

Module – 1 Introduction

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1.1 Introduction

Structure is an assemblage of various structural components which resists deformation caused due to external loads. Structures are the means of transferring forces and moments.

Structure can be classified based on

- Size
- Shape or geometry
- Type of material
- Methods adopted for analysis and design.

The design of structure mainly includes

- Functional planning
- Structural design

Firstly, structure should serve the purpose for which it is intended and this can be achieved by proper functional planning which are carried out by architects.

Secondly, structure should have adequate strength in order to resist deformations caused due to external loads. Therefore, the structural designer should consider

- 1) The forces those are likely to act on the structure,
- 2) Should estimate the strength of the material,
- 3) Should design the cross-sectional dimensions of the structural components and
- 4) Should select the design methods that are adopted for design a structure.

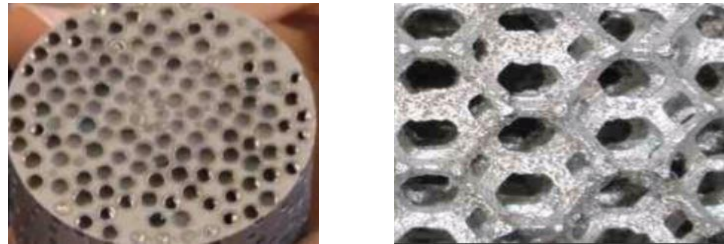
Steel structures are nothing but the assemblage of various structural steel components that are effectively joined or fastened to act as a single unit. When structural steel is used as a material for structures, structural design is known as “**Design of Steel Structures**”.

1.2 Objective

- To know the advantages and disadvantages of steel over other construction materials
- To know different types of structural steel products and their applications
- To understand the behavior of mild steel under stress.

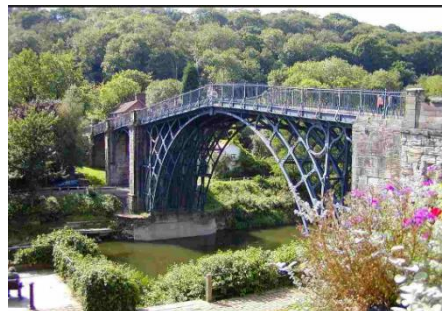
1.3 Historical Background

- Steel has been known since 3000BC.
- Ashoka pillar at Saranath was made with steel and iron joints are used in Puri Jaganath temples which are more than 1500 years old.
- In between 400-500BC, Foam steel was used in China & Europe.



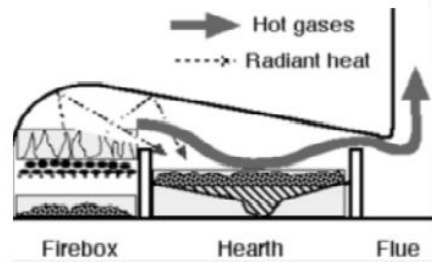
Note:- A metal foam is a cellular structure consists of solid metal aluminum or steel with gas filled pores comprising a large portion of volume which has low density and greater energy dissipation.

- Large scale use of iron for structural purpose was started in Europe in the later part of 18th century.
- Cast iron was obtained from its ore by melting it in furnace fired by charcoal.
- The first major application of cast iron was in 30.4m span Coalbrookdale Arch Bridge which was designed by Abraham Darby and it was constructed across the river Severn at England in the year 1779.



- Later Darby found out a way of converting coal to coke, which made some changes in iron making process.
- A later development in the process was the combination of limestone with impurities of ore and coke to form slag. The product obtained from this process was very brittle in nature and liable to cracks under strain.
- Due to this disadvantage, a person named Henry Cort invented a reverberatory furnace in the year 1784. This method reduces the carbon content in steel and they named the

product as Wrought iron which was stronger, flexible and has high tensile strength compared to cast iron.



Reverberatory Furnace

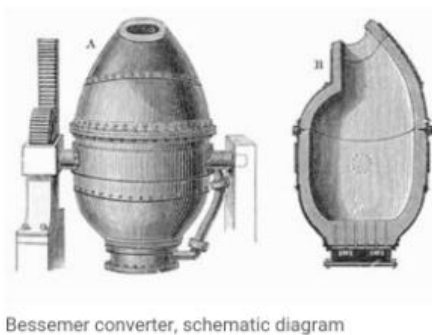
- Carbon content in case of cast iron may vary in the range of 2 to 4% and in case of wrought iron; it is less than 0.15%.
- During the year 1826, wrought iron chains were used in Menai Strait Suspension Bridge constructed across Menai Strait river which was designed by Thomas Telford.



- Britannia Bridge was the first Box- girder bridge made out of wrought iron material designed by Robert Stephenson and it was constructed across Menai Strait river in the year 1850.



- Structural steel was introduced in the year 1740, but it was not available in large quantities. Until Sir Henry Bessemer of England, invented and patented the iron making process in the year 1855.



- In the year 1865, Siemens and Martin invented an open-hearth process which was extensively used for manufacture of structural steel.



- Carbon content in case of structural steel may vary at about 0.25 to 1.5%.
- The first major application of structural steel was Fowler and Baker's Forth Railway Bridge which was constructed across Fifth of Fourth river at Scotland in the year 1890.



1.4 Steel as construction material

Steel is an alloy of iron and other elements primarily carbon, nickel, manganese, silicon, molybdenum etc., which is widely used material for building structures because of various properties such as,

- **High strength to weight ratio:-** Steel is 10 times stronger than concrete. Because of high strength to weight ratio, slender sections can carry heavy loads with possible reduction of dead load. Due to this reason, steel has become an affordable material which can be used for making long span structures like auditoriums, railway bridges (eg:- Howrah Bridge in Calcutta), high rise buildings (eg:- Eiffel Tower in Paris), electric transmission towers, industrial buildings, tanks, chimneys etc.
- **Good ductility**
- **Great stiffness:-** Steel deforms very less to external loads. Steel does not require any special considerations.
- **Easy fabrication:-** Steel allows itself to be worked easily in fabricating shops in various ways like bending, sawing, flame cutting and welding.
- **Low cost:-** Cheaper than other materials. Aluminum cost about 3 to 4 times than usual grade of steel.
- **Easy Erection:-** No need of formwork for erecting steel structures.

1.5 Structural steel

It is a category of steel which is used as a construction material for manufacturing structural steel shapes like beams, plates, channel sections, hollow sections, angles etc.

Types of structural steel

- **Carbon steel:-** Carbon and manganese are the two elements imparting strength to this steel.

Minimum yield strength $f_y = 230$ to 300MPa

Minimum tensile strength $f_u = 230$ to 300MPa

- **High strength carbon steel:-** If the carbon content increases in steel, strength increases but ductility, toughness and weldability of steel reduces. This type of steel is employed in transmission lines and microwave towers.

Minimum yield strength $f_y = 350$ to 400MPa

Minimum tensile strength $f_u = 480$ to 550MPa

- **Medium and high strength micro-alloyed steel:-** Even though the carbon content is low, strength is mainly due to additional micro-alloy like titanium, boron (up to 0.25%).

Minimum yield strength $f_y = 550$ to 700MPa

Minimum tensile strength $f_u = 440$ to 590MPa

- **High strength quenched and tempered steel:-** Steel was heated in order to develop high strength. Even though it tough, it requires some welding techniques.

Minimum yield strength $f_y = 550$ to 700MPa

Minimum tensile strength $f_u = 700$ to 950MPa

- **Weathering steel:-** These are low-alloy, corrosive resistant steels. Therefore no need to paint the surfaces.

Minimum yield strength $f_y = 350\text{MPa}$

Minimum tensile strength $f_u = 480\text{MPa}$

1.6 Properties of structural steel

Physical properties

Property	Value
1. Unit mass / density of steel (ρ)	7850 kg/m^3
2. Modulus of elasticity (E)	$2 \times 10^5 \text{ N/mm}^2$

3. Poisons ratio (μ)	0.3
4. Modulus of rigidity (G)	$0.769 \times 10^5 \text{ N/mm}^2$
5. Co-efficient of thermal expansion (α)	$12 \times 10^{-6} / ^\circ\text{C}$

Density:- defined as the ratio of mass by volume expressed in kg/m^3 or g/cm^3 .

Modulus of elasticity:- defined as the ratio stress to strain within elastic limit also know as young's modulus.

Poisons ratio:- defined as the ratio of lateral strain to axial strain.

Modulus of rigidity:- defined as the ratio of shear stress to shear strain and also called as shear modulus.

Co-efficient of thermal expansion:- defined as the ratio of unit change in length or volume to the unit change in temperature and expressed in F or $^\circ\text{C}$.

Mechanical properties

Property	Value
1. Yield stress (f_y)	220-450 N/mm^2
2. Ultimate or Tensile strength (f_u)	1.2 f_y
3. Percentage elongation	20
4. Notch toughness	-

Yield strength or yield stress:- defined as the minimum force required to cause permanent deformation in steel.

Ultimate or Tensile strength:- defined as the point at which permanent deformation takes place when the material is stretched or pulled along its length.

Percentage elongation:- defined as the ratio of increase in gauge length to original length expressed in percentage.

Notch toughness:- it is the ability of the material to absorb energy during rupture or fracture with the presence of notch or neck or flaws.

Other mechanical properties are

BHN (Brinell Hardness Number) = 150-190

VHN (Vickers Hardness Number) = 157-190

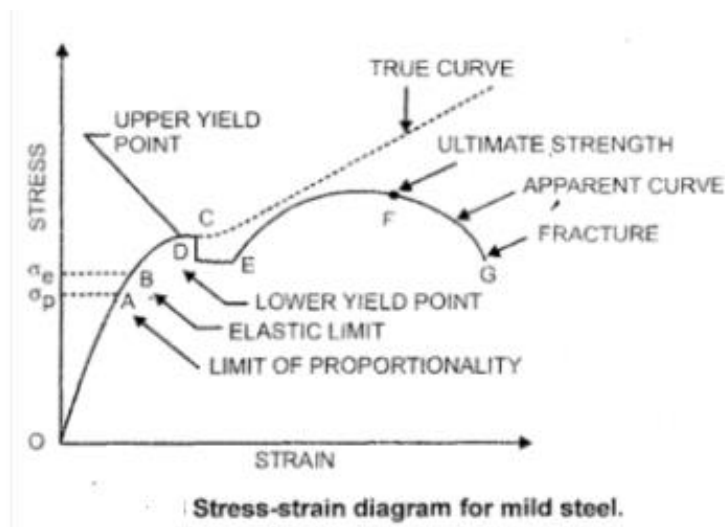
Approximate melting point = 1500°C

Thermal conductivity = 0.14cal/cm²

Chemical composition of structural steel

Grade	Designation	Carbon	Manganese	Sulphur	Phosphorous	Silicon
A	Fe410WA	0.23	1.5	0.05	0.05	0.4
B	Fe410WB	0.22	1.5	0.045	0.04	0.4
C	Fe410WC	0.20	1.5	0.04	0.04	0.4

1.7 Stress-Strain curve of mild steel



Stress-strain curve gives information to understand the behavior of material subjected to given loading condition.

OABCDEFG is a stress-strain curve of mild steel subjected to gradually increasing tensile load.

OA → represents a straight line, where stress is directly proportion to strain and it obeys Hooke's law.

A → represents limit of proportionality, stress beyond which linear variation ceases.

B → represents elastic limit, maximum stress up to which the material regains its original shape after removal of load.

AB → is not a straight line.

Stress beyond point B, material goes to plastic region and stress reaches to upper yield point C. At this stage material deforms and there will be reduction in cross-sectional area. Then stress reduces to a point D is known as lower yield point.

Between DE → when load remains constant, stress also remains constant. But there will be a considerable amount deformation.

From point E → strain hardening takes place, where the material starts to deform when it reaches yield point and becomes stronger. So addition stress is required to cause addition deformation of the material till it reaches to point F. This point is known as ultimate point and stress at this point is called ultimate strength.

At point F → necking takes place, where there will be a reduction of cross-section area of material at a rapid rate. Apparent stress reduces but true stress goes on increases until material breaks at point G and this pint is known as point of failure or fracture.

Failure of ductile material is cup and cone type and ultimate strength is calculated at 0.2% of maximum strain.

1.8 Structural steel products

- Flat hot rolled structural steel sections – plates, flat bars, sheets, strips etc.
- Hot rolled structural steel sections – hollow sections, rolled sections
- Cold rolled structural sections
- Bolts
- Welding electrodes

Rolled structural steel

They are two types 1) **Hot rolled structural steel** and 2) **Cold rolled structural steel**.

Hot rolled structural steel is a mill process in which rolling of steel at high temperature above recrystallization state where steel can be formed into shapes easily. This method is very cheaper and faster because there will be no delay in the process.

Cold rolled structural steel is a milling process in which steel is rolled at room temperature or below recrystallization state. This method increases the strength by 20% and having good surface finish with lesser dimensional tolerance.

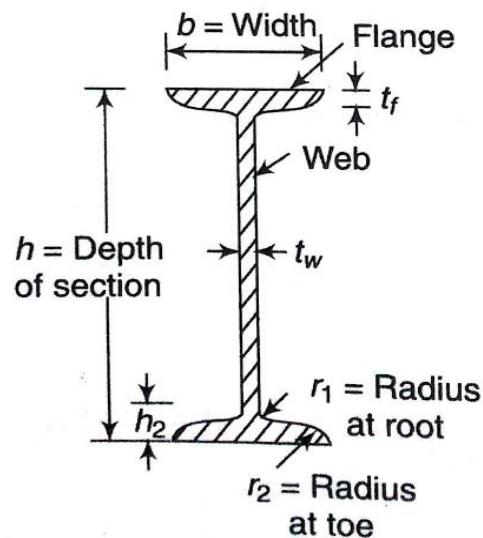
Hot rolled structural sections

➤ **I section:-**

These are used as columns and beams.

This section resists BM and SF; where 80% of BM will be resisted by flange and 20% will be resisted by columns. 95% of SF will be resisted by web and remaining will be resisted by flange.

Sometimes I sections with cover plates are used in order to resist large bending moments.



I sections are classified as

- 1) Indian Standard Joist/Junior Beams (ISJB)
- 2) Indian Standard Light Beams (ISLB)
- 3) Indian Standard Medium Beams (ISMB)
- 4) Indian Standard Wide Flange Beams (ISWB)

5) Indian Standard Heavy Beams (ISHB)

For example; ISMB 450@710.2N/m means ISMB with depth 450mm and having weight 710.2N per meter length.

ISMB, ISLB, ISJB are used as beams and ISWB & ISHB are employed as columns.

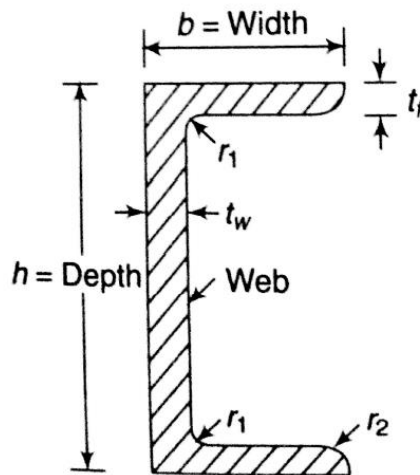
➤ Channel section:-

These are used as beams and columns.

Because of its shape, it allows to join angle sections to its web.

Built-up channels are conveniently used as columns.

Double channel members are used as bridge truss members.



Standard channel section are classified as

- 1) Indian Standard Junior Channels (ISJC)
- 2) Indian Standard Light Channels (ISLC)
- 3) Indian Standard Medium weight with sloping flange (ISMC)
- 4) Indian Standard Medium weight with Parallel flange (IMCP)

For example; ISMC [200@216.9N/m](#) means ISMC with height 200mm and weight 216.9N per meter length.

➤ Angle sections:-

Abbreviated as ISA followed by widths and thickness.

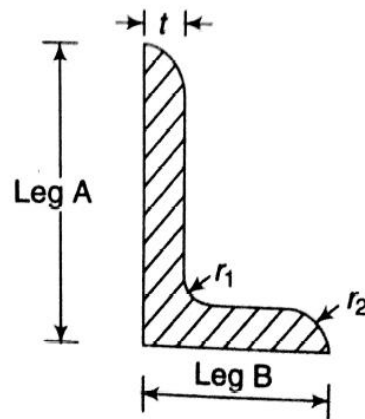
Available in equal and unequal sizes

Angles are used as connecting members for connecting structural elements.

Most employed in fabrication works.

Used as combination with plates to form plate girders.

Employed as tension and compression members of roof trusses.



Indian standard Angle sections are classified as

- 1) Indian Standard Equal Angles
- 2) Indian Standard Unequal Angles

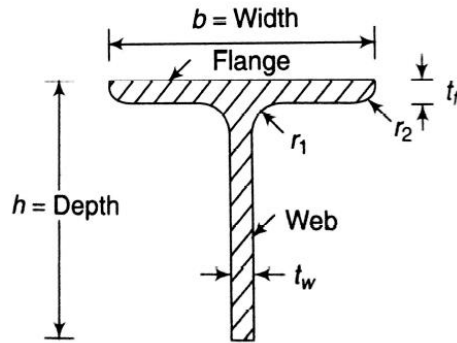
For example; ISA65X65X8 indicates an equal angle with legs of equal size 65mm and thickness 8mm.

ISA65X50X8 indicates an unequal angle with unequal leg widths with a thickness of 8mm.

➤ **T section:-**

Greater applications in furniture fabrication.

This type of section resists Bi-axial loadings.

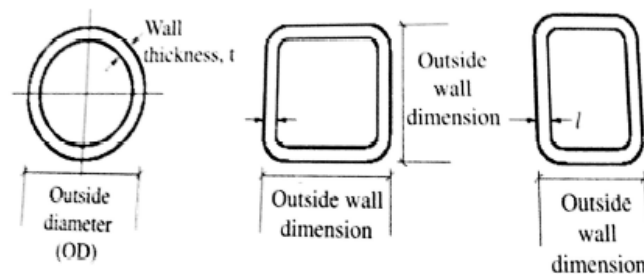


Indian standard T sections are classified as

- 1) Indian Standard Normal T bars (ISNT)
- 2) Indian Standard Long Legged T bars (ISST)
- 3) Indian standard Deep Legged T bars (ISDT)
- 4) Indian Standard Junior T bars (ISJT)
- 5) Indian Standard Light T bars (ISLT)
- 6) Indian Standard Wide T bars (ISWT)

➤ **Steel tubes:-**

These are used as scaffoldings, trusses, domes and columns.



The Structural Sections

➤ **Rolled steel flats:-**

Used for lacings in columns.

Used as ties.

Abbreviated with width followed by a letter F and thickness

For example; 50F8 where width ≤ 250 mm.

➤ **Plates:-**

Designated using symbol PL followed by length, width and thickness.

For example; PL200X1000X6mm where $T \geq 5\text{mm}$.

➤ **Steel sheets and strips:-**

Sheets are designated using symbol SH followed by length, width and thickness. $T > 5\text{mm}$

Strips are designated using symbol ST followed by width and thickness. $T < 5\text{mm}$

For example; SH2000X600X8 and ST200X3.

➤ **Square bars:-**

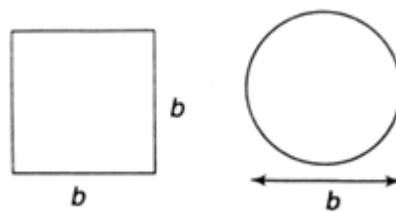
Steel bars of square section are made in various sizes from 5mm to 20mm and designated as ISSQ.

➤ **Round bars:-**

Round bars are used as reinforcements in RCC.

Solid circular bars are available in various from 5mm to 200mm.

Designated as ISRO.

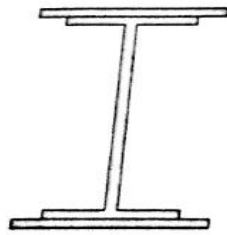


➤ **Built-up sections:-**

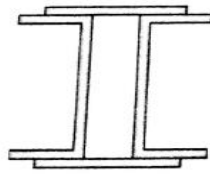
Rolled steel sections can be strengthened by welding cover plates on it.

Two rolled steel sections can be combined to resist loads in different directions

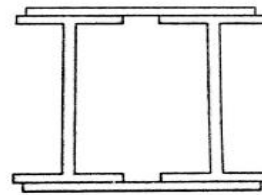
Two steel sections can be combined with lacing plates to act as single unit.



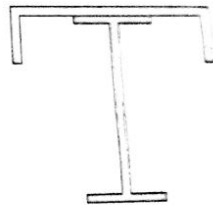
**I-Section With
Cover Plates**



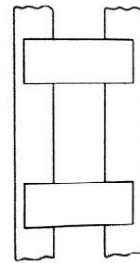
**Double Channel Section
With Cover Plates**



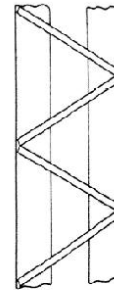
**Double I-Section
With Cover Plates**



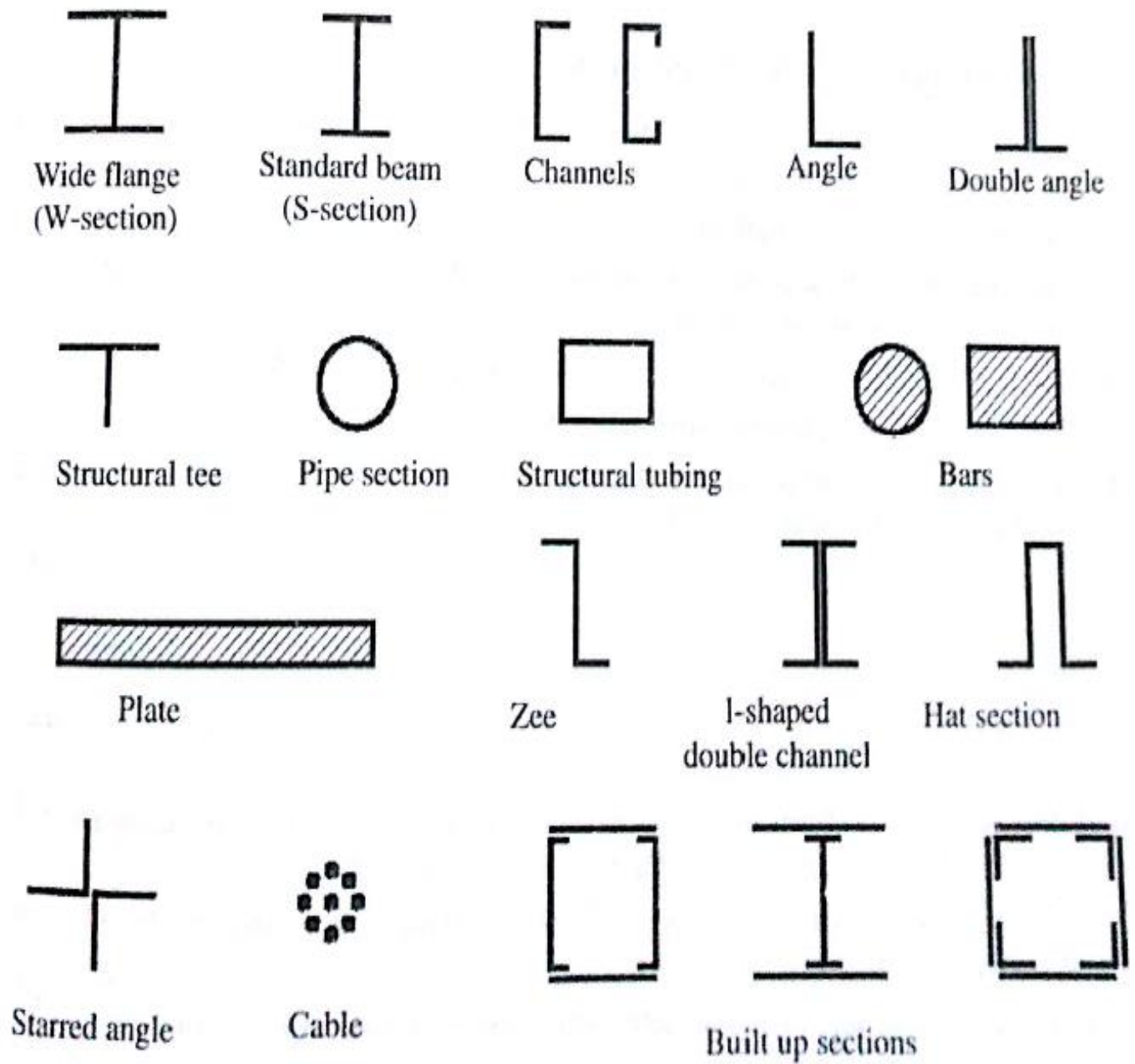
Crane Girder



Battened Member



Laced Member



Common structural shapes

1.9 Classification of cross-section

- 1) **Class I or Plastic section:-** This type of sections are subjected to bending moments equal to plastic moments and can undergo local rotation means plastic hinge is formed.

Plastic hinge refers to the deformation of part of beam wherever plastic bending takes place.

Plastic moment means moment at which entire cross section reaches to yield stress.

- 2) **Class II or Compact section:-** This type of sections are subjected to bending moments equal to plastic moment but cannot undergo local rotation.
- 3) **Class III or Semi-compact sections:-** This type of sections are subjected to bending moments equal to yield moments where cross section starts to buckle after outermost fiber yields.

Yield moment is the moment at which only exterior fibers reaches yield stress.

- 4) **Class IV or Slender sections:-** This type of section fails before yield moment is obtained.

1.10 Theories of failure / Failure criteria of steel sections

- 1) **Maximum stress theory (Rankin's theory):-** It states that, yield occurs when maximum principal stress reaches to uni-axial tensile stress.
- 2) **Maximum strain theory (St. Venant's theory):-** It states that, yield occurs when maximum principal tensile strain reaches uni-axial tensile strain at yield.
- 3) **Maximum shear stress theory (Guest's and Trescas's theory):-** It states that, yield occurs when shear stress reaches half of uni-axial tensile stress at yield.
- 4) **Distortion or shear strain energy theory (Von Mises theory):-** It states that yield occurs when shear strain energy reaches shear strain energy in simple tension.

1.11 Types of steel structures

Buildings:- These may include rigid, semi-rigid or simple connected frames, load-bearing walls, cable-stayed and cantilevered structures. Buildings may be single or multi-storeyed, single bay or multiple bays. Buildings are also classified according to use, such as residential, commercial, office, industrial etc. these buildings may include a steel frames or a steel roof supported by load-bearing walls.

Bridges:- Bridges may be classified as truss, plate-girder, arch, cantilever, cable-stayed or cable suspension. The truss and plate-girder bridges are commonly adopted for small to moderate spans and cable-stayed and suspension bridges for long spans. Bridges may be classified as railway, road, road-rail and pedestrian bridges depending upon use.

Towers:- Towers may be of different types, such as lighting towers, power transmission towers, observation towers, towers for radar and TV installation, telephone relay towers and windmill towers. Towers may be self-supporting or cable-stayed. Most towers are made of steel angles or tubes, which are bolted at site.

Storage tanks:- These may be rectangular, circular, or spherical. They can be used to store water or oil. They may rest on ground or be elevated on a staging.

Other structures:- Silos, bunkers, domes, folded plates, offshore platforms, chimneys, cooling towers.

1.12 Advantages of steel structures

- Steel members have high strength per unit weight. Therefore steel members of small section having less self-weight can able to carry heavy loads.
- It has assured quality and durability.
- Being light, steel members can be conveniently handled and transported. For this reason pre-fabricated products are frequently provided.
- Properly maintained steel structures give longer life. The properties of steel do not change with time. This makes the steel as most suitable material for a structure.
- Steel is a ductile material which does not fail suddenly, but gives visible signs of impending failure by large deflections.
- Speed of construction is the other advantage of steel structure. When pre-fabricated products are ready at the site can be erected easily. This saves the construction time.
- Additions and alterations can be done easily to steel structures.
- Steel has highest scrap value amongst all building material.
- If joints are taken care, it is the best water and gas resistant which can be used to store water.
- Material is reusable.

1.13 Disadvantages of steel structures

- They are costly.
- It is susceptible to corrosion.
- They require regular maintenance. So maintenance cost is high.
- They become more elastic when exposed to fire.

- Needs high degree of control to ensure proper fitting of various structural elements.

1.14 Applications of steel structures

- 1) **Construction:-** Because high strength to weight ratio, slender sections can carry heavy load and can be obtained at affordable price. So long span structures can be constructed at low cost.

Eg:- Many historical buildings like Empire State building primarily consists of steel as construction material. Steel also used in

- RCC
 - Auditoriums
 - Low and high rise buildings
 - Piers and suspension cables
 - Educational institutes and hospitals.
- 2) **Transportation:-** Used in Engineering and manufacturing sectors but bulk goes to transport vehicles. Different types of steel are used in car body, doors, engines, gear box, steering, wheel axels etc. Other than automotive market, steel is used to transport materials by trains, rails, ships, tracks and transmission lines.
 - 3) **Energy:-** Many sectors of power includes nuclear, natural gas, wind, electric needs steel for infrastructure. Many energy projects rely on steel for
 - Oil and gas wells and platforms
 - Electric transmission lines
 - Pipe lines
 - Transmission towers
 - Wind turbines
 - 4) **Packages:-** Used to protect from water, light and air exposure and fully recyclable. Used for packing food, beverages followed by general lines and closures. (Eg:- bottle caps)
 - 5) **Appliances:-** 75% of total weight of home appliances is from steel. Used in mixers, ovens, fridge, washing machine farm vehicles and machine tools etc.

1.15 Loads and load combinations

Loads:- For the purpose of safe designing a structure, it is essential to know what are the forces that are likely to act and the worst combination for which the structure should withstand during its life span.

- 1) **Dead load:-** These loads are nothing but self weight of the structure and this load is fixed in magnitude and in position. The load should be assumed as specified in IS 875(Part I) and checked after design is completed.
- 2) **Imposed load:-** These are also called as gravity loads other than dead load produced by occupancy of the structure. Imposed loads are different for different occupancy and should be selected as per IS 875(Part II). Crane load should be selected as per manufactures data. Snow load should be selected as per IS 875(Part IV). Different types of imposed are as follows

- | | |
|---------------|-------------------|
| a) Live load | e) Wave load |
| b) Crane load | f) Earth pressure |
| c) Snow load | |
| d) Dust load | |

Live loads:- these are also known as imposed loads which vary in magnitude or in position. Furniture loads, moving equipments, weight of persons in assembly halls/school, parts stored in ware house etc., are some of the examples for live loads.

- 3) **Wind loads:-** For the purpose of safe design of structure, wind loads should be adopted from the loops and tables gives in IS 875(Part III).
- 4) **Seismic loads:-** If the structure is situated in earthquake prone areas, seismic force should to considered for the safe design of structures. Earthquake force causes ground motion in both directions (ie. Horizontal and Vertical), which can create enormous damage to the buildings. As per IS 1893:2002, loads should be selected for the safe design of structures.
- 5) **Accidental loads:-** Accidentals loads like fire, explosion, impact or collusion should be selected as per IS 875(Part V).
- 6) **Errection loads:-** These are temporary loads, that are likely to act at the time of construction.

Load combination:-

Following are the load combinations considered for the safe design of structures.

DL	DL + IL + EL
DL + IL	DL + IL + TL
DL + WL	DL + WL + TL
DL + EL	DL + EL + TL
DL + TL	DL + IL + WL + TL
DL + IL + WL	DL + IL + EL + TL

DL – Dead load, IL – Imposed load, WL – Wind load, EL – Earthquake load, TL – Temperature load, SL – Snow load.

When snow load is deposited on roofs, replace IL by SL.

Structural Analysis:- It is necessary to find the internal forces developed in the members of structure. The required internal forces for design are 1) Axial force and 2) Moments.

Different types of structural analysis are as follows

1) Elastic Analysis:- No fiber of the members have yielded for the design load and stress is directly proportional strain.

I order analysis:- It is based on load acting on un-deformed shape of the structure.

II order analysis:- It is based on deformed shape of the structure.

2) Plastic analysis:- It is assumed that when every fiber of the section reaches the yield stress, a plastic hinge is formed. Member can freely rotate without resisting any additional moment.

3) Advanced analysis:- For frame which is fully laterally restrained, an advanced analysis is carried out.

4) Dynamic analysis:- Carried out in accordance with IS 1893 (Part I).

1.16 Design philosophy

The aim of the design is to decide the shape, size and connection details of the members and structure being designed should perform satisfactorily during its life span. Therefore, structure should

- Sustain all loads that are likely to act on it
- Sustain deformation during and after construction
- Adequate durability
- Resistance to fire
- Be stable and have alternate load path such that building should not collapse due to accidental loadings.

Design philosophies are listed below

- 1) Working Stress Method (WSM)
- 2) Ultimate Load Method (ULM)
- 3) Limit State Method (LSM)

- 1) Working stress method:-** This is the oldest, systematic and analytical design method. In this method stress is directly proportional to strain up to yield stress. Therefore, the stress caused due to loads should be checked against allowable or permissible stress which is a fraction of yield stress.

$$\text{Permissible stress} = \frac{\text{Yield stress}}{\text{Factor of safety}}$$

In working stress method, working stress $\leq$ permissible stress

Advantages of working stress method

- Method is simple
- As working stress are low, serviceability requirements are satisfied.

Disadvantages of working stress method

- This method provides un-economical sections.
- This method gives an idea that, factor of safety multiplied by working load gives failure load. But this not true.

- 2) **Ultimate load method:-** the limitation of working stress method in accessing the actual load carrying capacity leads to the development of ultimate load method or **load factor method**. This method is based on plastic analysis, means even when the fiber of the section reaches the yield, the member can resist load till sufficient plastic hinge is formed to develop a collapse mechanism.

