frac{F}{b \times d}\right)$ 

Average shear stress for rectangular beam is given by  $\tau_v = \left(\frac{F}{h \times t_w}\right)$ 

For rolled steel beams and channels, it is assumed that shear force is resisted by web only. The portion of shear resisted by the flanges is neglected. The average shear stress  $\tau_{va.cal}$ , in a member calculated on the gross cross-section of web (when web buckling is not a factor) should not exceed in case of unstiffened web of the beam,

$$\tau_{\rm va} = 0.4 f_y$$

The allowable shear stress as per AISC, AASHTO and AREA specifications are as follows:

Specifications	Allowable shear stress			
AISC	0.40 fy			
AASHTO	0.33 fy			
AREA	0.35 fy			

When the beams are subjected to co-existent bending stresses (tension or compression) and shear stress, then the equivalent stress,  $\sigma_{e,cal}$  is obtained from the following formula

$$\sigma_{e.cal} = [\sigma_{bt.cal}^2 + 3\tau_{vm.cal}^2]^{1/2} \qquad \text{or} \qquad \sigma_{e.cal} = [\sigma_{bc.cal}^2 + 3\tau_{vm.cal}^2]^{1/2}$$

The equivalent stress due to co-existent bending (tension or compression) and shear stresses should not exceed the maximum permissible equivalent stress  $\sigma_e$  found as under

$$\sigma_e = 0.90 f_y$$

When the bearing stress  $\sigma_p$  is combined with tensile or compressive bending and shear stresses under the most unfavourable conditions of loading, the equivalent stress  $\sigma_{e.cal}$  obtained as below should not exceed

E-NOTES , Subject : Design of Steel Structures I , Subject Code: CE-304B , Branch: CIVIL Engineering , Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

$$\begin{aligned} \sigma_e &= 0.90 f_y \\ \sigma_{e.cal} &= \left[\sigma_{bt.cal}^2 + \sigma_{p.cal}^2 + \sigma_{bt.cal} \cdot \sigma_{p.cal} + 3\tau_{vm.cal}^2\right]^{1/2} \\ \sigma_{e.cal} &= \left[\sigma_{bc.cal}^2 + \sigma_{p.cal}^2 + \sigma_{bc.cal} \cdot \sigma_{p.cal} + 3\tau_{vm.cal}^2\right]^{1/2} \end{aligned}$$

 $\sigma_{bc.cal}$ ,  $\sigma_{bt.cal}$ ,  $\tau_{vm.cal}$  and  $\sigma_{p.cal}$  are the numerical values of the co-existent bending (compression or tension), shear and bending stresses. When bending occurs about both the axes of the member,  $\sigma_{bt.cal}$  and  $\sigma_{bc.cal}$  should be taken as the sum of the two calculated fibre stresses,  $\sigma_e$  is the maximum permissible equivalent stress.

The bearing stress in any part of a beam when calculated on the net area of contact should not exceed the value of  $\sigma_p$  calculated as below

Where  $\sigma_p$  is the maximum permissible bearing stress and  $f_y$  is the yield stress.

# 14.3 EFFECTIVE SPAN AND DEFLECTION LIMITATION

The effective span of a beam shall be taken as the length between the centres of the supports, except in cases where the point of application of the reaction is taken as eccentric to the support, then, it shall be permissible to take the effective span as the length between the assumed points of application of reaction.

The stiffness of a beam is a major consideration in the selection of a beam section. The allowable deflections of beams depend upon the purpose for which the beams are designed. The maximum deflections for some standard cases are given below. In these formulae W is the total load on the beam in case of uniformly distributed load and each concentrated load in the case of concentrated loads.

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

Deflection in beam carrying uniformly distributed load over the whole span  $=\frac{5}{384} \frac{Wl^3}{El}$ Deflection in beam carrying a point load at the centre  $=\frac{1}{48} \frac{Wl^3}{El}$ Deflection in beam carrying two point loads (uniformly spaced)  $=\frac{22}{648} \frac{Wl^3}{El}$ Deflection in beam carrying three point loads (uniformly spaced)  $=\frac{19}{384} \frac{Wl^3}{El}$ Deflection in beam carrying a number of point loads W each (uniformly spaced)  $=n\left\{3-\frac{1}{2}\left(1+\frac{4}{n^2}\right)\right\}\frac{Wl^3}{El}$ 

Deflection in beam carrying a symmetrical triangular load  $=\frac{1}{60}\frac{Wl^3}{El}$ 

The large deflections of beams are undesirable for the following reasons:

- When the loads are primarily due to human occupants especially in the case of public meeting places, large deflections result in noticeable vibratory movement. This produces an uncomfortable sensation to the occupants.
- 2. The large deflections may result in cracking of ceiling plaster, floors or partition walls.
- **3**. The large deflection indicate the lack of rigidity. It may cause vibrations and overstresses under dynamic loads.
- 4. The large deflections may cause the distortions in the connections. The distortions cause secondary stresses.
- 5. The large deflections may cause poor drainage, which will lead to ponding of water and therefore increase the loads.

