ASHOKA INSTITUTE OF TECHNOLOGY & MANAGEMENT, VARANASI

DIGITAL EDUCATION

STUDY NOTES

Branch : Civil Engineering

Subject: Design of Steel Structure

Paper Code: KCE 075

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NOTES OF DSS (KCE 075) – B. TECH 7th Sem

UNIT 1

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 structures and in the structure, then the structures are known as statically indeterminate 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.

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)

6. 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/m3

2. Young's modulus of elasticity, E=2.04 to 2.18 x 105 N/mm2

3. Modulus of rigidity, G=0.84 to 0.98 x 105 N/mm2

4. Coefficient of thermal expansion (or contraction) α=12 x 10-6/˚C or 6.7 x 10-6/˚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 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 55HTW. 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.

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.

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 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 Isections 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 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 Secions

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

2. Medium weight channels MC

3. Medium weight channels with parallel flanges MCP

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

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 ∟ 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.

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., ∟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 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..

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.

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.

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
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 33⅓ 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/m2 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

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

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/m2 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/m2 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

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

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 stressstrain relationship for mild steel can be studied by plotting stress-strain curve. 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 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 crosssectional 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 (σat). The tensile stress is calculated on net cross-sectional area of the member:

$$\sigma at = (Pt/An)$$

Where Pt is the direct axial tensile load and An 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 (σac). The compressive stress is calculated on gross cross-sectional area of the member $\sigma ac = (Pc/Ag)$

Where Pc is the direct axial compressive load and Ag 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'(σb). the bearing stress is calculated on net projected area of contact

$\sigma b = (P/A)$

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 33⅓ 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.

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 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.

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.

UNIT 2

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/mm2 to 250 N/mm2. 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 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 t0.5

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.

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 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

2. Double riveted lap joint:

a. Chain riveted lap joint

b. Zig-zag riveted lap joint

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

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

1. Strength of a riveted joint against shearing - Ps

2. Strength of a riveted joint against bearing - Pb

3. Strength of plate in tearing - Pt

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

Ps = 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

Where N=Number of rivets on each side of the joint; D=Gross diameter of the rivet; ps=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

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

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.

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,

Pb=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 pb

Where D= Gross diameter of the rivet; t=thickness of the thinnest plate; pb= 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 Pb = N x D x t x pb

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 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

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.

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

$Pt=(p-D).t.pt$

Where p=pitch of rivets adopted, t=thickness of plate and pt = working stress in direct tension for plate. Tearing strength of plate should not exceed the rivet value R (Ps or Pb whichever is less)

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

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 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

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/mm2 (MPa). Working stress in bearing in rivets=250 N/mm2 (MPa). Working stress in axial tension in plates=156 N/mm2.

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 shear = (π/4) x 21.52 x 80/1000

Ps = 29.044 kN

Strength of rivet in bearing = 21.5 x 10 x 250/1000

Pb = 53.750 kN

Strength of plate in tension per gauge length = Pt=(p-D).t.pt

Pt = (55-21.5) x 10 x 156/1000

= 52.260 kN

Strength of joint is minimum of Ps, Pb or Pt

Therefore, the strength of joint is = 29.044 kN

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/mm2 (MPa).

Working stress in bearing in power driven rivets=300 N/mm2 (MPa).

For plates working stress in axial tension =156 N/mm2.

Solution

Nominal diameter of rivet from Unwin's formula

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

Strength of rivet in double shear =

Strength of rivet in bearing = D x t x pb = 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. Pt = (p-D) x t x pt = ((p-23.5) x 12 x 156/100) =1.872 (p-23.5) kN

In double riveted joint,

Strength of 2 rivets in shear Ps = 2 x 86.75 = 173.5 kN

Strength of 2 rivets in bearing Pb = 2 x 84.6 = 169.2 kN

The pitch of the rivets can be computed by keeping Pt = Ps or Pb whichever is less Therefore 1.872 (p-23.5) = 169.2 p-23.5 = (169.2/1.872) = 90.385 p= 90.385 + 23.5 = 113.885 mm

Adopt pitch, p= 100 mm

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

Solution

Nominal diameter of rivet from Unwin's formula

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/mm2 (MPa).