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

Advantages of load factor method

- It allows the selection of load factor based on load combination.
- The redistribution of internal forces are accounted.

Disadvantages of load factor method

- No guarantee for serviceability of structure.

- 3) **Limits State Method:-** This method take care of both strength and serviceability requirements. Limit state is the condition in which structure is considered to be failed to serve the purpose for which it is intended.

- **Limit state of strength:-** Refers to the loss of equilibrium or stability of structure. It deals with strength, overturning, sliding, buckling, fatigue and brittle fracture.
- **Limit state of serviceability:-** It is an acceptable limit of performance of structure. It deals with discomfort to occupancy or malfunction caused by excessive deflection, vibration, cracking, corrosion, durability and fire.

Characteristic strength:- It is the strength of the material below which not more than prescribed percentage of test results are expected to fall (allowable % is 95%)

Characteristic yield strength:- It is the stress below which not more than 5% of test results are expected to fall.

Characteristic load:- It is the value of load not being exceed during the life span of structure. (Loads on the structure does not exceed 95% times the characteristic loads)

Partial safety factor is regarded as overloading factor for which structure is designed. Partial safety factor is different for different material and loads.

$$\text{Design actions/loads } (Q_d) = \sum_k \gamma_{fk} Q_{ck}$$

$$\text{Design strength } S_d = \frac{S_u}{\gamma_m}$$

- 1.17 Design considerations:-** Aim of the structural designer is to design the structure such that it should perform satisfactorily during its life time.

- **Safety:-** This is one of the factor for any structure. The structure may collapse not only due to expected loads and also due to accidental or impact loads and other possibilities such as expose to load exceeding load bearing capacity, overturning, sliding, buckling, fatigue and brittle fracture. Another aspect of safety is stability means stable in its equilibrium configuration.
- **Serviceability:-** Directly related to the performance of the structure under service loads should not cause discomfort for the user due to excessive deformation, vibration, crackling, fire, durability and corrosion. In some cases design may satisfy the safety criteria but not satisfies the serviceability criteria.
- **Economy:-** By increasing the design standards, cost of the structure also increases. For overall economy, one should look after the initial cost and maintenance cost.
- **Environment friendly:-** While selecting a material, one should look after the long-term environmental effects such as dismantling, maintenance, recyclability, repair and retrofitting.
- **Aesthetics:-** External appearance should be good after the construction of structure.

1.18 Codes and specifications

A structural engineer is often guided in his efforts by the code of practice. A code represents the varieties of opinions of engineers and professionals. It may not cover in detail every situation a designer may encounter. Often the designer must exercise his own judgment in interpreting and applying the requirements of code. It has to be noted that, by strictly following the codes may often give number of innovative designs.

Codes are basically written for the purpose of protecting public. They provide guidelines for the design and construction of structures. They are revised at regular intervals to reflect new developments in research, materials, construction techniques, experience gained from past design practice, behavior of existing structures and failure of structures.

Codes contain recommended loads for a given locality and recommended fire and corrosion protection. They also contain rules governing the ways in which loads are to be applied and design rules for concrete, steel and other materials.

The code should serve the following functions

- They ensure adequate structural safety, by specifying minimum requirements for the design.
- They ensure consistency among different engineers.
- They protect engineers from disputes, though codes may not provide legal protection.

In India, the Bureau of Indian Standards issues the codes and standard handbooks. Some of them are listed below

- a) **IS 800: 2007** – Code of practice for general construction in steel.
- b) **IS 875: 1987** – Code of practice for design loads for buildings and structures.
 - IS 875:1987 Part I – Dead loads
 - IS 875:1987 Part II – Imposed loads
 - IS 875:1987 Part III – Wind loads
 - IS 875:1987 Part IV – Snow loads
 - IS 875:1987 Part V – Special loads and load combinations.
- c) **IS 1893: 2002** – Criteria for earthquake-resistant design of structures.
- d) **IS 808: 1989** – Dimensions for hot-rolled steel beams, columns, channels and angle sections.

1.19 Structural Elements

A steel frame building consists of a skeleton frame which supports all the loads to which the building is subjected. The various elements of steel framed buildings are as follows.

- 1) **Beams and girders:-** These support vertical loads and are therefore subjected to bending moments and shear forces.
- 2) **Ties:-** These are the members subjected to axial tension.
- 3) **Struts, columns and stanchions:-** These are the members subjected to compressive loads. They may in addition be subjected to bending moments.
- 4) **Truss and lattice girders:-** These are framed members consisting of compression and tension members. These framed units carry lateral loads.
- 5) **Purlins:-** These are the members meant to support roof sheeting.
- 6) **Bracings:-** These are diagonal ties and struts connected to columns and roof trusses. They are meant to support wind loads. They provide stability to the building.
- 7) **Joints:-** Joints are provided to connect members.
- 8) **Bases:-** Bases are meant to transmit loads from column to foundation.

1.19 Assignment questions

- 1) What are the advantages and disadvantages of steel structures?
- 2) Explain the failure criteria's of steel structures.
- 3) Write a note on load and load combinations.
- 4) Explain the classification of cross section of elements?
- 5) What are the applications of steel structures?

1.20 Outcome

- Gives an idea about the stress strain behavior of mild steel
- Gives an idea about different structural steel products
- Gives an brief note on advantages and disadvantages of steel structures over other constructions materials.

1.21 Future study

- One can study the design methodology of each structural steel products which are necessary to build a structure.

(Unit - 4) Plastic Behavior of Structural Steel

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Plastic hinge concept

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Introduction

The traditional analysis of structures is based on the linear elastic behavior of materials, implying that the material follows Hooke's law (Stress is proportional to strain). It is also assumed that the deformations are small, implying that the original dimensions of the structure can be used in the analysis. This is also known as first order elastic analysis. (Cl. 4.4.2 pp - 24)

IS 800 - 2007 permits plastic analysis as per the Cl. 4.5 (pp 25 and 26). However, the requirements specified in Cl. 4.5.2 shall be satisfied unless otherwise specified.

- The yield stress of the grade of structural steel used shall not exceed 450 MPa.
- The stress - strain characteristics of steel shall comply with IS: 2062 to ensure complete plastic moment redistribution.
- The stress - strain diagram shall have a plateau at the yield stress level extending for at least six times the yield strain.
- The ratio of ultimate tensile stress to the yield stress for the specified grade of steel shall not be less than 1.2
- The percentage elongation shall not be less than 15 and the steel shall exhibit strain - hardening capabilities. (Steel conforming to IS: 2062 shall be deemed to satisfy the above requirements)
- The members shall be hot - rolled or fabricated using hot - rolled plates and sections.
- The cross section of the members shall be plastic (class 1 section) at plastic hinges and elsewhere at least compact sections. (class 2 section) Table 2 shall be followed in this regard.
- The cross section shall be symmetrical about the axis perpendicular to the axis of the plastic hinge rotation indicating that the beams shall be symmetrical about y-y axis and columns shall be symmetrical about both y-y and z-z axes.
- The members shall not be subjected to impact and fluctuating loading requiring fracture and fatigue assessment.

Objective

1. To understand the plastic theory
2. To understand the concept of plastic hinge concept
3. To understand the concept of plastic moment
4. To understand the concept of plastic collapse load

Stress - strain curves of structural steel

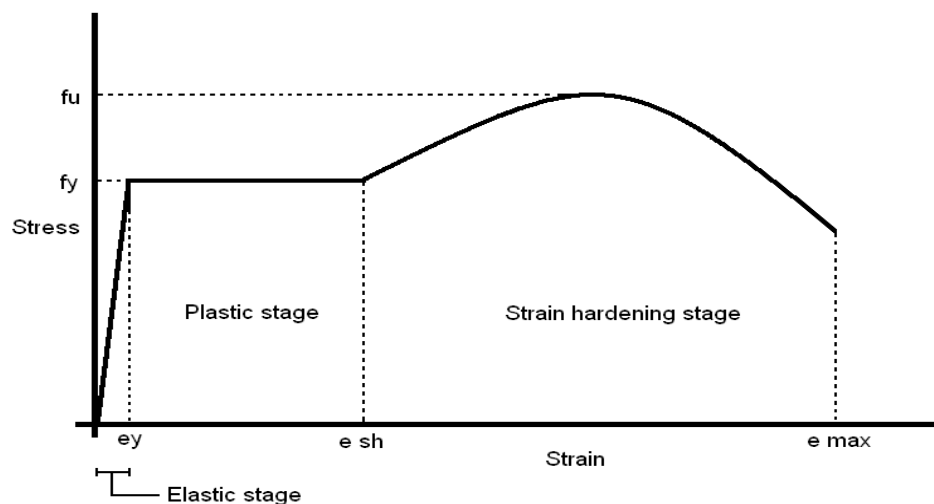
A typical stress - strain curve of steel conforming to IS: 2062 is shown in the figure. where,

f_y = yield stress in MPa e_y = yield strain

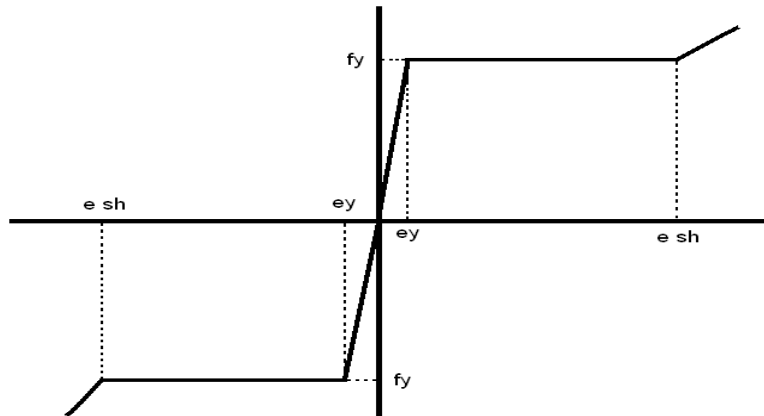
f_u = Ultimate stress in MPa e_{sh} = strain hardening strain e_{max} = ultimate strain

$e_{sh} = 6 * e_y$, $e_{max} = 180 * e_y$ and $f_u = 1.2 f_y$ (Typical)

From the stress - strain curve, steel yields considerably at a constant stress due to large flow of the material. This property known as ductility enables steel to undergo large deformations beyond the elastic limit without danger of fracture. This unique property of steel is utilized in plastic analysis of structures.



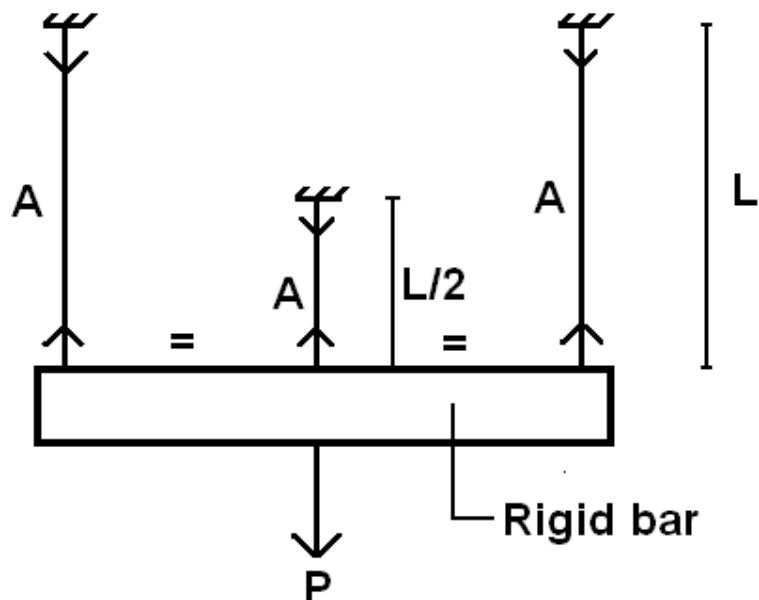
Stress - Strain Curve (Typical)



Perfectly Elasto - Plastic Material (Typical)

Calculation of failure loads in simple systems

Consider a three bar system shown below of length and area of C/S of each bar as indicated. E is the modulus of elasticity of the material.



Elastic analysis (Strength of Materials approach):

P_1 is the force in outer bars

P_2 is the force in the middle bar

Using $\sum V = 0$, (Vertical equilibrium equation)

$$2P_1 + P_2 = P \quad (1)$$

By compatibility, elongation of each bar is same -

$$P_1 L / AE = P_2 L / 2AE$$

from which

$$P_1 = P_2 / 2 \text{ or } P_2 = 2P_1 \quad (2) \text{ Substituting (2) in (1),}$$

$$2P_1 + 2P_1 = P \text{ or } P_1 = P/4 \text{ and } P_2 = P/2$$

In elastic analysis, as $P_2 > P_1$ the middle bar reaches the yield stress first and the system is assumed to fail.

$$P_2 = f_y A \text{ and } P_1 = f_y A / 2$$

$$\text{Yield load} = 2P_1 + P_2 = 2f_y A \text{ ----- Maximum load by elastic analysis}$$

Plastic analysis:

In plastic analysis, it will be assumed that even though the middle bar reaches the yield stress, they start yielding until the outer bars also reaches the yield stress. (Ductility of steel and redistribution of forces) With this, all the bars would have reached yield stress and the failure load (or ultimate load or collapse load) is given by

$$\text{Collapse load, } P_u = 2 f_y A + f_y A = 3f_y A \text{ ----- Maximum load by plastic analysis}$$

The collapse load calculated by plastic analysis is 1.5 times that of the elastic analysis. (Reserve strength) Plastic analysis can give economical solutions.

Plastic Theory of Beams:

The simple plastic theory makes use of the ductility of steel. (Large strain at collapse). The following assumptions are made in plastic bending of beams -

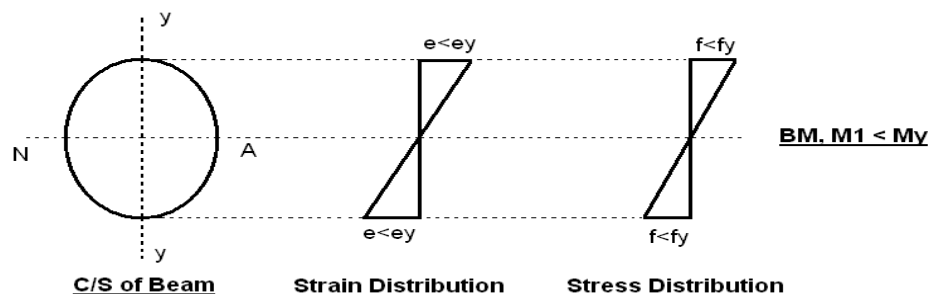
- Structural steel is a ductile material capable of deforming plastically without fracture.
- The material is homogeneous and isotropic obeying Hooke's law up to limit of proportionality (yield point) and then the stress is constant with increase in strain.
- The stress - strain curve can be represented by an ideal elasto - plastic material with properties of steel in compression and tension same. (yield stress and yield strain, modulus of elasticity etc.,)

- Cross - sections remain plane and normal to the longitudinal axis before and after bending. With this, the effect of shear force is ignored and the distribution of the strain across the depth of the c/s of the beam is linear.
- The effect of axial forces and residual stresses are ignored.
- The c/s of the beam is symmetrical about an axis parallel to the plane of bending. (y -y axis)
- Members are initially straight and instability does not develop before collapse occurs due to the formation of sufficient plastic hinges.
- Each layer of the beam is free to expand or contract independently with respect to the layer above or below it (each layer is separated from one another)
- Deformations are sufficiently small so that $\theta = \tan \theta$ can be used in the calculations of the collapse load.
- The connections provide full continuity so that plastic moment can develop and transmitted through the connections.
- Strain energy due to elastic bending is ignored.

Behavior of beam under an increasing BM:

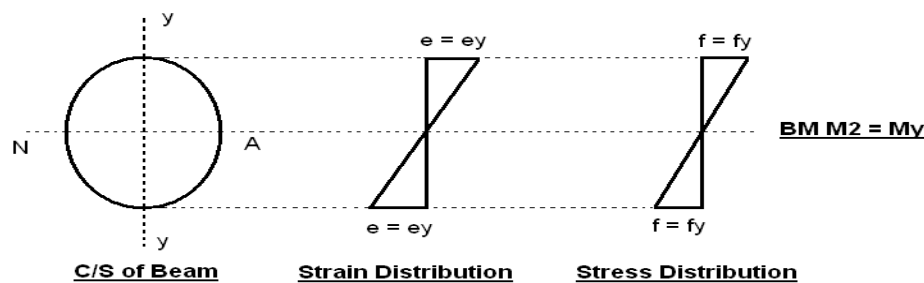
Consider a beam having a symmetrical C/S subjected to an increasing BM.

With BM $M_1 < M_y$ (yield moment) the stress and strain distributions across the depth will follow the elastic bending equation (Euler's - Bernoulli's equation) and is indicated in the figure below. All the fibres are stressed below the yield stress, f_y .

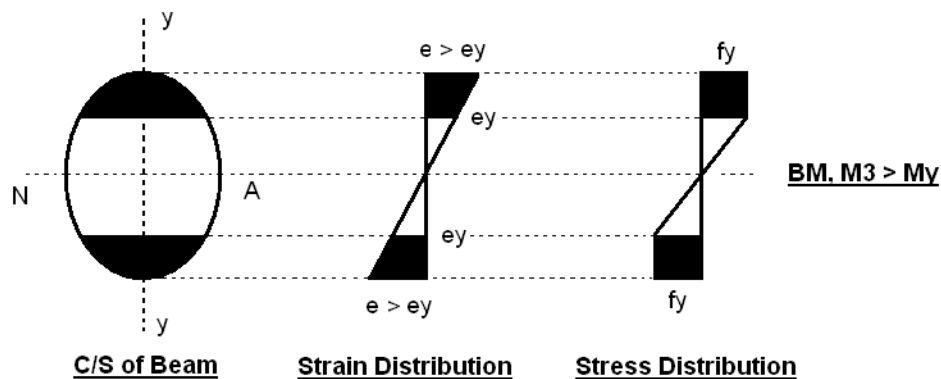


With BM, $M_2 = M_y$, the extreme fibres of the c/s begin to yield and the stress will be equal to f_y . The stress and strain distributions across the depth will still follow the elastic bending equation (Euler's - Bernoulli's equation) and is indicated in the figure below. All the fibres

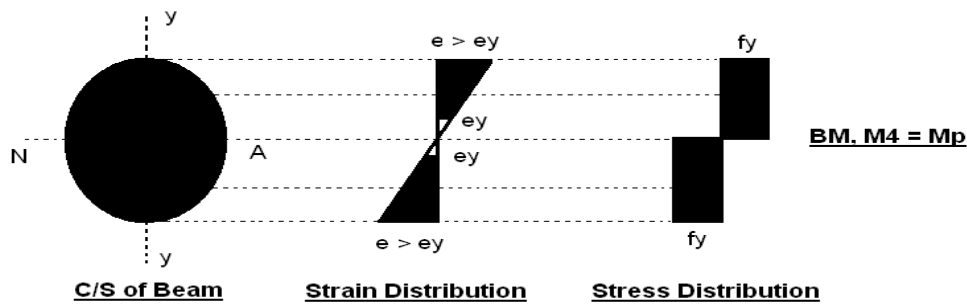
other than extreme fibres are stressed below the yield stress, f_y . The permissible stress (working stress) method is based on this behavior with suitable factor of safety.



With further increase in BM, $M_3 > M_y$, yielding spreads into inner fibres. The c/s is elasto - plastic and the internal moment of resistance can be computed, if required from the stress distribution shown below.



Finally, the yielding of fibres spreads almost for the entire c/s and the BM, M_4 at this stage is called the plastic moment M_p . The c/s is said to be fully plasticized. A plastic hinge is formed at this stage. After this, the deflection increases rapidly resulting in collapse or failure. The neutral axis, NA is now called the Equal Area Axis. An idealized stress distribution is shown below.



Basic Definitions: Plastic Moment, M_p :

It is defined as the moment of resistance of a fully plasticized or yielded c/s. The entire c/s is under yield stress, f_y .

Mathematically, $M_p = f_y Z_{pz}$

f_y is the yield stress of the material

Z_{pz} is the plastic section modulus of the c/s about z-z axis

Yield Moment, M_y :

It is defined as the moment of resistance of a c/s whose extreme fibres only has reached yield stress, f_y .

Mathematically, $M_y = f_y Z_{ez}$

f_y is the yield stress of the material

Z_{ez} is the elastic section modulus of the c/s about z-z axis

Plastic hinge:

It is defined a point in a flexural member where full plastic moment has developed and is rotating at a constant moment, M_p . The following characteristics are observed with respect to a plastic hinge :

- A plastic hinge is a zone of yielding in a flexural member, where the stress is equal to f_y throughout the c/s.
- The length of the plastic hinge depends on the type of loading, c/s and support conditions. However, in the analysis, it is assumed as point where all the plastic rotation takes place.

- At the point, where plastic hinge is formed, it will be rotating with a constant plastic moment, M_p
- Plastic hinges are formed at points where concentrated loads are acting, fixed or continuous supports in an indeterminate beam, joints in a rigid frame and at points of zero shear, when UDL or other varying loads are acting.
- Plastic hinge is formed in a member of lesser capacity, when two members meet as in the case of a continuous beam.
- The first plastic hinge is formed at the point of maximum BM.
- The formation of plastic hinges allow redistribution of moments until plastic hinges are formed at all the critical sections.
- The formation of sufficient number of plastic hinges convert the structure into a mechanism where the given structure breaks into rigid links with large deformations. (Collapse)

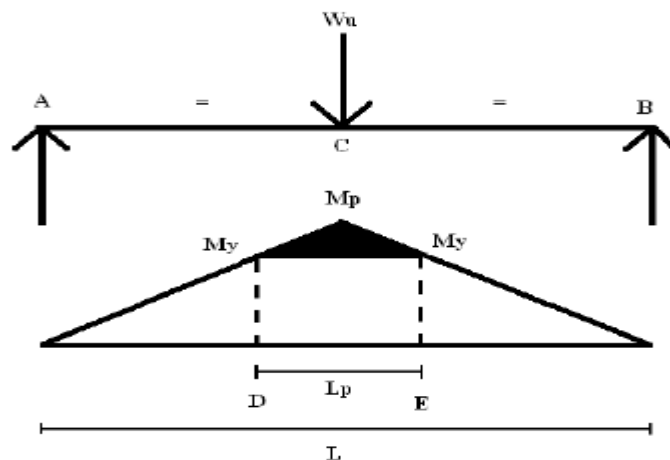
Length of plastic hinge:

It is the length in a beam over which the BM is greater than the yield moment, M_y

Problems

Example 1

Consider a simply supported beam subjected to a UDL which has yielded over the shaded area as shown. Let L_p and L be the length of the plastic hinge and span of the beam.



Consider a simply supported beam subjected to a central concentrated load which has yielded over the shaded area as shown. Let L_p and L be the length of the plastic hinge and span of the beam.

From similar triangles, $L_p / L = (M_p - M_y) / M_p$

$$= (1 - M_y / M_p)$$

$$= [1 - 1 / (M_p / M_y)]$$

$$= (1 - 1/S)$$

Where $S = (M_p / M_y)$ is called the shape factor

$$\text{Hence, } L_p = (1 - 1/S) L$$

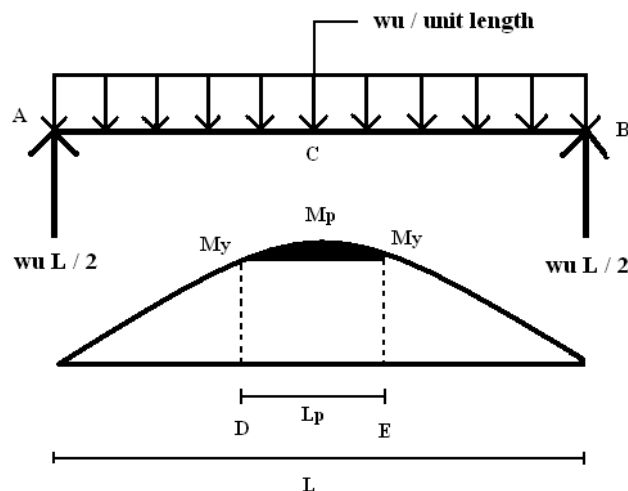
For a rectangular section, $S = 1.5$ and $L_p = 1/3 * L$

For typical I - sections, $S = 1.14$ and $L_p = 0.123 L$, say $1/8 * L$

Length of plastic hinge depends on the shape factor of the c/s

Example 2

Consider a simply supported beam subjected to a UDL which has yielded over the shaded area as shown. Let L_p and L be the length of the plastic hinge and span of the beam.



$$\text{Let } x = L_p / 2$$

$$M_y = w_u L / 2 * (L / 2 - x) - w_u (L / 2 - x)^2 / 2$$

$= w_u L^2 / 4 - (w_u L / 2) * x - w_u / 2 * (L^2 / 4 + x^2 - L x)$ Cancelling $(w_u L / 2) * x$, we have

$$M_y = w_u L^2 / 8 - w_u x^2 / 2$$

$$= w_u L^2 / 8 - w_u L^2 / 8 * (4 x^2 / L^2)$$

Noting $M_p = w_u L^2 / 8$, we have

$$M_y = M_p (1 - 4 x^2 / L^2)$$

$$M_y / M_p = (1 - 4 x^2 / L^2)$$

$$1 / S = (1 - 4 x^2 / L^2)$$

$$4 x^2 / L^2 = 1 - 1 / S$$

$$x^2 = L^2 / 4 (1 - 1 / S)$$

$$x = L / 2 \sqrt{(1 - 1 / S)}$$

$$L_p / 2 = L / 2 \sqrt{(1 - 1 / S)} \quad L_p = L \sqrt{(1 - 1 / S)}$$

For a rectangular section, $S = 1.5$ and $L_p = 0.577 * L$

For typical I - sections, $S = 1.14$ and $L_p = 0.35 * L$

Length of plastic hinge depends on the shape factor of the c/s

Note:-

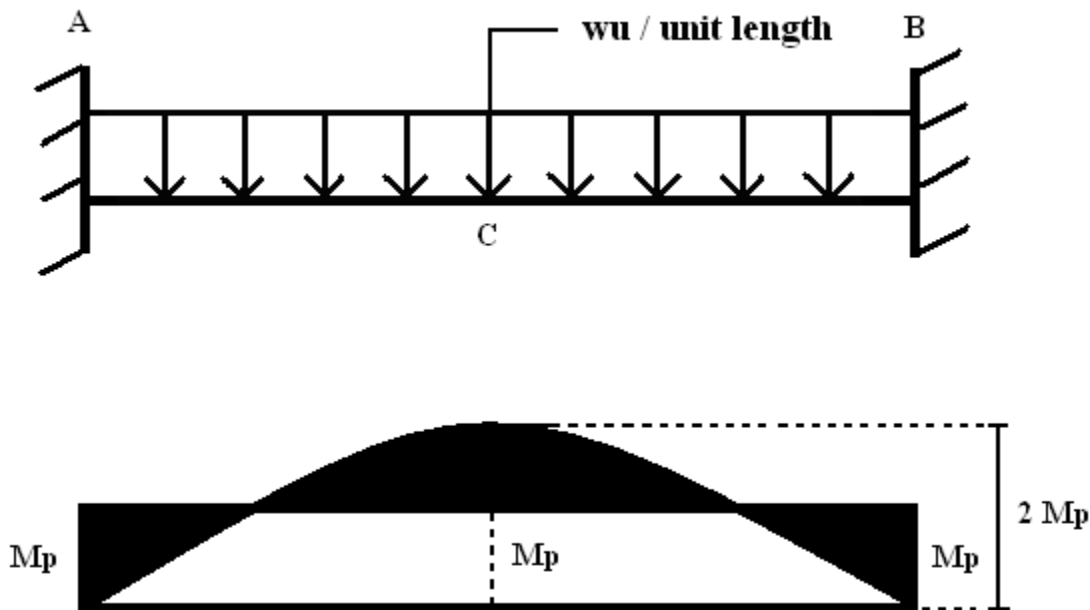
From the above examples, it is observed that the length of the plastic hinge is dependent on the loading and shape factor. It is also dependent on the support conditions. For propped cantilevers, fixed beams, continuous beams and rigid frames, it is still difficult to calculate the length of plastic hinges for different loadings and supports. Hence, it is customary to assume the plastic hinge in the analysis as a point where all the plastic rotation takes place.

Redistribution of Moments:

In determinate structural systems like simply supported beams, cantilevers and overhang beams, only one plastic hinge will be formed at the point of maximum BM and the structure is assumed to collapse. The increase in load carrying capacity is only marginal and depends on the shape factor. However, in indeterminate structural systems like propped

cantilevers, fixed beams, continuous beams and rigid frames, there is further increase in load carrying capacity due to the redistribution of moments. This is called the reserve strength of the beam. This is mainly due to the property of ductility of steel.

Redistribution of moments is explained with respect to a fixed beam subjected to UDL throughout. The beam is shown in the figure below.



The maximum elastic BM occurs at the supports and is equal to $wL^2 / 12$. As a first step, the plastic hinges are formed at the supports and complete plasticization of the support c/s takes place. But this does not mean failure or collapse occurs. This is because a plastic hinge at the centre should also be formed. (No. of plastic hinges in indeterminate structures = SI + 1) At this stage, the plastic hinges formed at the supports will be rotating at constant moment, M_p due to ductility of the material. Collapse of the beam occurs only when complete plasticization occurs at the centre resulting greater load carrying capacity.

At collapse, using equilibrium method, we have

$$w_u L^2 / 8 = 2 M_p$$

$$w_u = 16 M_p / L^2 \text{ ----- Collapse load for the beam}$$

At first yield, $M_y = w_y L^2 / 12$ at the supports, (only extreme fibres will yield)

$$w_y = 12 M_y / L^2$$

$$\text{Reserve strength} = w_u / w_y = (16 M_p / L^2) / (12 M_y / L^2)$$

$$= 4 / 3 * M_p / M_y$$

$$= 4 / 3 * S$$

where S is the shape factor.

For typical I - sections, $S = 1.14$

$$\text{Reserve strength} = 4 / 3 * 1.14 = 1.52$$

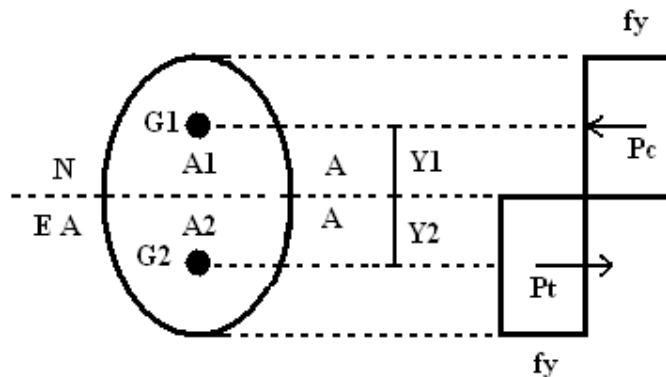
The beam is capable of carrying 52% more load beyond the first yield.

In general, increase in load carrying capacity occurs in plastic analysis due to - i) Plasticization of the section (formation of plastic hinges at critical sections)

ii) **Redistribution of moments**

Plastic Section modulus of a c/s:

Consider a c/s symmetrical about the vertical y-y axis as shown below with the section fully plasticized.



A_1 and A_2 are the areas above and below the Neutral Axis (Equal Area Axis)

f_y is the yield stress in the material

Compressive force above the NA = $f_y A_1$

Tensile force below the NA = $f_y A_2$

For equilibrium, the forces are equal -

$f_y A_1 = f_y A_2$ from which

$$A_1 = A_2$$

For a fully plasticized c/s, area above the NA = area below the NA

In other words, $A_1 + A_2 = A$ $A_1 + A_1 = A$ $A_1 = A_2 = A/2$

The neutral axis (NA) of a fully plasticized section is called the equal area axis (EAA)

The plastic moment of a fully plasticized section can be obtained by taking moments of the forces above and below the EAA about EAA.

$$M_p = f_y A_1 Y_1 + f_y A_2 Y_2$$

$$= f_y A/2 Y_1 + f_y A/2 Y_2$$

$$= f_y A/2 (Y_1 + Y_2)$$

$$= f_y Z_{pz}$$

where, Z_{pz} = plastic section modulus of the c/s about z-z axis = $[A/2 (Y_1 + Y_2)]$

The plastic section modulus of a c/s, Z_{pz} is defined as the moment of the areas above and below EAA about the EAA. It is the resisting modulus of a completely plasticized section and is a geometric property.

Shape factor, S:

It is defined as the ratio of plastic moment, M_p to yield moment, M_y . Also, called form factor.