## 14.3.1 Limiting vertical deflection

The deflection of a member is calculated without considering the impact factor or dynamic effect of the loads causing the deflection. The deflection of a member shall not be such as to impair the strength or efficiency of the structure and lead to damage to finishing. Generally, the maximum deflection for a beam shall not exceed 1/325 of the span. This limit may be exceeded in cases where greater deflection would not impair the strength or efficiency of the structure or

E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

lead to damage to finishing. The deflection of the beams may be decreased by increasing the depth of beams, decreasing the span, providing greater and restraint or by any other means.

# 14.3.2 Limiting horizontal deflection

At the caps of columns in single storey buildings, the horizontal deflection due to lateral force should not ordinarily exceed 1/325 of the actual length 'l' of the column. This limit may be exceeded in cases where the grater deflection would not impair the strength and efficiency of the structure or lead to damage to finishing. According to AISE specifications, the deflections of beams and girders for live load and plastered ceiling should not exceed 1/360 of the span.

# **14.4 LATERALLY SUPPORTED BEAMS**

The laterally supported beams are also called laterally restrained beams. When lateral deflection of the compression flange of a beam is prevented by providing effective lateral support (restraint), the beam is said to be laterally supported. The effective lateral restraint is the restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a beam from lateral buckling to either side at the point of application of the restraint. The concrete slab encasing the top flange, so that the bottom surface of the concrete slab is flush with the bottom of the top flange, is shown in Fig. 14.2.A. It provides a continuous lateral support to the top flange of the beam. When other beams frame at frequent intervals into the beam in questions as shown in Fig. 14.2.B, lateral support is provided at each point of connection but main beam should still be checked between the two supports.



E-NOTES, Subject: Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

In the laterally supported beams, the value of allowable bending compressive stress remains unaltered and the reduction in its value is not made. Bending comprehensive stress is taken equal to the allowable bending tensile stress, ( $\sigma_{bc}=\sigma_{bt}=0.66f_y$ ). The adequate lateral support is provided to safeguard against the lateral-torsional bucking. In case of doubt for adequate lateral support, the beams should be designed as laterally unsupported. In case the concrete slab holds the top flange (compression flange) of the beam from one side only, then, the lateral support is not credited. The concrete slab simply resting over the top flange of the beam without shear connectors also does not provide an lateral support. Sometimes, the plank or bar grating is attached to the top flange of beam by means of bolts. When the bolts are firmly fastened, then, they provide adequate lateral support temporarily. Even then, bolts have temporary nature of connections. It is possible that the bolts might be omitted or removed. As such, the top flange should not be considered laterally supported fully. The beams having lateral support from other members may buckle between points of lateral support. Therefore, the laterally unsupported length of beam is kept short.

# 14.5 DESIGN OF LATERALLY SUPPORTED BEAMS

The design of beams is generally governed by the maximum allowable bending stress and the allowable deflection. Its design is controlled by shear only when the spans are short and loads are heavy. The members are selected such that the sections are symmetrical about the plane of loading and the unsymmetrical bending and torsion are eliminated. The design of beams deals with proportioning of members, the determination of effective section modulus, maximum deflection and the shear stress. In general, the rolled steel sections have webs of sufficient thickness such that the criterion for design is seldom governed by shear. The following are the usual steps in design of laterally supported beams:

**Step 1.** For the design of beams, load to be carried by the beam, and effective span of the beam are known. The value of yield stress,  $f_y$  for the structural steel to be used is also known. For the rolled steel beams of equal flanges as given in ISI Handbook no.1, the ratio of mean thickness of the compression flange (T=t<sub>f</sub>) to the thickness of web used to be less than 2.00. Also the ratio of the depth of web d<sub>1</sub> to the thickness of web is also smaller than 85. The ends of compression flange of a laterally supported beam remain restrained against lateral bending (i.e., not free to rotate in plan at the bearings).

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

#### Course: B.tech,

## (Prepared By: Ms Kajal, lecturer, CE)

In the beginning of design, the permissible bending stress in tension,  $\sigma_{bt}$  or in compression,  $\sigma_{bc}$  may be assumed as 0.66  $f_y$ . The bending compressive stress,  $\sigma_{bc}$  and the bending tensile stress,  $\sigma_{bt}$  are equal for the laterally supported beam.