Bearing stress for power driven rivets=300 N/mm2 (MPa).

Strength of plate in tension =156 N/mm2.

Strength of rivet in double shear =

Strength of rivet in bearing = D x t x pb = 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. Pt = (p-D) x t x pt = ((p-23.5) x 16 x 156/100) =2.496 (p-23.5) kN

The pitch of the rivets can be computed by keeping Pt = Ps or Pb whichever is less

Therefore 2.496 (p-23.5) = 86.75

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

p= 34.756 + 23.5 = 58.256 mm

Adopt pitch, p= 55 mm

Adopt thickness of each cover plate t ≈ 5/8 x 16 ≈ 10 mm

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

Solution : Nominal diameter of rivet from Unwin's formula

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

Strength of rivet in double shear =

Strength of rivet in bearing = D x t x pb = 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 ≈ 9 rivets.

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

Pt = (b-d) x t x pt = ((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

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 =

Strength of power driven rivet in bearing = D x t x pb = 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 $P_t = (b-d) \times t \times p_t = ((250-23.5) \times t \times 156/100) = 500$ 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.

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.

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

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 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. When the cross section of fillet weld is 45˚, isosceles triangle 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 it is known as a special fillet weld. The 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.

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 recommended in IS:816-1969.

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. This avoids the washing down of edges of weld.

When fillet weld is applied to the round toe of rolled steel sections, the maximum size of the weld should not exceed ¾ of the thickness of the section at the toe. When fillet weld is used for lap joint, then overlap of the members connected, 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/mm2.

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 x 16 = 10 mm

Effective length of weld = 150 mm

Strength of single U butt weld = (10 x 150 x 142/1000) = 213.0 kN

Example 7.2. In a truss girder of a bridge, a tie 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 110/1000) = 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

Total length of two welds = 240 mm

Total load transmitted by 6 mm weld = (240 x 0.7 x 6 x 110/1000) = 110.88 kN

Maximum pull that can be transmitted by the plate = (120 x 10 x 0.6 x 250/1000) =180 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 x 30 x 15 x 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 mm2

UNIT 3

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

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 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.

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.

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 fy, where fy is the minimum yield stress of steel.

For example, if fy=250 N/mm2, Safe stress in axial tension = 0.6 x 250 = 150 N/mm2.

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.

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:

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.

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.

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. Rivets used for the connection are 20 mm diameter. Allowable tensile stress is 150 N/mm2.

Solution

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

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 x 73.7 = 103.18 kN

Number of rivets required = (103.18 x 1000/36,305) ≈ 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. 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

10.3 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

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, λ=l/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. 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.

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 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, ryy of the section, about yy-axis (axis parallel to the web) is much smaller than the radius of gyration rxx 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, lyy for bending about yy-axis is found by considering the length of column between one-storey only. The effective length of column lxx 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 lyy is much smaller than that of lxx. The values of slenderness ratio (lyy/ryy) and (lxx/rxx) 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

$\sigma ac = (Pa/A)$

Where Pa = 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 σc is known

$Areqd = (P/\sigma c)$

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/rmin). 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 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

Area to be provided by two cover plates = 17339.74 mm2

Area to be provided by one plate = 8669.87 mm2

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 = ½ (700-140)=560/2=280 mm

Thickness = 18 mm

Step 6: Check for load carrying capacity

Area to be provided by two cover plates = 17339.74 mm2

Area to be provided by one plate = 8669.87 mm2

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 = ½ (700-140)=560/2=280 mm

Thickness = 18 mm

Step 6: Check for load carrying capacity

UNIT 4

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 with their approximate radii of gyration. A column or a compression member may be made 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 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/r2. 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 (rmin) is taken. rmin for a built up section is calculated.

Step 3. The maximum slenderness ratio (l/ rmin) is determined for the compression member.

Step 4. The allowable working stress (σac) 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.