Mathematically, $S = M_p / M_y$

$$= f_y Z_{pz} / f_y Z_{ez}$$

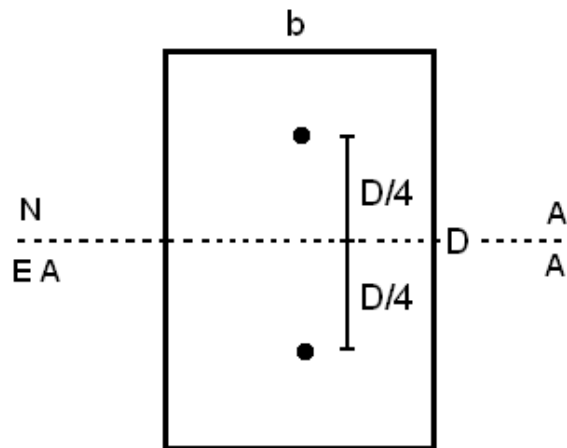
$$= Z_{pz} / Z_{ez}$$

It is also the ratio between the plastic section modulus to the elastic section modulus of a c/s and is a geometric property

Shape factor of simple sections:

1. Rectangle:

Consider a rectangle as shown



Elastic section modulus, $Z_{ez} = I_{NA} / \text{extreme fibre distance}$

$$= (bD^3/12) / (D / 2)$$

$$= bD^2 / 6$$

Plastic section modulus, $Z_{pz} = \text{Moments of the areas above and below the E A A (equal area axis) about the E A A.}$

$$Z_p = [(bD / 2) \times D/4] \times 2$$

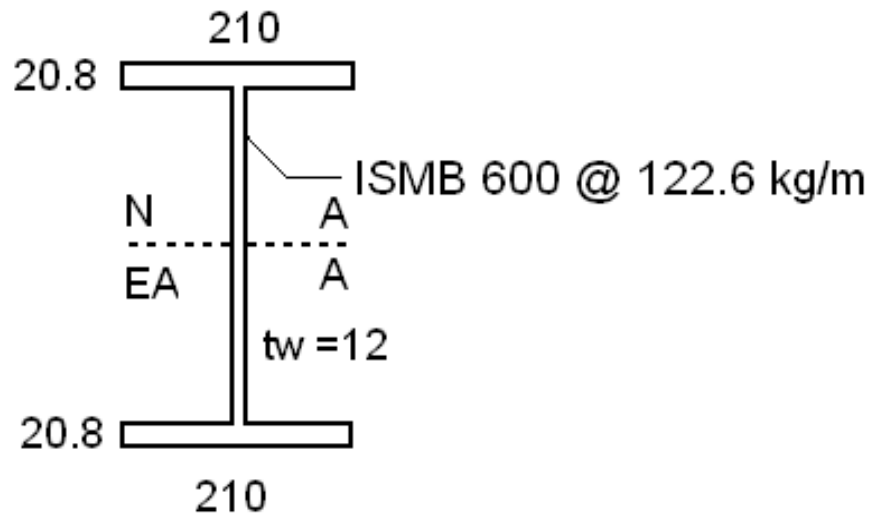
$$= bD^2 / 4$$

$$S = Z_{pz} / Z_{ez}$$

$$= 1.5$$

2. I - section, say ISMB 600 @ 122.6 kg/m:

Consider the I - section shown



$$z_{ez} = 3060.4 \text{ cm}^3 \text{ (From hand book; Table I, pp - 4,5)} = 3060.4 \times 10^3 \text{ mm}^3$$

$$z_{pz} = [210 \times 20.8 \times (600/2 - 20.8/2) + 12 \times (600/2 - 20.8) \times (600/2 - 20.8)/2] \times 2$$

$$= 3465.38 \times 10^3 \text{ mm}^3 \text{ (The value given in Table 46; pp - 138 is } 3510.63 \times 10^3 \text{ mm}^3 \text{)}$$

$$\text{Shape factor, } S = Z_{pz} / Z_{ez}$$

$$= 1.132 \text{ (The value given in Table 46; pp - 138 is 1.147)}$$

If the above c/s is used over an effective span of 8m, the permissible UDL on the beam is calculated as -

$$\text{Yield Moment, } M_Y = f_Y Z_{ez} \text{ (Assuming } f_Y = 250 \text{ MPa)}$$

$$= 250 \times 3060.4 \times 10^3$$

$$= 765.1 \times 10^6 \text{ N mm}$$

$$= 765.1 \text{ kN m}$$

$$\text{Plastic Moment, } M_P = f_Y Z_{pz}$$

$$= 250 \times 3465.38 \times 10^3$$

$$= 866.35 \times 10^6 \text{ N mm}$$

$$= 866.35 \text{ kN m}$$

Working bending moment, $M = M_p / 1.5 = 577.57 \text{ kN m}$

If the permissible UDL on the beam is w /unit length, $w L^2 / 8 = M$

$$w \times 8^2 / 8 = 577.57$$

$$w = 72.2 \text{ kN/m (including self weight)}$$

On similar lines, if central concentrated load is required, $W L / 4 + w_{\text{self weight}} * L^2 / 8 = M$

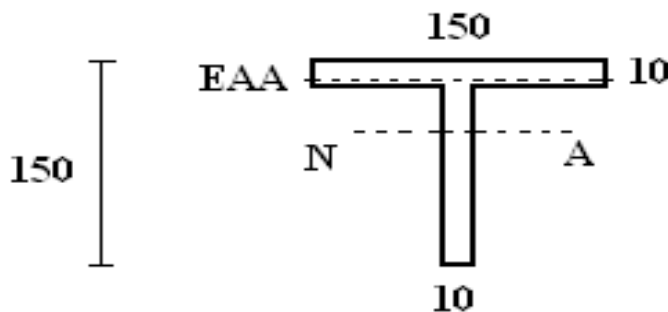
$$W * 8 / 4 + 1.23 \times 8^2 / 8 = 577.57$$

$$W = 283.87 \text{ kN}$$

If the c/s is symmetrical about the horizontal $z - z$ axis or NA, then both NA and EAA will co inside.

3. T - section say ISNT 150:

Consider the T section as shown



$$Z_{ez} = 54.6 \text{ cm}^3 \text{ (From hand book; Table VI pp - 18,19)}$$

The section is unsymmetrical about the NA and hence the EAA has to be located. Total area of the c/s = $29.08 \text{ cm}^2 = 2908 \text{ mm}^2$ (From hand book; Table VI, pp - 18,19) $A_1 = A_2 = A / 2 = 1454 \text{ mm}^2$

If EAA is at distance y from top, we have

$$150 * y = 1454 y = 9.69 \text{ mm}$$

The plastic section modulus is calculated by taking moments of the area above and below the EAA about EAA.

$$Z_{pz} = 150 * 9.69 * 9.69 / 2 + 150 * 0.31 * 0.31 / 2 + 10 * 140 * (140 / 2 + 0.31)$$

$$= 105.48 \times 10^3 \text{ mm}^3$$

$$(0.31 = 10 - 9.69 ; 140 = 150 - 10) S = Z_{pz} / Z_{ez} = 1.93$$

Yield Moment, $M_Y = f_Y Z_{ez}$ (Assuming $f_Y = 250 \text{ MPa}$)

$$= 250 \times 54.6 \times 10^3$$

$$= 13.65 \times 10^6 \text{ N mm}$$

$$= 13.65 \text{ kN m}$$

Plastic Moment, $M_p = f_Y Z_{pz}$

$$= 250 \times 105.48 \times 10^3$$

$$= 26.37 \times 10^6 \text{ N mm}$$

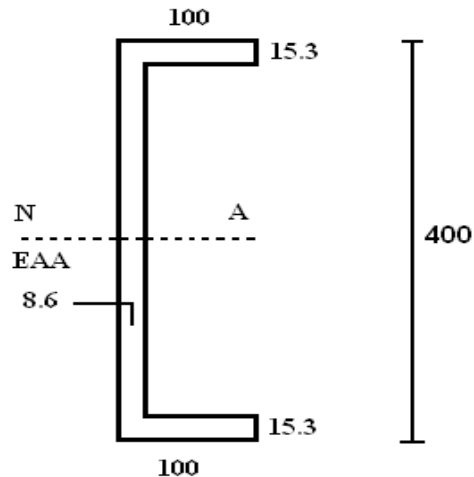
$$= 26.37 \text{ kN m}$$

Working bending moment, $M = M_p / 1.5 = 17.58 \text{ kN m}$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

4. Channel Section, say ISMC 400 @ 49.4 kg/m

Consider the channel section as shown.



The elastic properties of the section is obtained from Table II, pp 6 - 7 of ISI handbook [SP (6) - 1]

$$Z_{ez} = 754.1 \text{ cm}^3 = 754.1 \times 10^3 \text{ mm}^3$$

The beam is symmetrical about z - z axis and hence, NA and EAA co inside.

$$Z_{pz} = (100 * 15.3 * 192.35 + 8.6 * 184.7 * 184.7 / 2) * 2 = 881.97 \times 10^3 \text{ mm}^3$$

$$[192.35 = (400 / 2) - 15.3 / 2 ; 184.7 = (400 / 2) - 15.3]$$

$$S = Z_{pz} / Z_{ez} = 1.17$$

Yield Moment, $M_Y = f_Y Z_{ez}$ (Assuming $f_Y = 250 \text{ MPa}$)

$$= 250 \times 754.1 \times 10^3$$

$$= 188.53 \times 10^6 \text{ N mm}$$

$$= 188.53 \text{ kN m}$$

Plastic Moment, $M_P = f_Y Z_{pz}$

$$= 250 \times 881.97 \times 10^3$$

$$= 220.5 \times 10^6 \text{ N mm}$$

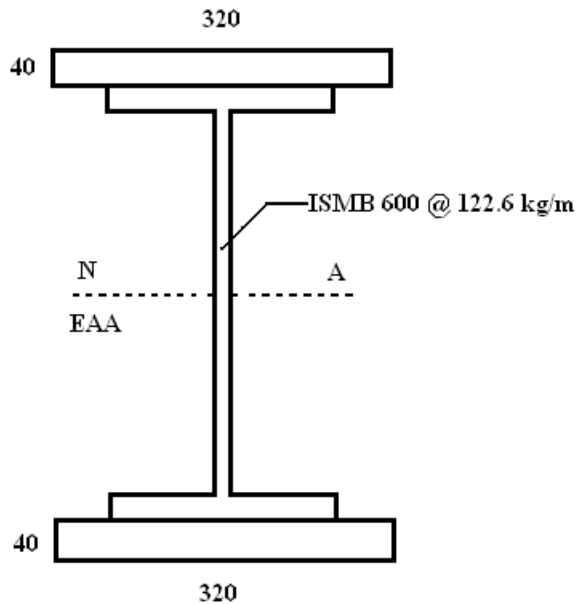
$$= 220.5 \text{ kN m}$$

Working bending moment, $M = M_P / 1.5 = 147 \text{ kN m}$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

5. Built up section, ISMB600 @ 122.6kg/m with 320 x 40 plates symmetrically placed

Consider the built up section as shown.



The elastic properties of the section is obtained from Table XIV, pp 54 - 55 of ISI handbook [SP (6) - 1]

$$Z_{ez} = 10420.5 \text{ cm}^3 = 10420.5 \times 10^3 \text{ mm}^3$$

The beam is symmetrical about z - z axis and hence, NA and EAA co inside

$$Z_{pz} = 3510.63 \times 10^3 + (320 \times 40 \times 320) \times 2$$

$$= 11702.63 \times 10^3 \text{ mm}^3$$

(The value of Z_{pz} of ISMB 600 @ 122.6kg/m is 3510.63 cm^3 obtained from Table 46 of IS : 800 - 2007 pp - 138; $320 = 600 / 2 + 40 / 2$)

$$S = Z_{pz} / Z_{ez} = 1.123$$

Mean thickness of flange = $(320 \times 40 + 210 \times 20.8) / 320 = 53.65 \text{ mm}$ (Table XIV gives mean thickness of flange = 53.3 mm)

(210 and 20.8 = width of flange and thickness of flange of ISMB 600 @ 122.6kg/m) Mean thickness > 40 mm

$f_y = 230 \text{ MPa}$ (Table 1 - IS: 800 - 2007, pp - 14)

Yield Moment, $M_Y = f_Y Z_{ez}$ (Assuming $f_y = 250 \text{ MPa}$)

$$= 230 \times 10420.5 \times 10^3$$

$$= 2396.72 \times 10^6 \text{ N mm}$$

$$= 2396.72 \text{ kN m}$$

Plastic Moment, $M_P = f_Y Z_{pz}$

$$= 230 \times 11702.63 \times 10^3$$

$$= 2691.6 \times 10^6 \text{ N mm}$$

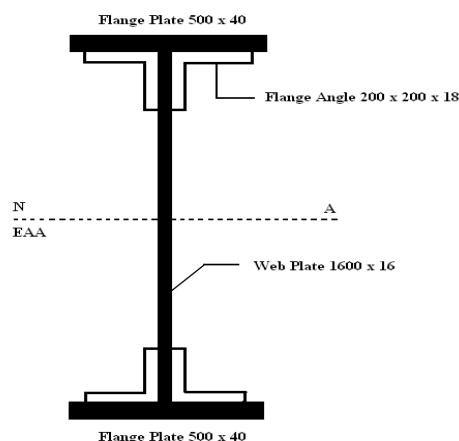
$$= 2691.6 \text{ kN m}$$

Working bending moment, $M = M_P / 1.5 = 1794.4 \text{ kN m}$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

6. Plate girder section, say Two 500 x 40 flange plates (one on each side), Four flange angles (two on each side) and One web plate 1600 x 16

Consider the section as shown.



The elastic properties of the section is obtained from Table XX, pp 84 - 85 of ISI

handbook [SP (6) - 1)]

$$Z_{ez} = 56782.9 \text{ cm}^3 = 56782.9 \times 10^3 \text{ mm}^3$$

The beam is symmetrical about z - z axis and hence, NA and EAA co inside

$$\begin{aligned} Z_{pz} &= (500 * 40 * 820 + 200 * 18 * 791 * 2 + 18 * 182 * 691 * 2 + 16 * 800 * 400) * 2 \\ &= 63485.26 \times 10^3 \text{ mm}^3 \end{aligned}$$

$$(820 = 1600 / 2 + 40 / 2 ; 791 = 1600 / 2 + 18 / 2 ; 182 = 200 - 18 ; 691 = 1600 / 2 - 18 - 182 / 2 ; 800 = 1600 / 2 ; 400 = 800 / 2) S = Z_{pz} / Z_{ez} = 1.118$$

$$\text{Mean thickness of flange} = (500 * 40 + 200 * 18 * 2) / 500 = 54.4 \text{ mm (Table XX gives}$$

$$\text{mean thickness of flange} = 55 \text{ mm) Mean thickness} > 40 \text{ mm}$$

$$f_y = 230 \text{ MPa (Table 1 - IS: 800 - 2007, pp - 14)}$$

$$\text{Yield Moment, } M_Y = f_y Z_{ez} \text{ (Assuming } f_y = 250 \text{ MPa)}$$

$$= 230 \times 56782.9 \times 10^3$$

$$= 13060.07 \times 10^6 \text{ N mm}$$

$$= 13060.07 \text{ kN m}$$

$$\text{Plastic Moment, } M_p = f_y Z_{pz}$$

$$= 230 \times 63485.26 \times 10^3$$

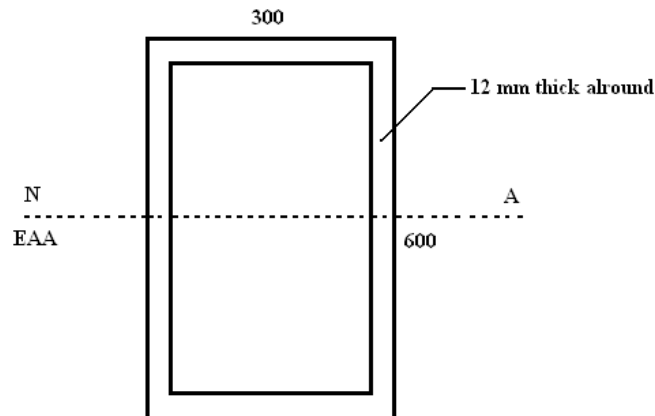
$$= 14601.61 \text{ N mm}$$

$$= 14601.61 \text{ kN m}$$

$$\text{Working bending moment, } M = M_p / 1.5 = 9734.41 \text{ kN m}$$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

6. Box section, say 300 x 600 outer dimensions with 12 mm as uniform thickness



Consider the section as shown.

The beam is symmetrical about z - z axis and hence, NA and EAA co inside

$$Z_{ez} = I_{zz} / y_{max}$$

$$= [300 * 600^3 / 12 - (300 - 12 * 2) * (600 - 12 * 2)^3 / 12] / (600 / 2)$$

$$= 3.35 \times 10^6 \text{ mm}^3$$

$$Z_{pz} = (300 * 12 * 294 + 12 * 288 * 144 * 2) * 2$$

$$= 4.11 \times 10^6 \text{ mm}^3$$

$$(294 = 600 / 2 - 12 / 2 ; 288 = 600 / 2 - 12 ; 144 = 288 / 2) S = Z_{pz} / Z_{ez} = 1.227$$

$$f_y = 250 \text{ MPa (Table 1 - IS: 800 - 2007, pp - 14)}$$

$$\text{Yield Moment, } M_Y = f_y Z_{ez} \text{ (Assuming } f_y = 250 \text{ MPa)}$$

$$= 250 \times 3.35 \times 10^6$$

$$= 837.5 \times 10^6 \text{ N mm}$$

$$= 837.5 \text{ kN m}$$

$$\text{Plastic Moment, } M_p = f_y Z_{pz}$$

$$= 250 \times 4.11 \times 10^6$$

$$= 1027.5 \times 10^6 \text{ N mm}$$

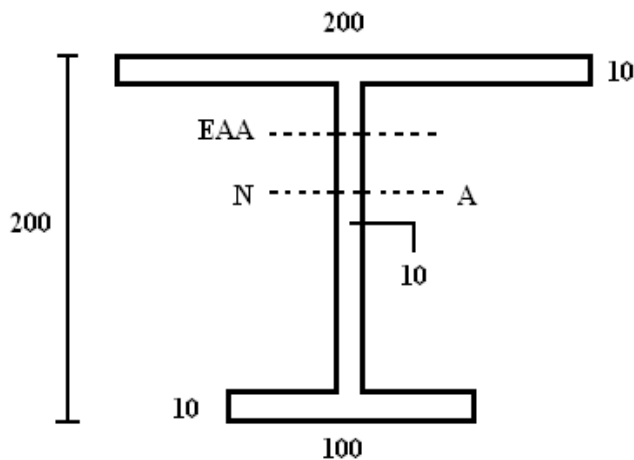
$$= 1027.5 \text{ kN m}$$

Working bending moment, $M = M_p / 1.5 = 685 \text{ kN m}$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

8. Unsymmetrical I section

Consider an Unsymmetrical c/s as shown



The section is unsymmetrical about the NA and hence the EAA has to be located. The NA has to be calculated as section properties are not available in SP (6)

Let y_t be the distance of NA from the top

$$y_t = [200 * 10 * 10 / 2 + 10 * 180 * (180 / 2 + 10) + 100 * 10 * (200 - 10 / 2)] /$$

$$\text{Total area, } A = (200 * 10 + 180 * 10 + 100 * 10)$$

$$= 80.21 \text{ mm}$$

$$y_b = \text{distance of NA from the bottom} = 200 - 80.21 = 119.79 \text{ mm}$$

$$I_{NA} = [200 * 10^3 / 12 + 200 * 10 * (80.21 - 5)^2] + [10 * 180^3 / 12 + 10 * 180 * (100 - 80.21)^2] + [100 * 10^3 / 12 + 100 * 10 * (195 - 80.21)^2]$$

$$= 30.08 \times 10^6 \text{ mm}^4$$

$$Z_{ez} = I_{NA} / y_{\max} = 30.08 \times 10^6 / 119.79 = 251.11 \times 10^3 \text{ mm}^3$$

Let the EAA be at distance y from the top

$$200 \times 10 + 10 \times (y - 10) = \text{Total area} / 2 = 4800 / 2 = 2400 \quad y = 50 \text{ mm}$$

$$Z_{pz} = 200 \times 10 \times 45 + 10 \times 40 \times 20 + 10 \times 140 \times 70 + 100 \times 10 \times 145$$

$$= 341 \times 10^3 \text{ mm}^3$$

$$S = Z_{pz} / Z_{ez} = 1.358$$

$$f_y = 250 \text{ MPa (Table 1 - IS: 800 - 2007, pp - 14)}$$

Yield Moment, $M_Y = f_y Z_{ez}$ (Assuming $f_y = 250 \text{ MPa}$)

$$= 250 \times 251.11 \times 10^3$$

$$= 62.78 \times 10^6 \text{ N mm}$$

$$= 62.78 \text{ kN m}$$

Plastic Moment, $M_p = f_y Z_{pz}$

$$= 250 \times 341 \times 10^3$$

$$= 85.25 \times 10^6 \text{ N mm}$$

$$= 85.25 \text{ kN m}$$

Working bending moment, $M = M_p / 1.5 = 56.83 \text{ kN m}$

If UDL or other loads are required, maximum BM which depends on the type of loading and span is equated to working bending moment.

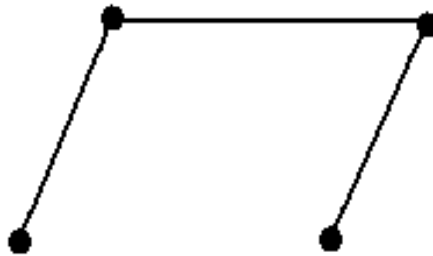
Mechanisms:

It can be defined as a situation in a structure in which sufficient number of plastic hinges are formed so as to cause collapse without further increase in the load carrying capacity. (collapse load) Mechanism breaks the structure into rigid links. When a mechanism is formed the structure is subjected to large rotations and deflections.

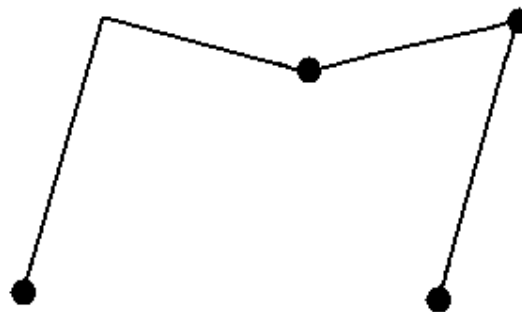
In an indeterminate structure, number of plastic hinges = SI makes the structure determinate and additional plastic hinge transforms the structure into a mechanism resulting in collapse. Mechanism results in instability of the structure with large deformations. The type of mechanism depends on the structure. Different types of mechanisms are indicated in the figures.



Beam Mechanism



Sway Mechanism



Combined Mechanism

4.7 Plastic Collapse load

It is the load corresponding to the collapse of a structure due to the formation of sufficient number of plastic hinges transferring the structure into a mechanism. It is also called ultimate load or limit load. The number of plastic hinges at collapse = $SI + 1$, where SI = Static indeterminacy of the structure.

The collapse of a structure can be a partial, complete or over complete collapse depending on number of plastic hinges

In a structural system, if only a part of the structure fails, making the structure as a whole useless, then the collapse is said to be partial and the corresponding load is taken as the partial collapse load. Number of plastic hinges in a partially collapsed structure is

$< SI + 1$. In rigid frames, if a beam fails, then it is called a partial collapse.

In a structural system, if complete collapse of the structure occurs, then the corresponding collapse load is taken as the true collapse load. Number of plastic hinges at collapse is $= SI + 1$. Usually, the value of the collapse load corresponds to any one mechanism which gives the lowest load.

In a structural system, if collapse of the structure occurs due to formation of more than the necessary plastic hinges, then the collapse is considered as over complete collapse and the corresponding load is said to be over complete collapse load. Number of plastic hinges at collapse is $> SI + 1$. Two or more mechanisms give the same collapse load. This can happen in symmetrically loaded structures, where the plastic hinges do not lie on the axis of symmetry.

4.8 Conditions in Plastic Analysis:

In elastic analysis of structures, the conditions to be satisfied are: Equilibrium conditions - $\sum V = 0$; $\sum H = 0$ and $\sum M = 0$

Compatibility conditions - Continuity of slopes and deflections

$BM < \text{Yield Moment}$ with suitable factor of safety

In plastic analysis of structures, the conditions to be satisfied are : Equilibrium conditions - $\sum V = 0$; $\sum H = 0$ and $\sum M = 0$

Mechanism condition or formation - Sufficient number of plastic hinges shall

be formed to transfer the structure into a mechanism

Plastic moment condition - BM at any point shall not exceed the plastic moment, M_p of the section ($M < \text{or} = M_p$)

If all the three conditions are satisfied in plastic analysis, then we get the true collapse load. However, if any two conditions are satisfied, we get a load which is either below or above the true collapse load.

4.9 Theorems in Plastic analysis:

Based on the number of conditions satisfied, we have three theorems in plastic analysis.

Static Theorem (Lower bound theorem):

This theorem states that the load computed from any distribution of BMDs in equilibrium with external loads (safe and statically admissible BMD) so that the maximum BM in any member shall not exceed its plastic moment, M_p ($M < \text{or} = M_p$) is less than or equal to the true collapse load. This theorem leads to equilibrium or static method of plastic analysis.

This theorem also called the safe theorem satisfies equilibrium and plastic moment condition.

Kinematic Theorem (Upper bound theorem):

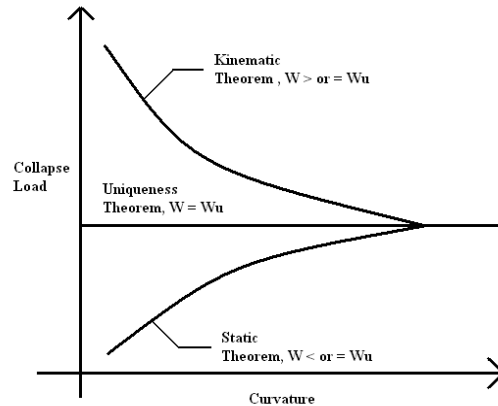
This theorem states that the load computed from any assumed kinematically admissible mechanism is greater than or equal to the true collapse load. This theorem leads to kinematic or mechanism method of analysis.

This theorem also called the unsafe theorem satisfies equilibrium and mechanism condition. A kinematically admissible mechanism (deformation) is one in which the deformation (rotation and deflection) under the load and supports satisfies the virtual work equation. (internal work by the plastic moment at plastic hinges = external work by the loads on the structure)

Uniqueness Theorem:

This theorem states that if the load evaluated by static and kinematic theorems is same, then it is the true collapse load. All the three conditions of plastic analysis are satisfied. According to this theorem, there is only one unique solution for a given structure, while there are innumerable possible solutions with other theorems.

All the three theorems are graphically shown in the figure below.



Methods of Plastic Analysis:

Equilibrium or Static Method:

This method is based on the lower bound theorem and consists of the following steps:

- i) The redundant forces are chosen.
- ii) Free BMD (Simply supported) of the structure is drawn.
- iii) Redundant BMD of the structure is drawn.
- iv) The two BMDs are combined and the peak moments are determined. The peak moment points are set the corresponding plastic moment, M_p . Also, check the number of plastic hinges formed.
- v) The collapse load is determined using the equilibrium condition by equating the maximum simply supported BM to the plastic moment at the corresponding point.
- vi) A check is applied so that plastic moment at any point is not $>$ or $= M_p$ at that point.
- vii) BMD and SFD are drawn if required.

Kinematic or Mechanism method:

This method is based on the upper bound theorem and consists of the following steps:

- i) The possible points of plastic hinges (N) formation is located. They are invariably formed at points of maximum BM, Continuous supports, Fixed supports, rigid joints, etc.,
- ii) The number of independent mechanisms and combined mechanisms are determined.

In beams, only independent mechanisms are required. No. of independent mechanisms = $N - SI$, where N = No. of plastic hinges possible and SI = Static indeterminacy

iii) Using the principle of virtual work, the equilibrium equation involving the internal work done by plastic moment, M_p and the external work done by loads are equated to determine the plastic collapse load.

iv) A check is applied so that plastic moment at any point is not $>$ or $= M_p$ at that point. v) BMD and SFD are drawn if required.

In this method, the elastic strain energy is ignored and the internal work done at only plastic hinges is considered.

Virtual Work Principle:

This principle is used for determining the collapse loads using mechanism method. This principle establishes the equilibrium relation between the work done by external loads and the internal work done by the plastic moment at specified location of plastic hinges. The principle is stated as follows:

If a structure is subjected to a set of loads which is in equilibrium, a virtual displacement (deflection or rotation) at specified points will not alter the equilibrium of the structure with total work done being zero. With this, the virtual work equation can be written as -

External Work done by the loads = Internal Work done by the plastic moment the location of the plastic hinges.

Plastic analysis of beams:

Beams can be analyzed by equilibrium or mechanism method. Usually, beams are subjected to transverse loads and only beam mechanisms occur. In continuous beam, each span is converted into a mechanism separately and lowest collapse load or the highest plastic moment is taken as the true value. It will be assumed that if one span collapses, the structure fails.

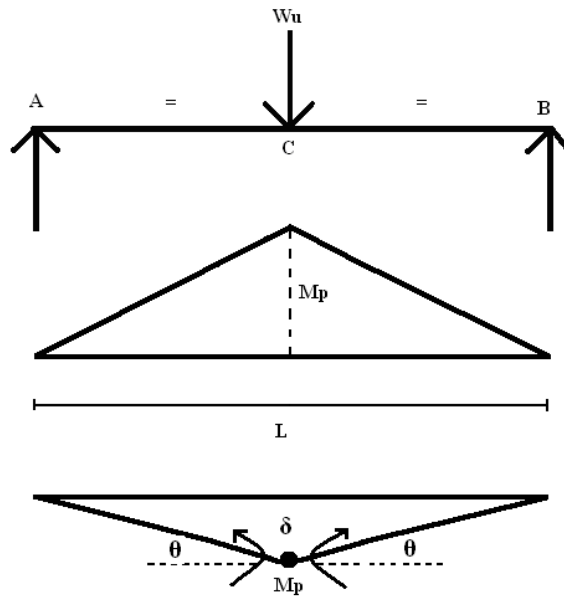
In beams, mechanisms can be relevant or irrelevant. Relevant mechanisms are the correct mechanisms, where plastic hinges are assumed at maximum BM locations such as fixed support, continuous support, concentrated loads, point of zero shear etc., Irrelevant mechanisms are incorrect mechanisms, where plastic hinges are assumed to be at incorrect locations. In analysis, only relevant mechanisms are considered.

No. of relevant mechanisms = $N - SI$ N = No. of plastic hinges possible SI = Static indeterminacy

4.10 Problems on Plastic moment

Problem 1

Simply supported beam with central concentrated load -



Equilibrium method -

Maximum free BM = $W_U * L / 4$

Plastic moment of the section = M_p

Equating, $W_U * L / 4 = M_p$

$$W_U = 4 * M_p / L$$

Mechanism method -

Internal work done = $2 * M_p * \theta$ External work done = $W_U * \delta$ From geometry, $\delta = (L / 2) * \theta$

Equating, $2 * M_p * \theta = W_U * (L / 2) * \theta$

$$W_U = 4 * M_p / L$$

Problem 2

Propped Cantilever with Central concentrated load

Equilibrium method -

Maximum free BM = $W_U * L / 4$

Plastic moment of the section = $M_p + M_p / 2 = 1.5 M_p$

Equating, $W_U * L / 4 = 1.5 M_p$

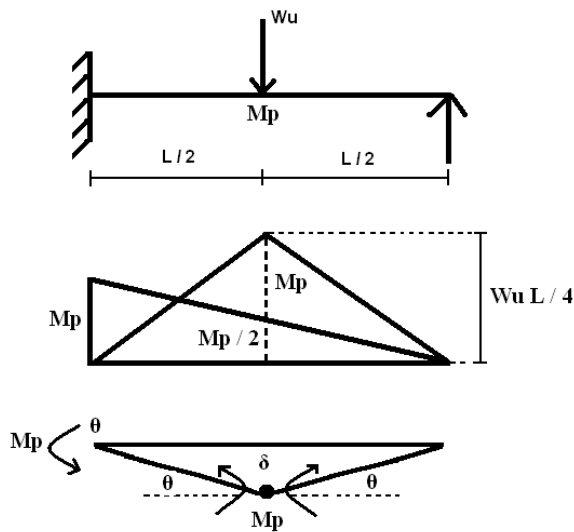
$$W_U = 6 * M_p / L$$

Mechanism method -

Internal work done = $3 * M_p * \theta$

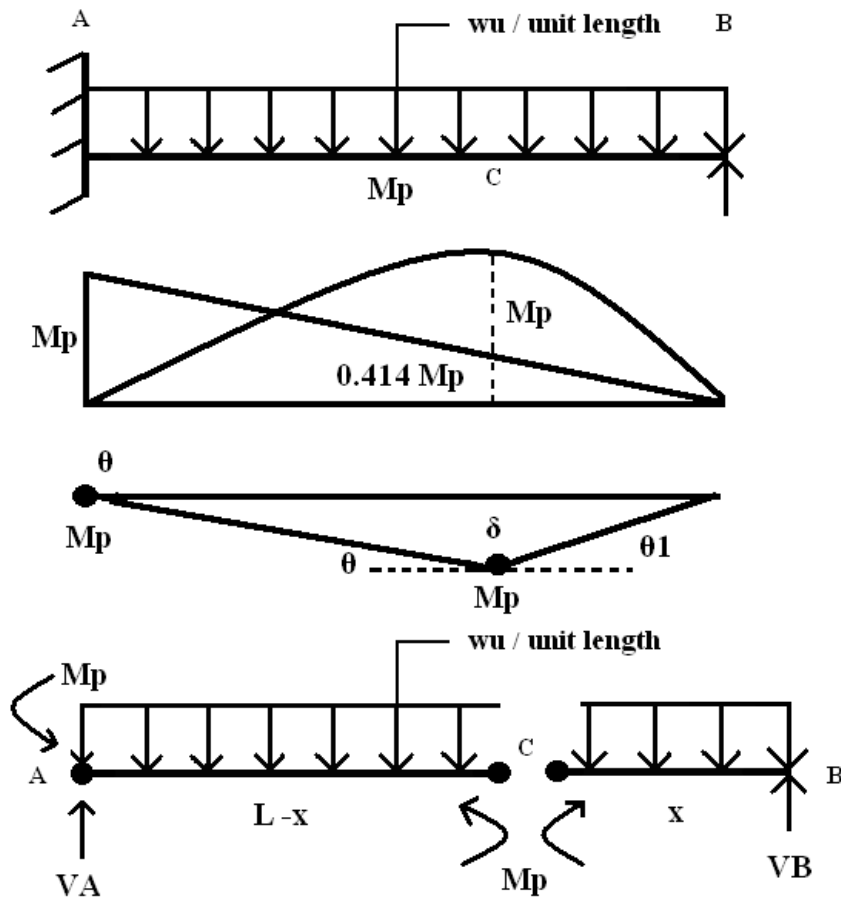
External work done = $W_U * \delta$

From geometry, $\delta = (L / 2) * \theta$ Equating, $3 * M_p * \theta = W_U * (L / 2) * \theta$ $W_U = 6 * M_p / L$



Problem 3

Propped Cantilever with UDL throughout



The location of the plastic hinge at C is carried out by segment equilibrium method. Let the hinge at C be at distance x from B.