**Step 2.** The maximum bending moment M and the maximum shear force F in the beam are calculated. The required section modulus for the beam is determined as  $Z=(M/\sigma_{bc})$ 

**Step 3.** From the steel section tables, a rolled steel beam section, a rolled steel beam section, which provides more than the required section modulus is selected. The steel beam section shall have (D/T) and  $(l/r_y)$  ratios more than 8 and 40 respectively. As such the trial section of beam selected may have modulus of section, Z more than that required. Some of the beam sections of different categories have almost the same value of the section modulus Z. It is necessary to note the weight of beam per meter length and the section modulus, Z. The beam section selected should be such that it has minimum weight and adequate section modulus, Z.

**Step 4.** The rolled steel beam section is checked for the shear stress. The average and maximum shear stresses should not exceed the allowable average and maximum values of shear stresses.

**Step 5.** The rolled steel beam is also checked for deflection. The maximum deflection should not exceed the limiting deflection.

ISI Handbook no.1 provides tables for allowable uniform loads on beams and channels used as flexural members with adequate lateral support for compression flange. The values of allowable uniform loads corresponding to respective effective spans are given for various beams and channel sections. For given span and total uniformly distributed load found, rolled beam or channel section may be selected from these tables. The rolled steel I-sections and wide flange beam sections are most efficient sections. These sections have excellent flexural strength and relatively good lateral strength for their weights.

**Example 14.1** The effective length of compression flange of simply supported beam MB 500,@0.869 kN/m is 8 m. Determine the safe uniformly distributed load per meter length which can be placed over the beam having an effective span of 8 meters. Adopt maximum permissible stresses as per IS 800-1984. The ends of beam are restrained against rotation at the bearings.

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

(Prepared By: Ms Kajal, lecturer, CE)

## Solution:

## **Step 1: Permissible bending stress**

MB 500,@0.869 kN/m has been used as simply supported beam. The effective span of beam is 8 m. The effective length of compression flange is also 8 m.

From the steel section table, the section modulus of beam	$Z=1808.7 \text{ x } 10^3 \text{ mm}^3$
Mean thickness of compression flange	$t_{\rm f}{=}T{=}17.2~mm$
Thickness of web	t <sub>w</sub> =10.2 mm

It is assumed that the value of yield stress,  $f_y$  for the structural steel of MB 500, @0.869 kN/m is 250 N/mm<sup>2</sup>(MPa).

Ratio  $\left(\frac{T}{t_w} = \frac{17.2}{10.2}\right) = 1.686 < 2.00$ Ratio  $\left(\frac{d_t}{t_w}\right) = \left(\frac{h-2h_2}{t_w}\right) = \left(\frac{500-2\times37.95}{10.2}\right) = \left(\frac{h_1}{t_w}\right) = \left(\frac{424.1}{10.2}\right) = 41.578 < 85$ Ratio  $\left(\frac{D}{T}\right) = \left(\frac{500}{17.0}\right) = 29.07$ 

The effective length of compression flange is 8 m.

Ratio  $\frac{l}{r_y} = \left(\frac{0.7 \times 8 \times 1000}{35.2}\right) = 159.1$ 

From IS: 800-1984, the maximum permissible bending stress, for above ratios (by linear interpolation)  $\sigma_{bc}$ =65.121 N/mm<sup>2</sup>(MPa)

## Step 2: Load supported over beam

$$M_r = (\sigma_{bc} \times Z) = \left(\frac{88.566 \times 1808.7 \times 10^3}{1000 \times 1000}\right) = 160.189 \ m - kN$$

MB 500, @0.869 kN/m can resist maximum bending moment equal to moment of resistance. Therefore the maximum bending moment M=160.189 m-kN

## Step 3: Load supported over beam
### INTERNATIONAL INSTITUTE OF TECHNOLOGY & MANAGEMENT, MURTHAL SONEPAT

# E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

## (Prepared By: Ms Kajal, lecturer, CE)

The effective span of the beam is 8 meters. Let w be the uniformly distributed load per meter length. The maximum bending moment, M for the beam occurs at the centre..

$$M = \left(\frac{w \cdot l^2}{8}\right) = 160.189 = \left(\frac{w \times 8 \times 1000}{8}\right)$$
  
Therefore  $w = \left(\frac{160.189 \times 8}{8 \times 8 \times 1000} \times 1000\right) = 20.02 \ kN/m$ 

The self-weight of the beam is 0.869 kN/m. Therefore, the safe uniformly distributed load which can be placed over the beam (20.02-0.869)=19.15 kN.

**Example 14.2** Design a simply supported beam to carry a uniformly distributed load of 44 kN/m. The effective span of beam is 8 meters. The effective length of compression flange of the beam is also 8 m. The ends of beam are not free to rotate at the bearings.