Step 6. The safe load carrying capacity of the member is determined as P=(σac.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

1. When single angle discontinuous strut is connected to a gusset plate with one rivet, 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, 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.

1. 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, 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’ 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, 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.

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 mm2

Radius of gyration rxx= ryy=149.3 mm

Radius of gyration ruu= 58.3 mm, rvv=29.3 mm

Step 2: Slenderness ratio,

Minimum radius of gyration rmin= 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, fy=260 N/mm2, allowable working stress in compression σac =64.45 N/mm2 (MPa)

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

0.80 σac = (0.80 x 64.45) = 51.56 N/mm2.

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 mm2

Radius of gyration rxx= ryy=45.7 mm

Radius of gyration ruu= 57.6 mm, rvv=29.3 mm

Step 2: Slenderness ratio,

Minimum radius of gyration rmin= 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, fy=260 N/mm2, allowable working stress in compression σac =64.45 N/mm2 (MPa)

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

0.80 σac = (0.80 x 64.45) = 51.56 N/mm2.

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 mm2

Radius of gyration rxx= ryy=149.3 mm

Radius of gyration ruu= 58.3 mm, rvv=29.3 mm

Length of strut between centre to centre of intersection L=3.50 m

Step 2: Slenderness ratio,

Minimum radius of gyration rmin= 29.3 mm

Effective length of discontinuous strut double riveted 0.85 x L= 0.85 x 3.5 = 2.975 m

Slenderness ratio of the strut

Step 3: Safe load

From IS:800-1984 for l/r=101.5 and the steel having yield stress, fy=260 N/mm2, allowable working stress in compression σac =71.65 N/mm2 (MPa)

Allowable working stress for discontinuous strut double riveted is not reduced.

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:
Sectional area A = 4204 mm2

Radius of gyration rxx= 39.4 mm

Angles are 10 mm apart

Radius of gyration ryy= 40.1 mm

Length of strut between centre to centre of intersection L=4 m

Step 2: Slenderness ratio,

Minimum radius of gyration rmin= 39.4 mm

Effective length of discontinuous strut 0.85 x L= 0.85 x 4.0 = 3.40 m

Slenderness ratio of the strut

Step 3: Safe load

From IS:800-1984 for l/r=86.3 and the steel having yield stress, fy=260 N/mm2, allowable working stress in compression σac =95.96 N/mm2 (MPa)

Minimum radius of gyration rmin= 36.7 mm

Slenderness ratio of the strut

Step 3: Safe load

From IS:800-1984 for l/r=109 and the steel having yield stress, fy=260 N/mm2, allowable working stress in compression σac =73.9 N/mm2 (MPa)

For above strut, allowable working stress 0.80 σac = (0.80 x 73.9) = 59.12 N/mm2.

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:

From IS:800-1984 for l/r=101.5 and the steel having yield stress, fy=260 N/mm2, allowable working stress in compression σac =71.65 N/mm2 (MPa)

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 x 3.25 x 1000) = 2762.5 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/mm2, respectively.

Therefore, allowable stress in compression for strut σac =57 N/mm2 (MPa)

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 mm2, rxx=ryy=33.6 mm, ruu=42.5 mm, rvv=21.4 mm

Therefore rmin=21.4 mm

Step 3: Slenderness ratio

Slenderness ratio

Step 4: Safe load

From IS:800-1984, allowable working stress in compression for the steel having yield stress as 260 N/mm2 σac =57.56 N/mm2 (MPa)

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

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 mm2, rxx=ryy=36.7 mm, ruu=46.3 mm, rvv=23.6 mm

Step3: Slenderness ratio

Effective length of strut is 2762.5 mm

Minimum radius of gyration rmin=23.6 mm

Slenderness ratio

Step 4: Safe load carrying capacity

From IS:800-1984 for l/r=117.055 and the steel having yield stress, fy=260 N/mm2, allowable working stress in compression

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/mm2, respectively.