BC = x and AC = $(L-x)$ At C, SF = 0

Consider the segment BC,

$$\Sigma M_B = 0,$$

$$M_p - w_u x^2 / 2 = 0$$

$$w_u = 2 M_p / x^2 \text{ ----- (1)}$$

Consider the segment AC,

$$\Sigma M_A = 0,$$

$$w_u (L-x)^2 / 2 - 2 M_p = 0$$

$$w_u = 4 M_p / (L-x)^2 \text{ ----- (2)}$$

Equating (1) and (2),

$$2 M_p / x^2 = 4 M_p / (L-x)^2$$

Simplifying, we get $x^2 + 2Lx - L^2 = 0$

$$x = -L + \sqrt{2} * L$$

$$x = 0.414 L \text{ (BC) and } L - x = 0.586 L \text{ (AC)}$$

$$w_u = 11.66 M_p / L^2$$

Equilibrium method -

Simply supported BM at 0.414 L from B

$$= w_u L / 2 * 0.414L - w_u * (0.414L)^2 / 2$$

$$= 0.1213 w_u L^2$$

$$\text{Plastic moment of the section} = M_p + 0.414 M_p = 1.414 M_p$$

Equating,

$$1.414 M_p = 0.1213 w_u L^2$$

$$w_u = 11.66 M_p / L^2$$

Mechanism method -

$$\text{Internal work done} = 2 M_p \theta + M_p \theta_1$$

External work done = $0.5 * L * \delta * w_u$ (Area under the mechanism * intensity of loading)

From geometry,

$$\delta = 0.586L \theta = 0.414L \theta_1$$

Equating,

$$2 M_p \theta + M_p \theta_1 = 0.5 * L * \delta * w_u$$

$$2 M_p \theta + M_p * 1.415 \theta = 0.5 * L * 0.586L \theta * w_u$$

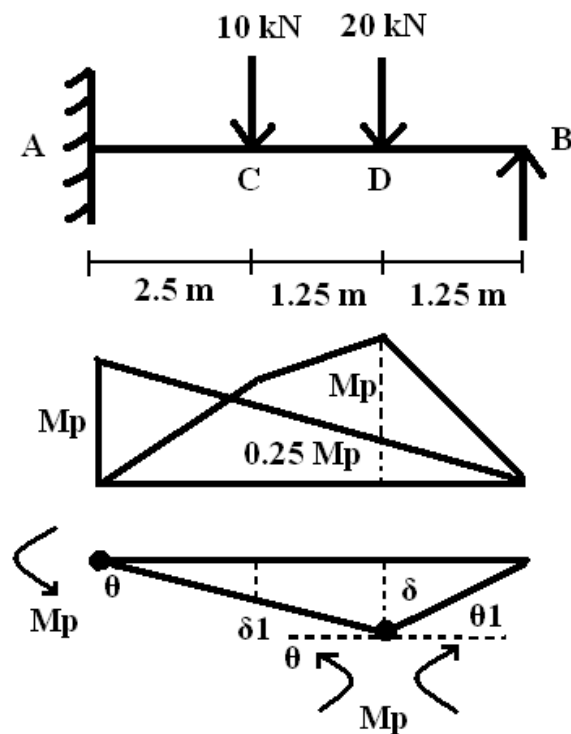
$$3.415 M_p = 0.293 w_u L^2$$

$$w_u = 11.66 M_p / L^2$$

SFD if required can be drawn either in terms of M_p or w_u

Problem 4

Propped Cantilever with two concentrated loads at collapse. Find M_p



Two plastic hinges can be formed at collapse. No. of possible plastic hinges = 3 (A, C and D)
Two Relevant mechanisms are possible.

Equilibrium method - Plastic hinges at A and C

$$\text{Maximum free BM} = 10 * 2.5 = 25 \text{ kNm}$$

$$\text{Plastic moment of the section} = M_p + 0.5 M_p = 1.5 M_p$$

$$\text{Equating, } 25 = 1.5 M_p$$

$$M_p = 16.67 \text{ kNm}$$

Plastic hinges at A and D

$$\text{Maximum free BM} = 20 * 1.25 = 25 \text{ kNm}$$

$$\text{Plastic moment of the section} = M_p + 0.25 M_p = 1.25 M_p$$

$$\text{Equating, } 25 = 1.25 M_p$$

$$M_p = 20 \text{ kNm}$$

Adopt the largest value of $M_p = 20 \text{ kNm}$ for design. Mechanism method -

$$\text{Internal work done} = 2 * M_p * \theta + M_p * \theta_1$$

$$\text{External work done} = 20 * \delta + 10 * \delta_1$$

$$\text{From geometry, } \delta = 3.75 \theta, \delta = 1.25 \theta_1, \theta_1 = 3 \theta \text{ and } \delta_1 = 2.5 \theta$$

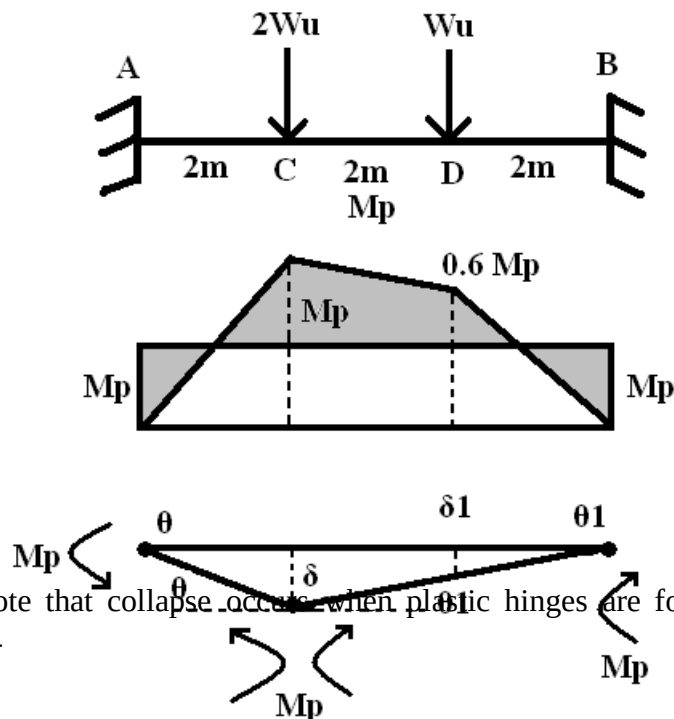
$$\text{Equating, } 2 * M_p * \theta + M_p * \theta_1 = 20 * \delta + 10 * \delta_1$$

$$2 * M_p * \theta + M_p * 3 \theta = 20 * 3.75 \theta + 10 * 2.5 \theta$$

$$M_p = 20 \text{ kNm}$$

Problem 5

Calculate the collapse load for the fixed beam shown with $M_p = 100 \text{ kNm}$



By inspection, we note that collapse occurs when plastic hinges are formed at A, C and D.
Equilibrium method -

Plastic hinges at A, B and C

$$\text{Maximum free BM at C} = (10 / 6) W_U * 2 = (10 / 3) W_U$$

$$\text{Plastic moment of the section} = M_P + M_P = 2 M_P$$

$$\text{Equating, } (10 / 3) W_U = 2 M_P = 200 \text{ kNm}$$

$$W_U = 60 \text{ kN}$$

Mechanism method -

$$\text{Internal work done} = 2 * M_P * \theta + 2 * M_P * \theta_1$$

$$\text{External work done} = 2 W_U * \delta + W_U * \delta_1$$

$$\text{From geometry, } \delta = 2 \theta, \delta = 4 \theta_1, \theta_1 = 0.5 \theta \text{ and } \delta_1 = 2 \theta_1 = \theta$$

Equating internal and external work done,

$$2 * M_P * \theta + 2 * M_P * \theta_1 = 2 W_U * \delta + W_U * \delta_1$$

$$2 * M_P * \theta + 2 * M_P * 0.5 \theta = 2 W_U * 2 \theta + W_U * \theta$$

$$3M_P = 5 W_U$$

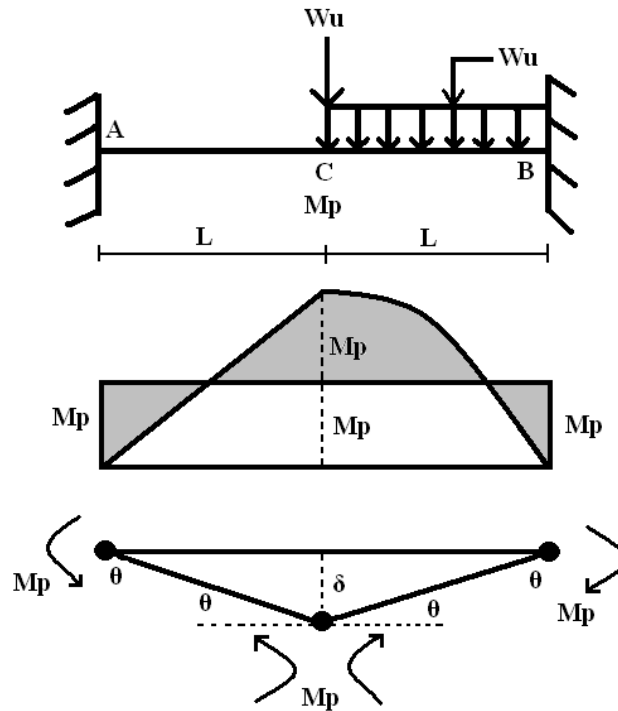
$$W_U = 0.6 M_P = 60 \text{ kN}$$

The vertical reactions will be as in the case of simply supported beam.

Problem 6

Calculate the collapse load for the fixed beam shown.

The plastic hinges are formed at A , B and C based on crossing of shear force at C



The Simply supported reactions at A and B are $1.25W_u$ and $0.75W_u$ respectively.

Equilibrium method -

Plastic hinges at A, B and C

Maximum free BM at C = $0.75 W_u * L$

Plastic moment of the section = $M_p + M_p = 2 M_p$

Equating, $0.75 W_u * L = 2 M_p$

$W_u = (8 / 3) M_p / L$

Mechanism method -

Internal work done = $4 * M_p * \theta$

External work done = $W_u * \delta + 0.5 * L * \delta * W_u / L$

From geometry, $\delta = L \theta$

Equating internal and external work done,

$$4 * M_p * \theta = W_U * \delta + 0.5 * L * L \theta * W_U / L$$

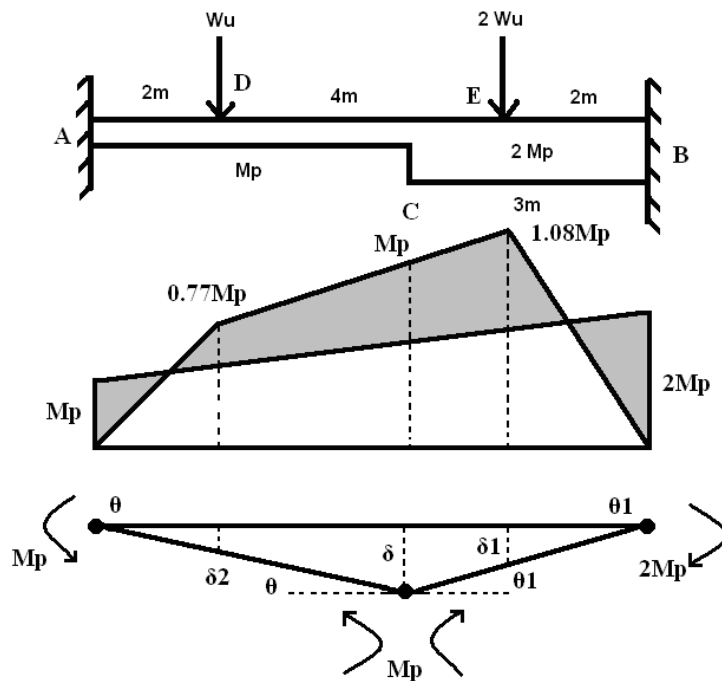
$$4 * M_p * \theta = W_U * L \theta + W_U * L \theta / 2$$

$$4M_p = 1.5 W_U L$$

$$W_U = (8 / 3) M_p / L$$

Problem 7

Fixed beam with different plastic moments. The beam is shown in the figure. The Simply supported reactions at A and B are $1.25W_U$ and $1.75W_U$ respectively.



Equilibrium method -

Plastic hinges at A, B and D

Maximum free BM at C = $1.25 W_U * 2 = 2.50 W_U$ Plastic moment of the section = $M_p + 1.25 M_p = 2.25 M_p$ Equating, $2.50 W_U = 2.25 M_p$

$$W_U = 0.9 M_p$$

Plastic hinges at A, B and C

Maximum free BM at C = $1.25 W_U * 5 - W_U * 3 = 3.25 W_U$ Plastic moment of the section = $M_p + 1.625 M_p = 2.625 M_p$ Equating, $3.25 W_U = 2.625 M_p$

$$W_U = 0.8077 M_p$$

Plastic hinges at A, B and E

$$\text{Maximum free BM at C} = 1.75 W_U * 2 = 3.5 W_U$$

$$\text{Plastic moment of the section} = 2 M_p + 1.75 M_p = 3.75 M_p$$

$$\text{Equating, } 3.5 W_U = 3.75 M_p$$

$$W_U = 1.0714 M_p$$

The lowest load of $W_U = 0.8077 M_p$ is the true collapse load.

Mechanism method -

$$\text{Internal work done} = 2 * M_p * \theta + 3 * M_p * \theta_1$$

$$\text{External work done} = W_U * \delta_2 + 2W_U * \delta_1$$

$$\text{From geometry, } \delta = 5 \theta = 3 \theta_1, \delta_1 = 2 \theta_1, \delta_2 = 2 \theta, \theta_1 = (5 / 3) \theta$$

Equating internal and external work done,

$$2 * M_p * \theta + 3 * M_p * \theta_1 = W_U * \delta_2 + 2W_U * \delta_1$$

$$2 * M_p * \theta + 3 * M_p * (5 / 3) \theta = W_U * 2 \theta + 2W_U * 2 * (5 / 3) \theta$$

$$7M_p = (26 / 3)W_U$$

$$W_U = 0.8077 M_p$$

The support reactions in the fixed beam can be obtained as follows :

$$2W_U * 6 + W_U * 2 + 2 M_p - M_p - V_B * 8 = 0$$

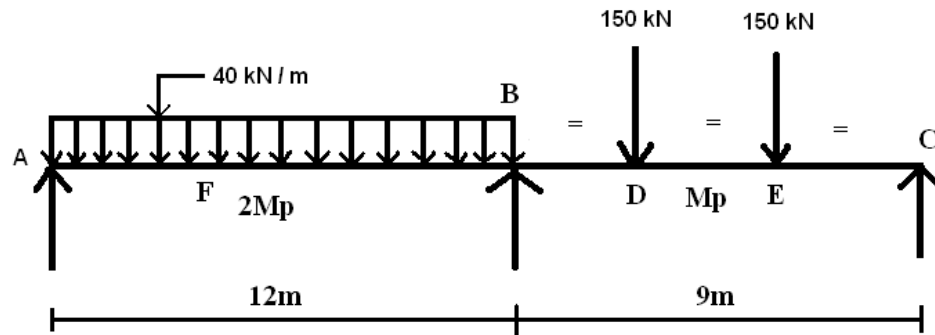
$$V_B = 1.538 M_p$$

$$V_A = 0.885 M_p$$

SFD can be drawn.

Problem 8

A two span continuous beam is shown below. All the loads are service loads.



No. of possible plastic hinges = 4 (B, Between A and B, under each concentrated load. No. of relevant mechanisms = 4 - 1 = 3

Each mechanism is similar to a propped cantilever. Mechanism 1 -

Plastic hinges will be at B and in between A and B. The distance x where the SF is zero is worked as in propped cantilever using segment equilibrium method. However, the value of plastic moment at B shall be chosen lesser of the two values ie M_p . The plastic moment in between A and B is $2M_p$.

The location of the plastic hinge at F is carried out by segment equilibrium method. Let the hinge at F be at distance x from A.

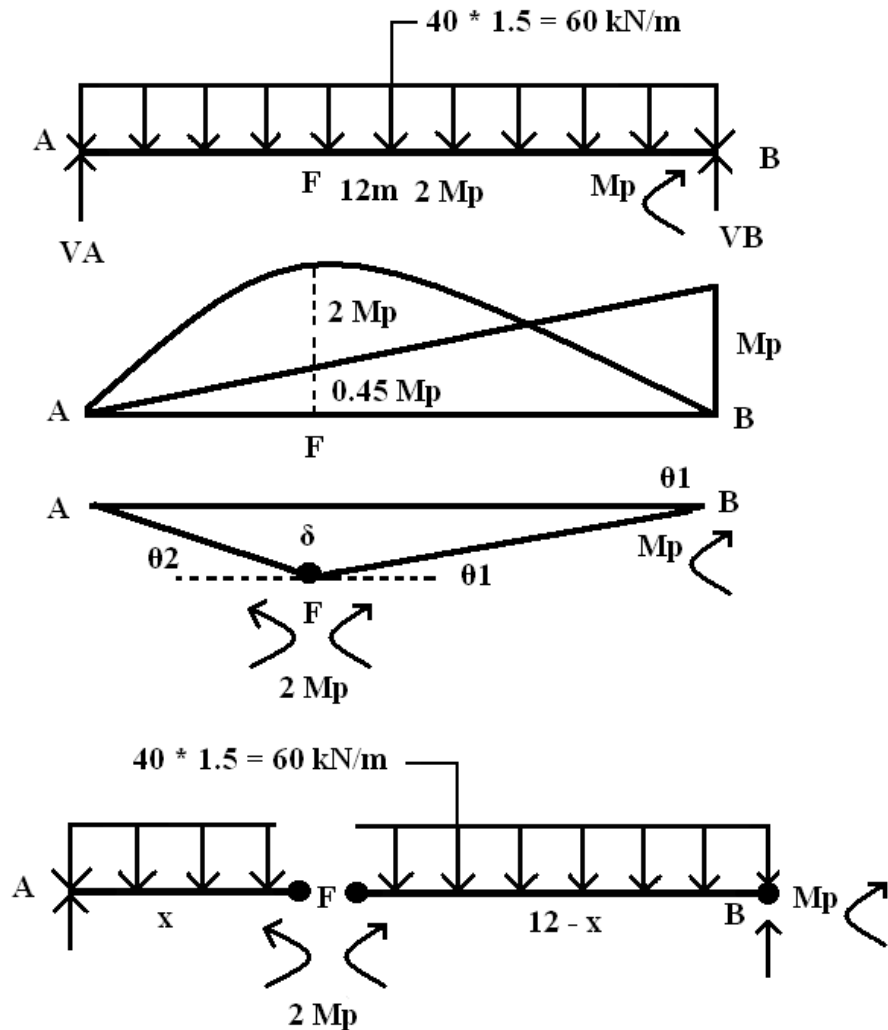
$$AF = x \text{ and } BF = (12 - x) \text{ At F, } SF = 0$$

Consider the segment AF,

$$\Sigma M_A = 0,$$

$$2M_p - 60 x^2 / 2 = 0$$

$$M_p = 15 x^2 \text{ ----- (1)}$$



Consider the segment BF,

$$\Sigma M_B = 0,$$

$$60 (12 - x)^2 / 2 - 3 M_p = 0$$

$$M_p = 10 (12 - x)^2 \text{ ----- (2)}$$

Equating (1) and (2),

$$15 x^2 = 10 (12 - x)^2$$

$$1.5 x^2 = (12 - x)^2$$

Simplifying, we get $x = 5.394 \text{ m}$

$$M_p = 436.43 \text{ kNm}$$

Equilibrium Method -

Simply supported BM at 5.394 m from A –

$$\text{SS BM} = 360 * 5.394 - 60 * 5.394^2 / 2 = 1068.98 \text{ kNm}$$

$$\text{Total Plastic Moment at the section} = (2.0 + 0.45) M_p = 2.45 M_p$$

Equating SS BM to the total plastic moment,

$$1068.98 = 2.45 M_p$$

$$M_p = 436.32 \text{ kNm}$$

Mechanism method -

$$\text{Internal work done} = 2 M_p \theta_2 + 3 M_p \theta_1$$

$$\text{External work done} = 0.5 * 12 * \delta * 60 = 360 \delta$$

From geometry,

$$\delta = 5.394 \theta_2 = 6.606 \theta_1$$

Equating,

$$2 M_p \theta_2 + 3 M_p \theta_1 = 360 \delta$$

$$2 M_p \theta_2 + 3 M_p * (5.394 / 6.606) * \theta_2 = 360 * 5.394 \theta_2$$

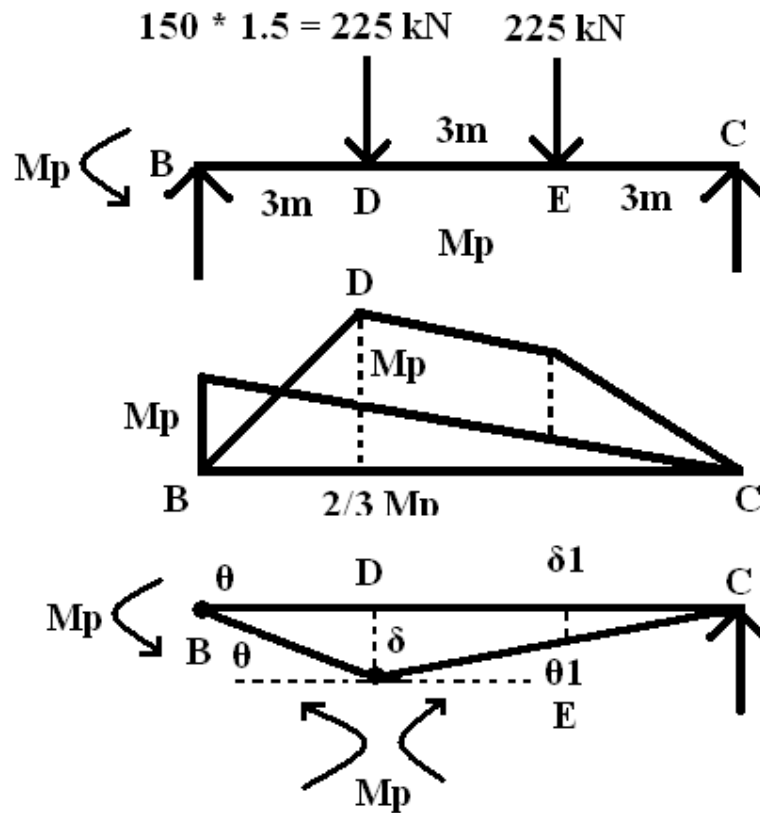
$$4.45 M_p = 1941.84$$

$$M_p = 436.37 \text{ kNm}$$

Note : All methods give the same answer. The student shall check the question and accordingly answer. If no method is specified, then the student can have his /her choice.

Mechanism 2 -

Plastic hinges will be formed at B and D (under 150 kN load near B).



Equilibrium Method -

Simply supported BM at 3.0 m from B - SS BM = $225 \times 3.0 = 675 \text{ kNm}$

Total Plastic Moment at the section = $(1.0 + 2/3) M_p = 5/3 M_p$

Equating SS BM to the total plastic moment,

$$675 = 5/3 M_p$$

$$M_p = 405 \text{ kNm}$$

Mechanism method -

$$\text{Internal work done} = 2 M_p \theta + M_p \theta_1$$

$$\text{External work done} = 225 \times \delta + 225 \times \delta_1$$

From geometry,

$$\delta = 3 \theta = 6 \theta_1 ; \theta_1 = 0.5 \theta \text{ and } \delta_1 = 3 \theta_1 = 1.5 \theta \text{ Equating,}$$

$$2 M_p \theta + M_p \theta_1 = 225 \times \delta + 225 \times \delta_1$$

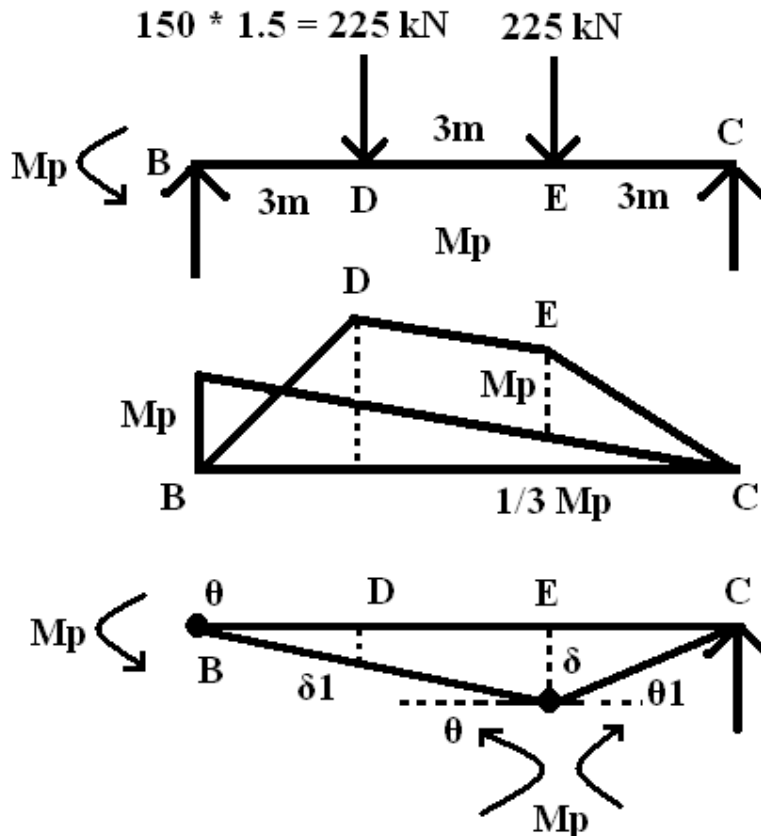
$$2 M_p \theta + M_p * 0.5 \theta = 225 * 3 \theta + 225 * 1.5 \theta$$

$$2.5 M_p = 1012.5$$

$$M_p = 405 \text{ kNm}$$

Mechanism 3 -

Plastic hinges will be at B and E (under 150 kN load near C)



Equilibrium Method -

Simply supported BM at 3.0 m from C - SS BM = $225 * 3.0 = 675 \text{ kNm}$

Total Plastic Moment at the section = $(1.0 + 1/3) M_p = 4/3 M_p$

Equating SS BM to the total plastic moment,

$$675 = 4/3 M_p$$

$$M_p = 506.25 \text{ kNm}$$

Mechanism method -

$$\text{Internal work done} = 2 M_p \theta + M_p \theta_1$$

$$\text{External work done} = 225 * \delta + 225 * \delta_1$$

From geometry,

$$\delta = 6 \theta = 3 \theta_1 ; \theta_1 = 2 \theta$$

$$\text{and } \delta_1 = 3 \theta$$

Equating,

$$2 M_p \theta + M_p \theta_1 = 225 * \delta + 225 * \delta_1$$

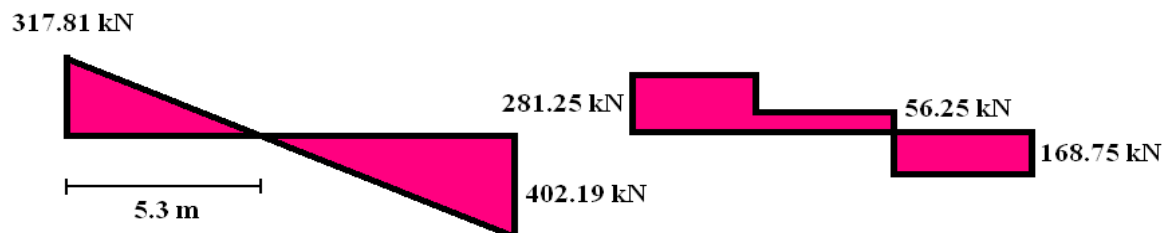
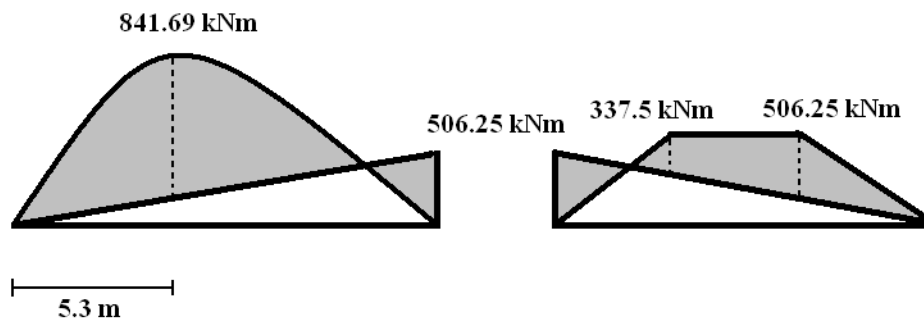
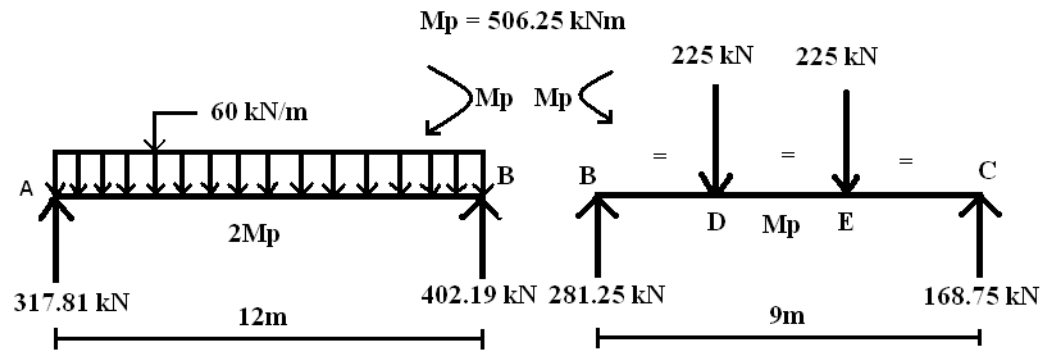
$$2 M_p \theta + M_p * 2 \theta = 225 * 6 \theta + 225 * 3 \theta$$

$$4 M_p = 2025$$

$$M_p = 506.25 \text{ kNm}$$

The largest value of M_p shall be chosen. For the given problem, $M_p = 506.25 \text{ kNm}$

The reactions for the beams are calculated as in elastic analysis by taking moments about any support and using the vertical equilibrium equation. The reactions are shown are below. BMD and SFD are also indicated. The BM has not exceeded the relevant plastic moment of the beam.



Note

The student shall work similar problems for practice with two and three span continuous beams with different support and loadings. If required BMD and SFD can be drawn. If end supports are fixed, plastic hinges are formed and the span behaves as fixed beam.

Sometimes, the value of M_p will be given for each span and the collapse load will be required to be calculated. In such problems, the collapse load is worked for each span and the lowest load is chosen as the true collapse load.

4.11 Assignment questions

1. What is plastic moment?

2. What is shape factor?
3. Explain plastic theory.
4. Explain plastic hinge concept.
5. Explain collapse load mechanism.

4.12 Outcome

Gives an idea about plastic moment

Gives an idea about plastic hinge

Gives knowledge about collapse load mechanism

4.13 Future Scope

By studying this concept one can analyze the frame sections which are subjected to plastic moments.

Module – 2 Bolted connections

Contents:-

- 2.1 Introduction
- 2.2 Objective
- 2.3 Types of structural steel connections
- 2.4 Types of bolts
- 2.5 Advantage and disadvantages of bolted connections
- 2.6 Terminologies
- 2.7 Types of bolted connections
- 2.8 Types of bolted joints
- 2.9 Failure of bolted joint
- 2.10 Design strength of bolt
- 2.11 Problems
- 2.12 Assignment questions
- 2.13 Outcome
- 2.14 Future study

2.1 Introduction

Any structure is an assemblage of different members such as beams, columns and tension members which are connected or joined to one another at the member ends. Many members of steel structures are made of different components such as I-sections, T-sections, channels and angles which are connected by fasteners or connectors so as to act as a single composite unit.

Connection between different members not only facilitates the flow of forces and moments from one to another but also to transfer the forces up to foundation level. Connections are also required in order to extend the length of different members, to connect steel columns to footings, to connect two different parts of structure at the time of Erection. If it is not properly designed and detailed, connections may become weak than member. Therefore it is desirable to avoid the connection failure before the member fails.

2.2 Objective

- To know the different types of connections
- To know the advantages and disadvantages of bolted connections
- To determine the strength of bolted joints

Requirements of structural steel connection

- Speed, easy to handle and erect.
- Less maintenance cost.
- Easy to inspect and replace.
- It should serve its function effectively and efficiently.
- It should have adequate strength.
- It should transmit the loads and at the same time it should be rigid.
- Connections should satisfy the structural behavior requirements.