### **Design:**

# Step 1: Load supported, bending moment and shear force

Uniformly distributed load	= 44  kN/m
----------------------------	------------

Assume self weight of beam = 1.0 kN/m

Total uniformly distributed load w = 45 kN/m

The maximum bending moment, M occurs at the centre

$$M = \left(\frac{w.\,l^2}{8}\right) = \left(\frac{45 \times 8 \times 8 \times 1000}{8 \times 1000}\right) = 360 \ m - kN$$

The maximum shear force, F occurs at the support  $F = \left(\frac{w.l}{2}\right) = \left(\frac{45 \times 3}{2}\right) = 180 \ kN$ 

# Step 2: Permissible bending stress

It is assumed that the value of yield stress,  $f_y$  for the structural steel is 250 N/mm<sup>2</sup> (MPa). The ratios (T/t<sub>w</sub>) and (d<sub>1</sub>/t<sub>w</sub>) are less than 2.0 and 85 respectively. The maximum permissible stress in compression or tension may be assumed as  $\sigma_{bc} = \sigma_{bt} = (0.66 \text{ x } 250) = 165 \text{ N/mm}^2$ 

#### INTERNATIONAL INSTITUTE OF TECHNOLOGY & MANAGEMENT, MURTHAL SONEPAT

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Branch: CIVIL Engineering, Sem-6<sup>th</sup> Course: B.tech,

### (Prepared By: Ms Kajal, lecturer, CE)

Section modulus required,  $Z = \frac{M}{\sigma_{bc}} = \left(\frac{360 \times 1000 \times 1000}{165}\right) = 2181.82 \times 10^3 \ mm^3$ 

The steel beam section shall have (D/T) and  $(l/r_y)$  ratios more than 8 and 40 respectively. The trial section of beam selected may have modulus of section, Z more than that needed (about 25 to 50 per cent more).

## **Step 3: Trial section modulus**

1.50 x 2181.82 x 10<sup>3</sup> mm<sup>3</sup>=3272.73 x 10<sup>3</sup> mm<sup>3</sup>

From steel section tables, try WB 600,@1.337 kN/m

Section modulus,  $Z_{xx}=3540.0 \times 10^3 \text{ mm}^3$ 

Moment of inertia,  $I_{xx}=106198.5 \times 10^4 \text{ mm}^4$ 

Thickness of web, t<sub>w</sub>=11.2 mm

Thickness of flange,  $T=t_f=21.3 \text{ mm}$ 

Depth of section, h=600 mm

# **Step 4: Check for section modulus**

$$\frac{D}{T} = \frac{600}{21.3} = 28.169$$
$$\frac{T}{t_w} = \frac{21.3}{11.2} = 1.901 < 2.0, also \left(\frac{d_1}{t_w} < 85\right)$$

The effective length of compression flange of beam is 8 m.

$$\frac{l}{r_y} = \left(\frac{0.7 \times 8 \times 10.0}{52.5}\right) = 106.66$$

From IS: 800-1984, the maximum permissible bending stress  $\sigma_{bc}$ =118.68 N/mm<sup>2</sup>(MPa)

### INTERNATIONAL INSTITUTE OF TECHNOLOGY & MANAGEMENT, MURTHAL SONEPAT

E-NOTES, Subject : Design of Steel Structures I, Subject Code: CE-304B, Course: B.tech, Branch: CIVIL Engineering, Sem-6<sup>th</sup>

### (Prepared By: Ms Kajal, lecturer, CE)

Section modulus required

 $\left(\frac{360 \times 1000 \times 1000}{118.68}\right) = 3033.34 \times 10^{3} mm^{3} < 3540 \times 10^{3} mm^{3} provided$ 

Further trial may give more economical section.

# **Step 5: Check for shear force**

 $\tau_{v.cal} = \left(\frac{F}{h \times t_w}\right) = \left(\frac{180 \times 1000}{600 \times 11.2}\right) = 26.78 \, N/mm^2$ 

Average shear stress,

Permissible average shear stress

 $0.4 \text{ x} f_y = (0.4 \text{ x} 250) = 100 \text{ N/mm}^2 > \text{Actual average shear stress}$ 

# **Step 6: Check for deflection**

Maximum deflection of the beam

$$y_{max} = \left(\frac{5}{384} \times \frac{w.l^4}{El}\right) = \frac{5}{384} \times \left(\frac{45 \times 8 \times 8^3 \times 1000^3 \times 1000}{2.047 \times 10^5 \times 106198.5 \times 10^4}\right) = 24.53 \ mm$$

Allowable deflection

$$\frac{1}{325} \times span = \left(\frac{1}{325} \times 8000\right) = 24.60 \, mm$$

The maximum deflection is less than allowable deflection, hence the beam is safe. Provide WB 600,@1.337 kN/m