Therefore, allowable stress in compression σac =64 N/mm2 (MPa)

Effective sectional area required

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 mm2, rxx=31.6 mm,

For angles having 10 mm distance back to back and long legs vertical ryy=27.5 mm

Therefore rmin=27.5 mm

Step 3: Slenderness ratio

Slenderness ratio

Step 4: Safe load

From IS:800-1984, allowable working stress in compression for the steel having yield stress as 260 N/mm2 σac =61.48 N/mm2 (MPa)

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.

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 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,

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 σbs= Allowable bending stress in the slab bases for all steels, it shall be assumed as 185 N/mm2 Consider a strip of unit width.

Along the xx-axis

Along the yy-axis

If Poison ratio is adopted as ¼ the effective moment for width D

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. 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 fy 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 N/mm2. The allowable bending stress in the slab base is 185 N/mm2(MPa).

Design:

Step 1: Area of slab base required

Axial load of column = 600 kN

It is assumed uniformly distributed under the slab

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

Area of slab base =(250+2a)(250+2b) mm2

Step 2: Projections of base plate

Let projections a and b are equal

Area of slab (250+2a)2 = 15 x 104. 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 x 390 = 1,52,100 mm2

Intensity of pressure from concrete under slab

Step 3: Thickness of slab base:

Thickness of slab

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/mm2. The allowable bending stress in the slab base is 185 N/mm2(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

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) mm2

Step 2: Projections of base plate

Let the projections a and b be equal.

Area of slab (250+2a)2 = 15 x 104. 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 x 390 = 1,52,100 mm2 Intensity of pressure from concrete under slab Step 3: Thickness of slab base:

Thickness of slab

Provide 16 mm thick slab base. The fastenings are provided to keep the column in position.

UNIT 5

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. 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. 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, 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 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 are also used as 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 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 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 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. 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:

Where, M = Bending moment

I = Moment of inertia

σ = 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. 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.

Where, Z is the section modulus (Z=I/ymax), ymax 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 ymax 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 y1 and y2 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 Z1=(I/y1) and Z2=(I/y2) 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 'Mr'. 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.

Mr=(C x Lever arm)=(T x Lever arm)

For the beam in equilibrium, the moment of resistance 'Mr' would be equal to the maximum bending moment 'M' at any section (Mr=M).

13.4. ALLOWABLE STRESS IN BENDING

The allowable bending stress, σbc 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 (½bf/tf) and (h/tw) 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 (σbc) or in tension σbt as below. σbc or $\sigma bt = 0.66$ fy

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/mm2 (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 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, σbc.

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 d1=Clear distance between the flanges l=Effective length of the compression flange (Table 13.1) ry=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 = K1 x mean thickness of the horizontal portion of the compression flange at the point of maximum bending moment where K1 = a co-efficient

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.

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 σbc calculated from the following formula.

K1= 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 K1 for different values of ψ.

K2= a coefficient to allow for the inequality of flanges, and depends on ω, 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 K2 for different values of ω.

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

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. 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 behaviour of the member in flexure (bending) should not exceed the value of maximum permissible shear stress, τvm found as follows.

τvm=0.45fy

Where fy 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.

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 τva.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,

τva= 0.4 fy

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.

1. 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 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, 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, lateral support is provided at each point of connection but main beam should still be checked between the two supports.

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, ($\sigma bc=\sigma bt=0.66fy$). 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, fy 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=tf) to the thickness of web used to be less than 2.00. Also the ratio of the depth of web d1 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).

In the beginning of design, the permissible bending stress in tension, σbt or in compression, σbc may be assumed as 0.66 fy. The bending compressive stress, σbc and the bending tensile stress, σbt 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/σbc)

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/ry) 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 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

Step 2: Permissible bending stress

It is assumed that the value of yield stress, f_y for the structural steel is 250 N/mm2 (MPa). The ratios (T/t_w) and (d_1/t_w) are less than 2.0 and 85 respectively. The maximum permissible stress in compression or tension may be assumed as $\sigma_{bc} = \sigma_{bt} = (0.66 \times 250) = 165$ N/mm2

Thank You

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