Necessity for connections

1) When there is a change in direction occurs:-

Eg:- Beam - Beam connection, Beam – column connection and connection between different members of truss.

2) To ensure manageable sizes of steel work for transportation and Erection:-

Eg:- Column splices at every two or three storey's.

3) When there is a change in component occurs:-

Eg:- Column base, connection to concrete cores and connections with walls, floors and roofs.

2.3 Types of structural steel connections

- Riveted connections
- Bolted connections
- Welded connections
- Pinned connections

Bolted joint connections

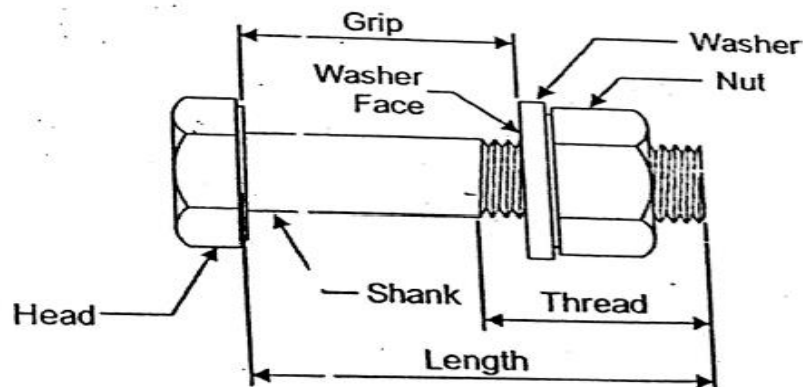
Bolt are used for structures not subjected to vibration. The cost of bolts is more but less time to fabricate structure with bolted connections. The fabrication works with bolts are noiseless and even less skilled labour can handle it.

A **bolt** is a metal pin with head at one end and shank or body threaded at another end to receive nut. Steel washers are provided under the bolt head as well as under nut.

Steel washers are provided

- To distribute the clamping pressure on the bolted member.
- To prevent the threaded portion of bolt from bearing on connecting pieces.

Parts of bolt assembly



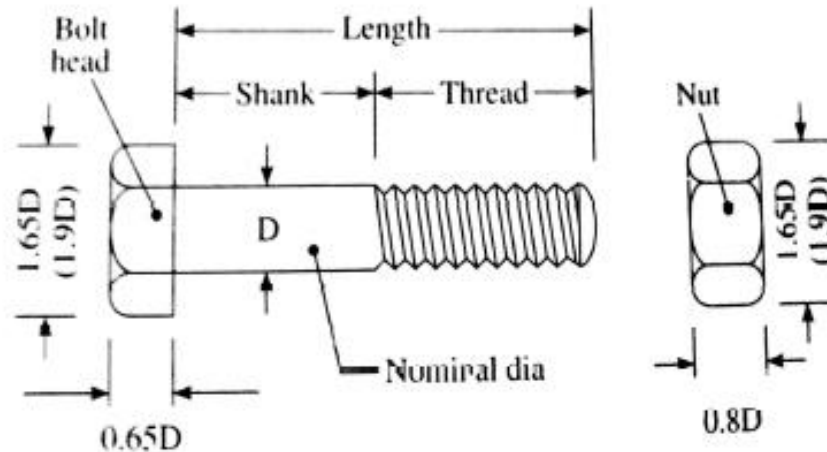
Grip:- It is the distance from behind the bolt head to the back of nut or washer.

Thread length:- It is the threaded portion of the bolt.

Bolt length:- It is the distance from behind the bolt head to the end of bolt.

2.4 Types of bolts

1) Unfinished or black bolt (IS 1363:2002):-



Black bolts are also referred as ordinary, unfinished and rough bolts which are less expensive. These are used for light structures which are subjected to static loads such as small trusses but not for structures which are subjected to dynamic or impact loads. These are made from structural steel rods (mild steel) with square or hexagonal head with nut. Nominal diameters of 16, 20, 24, 30 and 36mm of 4.6 grades are usually adopted.

2) Turned bolts (IS 1364:2002):-

These are similar to that of ordinary bolts but only difference is that the shank is made of hexagonal rods. Usually grades of 4.6 to 8.8 are adopted. These type of bolts have high shear and bearing resistance.

3) Ribbed bolts:-

The head of these bolts are round and similar to that of rivets. The actual diameter is slightly larger than the hole into which it is driven. While driving the bolts through the hole, the edges of a hole tears out and gives a tight fit. These types of bolts are rarely used in steel structures but have high resistance to vibrations.

4) High Strength Friction Grip Bolts (HSFG):-

These are made from medium carbon steel rods. They achieve strength by quenching or tempering or alloying of steel. Steel washers are made of hard steel which will distribute the clamping pressure evenly on bolting member. Tensile strength is several times than ordinary bolts. These types of bolts replace rivets which are widely used for structures such as bridges, machines and high-rise buildings.

Advantages of HSFG bolts

- Since load is transferred by friction. No bolt will be subjected to shear and bearing stresses.
- Since joints are rigid, no slip will occur between connecting members.

Disadvantages of HSFG bolts

- High material cost.
- Rigidity of joint is reduced due to loss of fit, resulting into excessive deformation.

Bolt grades

International Organization for Standardization ISO has designated bolts in India. Usually, the bolts are indicated by two numbers. The common grades of bolts are 4.6, 5.6, 6.5, 6.8, 8.8, 10.9 and 12.9.

The design yield stress f_{yb} can be derived from grade by multiplying two numbers and the second number by 10. OR 1st number indicates 1/100th of nominal ultimate strength in N/mm².

Design ultimate stress f_{ub} can be obtained by multiplying first number by 100. OR second number indicates the ratio of yield stress to ultimate stress in percentage.

Grade	4.6	5.6	6.5	6.8	8.8	10.9	12.9
f_{yb} (N/mm ²)	240	300	300	480	640	940	1100
f_{ub} (N/mm ²)	400	500	600	600	800	1040	1220

Assumptions made in the theory of bolted connections are

- Shear is uniform on the cross-section of the bolt.
- Distribution of tensile stress on the portions of the plate between the bolt holes is uniform.
- Bolts in a groove subjected to direct load through their centroid share the load equally.
- Bending stresses in the bolt are neglected.

2.5 Advantages and Disadvantages of bolted connections

Advantages of bolted connections

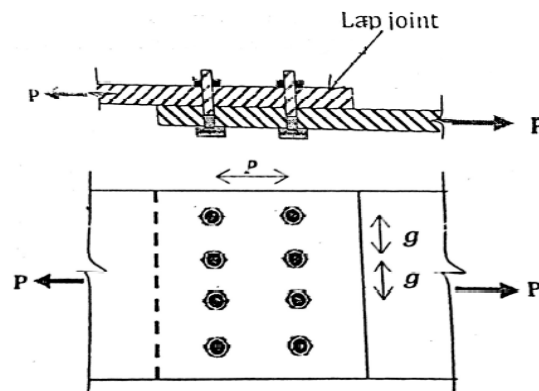
- Economical, speedy and easy to erect.
- Easy to inspect

- Less skilled labors can handle or operate it.
- It is noiseless.
- It can be replaced easily.

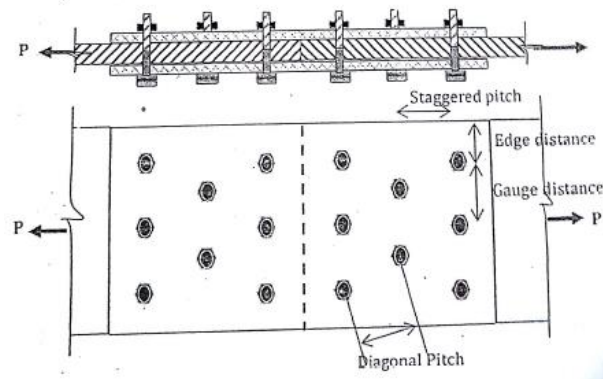
Disadvantages of bolted connection

- High material cost.
- Loosening of components to vibration.
- Reduction in strength due to loss of fit in holes.
- Reduction at the root of thread.
- Black bolts are having less strength because of non-uniform diameter.

2.6 Terminologies



- 1) **Gross sectional area (A_g):** It is the area of unthreaded portion of bolt or shank.
- 2) **Net effective cross-sectional area (A_n):** It is the cross-sectional area at the root of thread.
$$A_n = 0.78 \text{ to } 0.8 A_g.$$
- 3) **Pitch (p):** It is the distance between centers of adjacent bolts in the direction of load.
- 4) **Gauge distance (g):** It is the distance between two consecutive bolts of adjacent rows and is measured at right angles to the applied load.
- 5) **Edge distance (e):** It is the distance between center of bolt to adjacent edge measured in the direction at right angle to direction of stress.
- 6) **End distance (e'):** It is the distance between center of bolt to the end of element.
- 7) **Staggered pitch (S):** It is the center to center distance of any two consecutive bolts in zigzag bolting measured parallel to the direction of stress.
- 8) **Diagonal pitch (D):** It is the distance between centers of two adjacent bolts measured in diagonal direction.



2.7 Types of bolted connections

1) Simple connections

In many cases, a connection is required to transmit a force only and there may not be any moment acting on the group of connectors, even though the connection may be capable of transmitting some amount of moment. Such a connection is referred as simple, force, pinned or flexible connection.

The different types of simple connections found in steel structures may be classified as follows

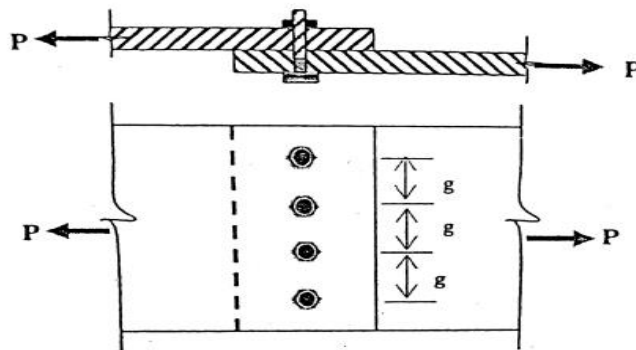
- Lap and Butt joints
- Truss joint connections
- Connections at beam-column junctions
- Seat angle connections
- Web angle connections
- Stiffened seat angled connections
- Tension and flange splices

2.8 Types of bolted joints

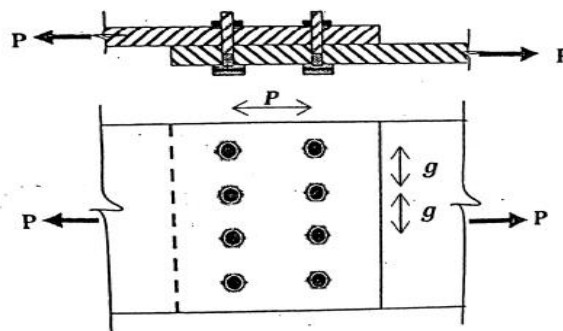
- 1) **Lap joint:-** When two members which are to be connected are simply overlapped and are connected by means of bolts or welds and this type of joint is called lap joint. It may have single row, chain bolting or staggered.

There are two types of lap joints

- Single bolted lap joint.



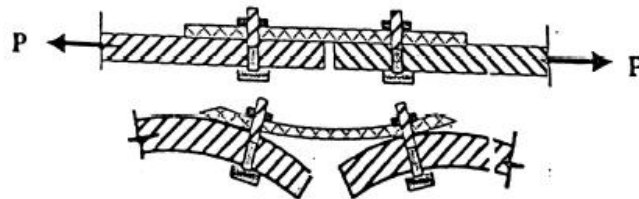
- Double bolted lap joint → chain bolting and zigzag bolting.



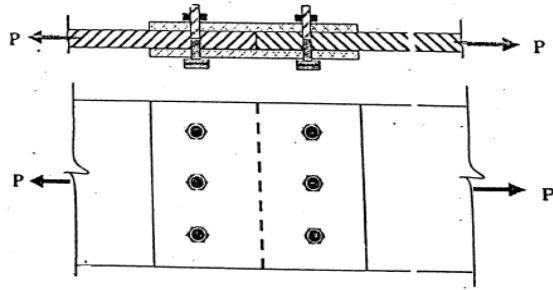
- 2) **Butt joint:-** In this type of joint, two members are connected against each other and connection is made by providing a cover plate on one side or both the sides.

Types bolted butt joint

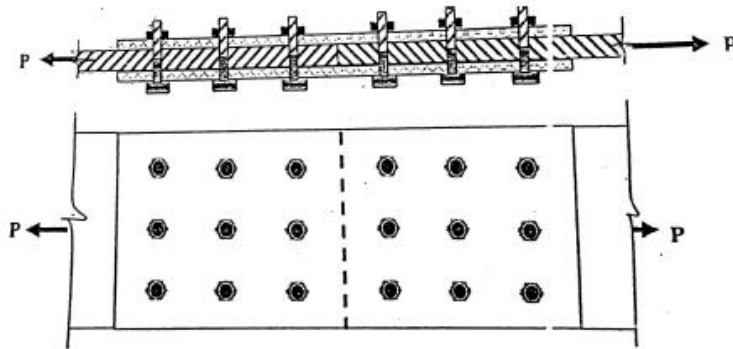
- a) **Single cover single cover butt joint:-** In this type bending stress will be more which may lead to failure of joint. So it is desirable to provide double cover plates.



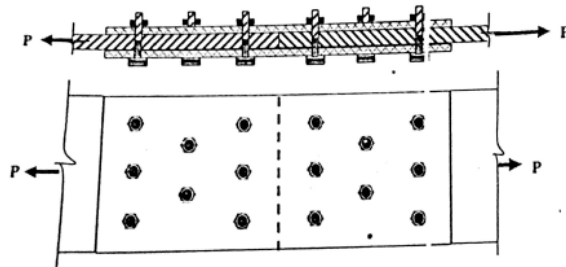
- b) **Double cover single bolted butt joint:-**



c) Double cover chain bolt butt joint:-

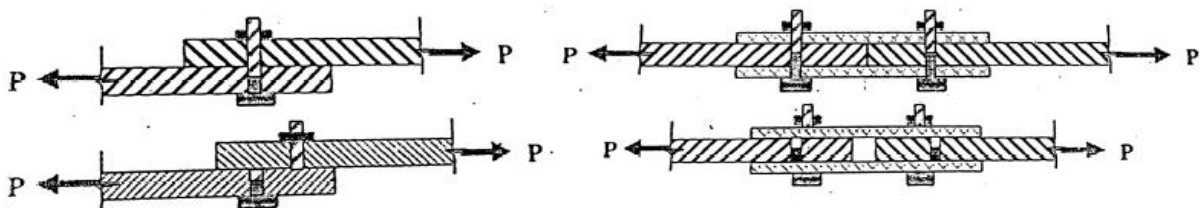


d) Double cover staggered or zigzag bolt butt joint:-

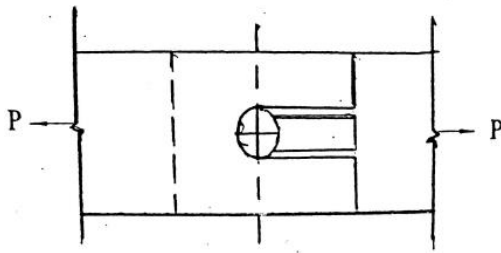


2.9 Failure of bolted joint

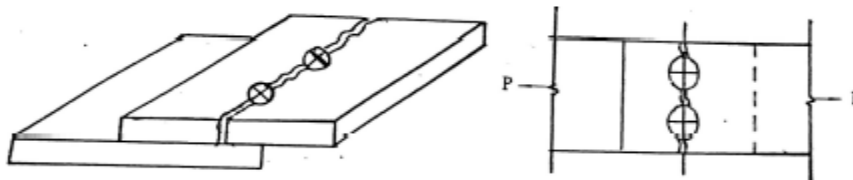
- 1) **Shear failure of bolt:-** Plates are bolted together and subjected to tensile load may results in shear of bolt and shear occurs at cross-section of bolt. Single shear occurs in case of lap joint and double shear occurs in butt joint.



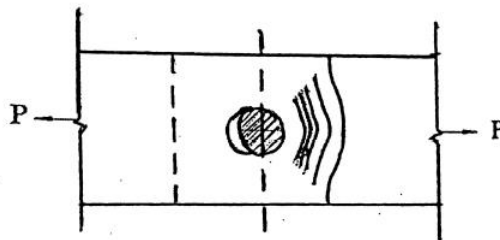
- 2) **Shear failure of plates:-** Plates may fail in shear along two lines. This may occur when minimum edge distance is not provided.



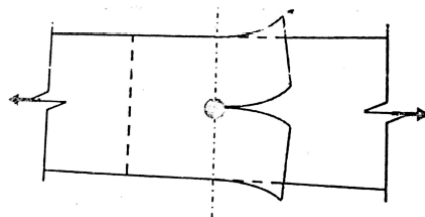
- 3) **Tearing failure of plates:-** When plates are bolted and subjected to tensile load, tearing failure may occur when the tensile strength of plate is less than tensile strength of bolt. Tearing failure occurs at net cross-section area of plate.



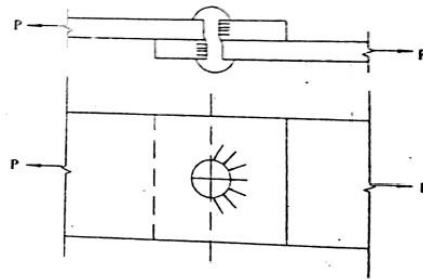
- 4) **Bearing failure of plates:-** The bearing failure of plates may take place when insufficient edge distance in case of bolted joint when result in crushing of plate against bolt bearing.



- 5) **Splitting failure of plate:-** Splitting failure of the plate may occur due to insufficient edge distance in bolted joint results in splitting or cracking of plates.



- 6) **Bearing failure of bolt:-** The bearing failure of bolt occurs when bolt is crushed against the plate.



Spacing and edge distance requirements

- Pitch should not be less than $2.5d$.
- Pitch should not be greater than
 - a) $16t$ or 200mm whichever is less in case of tensile member.
 - b) $12t$ or 200mm whichever is less in case of compressive member.
 - c) In case of staggered bolting pitch can be increased by 50% of value specified above but gauge distance $< 75\text{mm}$.
- In case of butt joint, maximum pitch should be restricted to $4.5d$ for a distance of 1.5 times of width of plate from building surface.
- The gauge length should not be more than $(100+4t)$ or 200mm whichever is less.
- The maximum edge distance shall not be less than
 - a) 1.7 times hole dia in case of shear or hard flame cut edge.
 - b) 1.5 times dia in case of rolled, machine flame cut, sawn and planed edges.
- Maximum edge distance should not exceed
 - a) $12t \epsilon$ where $\epsilon = \sqrt{250/f_y}$
 - b) $40 + 4t$ where t is thickness of thinner plate.

Pinned connection

A pinned connection is provided to allow free rotation of the end of a connected member. Some of the design considerations of a pinned connection are the same as those of bolted connections. A pin in a connection acts similar to the shank of a bolt. But since a single pin is used for the connection, forces acting on it are much greater than those on a bolt. Large shear forces can be resisted by pins of large diameter and of a high grade steel.

Pins are used in connections of braced girders, chain link cables of suspension bridges, diagonal bracings etc.

Pin connections can be made in two ways. In the type of connection shown in fig, an end of one of the bars is forged to form a fork with a drilled hole in it. The end of the other bar called eye bar is forged with a drilled hole in it. The eye bar is inserted between the jaws of the fork with all

the holes matching. A forged steel pin is driven through the holes of the bars. The pin is secured by a cotter pin. The bar ends are made octagonal.

Moment resistant connections

Moment resistant connections are used to transfer moment, axial force and shear force from one member to another. There are many practical structures in which moment resisting connections are necessary. Un-braced frames are an obvious example.

Moment resistant connections includes

- Eccentric connections.
- Split-beam T-stub connections
- Flange angle connections.

Bolt connections may be classified into three types based on the analysis and design method adopted.

- 1) **Rigid connections:-** Fully welded connections, haunched end plate connections and flush plate or extended plate connections are considered as rigid connections.
- 2) **Flexible connections:-** Simple connection will be considered as flexible connections.
- 3) **Semi-rigid connections:-** Falls in between the rigid and flexible connections.

In fact simple connections do have some degree of rotational restraint and rigid connections do experience some degree of joint deformation. However due to the difficulty of evaluating and establishing the degree of restraint and analyzing the frames with such joints, the effects of semi-rigid connections is not considered in practice. Their effect is also not considered in the plastic design method.

Beam-Beam connections

A beam may be supported at each end by a masonry wall or column or by a heavier beam. When a lighter beam is supported at its end by a heavier beam, two types of connections are adopted namely,

- Framed connections
- Seated connections

Framed connections

In this case, the connection is made by connecting two angles one on either side of the web of the beam to be connected. The outspread arms of the angles are connected to the web of the supporting beam. Angles of sizes, 100x75mm, 150x75mm, 100x100mm, 150x150mm may be used as connecting angles. Thickness of angles may be 8mm, 10mm or 12mm.

Seated connections

Suppose a beam has to transmit a large reaction to a supporting beam. In such a case we provide a seated connection. In this arrangement, the bottom flange of the beam to be supported rests over the outstanding leg of a seat angle. The other leg of the seat angle connected to the web of the supporting beam. This arrangement is possible if the depth of web of the supporting beam is deep enough to accommodate the seat angle.

Beam-column connections

A beam can be connected either to the flange or the web of a column. In this case, these are two types of connections,

- Framed connections
- Seated connections

Framed connections

In this case, for instance if a beam has to be connected to the flange of a column the connection is made by connecting two angles one on either side of the web of the beam and the outspread arms of the angles are connected to the flange of the column. The bolts connecting the web of the beam and the angles are in double shear while the bolts connecting the angles and the flange of the column are in single shear. The bolts are designed to resist the end reaction of the supported beam. For design purposes, the connection is assumed to act as hinged connection offering no resistance against rotation of the beam, at the connected end. To nearly satisfy this condition the connecting angles should not be thick. Angles of thickness over 10mm may be avoided. The depth of the connecting angles should be less than 0.6 times the depth of the beam.

2.10 Design strength

Plates may fail in shear, bearing, tearing, and splitting can be avoided by providing edge distance (e). Therefore, design strength of bolted connection is calculated by considering shear and bearing failure of bolt and tearing failure of plate. The least among these 3 will be the design strength of joint or bolted connection.

Design strength of bolt (V_{db}):-

A bolted connection should be designed such that factored shear (V_{sb}) should not exceed the design strength (V_{db}).

$$V_{sb} \leq V_{db}$$

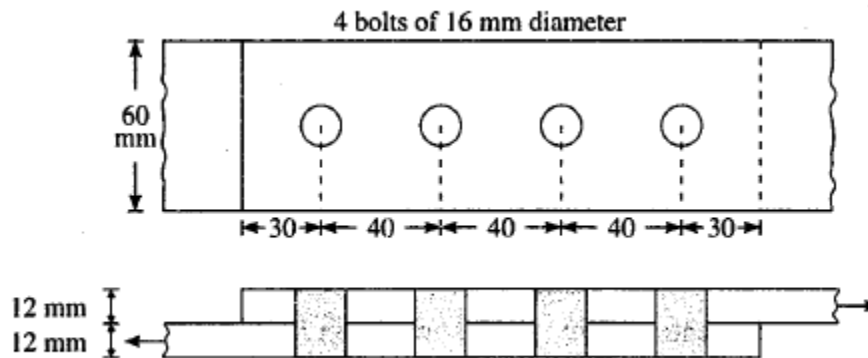
The design strength of bolt (V_{db}) is lesser of

- 1) Design shear strength V_{dsb} of the bolt

2) Design bearing strength V_{dpb} of the bolt.

2.11 Problems

- Two plates 12mm X 60mm are connected in a lap joint with 4 bolts of 16mm diameter as shown in figure. Determine the strength of the joint.



Ultimate strength of bolt material = 400 N/mm^2

Ultimate strength for plate = 410 N/mm^2 .

Solution.

Bolt diameter = 16 mm

Diameter of bolt hole $d_o = 16 + 2 = 18 \text{ mm}$

$$\text{Design shear strength of one bolt} = V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \right]$$

$$= \frac{1}{1.25} \cdot \frac{400}{\sqrt{3}} \left[1 \times 0.78 \times \frac{\pi \times 16^2}{4} \right] \text{ N}$$

$$= 28975 \text{ N}$$

Design bearing strength of one bolt

$$V_{db} = \frac{1}{\gamma_{mb}} [2.5 K_b d t f_u]$$

K_b is the least of the following:

$$\left. \begin{array}{l} (i) \frac{e}{3d_o} = \frac{30}{3 \times 18} = 0.56 \\ (ii) \frac{p}{3d_o} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49 \\ (iii) \frac{f_{u,bolt}}{f_{u,plate}} = \frac{400}{410} = 0.98 \\ (iv) 1 \end{array} \right\} \therefore K_b = 0.49$$

$$\therefore V_{db} = \frac{1}{1.25} [2.5 \times 0.49 \times 16 \times 12 \times 400] = 75264 \text{ N}$$

Lesser design strength of one bolt = 28975 N

Total design strength of four bolts = $4 \times 28975 = 115900 \text{ N}$

Design tensile strength of plate (yielding consideration)

$$T_{dt} = \frac{1}{\gamma_{mt}} [A_t \cdot f_y]$$

$$\frac{1}{1.10} (60 \times 12 \times 250) = 163636 \text{ N}$$

Design tensile strength of plate (rupture consideration)

$$T_{dr} = \frac{1}{\gamma_{mt}} [0.9 A_n f_u]$$

$$= \frac{1}{1.25} [0.9(60-18)(12 \times 410)] = 148780 \text{ N}$$

$\therefore$ Design strength of the joint = 115900 N.

2. Investigate the safety of lap joint connecting two plates to the following particulars; the plates are 12mm and 15mm thick. Each plate is 75mm wide. Use 16mm diameter bolts of 4.6 grade and factored load is 75KN.

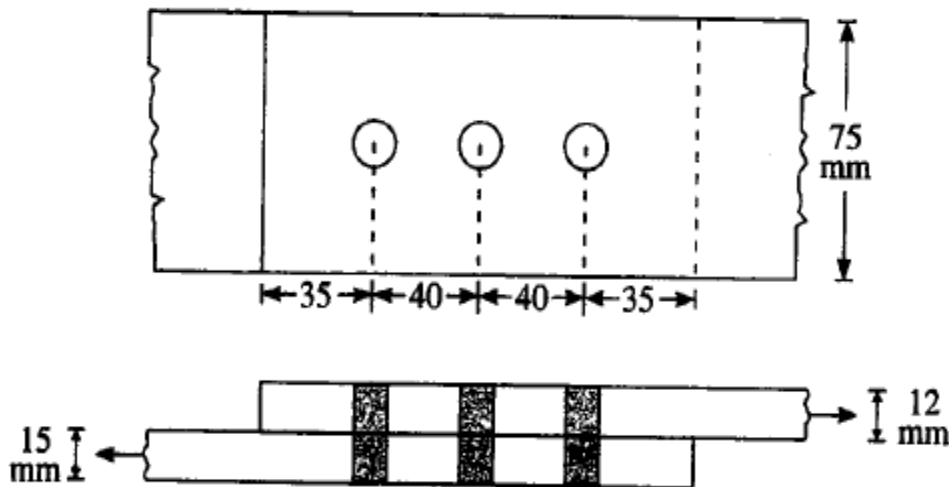
Solution. 4.6 grade bolts are used. Ultimate strength of bolt material

$$f_{ub} = 4 \times 100 = 400 \text{ N/mm}^2$$

$$\text{Yield stress of bolt material} = f_y = 0.6 \times 400 = 240 \text{ N/mm}^2$$

$$\text{For plates, } f_u = 410 \text{ N/mm}^2$$

$$f_y = 250 \text{ N/mm}^2$$



Diameter of bolt = 16 mm

Diameter of bolt hole = 16 + 2 = 18 mm

Design strength of a bolt

$$\text{Design shear strength of a bolt} = V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \right]$$

$$\text{The bolt is in single shear, } n_n = 1, n_s = 0, \gamma_{mb} = 1.25, A_{nb} = 0.78 \frac{\pi d^2}{4}$$

$$\therefore V_{dsb} = \frac{1}{\gamma_{mb}} \cdot \frac{f_{ub}}{\sqrt{3}} A_n = \frac{1}{1.25} \cdot \frac{400}{\sqrt{3}} \left[\frac{0.78 \pi \times 16^2}{4} \right] = 28975 \text{ N}$$

$$\text{Design bearing strength of a bolt} = V_{dpb} = \frac{1}{\gamma_{mb}} [2.5 K_b d t f_u]$$

k_b is the least of the following :

$$\left. \begin{array}{l} \text{(i)} \quad \frac{e}{3d_o} = \frac{35}{3 \times 18} = 0.65 \\ \text{(ii)} \quad \frac{p}{3d_o} - 0.25 \quad [\text{Provide a pitch } p = 40 \text{ mm}] \\ \quad \quad = \frac{40}{3 \times 18} - 0.25 = 0.49 \\ \text{(iii)} \quad \frac{f_{ubolt}}{f_{uplate}} = \frac{400}{410} = 0.98 \\ \text{(iv)} \quad 1 \end{array} \right\} \therefore K_b = 0.49$$

$$\therefore V_{dpb} = \frac{1}{1.25} [2.5 \times 0.49 \times 16 \times 12 \times 400] = 75264 \text{ N}$$

$$\therefore \text{Bolt value} = 28975 \text{ N}$$

$$\text{Number of bolts required} = \frac{75 \times 10^3}{28975} = 2.6 \text{ say } 3$$

Provide 3 bolts of 16 mm diameter

Fig. 4.24 shows the arrangement of the bolts

Check for the strength of the plate

Design strength of the plate (rupture consideration)

$$T_{dn} = \frac{1}{\gamma_{mt}} (0.9 A_n f_u) = \frac{1}{1.25} [0.9 (75 - 18) 12 \times 410] = 201917 \text{ N}$$

Design strength of the plate (yielding consideration)

$$T_{dg} = \frac{1}{\gamma_{mo}} [A_g f_y] = \frac{1}{1.10} [75 \times 12 \times 250] = 204545 \text{ N}$$

Factored load on the connection is 75000 N

$\therefore$ The design is safe.

3. Design a lap joint connecting two plates 120mm X 8mm to transmit a factored load of 120KN. Use 12mm diameter bolts of grade 4.6 and plate grade of 410.

Solution.

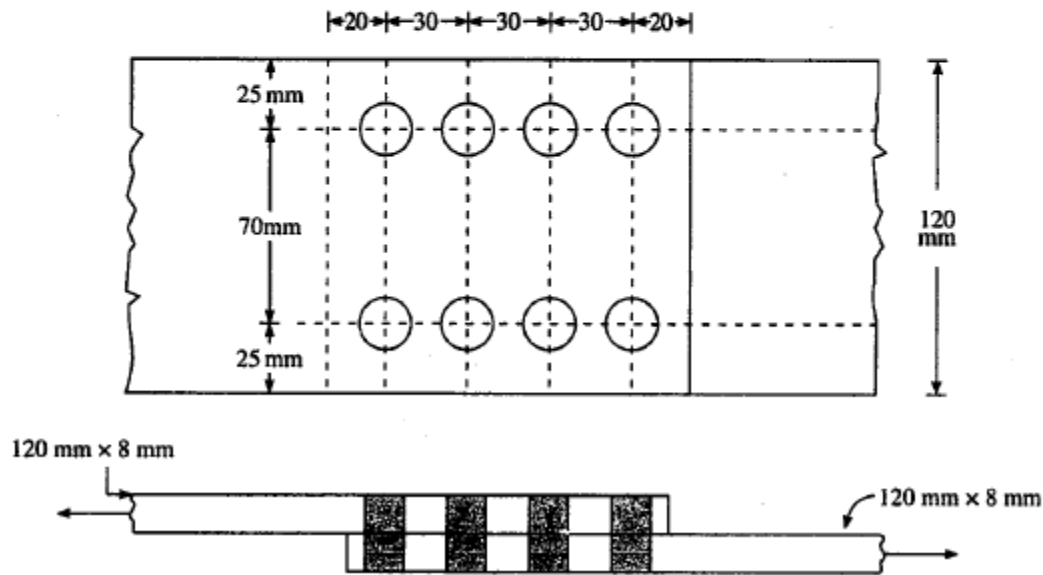
For bolts of 4.6 grade,

Ultimate strength of bolt material $f_{ub} = 4 \times 100 = 400 \text{ N/mm}^2$

Yield stress of bolt material $= f_y = 0.6 \times 400 = 240 \text{ N/mm}^2$

For plates, $f_u = 410 \text{ N/mm}^2$

$f_y = 250 \text{ N/mm}^2$



Minimum edge distance (For 12 mm bolts) = 20 mm

$\therefore e = 20 \text{ mm}$

Minimum pitch = $2.5 \times \text{bolt diameter} = 2.5 \times 12 = 30 \text{ mm}$

$\therefore p = 30 \text{ mm}$

Design shearing strength of the bolt

$$V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \right]$$

$$n_n = 1, n_s = 0$$

$$\therefore V_{dsb} = \frac{1}{\gamma_{mb}} \cdot \frac{f_{ub}}{\sqrt{3}} A_n$$

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi \times 12^2}{4} = 16298 \text{ N}$$

$$\text{Design bearing strength of a bolt} = V_{dpb} = \frac{1}{\gamma_{mb}} [2.5 K_b d t f_u]$$

K_b is the least of the following:

$$\left. \begin{array}{l} \text{(i)} \quad \frac{e}{3d_0} = \frac{20}{3 \times 13} = 0.513 \\ \text{(ii)} \quad \frac{p}{3d_0} - 0.25 = \frac{30}{3 \times 13} - 0.25 = 0.519 \\ \text{(iii)} \quad \frac{f_{ub}}{f_{up}} = \frac{400}{410} = 0.976 \\ \text{(iv)} \quad 1 \end{array} \right\} \therefore K_b = 0.519$$

$$\therefore V_{dpb} = \frac{1}{1.25} [2.5 \times 0.513 \times 12 \times 8 \times 400] = 39398 \text{ N}$$

$$\therefore \text{Bolt value} = 16298 \text{ N}$$

$$\text{Number of bolts required} = \frac{120 \times 10^3}{16298} = 7.4$$

Provide 8 bolts of 12 mm diameter.

Fig. 4.25 shows the arrangement of the bolts.

Check for the strength of the plate

Design strength of the plate from rupture consideration

$$\begin{aligned} &= T_{dn} = \frac{1}{\gamma_{ml}} [0.9 A_n f_u] \\ &= \frac{1}{1.25} [0.9 (120 - 2 \times 13) 8 \times 410] = 221990 \text{ N} \end{aligned}$$

Design strength of the plate from yielding consideration

$$\begin{aligned} &= T_{dg} = \frac{1}{\gamma_{mo}} [A_g f_y] \\ &= \frac{1}{1.10} [120 \times 8 \times 250] = 218182 \text{ N} \end{aligned}$$

Factored load on the connection = 120 kN = 120 000 N

$\therefore$ The design is safe.

4. Two plates 160mm X 10mm each are connected by a double cover butt joint with 16mm diameter bolts as shown in figure. The cover plates are 6mm thick. Determine the strength of the joint.

Solution.

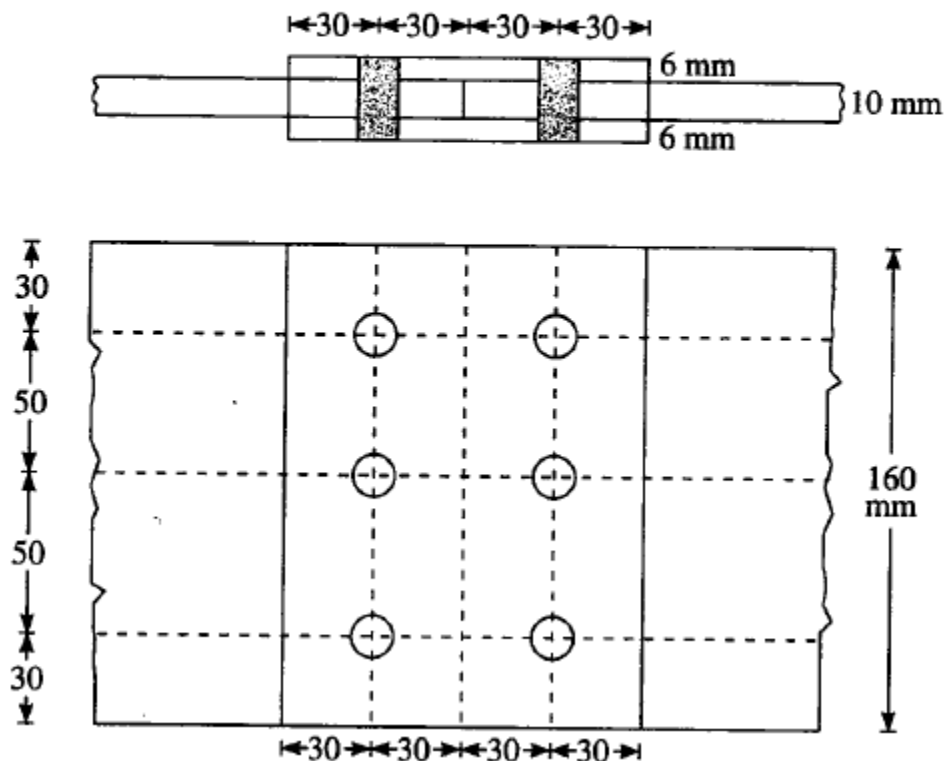
Diameter of bolt = $d = 16 \text{ mm}$

Diameter of bolt hole = $d_o = 16 + 2 = 18 \text{ mm}$

The bolts are in double shear

$$\text{Design strength of 1 bolt in double shear} = 2 \frac{1}{\gamma_{mb}} \frac{f_u}{\sqrt{3}} A_n$$

$$2 \times \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \times 0.78 \pi \times \frac{16^2}{4} \text{ N} = 57950 \text{ N}$$



Total design strength of 3 bolts = $3 \times 57950 = 173850 \text{ N}$

Design bearing strength of 1 bolt = $\frac{1}{\gamma_{mb}} [2.5 k_b d t f_u]$

$K_b = \text{least of the following}$

$$\left. \begin{array}{l} \text{(i)} \quad \frac{e}{3d_o} = \frac{30}{3 \times 18} = 0.556 \\ \text{(ii)} \quad \frac{f_{ubolt}}{f_{uplate}} = \frac{400}{410} = 0.976 \end{array} \right\} \therefore K_b = 0.556$$

$$\begin{aligned} \text{Design bearing strength of 1 bolt} &= \frac{1}{1.25} [2.5 \times 0.556 \times 16 \times 10 \times 400] \text{ N} \\ &= 71168 \text{ N} \end{aligned}$$

$$\text{Design bearing strength of 3 bolts} = 3 \times 71168 = 213504 \text{ N}$$

Design strength of the plate from yielding consideration

$$= \frac{1}{\gamma_{mo}} [A_g f_y] = \frac{1}{1.10} [160 \times 10 \times 250] = 363636 \text{ N}$$

Design strength of the plate from rupture consideration

$$= \frac{1}{\gamma_{mi}} [0.9 A_n f_u] = \frac{1}{1.25} [0.9 (160 - 3 \times 18) 10 \times 410] = 312912 \text{ N}$$

From the above considerations, the design strength of the connection is the least of the strength calculated above

$$= 173850 \text{ N or } 173.85 \text{ kN}$$

5. Two plates 250mm X 12mm are to be connected in a double cover butt joint with 20mm diameter bolts. The factored load on the plates is 500kN. Design the connection. The cover plates are 8mm thick. The bolts may be arranged in a diamond pattern.

Solution

$$d = 20 \text{ mm}$$

Diameter of bolt hole $= d_o = 20 + 2 = 22 \text{ mm}$

The bolts are in double shear.

Design shearing strength of 1 bolt in double shear

$$= 2 \times \frac{1}{\gamma_{mb}} \left[\frac{f_u}{\sqrt{3}} A_n \right] = 2 \times \frac{1}{1.25} \times \frac{400}{\sqrt{3}} (0.78) \frac{\pi \times 20^2}{4} \times 10^{-3} \text{ kN}$$

$$= 90.547 \text{ kN}$$

Based on the shearing strength of the bolt

$$\text{Number of bolts required} = \frac{500}{90.547} = 5.52$$

Provide 6 bolts.

Fig. 4.29 shows the proposed arrangement of bolts.

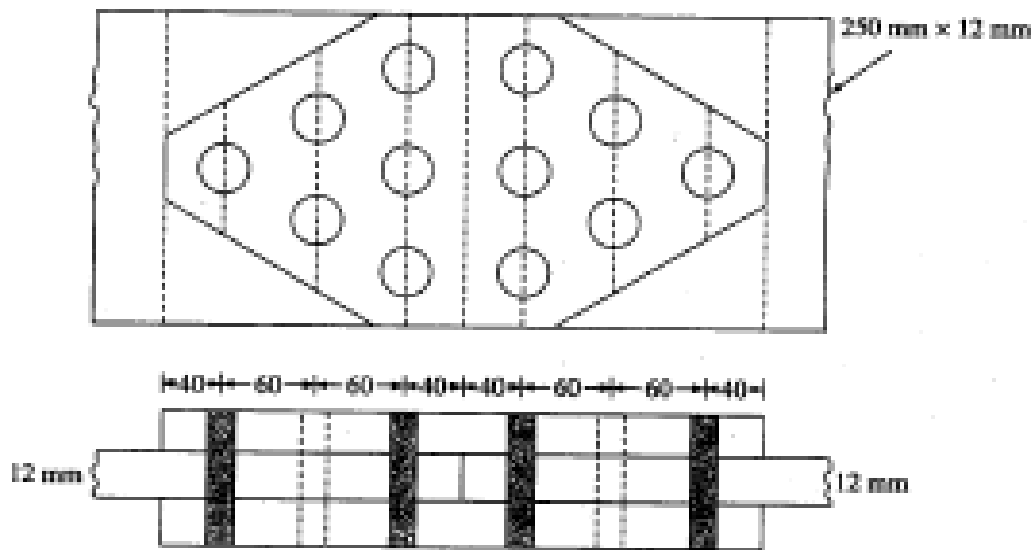


Fig. 4.29

$$\text{Design bearing strength of 1 bolt} = \frac{1}{\gamma_{mb}} [2.5 K_b d r f_u]$$

K_b = least of the following:

$$(i) \frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.606$$

$$(ii) \frac{f_u}{f_y} = \frac{400}{410} = 0.975 \quad \therefore K_b = 0.606$$

$$\therefore \text{Design bearing strength of 1 bolt} = \frac{1}{1.25} [2.5 \times 0.606 \times 20 \times 12 \times 400] 10^{-3} \text{ kN}$$

$$= 116.352 \text{ kN}$$

6. A line shaft transmits a load of 25kN at an eccentricity of 500mm across a bracket plate bolted to a stanchion. Two rows of bolts 100mm apart are provided with five bolts per row. The pitch of the bolts in each row is 60mm. Find the force induced in any bolt.

Solution.

Resistance against translation per bolt

$$= \frac{P}{n} = \frac{25 \times 10^3}{10} = 2500 \text{ N}$$

Fig. 4.45 shows the arrangement of the bolts.

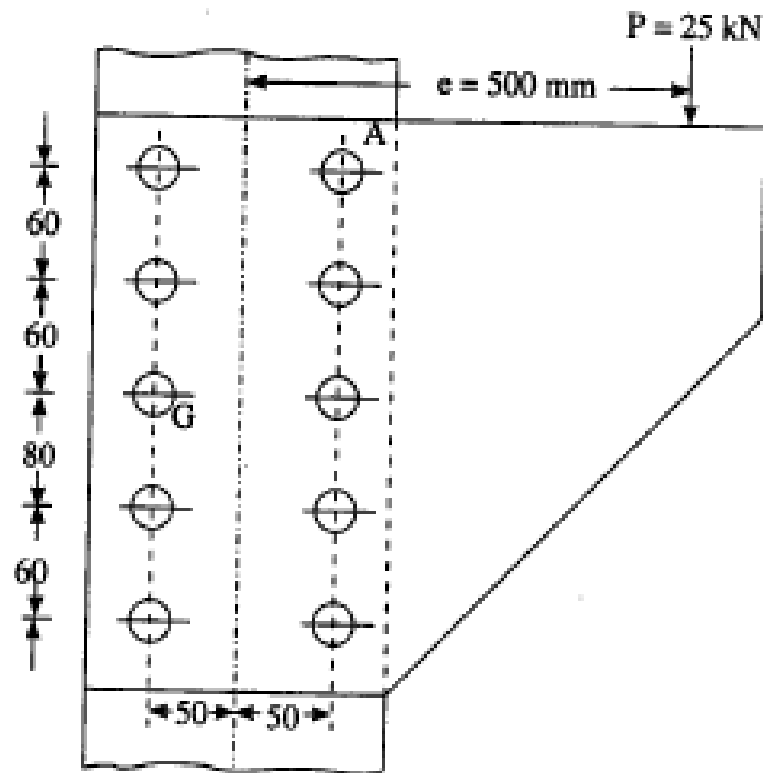
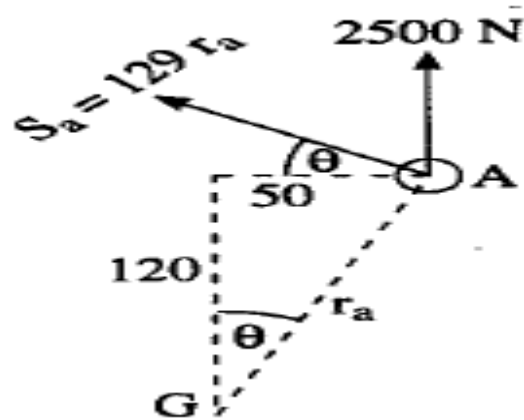


Fig. 4.45

With respect to the centroid G of the bolt group

$$\begin{aligned} \Sigma x^2 + \Sigma y^2 &= 10(50)^2 + 4(120)^2 + 4(60)^2 \text{ mm}^2 \\ &= 97000 \text{ mm}^2 \end{aligned}$$

$$K = \frac{Pe}{\Sigma x^2 + \Sigma y^2} = \frac{25000 \times 500}{97000} = 129 \text{ N/mm}$$



Now consider the bolt A

Resisting force against rotation

$$= S_a = K r_a = 129 r_a$$

See Fig. 4.46.

Total vertical component on the bolt A

$$\begin{aligned} &= V = 2500 + 129 r_a \sin \theta \\ &= 2500 + 129 \times 50 = 8950 \text{ N} \end{aligned}$$

Horizontal component on the bolt A

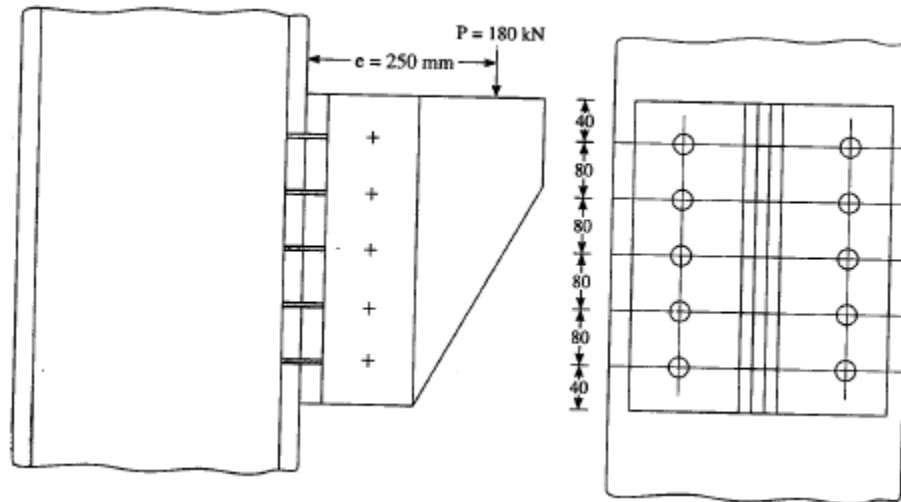
$$\begin{aligned} &= H = 129 r_a \cos \theta \\ &= 129 \times 120 = 15480 \text{ N} \end{aligned}$$

Resultant resistance offered by the bolt A

$$\begin{aligned} &= \sqrt{V^2 + H^2} \\ &= \sqrt{8950^2 + 15480^2} = 17881 \text{ N} \end{aligned}$$

This is the greatest force transmitted to a bolt.

7. Investigate the safety of the bolts connecting the angles and the column flange in the arrangement shown in figure. The bolts are 20mm in diameter. The factored load on the bracket is 180kN at an eccentricity of 250mm from the column flange.



Solution.

Bolt diameter $= d = 20 \text{ mm}$

Design shearing strength of the bolt

$$V_{db} = \frac{1}{\gamma_{mb}} \left[\frac{f_u}{\sqrt{3}} (0.78) \frac{\pi d^2}{4} \right]$$

$$= \frac{1}{1.25} \left[\frac{400}{\sqrt{3}} (0.78) \frac{\pi \times 20^2}{4} \right] = 45274 \text{ N}$$

Design tensile strength of the bolt

$$T_{db} = 0.9 f_u (0.78) \frac{\pi d^2}{4}$$

$$= 0.9 \times 400 (0.78) \frac{\pi \times 20^2}{4} = 88216 \text{ N}$$

h = Height of the upper most bolts from the lower edge of the bracket
 $4 \times 80 + 40 = 360 \text{ mm}$

Height of neutral axis above the lower edge of the bracket

$$= \frac{h}{7} = \frac{360}{7} = 51.43 \text{ mm}$$

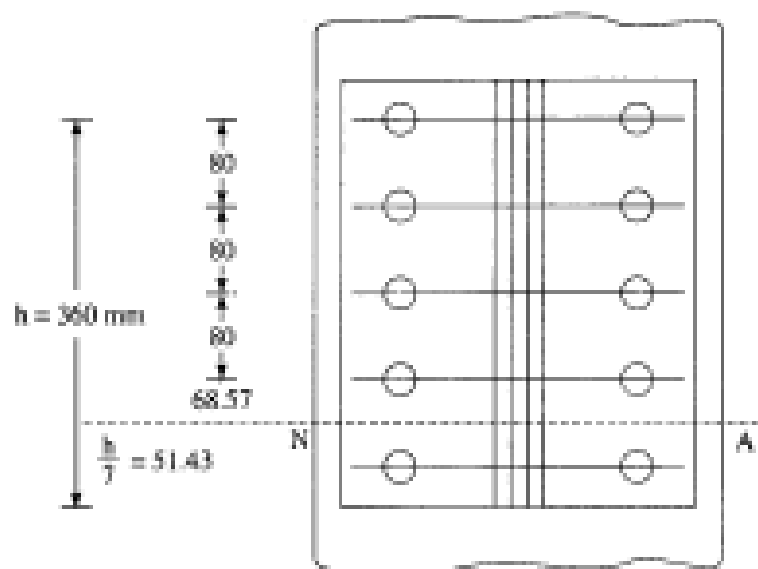


Fig. 4.66

There are 4 pairs of bolts above the neutral axis

$$\begin{aligned}\Sigma y &= 68.57 + 148.57 + 228.57 + 308.57 \\ &= 754.28 \text{ mm}\end{aligned}$$

$$\begin{aligned}\Sigma y^2 &= 68.57^2 + 148.57^2 + 228.57^2 + 308.57^2 \\ &= 174234.58 \text{ mm}^2\end{aligned}$$

$$K = \frac{Pe}{\Sigma y^2 + \frac{2h}{21}\Sigma y}$$

$$= \frac{180 \times 10^3 \times 250}{174234.58 + \frac{2 \times 360}{21}(754.28)} = 224.89 \text{ N/mm}$$

Maximum tension in the upper most bolt $T_{\max} = Ky_{\max}$

$$\begin{aligned}&= 224.89 \times 308.57 \text{ N} \\ &= 69394 \text{ N}\end{aligned}$$

$$\text{Shear force on the bolt} = V_b = \frac{180000}{10} = 18000 \text{ N}$$

$$\begin{aligned}\left[\frac{V_b}{V_{db}} \right]^2 + \left[\frac{T_{\max}}{T_{db}} \right]^2 &= \left[\frac{18000}{45274} \right]^2 + \left[\frac{69394}{88216} \right]^2 \\ &= 0.158 + 0.619 = 0.777 < 1\end{aligned}$$

∴ The design is safe.

2.12 Assignment Questions

1. Name the types of connections.
2. Define bolted connection.
3. Define a bolt and name the types of types.
4. What are the advantages and disadvantages of bolted connection?
5. What are the different types of bolted joints?
6. Explain the failure modes of bolted joints.

2.13 Outcome

- Gives an idea about the necessity of connection
- Give an brief note on advantages and disadvantages of bolted connection
- Based on the requirement one can select the bolted joint and arrangement of bolts

2.14 Future Scope

- By understanding the bolted connections, one design the connection roof truss, girders column splices.

Chapter – 3 Welded Connections

Contents:-

Introduction

Objective

Welding process

Advantages and disadvantages of welded connections

Welding electrodes

Types of welds

Defects in welds

Weld symbols

Types of welded joints

Problems

Assignment questions

Outcome

Future study

Introduction

When two structural members are to be connected by means of welds, then that structural connection is known as **Welded connection**.

Welding is a process of connecting two pieces of metals by establishing a metallurgical bond between them by fusion or pressure or both.

Objective

- To know different types of welds and weld connections
- To study different types of weld defects

Welding process

Commonly used welding methods are

- Electric Arc Method
- Oxy-Acetylene gas Method

In case of electric arc method, the molten or fused metal is deposited in between two parts of metal need to be connected. The metal parts are also fused to a specified depth. When the fused metal is deposited, two parts of metal are joined to a new metal. The ends of the metal part and the tip of the electrode are fused by electric arc which causes heat and develops a high temperature of 3300°C . The electrode or weld rod is coated with flux which vaporizes due to the action of heat and protective shield is formed which prevent the molten metal from absorbing atmospheric gas.

In case of Oxy-Acetylene gas welding, jet of oxygen and acetylene gas are the source of heat. Gas welding is used to cut metal pieces to shapes. Due to mechanically controlled gas welding equipment, it produces a smooth cut similar to that of sawed cut. Even though gas welding is simple and inexpensive, it is commonly adopted for repair and maintenance works.

Different processes of arc welding are

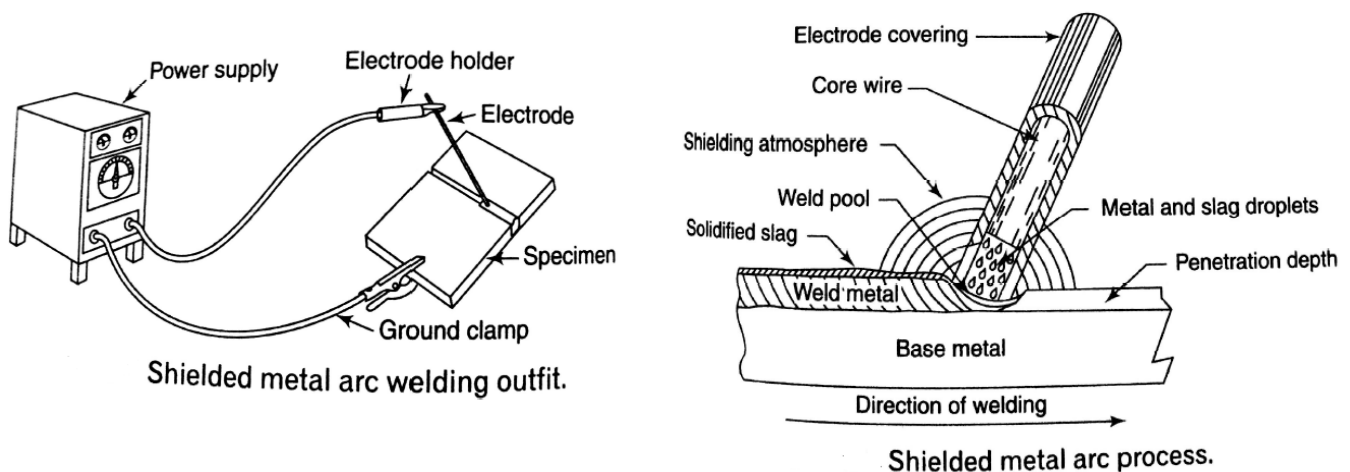
- Shielded metal arc welding (SMAW)
- Submerged arc welding (SAW)
- Gas-shielded metal arc welding (GMAW)
- Flux core arc welding (FCAW)
- Electro slag welding (ESW)
- Stud welding (SW)

Shielded metal arc welding (SMAW)

This also called as stick welding which is a manual process commonly adopted for structural welded connections having low capital cost and flexibility.

The SMAW requires the following setup as shown in figure.

1. Constant current (CC) welding power supply
2. Electrode holder, lead and its terminals
3. Ground clamp, lead and its terminals
4. Welding electrodes



The electrons flows through the gap between the ends of electrode and the metal produce an electric arc which causes heat and develops a high temperature of 3300°C . The metal electrode and base metals fuses and tiny granules of metal form at the tip of the electrode and are transferred in to the molten weld pool formed on the base metal. As the electrode moves away from the molten metal pool, the molten mixture of electrode and base metal solidifies and weld is completed.

SMAW can be both a low-hydrogen and non-hydrogen process depending on type of electrode used. The low-hydrogen electrodes are identified by 5, 6 or 8 as the last number in their designation and all other are non-hydrogen electrodes.

Using SMAW, welds from 3 to unlimited thickness are possible. Thickness less than 3 can be made but it requires special skills.

Advantages of SMAW

- 1) Low capital cost
- 2) Relatively portable
- 3) Welds can be performed in any position.

- 4) Can be performed in any weather conditions except rainy season.
- 5) Frequent stops and starts in between the process leads to weld defects.

Advantages and Disadvantages of welded connections

Advantages of welded connections

- The welding process offers air-tight and water-tight joints.
- Welded connections eliminate the need of making holes in the members.
- Welded structures are comparatively lighter than corresponding riveted or bolted connections.
- Welded connections provide rigidity. Welded truss deflects very less than compared to bolted connections and they are subjected to less bending moments.
- Repair and further new connections can be done easily and faster than bolting.
- Welded joints are more economical than bolted joints. For a welded structure, maintenance and painting can be done easily.
- Efficiency of welded joints is more when compared to bolted joints and we can obtain 100% efficiency in case of welding.
- Welded structure gives better finish and appearance than bolted or rivet structures.
- Connecting angles, gusset plates and splice plates can be minimized in many cases.
- The welding process is noiseless than compared to riveting.

Disadvantages of welding

- Requires highly skilled labors and supervision.
- Inspection of welded joints is difficult and expensive.
- Costly equipment is required.
- Members jointed by welding may distort, unless proper precautions are taken.
- Proper welding cannot be done in field condition especially in vertical and overhead positions.
- Welding at site is not feasible due to lack of power supply.
- Welded connections are more prone to cracking under fatigue loads.
- Welding done at field is more expensive than performed in shops.

Welding electrodes

A variety of electrodes are available so that a proper match of base metal strength and metallurgical properties of weld metal can be chosen.

If the molten metal is not shielded properly, it will absorb hydrogen and other atmospheric gases and due to oxidation will result in weld defects such as porosity, brittleness and susceptibility to corrosion. Hence only coated electrodes are used in the structural welding. Welding electrodes

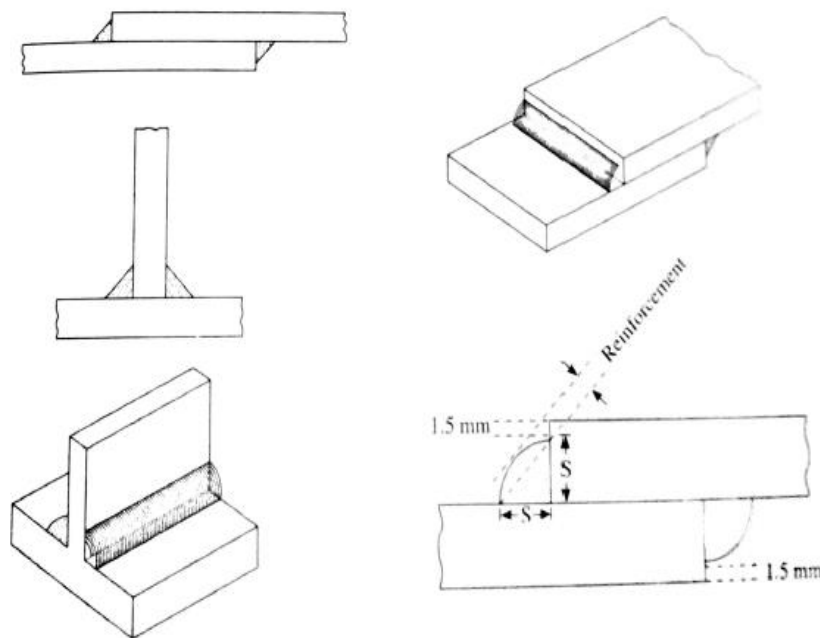
are by American welding society in cooperation with ANSI using the following numbering system for shielded metal arc welding (SMAW).

Exxxbc

In this numbering system, E stands for electrodes, xxx stands for 2 or 3 digit number establishing ultimate tensile strength of weld metal. As per IS 814 the following values are available; 40, 41, 42, 43, 44, 50, 51, 53, 54, 55, 56 kg/cm². The value b indicates the suitability of welding positions, which may be flat, horizontal, vertical and overhead; b = 1 for all positions; b = 2 for flat positioning work; b = 4 for flat, horizontal and vertical down and c stands for coating and operating characteristics. The value of c equal to 5, 6 or 8 indicates low hydrogen.

Types of welds

1) Fillet weld:-



- These are commonly used type of welds.
- Fillet welds are theoretically triangular in cross-section.
- Fillet welds join two surfaces at approximately right angles to each other in case of lap, tee and corner joints.

Advantages of fillet weld

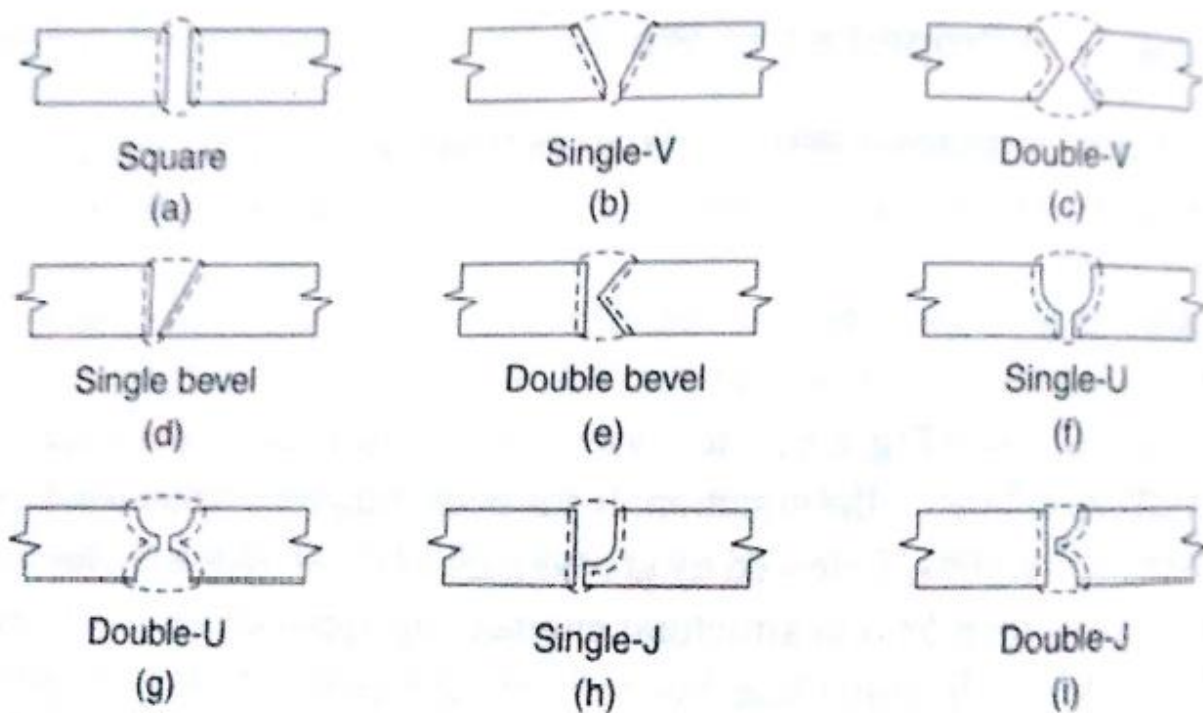
- Edge penetration is not necessary.
- Simple and economical.
- No need of skill labours.

Disadvantages of fillet welds

- These types of welds cannot carry high magnitude of loads.
- Poorer for fatigue loads.
- Less attractive in appearance.

2) Groove welds:-

- Groove welds are made in a groove between the edges of two plates of members to be connected.
- Groove welds are used to connect structural members that lying in the same plane i.e., often used as Butt joint. It may also be used to connect T- sections.
- It has a slope of 30° to 60° .
- A groove weld will transmit the full load of the member they join and it should have same strength as that of members they join.
- Fully penetration groove welds are commonly employed for structural connections.



Types of groove welds

Groove welds are classified as

- 1) Complete or fully penetration groove welds.
- 2) In –complete or partial penetration groove welds.

- 1) **Complete or fully penetration groove welds:-** In this case, weld metal and base metal are fused for the whole depth of the joint. This weld is made by welding on both sides or welding one side up to backing bar.
- 2) **In-complete or partial penetration groove welds:-** In this case, the weld metal is intentionally not deposited through the full thickness of the joint. These types of welds are used when the forces to be transmitted are less compared to complete penetration welds. These welds are useful when access from both the sides is impractical.

Advantages of groove welds

- Better fatigue resistance.
- These types of welds have better appearance.

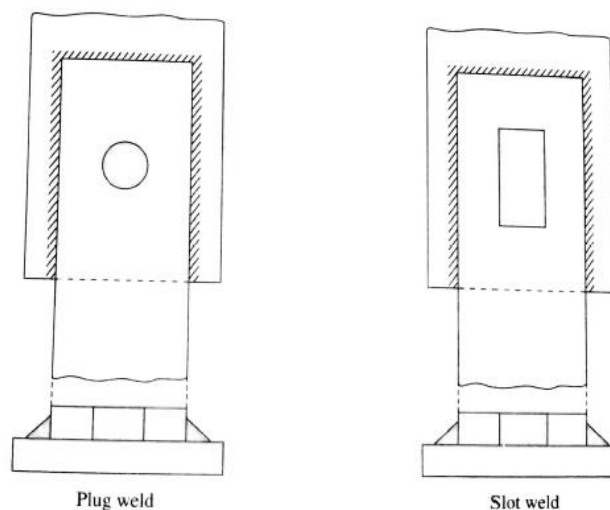
Disadvantages of groove welds

- This type of welds is more expensive.
- These types of welds require more skills.

Plug and slot welds

In situation where, necessary length of weld cannot be provided due to limitations in the overlap region, plug and slot welds are used which acts as supplement or additional strength requirements. These welds are used to resist shear like fillet welds and prevents buckling of compression members.

The welds are made with lapped parts in contact by depositing weld metal in circular or slotted holes made with work path. The openings are partly or completely filled.



The dimensions of the hole or slot should be done with following limits

- The width or diameter should not be less than 3 times the thickness (t).
- Corners at the enclosed ends of the slot should be rounded with a radius should not be less than 2 times the thickness.
- Distance between the edge of the part and edges of the slot are between the adjacent slots or holes should not be less than 2 times the thickness.
- For slot welds, the length of the slot shall not exceed 10 times the plate thickness. The width of the slot should not be greater than 2.25 times the thickness and should be at least equal to the diameter plus 8mm.

Note:-

- 1) **Fillet welds are most commonly used for light loads.**
- 2) **Groove welds are most efficient to transmit heavy loads due to high strength of base metal.**
- 3) **Plug and slot welds have limited applications and are used in special conditions.**

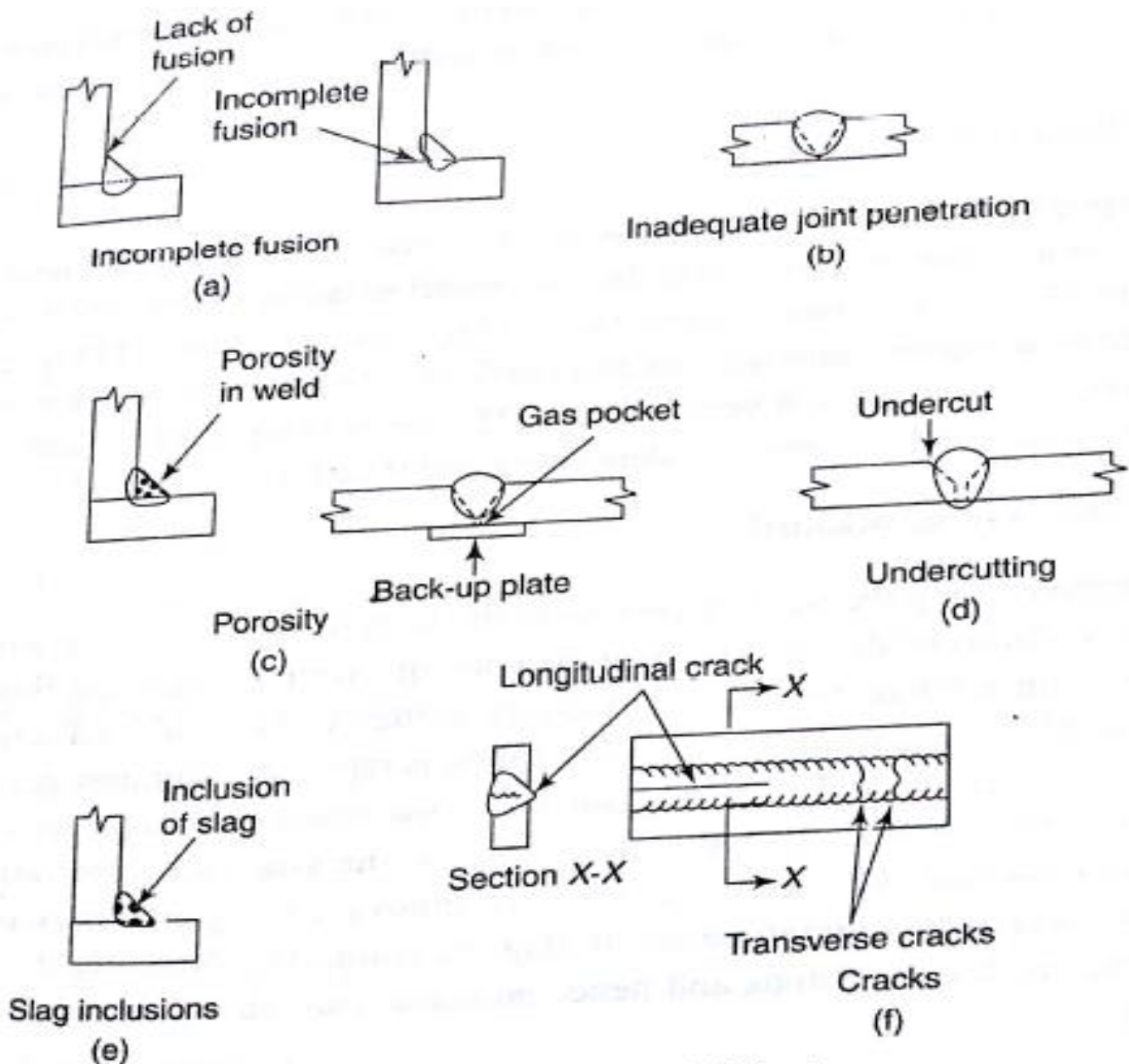
Defects in welds

- 1) **Incomplete fusion:-** Incomplete fusion may result if the surfaces to be jointed have not been cleaned properly and are coated with oxides, mill scale, and other foreign materials. Insufficient current supplied by the welding equipment and high rate of welding will also results in incomplete welding.
- 2) **Incomplete penetration:-** This effect is due to the failure of weld metal to penetrate the complete depth of the joint, where full penetration has been specified. This defect found in groove welds is due to unsuitable groove design for the selected welding process, use of large size of electrodes, insufficient welding current and excessive welding rates.
- 3) **Porosity:-** Porosity is formed when gas pockets or voids are entrapped during cooling process. It may be due to excessive high current. The latter is due to poor welding procedure and careless use of back up strips. Porosity results in stress concentration and reduces ductility of the metal.
- 4) **Slag inclusion:-** Slag is formed during the welding process in the form of metal oxides as a result of chemical reactions of the melted electrode coating. Normally slag floats over the surface and is removed after the weld is cooled. However, if the cooling is rapid, the slag may get trapped in the weld before it rises up to the surface. When several passes of welds are made to achieve desired weld thickness, slag that forms after each successive pass should be removed. The non removal of slag after each pass in a multipass weld is common cause of slag inclusion.

- 5) **Cracks:-** Cracks occur in the form of breaks in the weld metal. They result from internal stresses in the welds and may be either longitudinal or transverse to the line of weld. These are the most harmful of weld defects.

Cracks are classified as hot and cold cracks. Hot cracks occur due to the presence of sulphur, carbon, silicon and hydrogen in the weld metal. Phosphorous and hydrogen trapped in the hallow spaces of the metal structure give rise to the formation of cold cracks. Preheating of metals to be welded will eliminates the formation of cracks.

- 6) **Under cutting:-** It is the local decrease of the thickness of the metal at the weld toe. Undercut may result in loss of gross section and will act as a notch this may be due to excessive current.

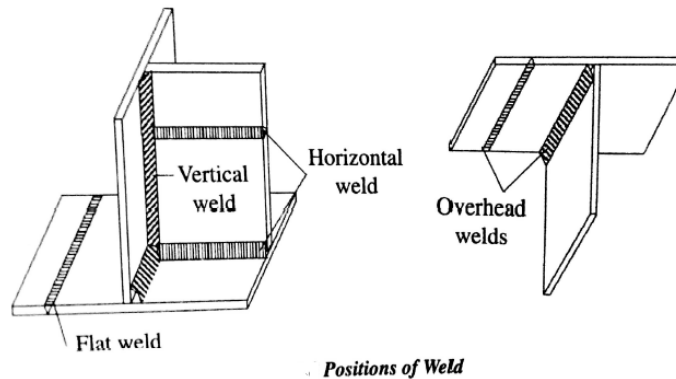


Possible weld defects

Positions of welds

Based on the position of weld, it can be classified as

- Flat welds
- Horizontal welds
- Vertical welds
- Overhead welds



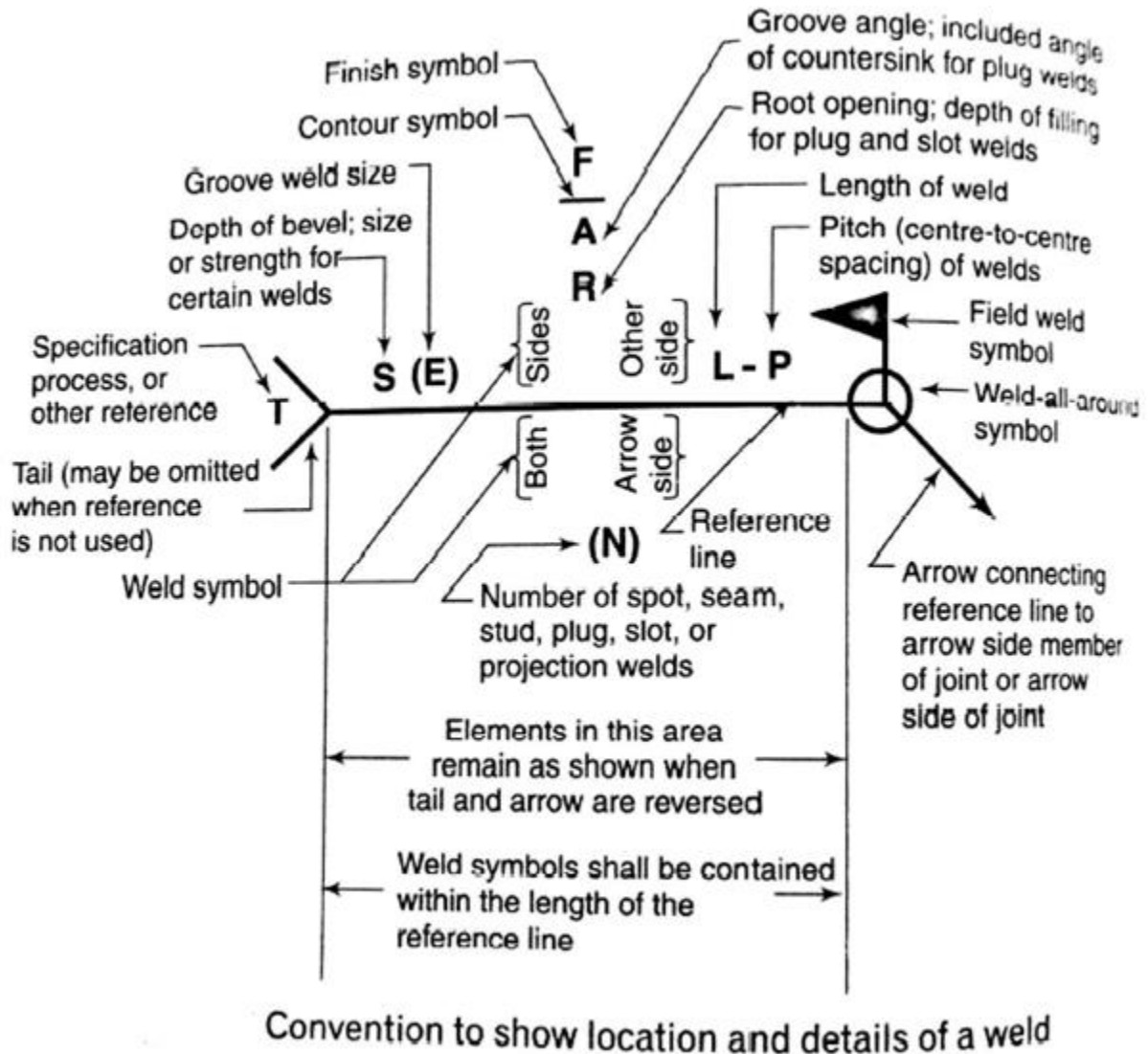
Weld symbols

The weld symbol is the basic symbol used to indicate the type of weld. The standard weld symbols are used in the drawing sheet for different types of welds are shown in figure. Symbols save a lot of space as descriptive notes can be omitted. These weld symbols are standardized by the American Welding Society (AWS).

BASIC ARC AND GAS WELD SYMBOLS							
Bead	Fillet	Plug or slot	Groove				
			Square	V	Revel	U	J

Basic weld symbols				Supplementary weld symbols				
Spot	Projection	Seam	Flash or upset	Weld all around	Field weld	Contour		
						Flush	Convex	Concave

Basic weld symbols

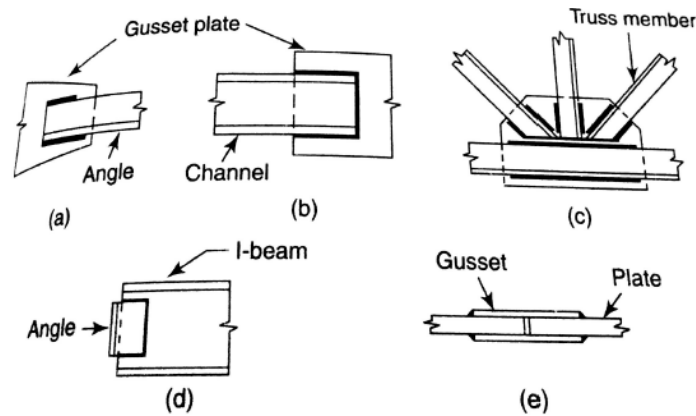


Types of welded joints

- **Simple joints:-** Simple connections are defined as joints between members that have not been designed with the intension that they transmit significant moments. Their purpose is to transfer load from the supported member into the supporting member in such a way that essentially only direct forces are involved. For example, beam – beam connection, column splice and column base connections.

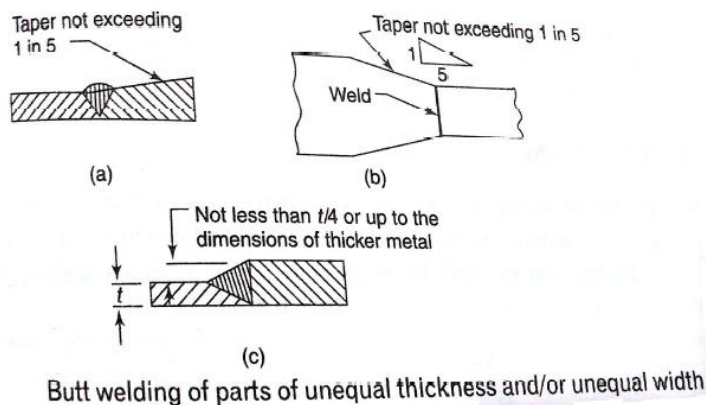
Five basic types are

- a) Lap joints:-** Lap joints are commonly used because they offer ease of fitting and ease of jointing. Thus they don't require any special preparation and can accommodate minor errors in fabrication or minor adjustments in length.

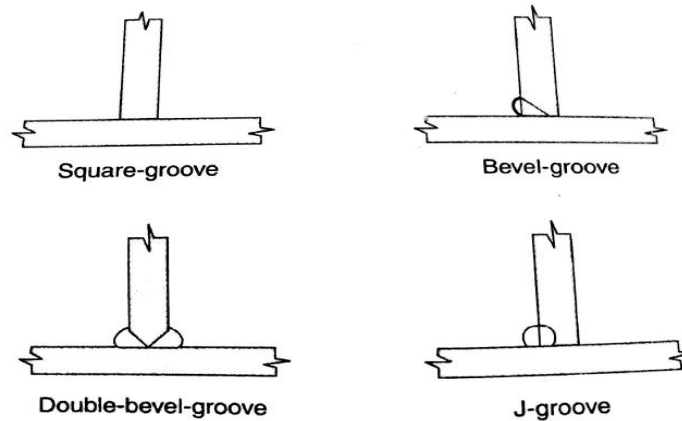


Some examples of welded lap joints

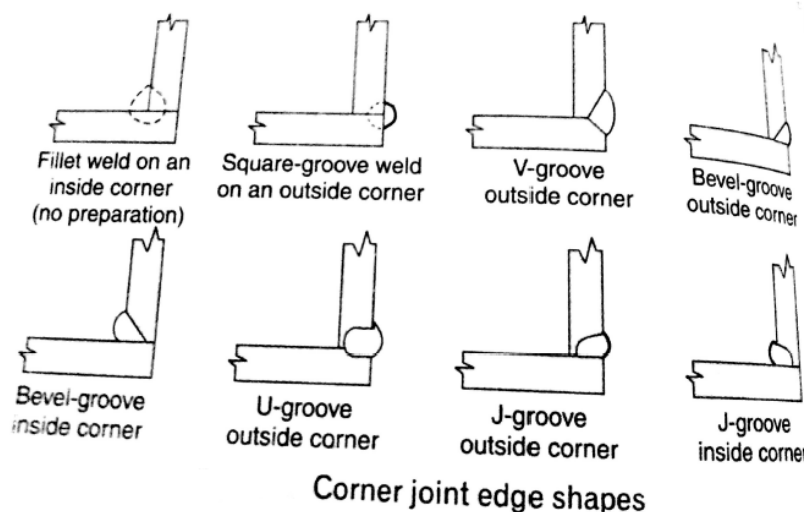
- b) Butt joint:-** A butt joint is used to join the ends of flat plates of nearly equal thickness. This type of joints eliminates the eccentricity developed using lap joints. The butt joint obtained from a full penetration groove welds has 100% efficiency. Such butt joints also minimizes the size of the connection and are aesthetically pleasing than lap joints.



- c) Tee joints:-** T-joints are often used to fabricate built-up sections such as T-shapes, I-shapes, plate girders, hangers, brackets and stiffeners, where two plates are joined at right angles. T-joints can be made either fillet or groove welds.



- d) **Corner joints:-** Corner joints are used to form built-up rectangular box sections, which may be used as columns or beams to resist high torsional forces.



Corner joint edge shapes

- e) **Edge joints:-** Edge joints are not used in structural engineering applications. They are used to keep two or more plates in a given plane.
- **Moment resistant connection:-** Moment resistant connections are used to transferring moment, axial forces and shear forces from one member to another. Moment resistant connections with welds are made using plates only. A plate is provided over the top of the flange of beam. The plate is tapered to facilitate fillet weld.
Moment resistant connection includes
 - Eccentric connections,
 - Stiffened beam seated connections and
 - Continuous beam – beam connections.

Problems

1. A groove weld is provided to connect two plates of thickness 18mm and 16mm. the factored tensile force on the joint is 450kN. The length of the weld provided is 160mm. investigate the safety of joint if,
 - a) Single V-groove weld is provided
 - b) Double V-groove is provided. Assume shop welding.

Solution. Design stress for a groove weld $= f_{wd} = \frac{f_y}{\gamma_{mw}} = \frac{250}{1.25} = 200 \text{ N/mm}^2$

(a) When a single V-groove weld is provided



Fig. 5.8

For this case the throat thickness of the weld,

$$t_t = \frac{5}{8} \text{ (thickness of thinner plate)}$$

$$= \frac{5}{8} (16) = 10 \text{ mm}$$

Design strength of the weld $= L_w t_w f_{wd}$

$$= 160 \times 10 \times 200 = 320000 \text{ N} = 320 \text{ kN}$$

This is less than the factored load 450 kN

$\therefore$ The connection is not safe

(b) When a double V-groove weld is provided



Fig. 5.9

For this case, the throat thickness =

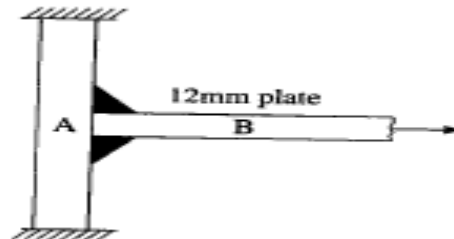
$$t_t = \text{Thickness of the thinner plate} = 16 \text{ mm}$$

Design strength of the weld $= l_w t_w f_{wd} = 160 \times 16 \times 200$

$$= 512000 \text{ N} = 512 \text{ kN} > 450 \text{ kN}$$

This connection is safe.

2. Two plates A and B are connected by a pair of fillet weld as shown in fig. Find the size of the weld required if the plate B is subjected to a factored pull of 2500N per mm width. Assume shop welding.



Solution. Design stress for the weld $= f_{wd} = \frac{f_u}{\sqrt{3} \gamma_{mw}}$

$$= \frac{410}{\sqrt{3} \times 1.25} = 189.4 \text{ N/mm}^2$$

Let the size of the weld be $s \text{ mm}$

Design strength of two fillet welds per mm length

$$= 2[189.4 \times 1 \times 0.7 s] = 2500$$

$\therefore$

$$s = 9.42 \text{ mm}$$

3. A splice has become necessary in a long tension member carrying a factored load of 80kN. The member is a single angle 60mm X 40mm X 8mm. Design a single V-groove welded joint.

Solution. Throat thickness of the V-groove weld $= \frac{5}{8}(t) = \frac{5}{8} \times 8 = 5 \text{ mm}$

Length of weld = Sum of the lengths of arms of the angle

$$= 60 + 40 = 100 \text{ mm}$$

Design strength of the weld $= \frac{f_y}{\gamma_{mw}} (\text{area of weld}) = \frac{250}{1.25} (5 \times 100) \times 10^{-3} \text{ kN}$

$$= 100 \text{ kN} > 80 \text{ kN}$$

Provide 10 mm weld

4. A 80mm X 8mm plate is to be connected to a 120mm X 8mm plate in a lap joint to transmit a factored load of 125kN and providing 6mm site welds, design the connection.

Solution

Thickness of the plate $= t = 8 \text{ mm}$
 Throat thickness of weld $= t_t = 0.7 s = 0.7 \times 6 = 4.2 \text{ mm}$
 Design stress for the weld $= f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.5} = 157.8 \text{ N/mm}$
 Design strength per mm length of weld $= 157.8 \times 1 \times 0.7 \times 6 = 662.8 \text{ N/mm}$
 Factored load $= 125 \text{ kN}$

Effective length of weld required $= \frac{125 \times 10^3}{662.8} = 189 \text{ mm}$

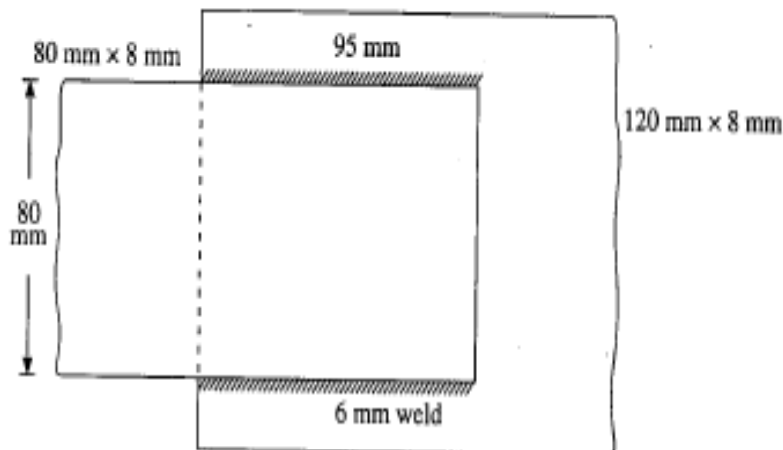
Providing only longitudinal welds, length of weld on each side
 $= \frac{189}{2} = 94.5 \text{ mm}$ say 95 mm

Minimum length of each longitudinal weld = perpendicular distance between the longitudinal welds = 80 mm

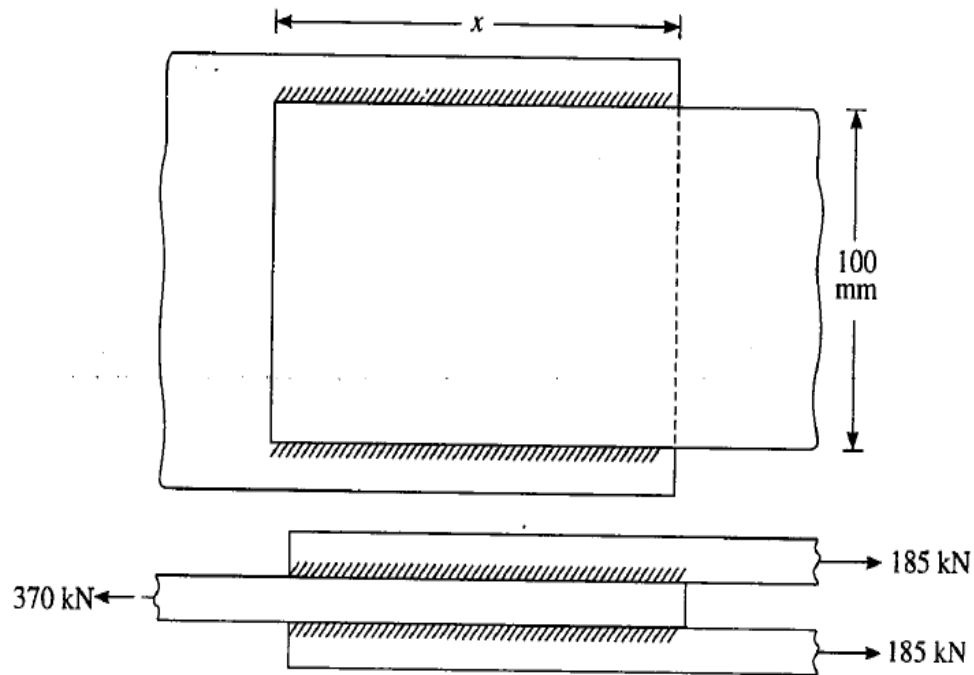
See Fig. 5.11.

The figure does not show end returns

Each end return $= 2 s = 2 \times 6 = 12 \text{ mm}$



5. For the welded joint shown in figure, find the minimum overlap x for a factored pull of 370 kN if 6 mm size welds are used.



Solution.

Throat thickness of the weld

$$t_t = 0.7s = 0.7 \times 6 = 4.2 \text{ mm}$$

Design stress for the weld

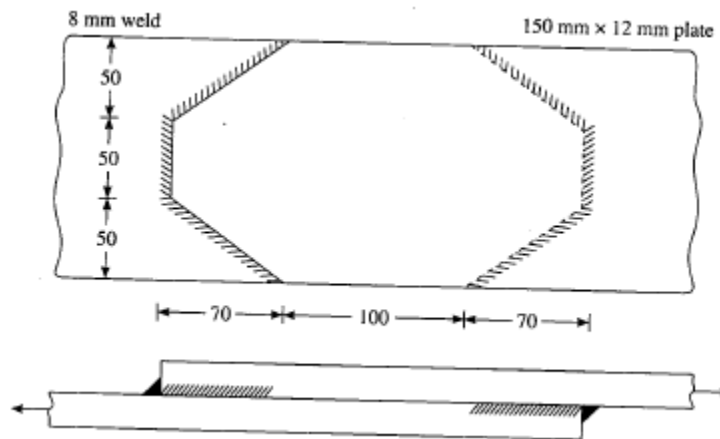
$$f_{wd} = \frac{f_u}{\sqrt{3} \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.5} = 157.8 \text{ N/mm}^2$$

Design strength per mm length of weld

$$= 157.8 \times 1 \times 4.2 = 662.8 \text{ N/mm}$$

Factored pull = 370 kN

6. Figure shows a welded lap joint provided to connect two plates 150mm X 12mm with 8mm fillet weld. Investigate the safety of the design. Assume shop welding.



$$\text{Throat thickness of weld} = t_t = 0.7 \times 8 = 5.6 \text{ mm}$$

$$\text{Design stress for the weld} = f_{wd} = \frac{f_u}{\sqrt{3} \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.4 \text{ N/mm}^2$$

$$\text{Design strength per mm length of weld} = 189.4 \times 1 \times 5.6 = 1060.6 \text{ N/mm}$$

$$\text{Total length of weld provided} = (2 \times 50) + 4\sqrt{50^2 + 70^2} = 444.1 \text{ mm}$$

$$\text{Design strength of the weld} = 1060.6 \times 444.1 = 471012 \text{ N}$$

Design strength of the plate

$$\frac{f_y A_g}{\gamma_{m0}} = \frac{250(150 \times 12)}{1.10} = 409091 \text{ N}$$

Since the design strength of the weld is greater than the design strength of the plate, the connection is safe.

Assignment questions

1. Define weld
2. What are the different types of welds?
3. What are the different types of welded connection?
4. What are the advantages and disadvantages of welded connections?
5. What are the different types of weld defects?

Outcome

- Gives an idea about different types of weld.
- One can select the type of connection based on the loads

Future study

- By knowing this, one can able to design welded plate girder, column splices and roof truss connections.

Chapter – 6 Design of Compression members

Contents:-

- Introduction
- Objective
- Types of column failures
- Different types of sections used as compression members
- Column buckling
- Design compressive strength
- Built up columns
- Lacings
- Battening
- Problems
- Assignment questions
- Outcome
- Future study

Introduction

- Structural Members subjected to axial compression/compressive forces
- Design governed by strength and buckling
- Columns are subjected to axial loads through the centroid.
- The stress in the column cross-section can be calculated as

$$f = \frac{P}{A}$$

Where, f is assumed to be uniform over the entire cross-section.

Objective

- To know the design criteria of compression members
- To study Lacings and battening

Failure modes of an axially loaded column

- Local buckling
- Squashing
- Overall flexure buckling
- Torsional buckling

Elastic buckling of slender compression members

- Slender columns have low crippling load carrying capacity.
- Consider one such column having length 'L' and uniform cross section A hinged at both ends A and B. Let P be the crippling load at which the column has just buckled.

Compression Members Vs Tension Members

- The longer the column, for the same x-section, the greater becomes its tendency to buckle and smaller becomes its load carrying capacity.
- The tendency of column to buckle is usually measured by its slenderness ratio

$$\text{Slenderness Ratio} = \frac{L}{r}$$

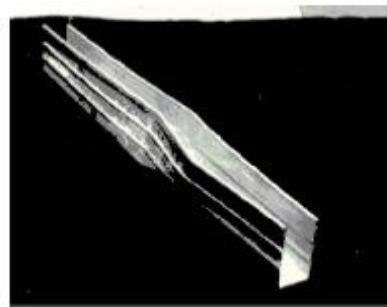
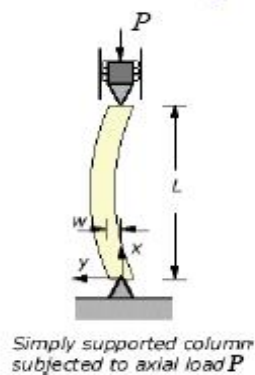
$$\text{where } r = \sqrt{\frac{I}{A}} = \text{radius of gyration}$$

Compression Member Failure

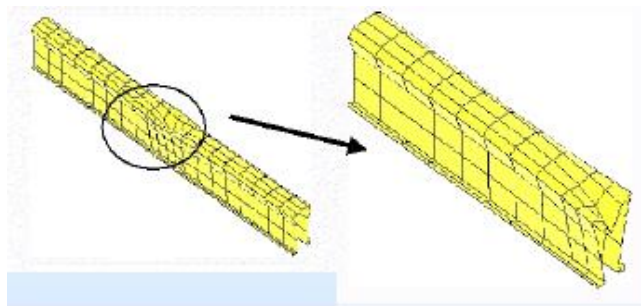
There are three basic types of column failures.

- One, a compressive material failure (very short and fat).
- Two, a buckling failure (very long and skinny).
- Three, a combination of both compressive and buckling failures (length and width of a column is in between a short and fat and long and skinny column).

Flexural Buckling (also called Euler Buckling) is the primary type of buckling members which are subjected to bending or flexure when they become unstable.



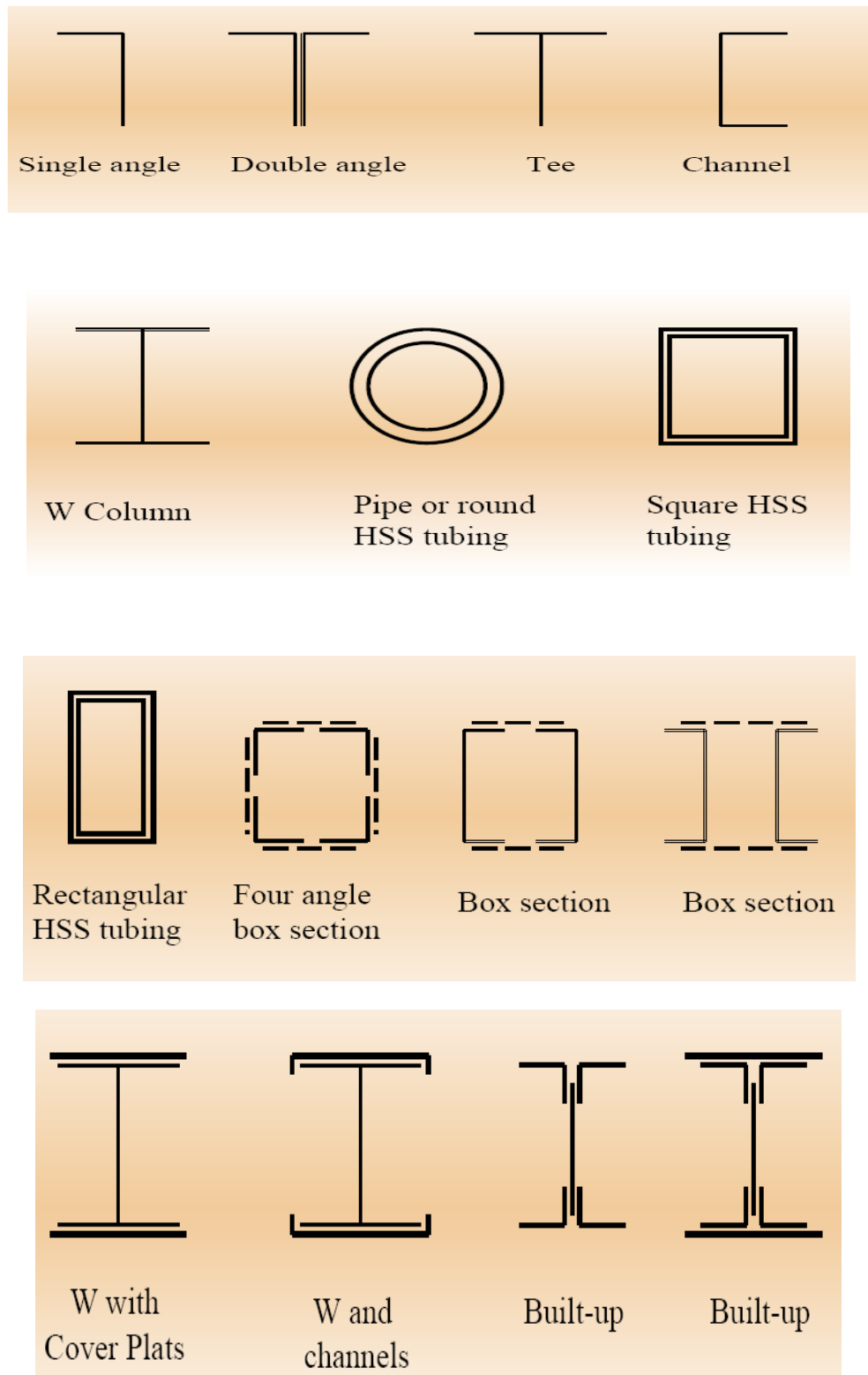
Local Buckling This occurs when some part or parts of x-section of a column are so thin that they buckle locally in compression before other modes of buckling can occur



Torsional Buckling These columns fail by twisting (torsion) or combined effect of torsional and flexural buckling.

Sections used for Compression Member

Figure 1. Types of Compression Members

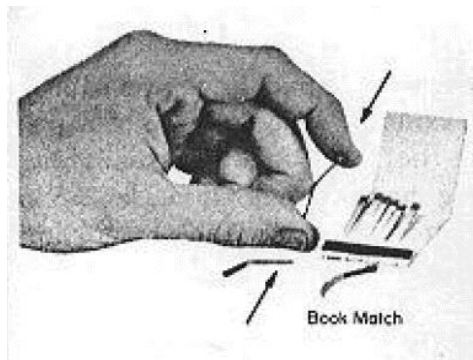
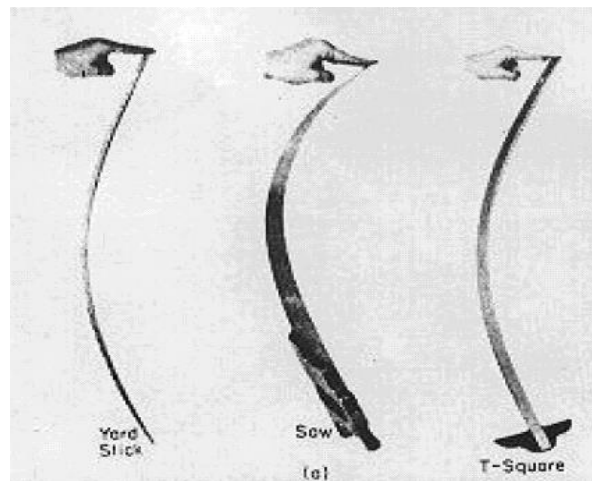


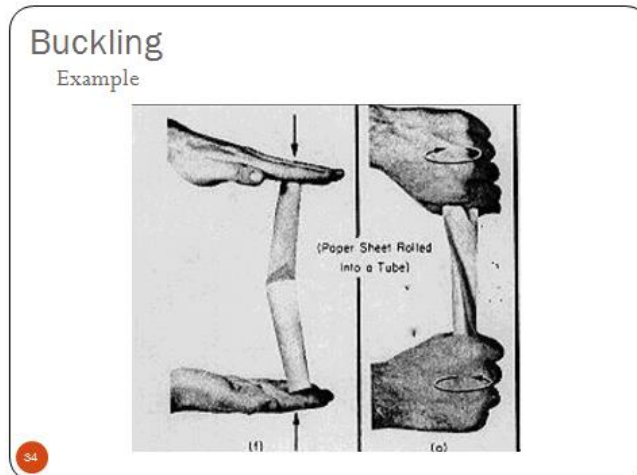
Column Buckling

- Buckling
- Elastic Buckling
- Inelastic Buckling

Buckling is a mode of failure generally resulting from structural instability due to compressive action on the structural member or element involved.

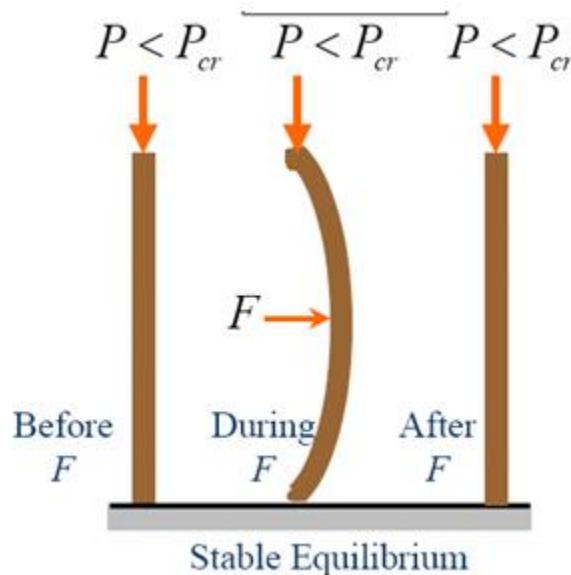
Examples of commonly seen and used tools are provided.





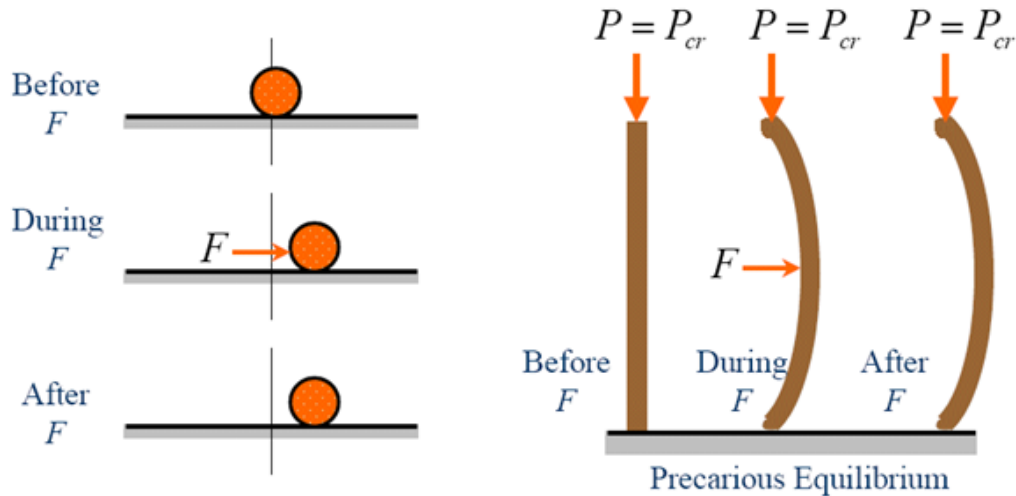
Mechanism of Buckling

Let us consider Fig 1, 2, 3 and study them carefully. In fig1 some axial load P is applied to the column. The column is then given a small deflection by giving a small force F . If the force P is sufficiently small, when the force F is removed, the column will go back to its original straight position.

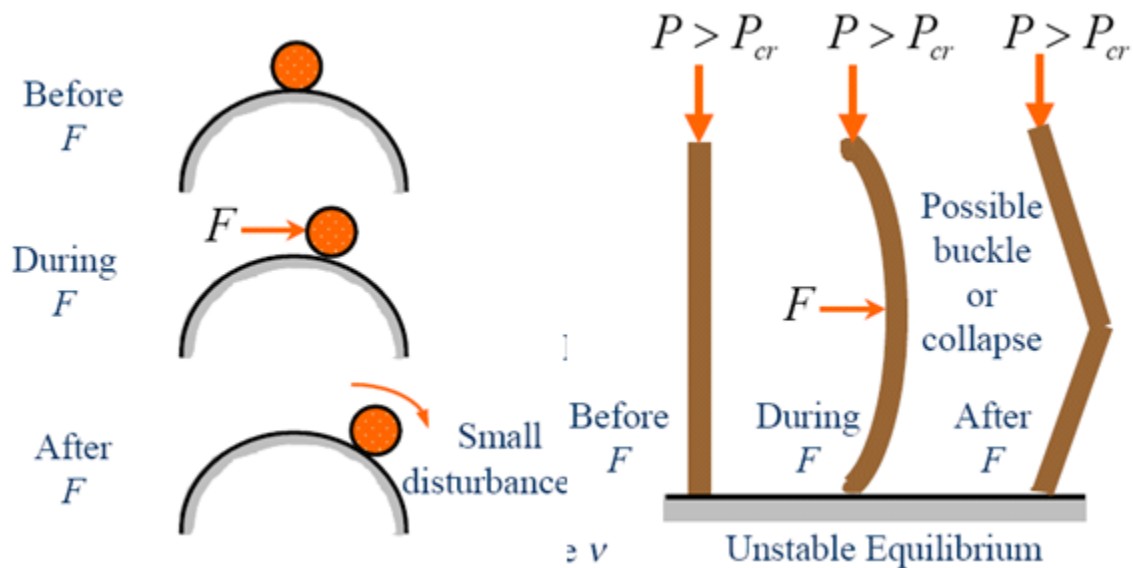


- The column will go back to its original straight position. Just as the ball returns to the bottom of the container.
- Gravity tends to restore the ball to its original position while in columns elasticity of column itself acts as a restoring force.
- This action constitutes stable equilibrium.

- The same procedure can be repeated with increased load until some critical value is reached.



- The amount of deflection depends on amount of force F .
- The column can be in equilibrium in an infinite number of bent positions.



- The elastic restoring force was not enough to prevent small disturbance growing into an excessively large deflection.
- Depending on magnitude of load P , column either remain in bent position, or will completely collapse or fracture.

- This type of behavior indicates that for axial loads greater than P_{cr} the straight position of column is one of unstable equilibrium in that a small disturbance will tend to grow into an excessive deformation.
- Buckling is unique from our other structural elements considerations in that it results from state of unstable equilibrium.
- Buckling of long columns is not caused by failure of material of which column is composed but by determination of what was stable state of equilibrium to an unstable one.
- The value of P at which a straight column becomes unstable is called the Critical Load.
- When column bends at critical load, it is said to have buckled.
- Therefore, critical load is also called the buckling load.

Design compressive strength

7.1.2 The design compressive strength P_d , of a member is given by:

$$P < P_d$$

where

$$P_d = A_e f_{cd}$$

where

A_e = effective sectional area as defined in 7.3.2, and

f_{cd} = design compressive stress, obtained as per 7.1.2.1.

7.3.2 Effective Sectional Area, A_e

Except as modified in 3.7.2 (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

7.1.2.1 The design compressive stress, f_{cd} , of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

$$\phi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r} \right)^2 / \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2}$$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r ;

α = imperfection factor given in Table 7;

χ = stress reduction factor (see Table 8) for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{\left[\phi + (\phi^2 - \lambda^2)^{0.5} \right]}$$

γ_{m0} = partial safety factor for material strength.

Table 7 Imperfection Factor, α
(Clauses 7.1.1 and 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Assumptions made in the design of compression members

- The column is assumed to be absolutely straight.
- The modulus of elasticity is assumed to be constant in a built- up column
- Secondary stresses are neglected

Design steps

- For beginners, for an average column size of 3-5 m the slenderness ratio of 40 to 60 is selected. For very long column a λ of 60 may be assumed. For column with very heavy factored load a smaller value of slenderness ratio should be assumed.
- Choose a trial section by assuming an appropriate slenderness ratio from following table

Type of member	slenderness ratio
Single angle	100-50
Single channel	90-110
Double angles	80-120
Double channels	40-80
Single I -Section	80-100
Double I - section	30-60

- Select a trial section by referring the table above and from steel tables
- Calculate KL/r for the section selected. The calculated value of slenderness ratio should be within the max limiting value given by IS 800-2007 (page 20)

Built-up Column members

- Laced member
- Struts with batten plates
- Battened struts
- Members with perforated cover plates

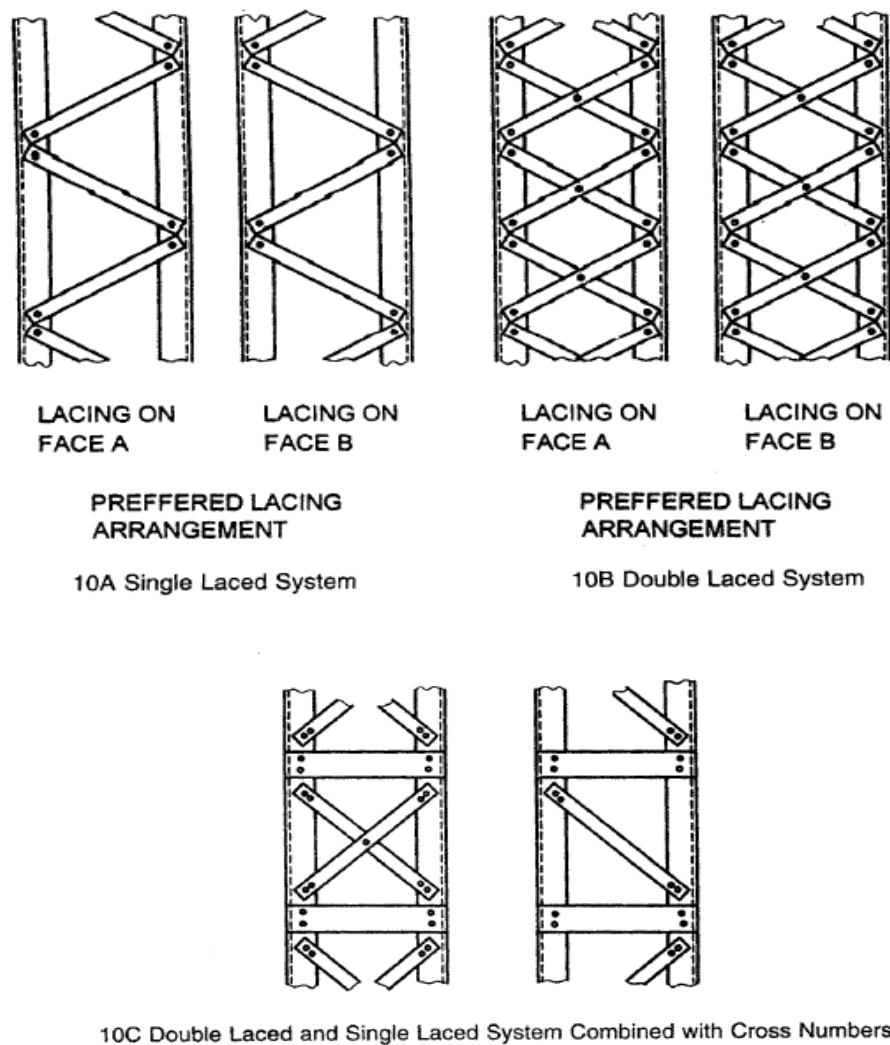
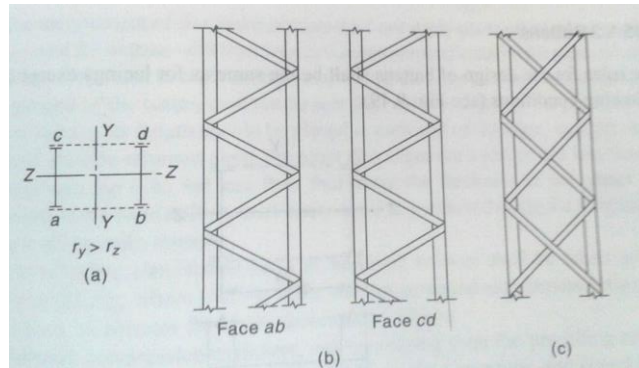


FIG. 10 LACED COLUMNS

Lacings: rules specified in IS 800: 2007

- Radius of Gyration of combined column @ axis perpendicular to plane of lacing $>$ radius of gyration @ axis parallel to plane of lacing (i.e. $r_y > r_z$) figure (a)
- Lacing system should be uniform throughout the length of column
- Single and double laced systems should not be provided on opposite sides of the same member (fig. b and c)



- Lacing shall be designed to resist a total transverse shear V_t at any point in the member, equal to 2.5% of the axial force in the member, and this shear shall be divided among the lacing systems in parallel planes.
- Lacings in addition should be designed to resist any shear due to bending moment or lateral load on the member.
- Slenderness ratio of lacing shall not exceed 145
- Effective length shall be taken as the length between inner end bolts/rivets of the bar for single lacings and 0.7 times the length for double lacings effectively connected at intersections. For welded bars the effective length is taken as 0.7 times the distance between the inner ends of the welds connecting the single bars to the members.
- Min width of lacing bar shall not be $<$ than app 3 times dia of the connecting rivet /bolt; the thickness shall not $<$ than $1/40^{\text{th}}$ of effect length for single and $1/60^{\text{th}}$ for double lacing
- Spacing of lacing bars shall be such that the max slenderness ratio of the components of main member between two consecutive lacing connections is not $>$ than 50 or 0.7 times the most unfavourable slenderness ratio of the combine column.
- When welded lacing bars overlap the main members the amount of lap should not be $<$ than 4 times the thickness of the bar and the welding is to be provided along each side of

bar for the full length of the lap. Where lacing bars are fitted between main members, they should not be connected by fillet weld or by full penetration butt weld.

- Plates shall be provided at the ends of laced compression members and shall be designed as battens.
- Flats, angles, channels or tubes may be used as lacings
- Whether double or single the angle of inclination shall be between 40deg to 70deg to axis of the built-up member.
- The effective slenderness ratio $(KL/r)_e$ of the laced column shall be taken as 1.05 times $(KL/r)_0$ where $(KL/r)_0$ is the max actual slenderness ratio of the column, to account for shear deformations effects.
- The required sections of lacing bars as compression/tension members may be determined using the appropriate design stresses f_{cd} as given before.

Battens: rules as per IS 800:2007(Cl. 8.8,page 50)

- No of battens shall be such that the member is divided into not < than three bays.(i.e there should be min of three bays)
- Battens are designed to resist simultaneously;

Longitudinal shear

$$V_b = V_t \cdot L_o / n_s$$

And moment

$$M = V_t \cdot L_o / 2n$$

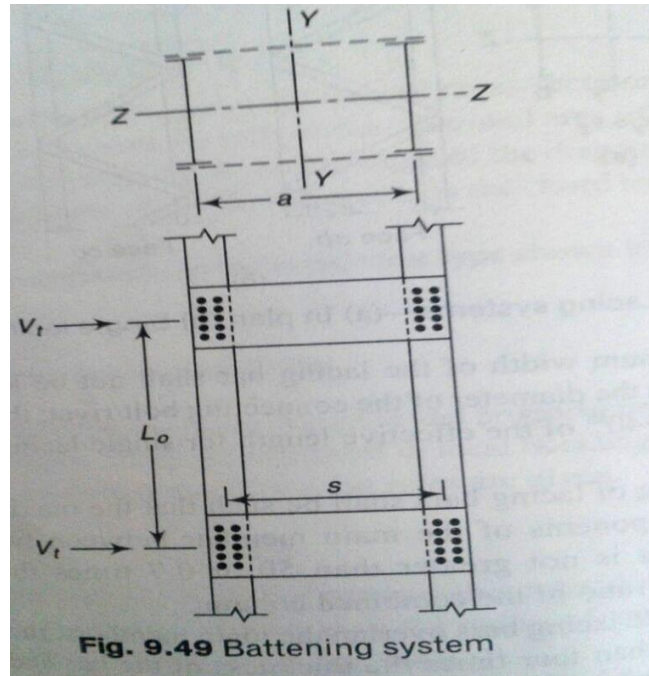
Where

L_o = distance bet c/c of battens, longitudinally

n = no of parallel planes of battens

s = min transverse distance bet centroids of the bolt/rivet group/welding connecting the batten to the main member.

Battens shall be designed to carry BMs and SFs arising from transverse SF, V_t equal to 2.5% of the total axial force on the whole compression member



Problems

An ISHB 400 @ 806.4 N/m is to be used as column 3.5 m long with both ends restrained against rotation and translation. Determine the design axial load on the column section. Also assume the following data : $f_y = 250 \text{ N/mm}^2$, $f_u = 410 \text{ N/mm}^2$ and $E = 2 \times 10^5 \text{ N/mm}^2$.

Solution:

Method 1

Properties of ISHB 400 from handbook/steel table:

$$A = 10466 \text{ mm}^2 \quad b_f = 250 \text{ mm} \quad t_w = 10.6 \text{ mm} \quad h = 400 \text{ mm}$$

$$t_f = 12.7 \text{ mm} \quad r_z = 166.1 \text{ mm} \quad r_y = 51.6 \text{ mm} \quad L = 3.5 \text{ m} = 3500 \text{ mm}$$

Refer table 10 page 44

$$h/b_f = \quad \text{if } > 1.2 \text{ and } t_f = \quad < 40 \text{ mm}$$

From above condition the buckling curve to be used along z-z axis is curve a and that about y-y axis is curve b.

Since $r_y < r_z$ the design compressive strength is governed by the effective slenderness ratio λ_y (column will buckle about y-y axis)

$\lambda_y=40$ and $f_y = 250$ $f_{cd} = 206$ MPa and

$y = KL / r_y$

$$= (0.65 \times 3500) / 51.6$$

$$= 44.09 \quad \text{for both ends fixed } k = 0.65$$

For buckling class b and $f_y = 250$ MPa, design compressive stress = 200 MPa = f_{cd} (refer table 9(b) of IS code) This is obtained by interpolating between the two values of λ_y

$\lambda_y=40$ and $f_y = 250$ $f_{cd} = 206$ MPa and so $(50-40) \rightarrow (194-206)$

$\lambda_y=50$ and $f_y = 250$ $f_{cd} = 194$ MPa $(50 - 44.09) \rightarrow (194 - x)$

Cross multiplying $10 \times (194 - x) = (-12) \times (5.91)$

$$x = 200 \text{ MPa}$$

Method 2

we can use the formulae

Non-dimensional slenderness ratio

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{ce}} = \sqrt{f_y \left(\frac{KL}{r} \right)^2 / \pi^2 E}$$

$$= \sqrt{[(250 \times (44.09)^2) / (\pi^2 \times 2 \times 10^5)]}$$

$$= 0.5$$

For $\lambda = 0.5$, for buckling curve b, $f_{cd}/f_y = 0.8$

$$\text{therefore} \quad f_{cd} = 0.8 \times 250 = 200 \text{ MPa}$$

Therefore $P_d = A_e \times f_{cd} = 10466 \times 200 = 2093 \times 10^3 \text{ N}$ or 2093 kN

Method 3

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

$$\phi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r} \right)^2 / \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2}$$

- Imperfection factor

$\alpha = 0.34$ (for buckling class b)

$$\text{Find } \phi = 0.5 [1 + 0.34(0.5 - 0.2) + 0.5^2]$$

$$= 0.676$$

$$\chi = 1 / \{ 0.676 + [0.676^2 - 0.5^2]^{1/2} \}$$

$$= 0.884$$

$$F_{cd} = (250 \times 0.884) / 1.1$$

$$= 200.9 \text{ N/mm}^2$$

$$P_d = A_e \times f_{cd} = 10466 \times 200.9$$

$$= 2102.71 \text{ kN}$$

Assignment questions

1. What are compression members?
2. What are the different types of section used as compression members?
3. What are the failure modes of compression members?
4. Write a note on column buckling
5. Write a note on lacing.
6. Write a note on battening.

Outcome

- Gives knowledge about design criteria of compression members
- Gives knowledge about Lacings and battening

Future study

By study this one can apply these for designing columns to support roof truss in railways stations, industrial building etc.

Module - 4 Tension Members

Contents:-

- 4.1 Introduction
- 4.2 Objective
- 4.3 Types of tension members
- 4.4 Behavior of tension members
- 4.5 Modes of failure of tension members
- 4.6 Factors affecting the strength of tension members
- 4.7 Slenderness ratio
- 4.8 Lug Angle
- 4.9 Gusset plate
- 4.10 Problems
- 4.11 Assignment questions
- 4.12 Outcome
- 4.13 Future study

4.1 Introduction

Tension member is a member which transmits a direct axial pull between two points in a structural frame.

Ex:- Cable in a suspension bridge.

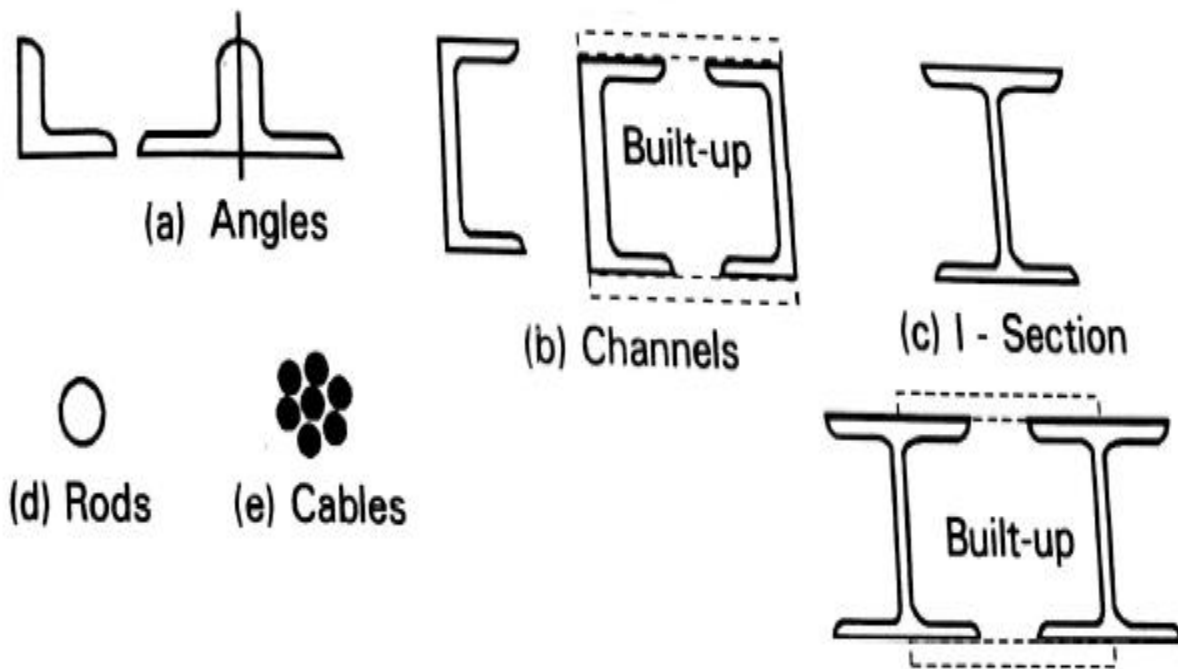
A tension member may be subjected to a bending moment either due to eccentricity of the longitudinal load or due to transverse load acting in addition to the main longitudinal load. A tension member may have bolted or welded. When the tensile load reaches the ultimate tension, member fails due to excessive elongation or by rupture of its section.

4.2 Objective

To study the behavior of tension members under loading

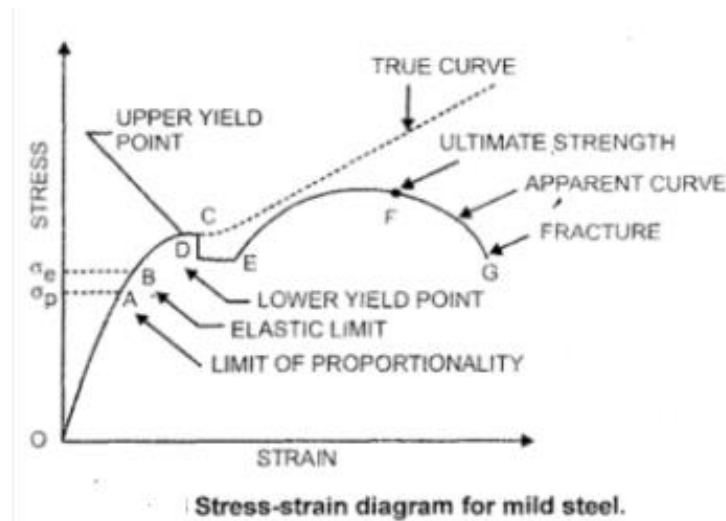
4.3 Types of tension members

- Open sections like angles, channels and tees.
- Closed sections like circular, square and rectangular hollow sections.
- Built-up sections like double angles, double channels. Built sections like four angle members are also used as tension members.



4.4 Behavior of tension members

Behavior of Mild steel subjected to axial tension



Stress-strain curve gives information to understand the behavior of material subjected to given loading condition.

OABCDEFGG is a stress-strain curve of mild steel subjected to gradually increasing tensile load.

OA → represents a straight line, where stress is directly proportion to strain and it obeys Hooke's law.

A → represents limit of proportionality, stress beyond which linear variation ceases.

B → represents elastic limit, maximum stress up to which the material regains its original shape after removal of load.

AB → is not a straight line.

Stress beyond point B, material goes to plastic region and stress reaches to upper yield point C. At this stage material deforms and there will be reduction in cross-sectional area. Then stress reduces to a point D is known as lower yield point.

Between DE → when load remains constant, stress also remains constant. But there will be a considerable amount deformation.

From point E → strain hardening takes place, where the material starts to deform when it reaches yield point and becomes stronger. So addition stress is required to cause addition deformation of the material till it reaches to point F. This point is known as ultimate point and stress at this point is called ultimate strength.

At point F → necking takes place, where there will be a reduction of cross-section area of material at a rapid rate. Apparent stress reduces but true stress goes on increases until material breaks at point G and this point is known as point of failure or fracture.

Failure of ductile material is cup and cone type and ultimate strength is calculated at 0.2% of maximum strain.

Design of strands

Strands are nothing but group of twisted wires or ropes. The advantages of wires and cables are flexibility and strength. Since the cables are long and their stiffness is negligible, initial sag and other geometrical effects must be accounted for in the design

The cross-sectional area of strands is given by

$A_s = 0.6D^2$ where, D = Nominal diameter.

The modulus of elasticity is given by $E = 0.65 \cdot E_s$

Where, $E_s = 2 \cdot 10^5$ MPa.

The ultimate stress of wire ropes = 1400-2000 MPa

The proof stress (0.7% residual strain) = 1000 -1400 MPa

Percentage of elongation = 3 to 4%.

4.5 Modes of failure of tension members

- Failure due to yielding of gross section
- Failure due to rupture of cross-section
- Failure due to block shear (failure along cross-section of bolt due to shear in the direction of applied load and failure due to tension along perpendicular to the direction of applied load)

4.6 Factors affecting the strength of tension members

1. **Effect of bolt holes:-** Holes in a member causes stress concentration.
2. **Net area of cross-section:-** Tension members having bolt holes will have a reduced cross section called net area of cross section, which reduces the strength. When more than one bolt hole is present, the failure path may occur along sections normal to the axis of the member, or they may include zigzag sections, if the fasteners are staggered.
3. **Effect of bearing stress:-** Bearing stress developed in a member adjacent to the hole and if the fasteners slip between plates and the connected bolts. This effect can be neglected, if the bearing stress is less than 2.25 times the tensile stress.
4. **Effect of shear lag:-** The force transmitted to a tension member through a connected member is always non-uniform. The bolts or weld which is near to the transfer point will experience more stress than bolts or welds at the farthest point. For example: the internal

transfer of forces from flange to web will be by shear and because one member lags behind the other, the phenomenon is called “shear lag”. The shear lag reduces the effectiveness of the connected members. The effect can be prevented by increasing the connection length.

5. **Geometry of section:-** Lack of fit in black bolts in clearance holes. Gaps in the connecting plate and the surface of the member to be connected, due to fabrication errors, welding distortions etc. If the ratio of gauge length to diameter is small than the efficiency of the connection will be higher.
6. **Ductility:-** Higher the ductility of the connected members, higher the strength.
7. **Spacing of fasteners:-** Block shear failure will occur due to closer spacing of the fasteners.
8. **Method of fabrication:-** Punched holes will have 10-15% lesser strength than the drilled holes because, punched holes undergoes strain hardening and will exhibit less ductility.

4.7 Slenderness ratio

It is defined as the ratio of effective length to the minimum radius of gyration.

Slenderness ratio should be within certain limits to avoid,

- Excessive sag due to self-weight of the member.
- Flutter due to wind.
- Vibration caused by moving loads.

Limits for slenderness ratios for tension members are as follows,

Sl No	Type of Tension member	Maximum slenderness ratio
1	A tension member in which a reversal of direct stress due to loads other than wind or seismic forces occur	< 180
2	Members subjected to compressive forces due to combination of wind or earthquake forces	< 250
3	A member normally acting as a tie in a roof truss or a bracing system but subjected to possible reversal of stress resulting from the action of wind or seismic forces	< 350
4	Tension members other than pre-tensioned members	< 400

4.8 Lug Angles

To accommodate the necessary number of bolts for the connection of a member to a gusset plate a certain length of the member is utilized. When load on the member is large many bolts may be required and the length of the member utilized for the connection itself may be large and may

involve a considerable length of the gusset plate. Lug angle is provided to reduce the length of the joint.

Advantage

The only advantage is to reduce the length of the joint.

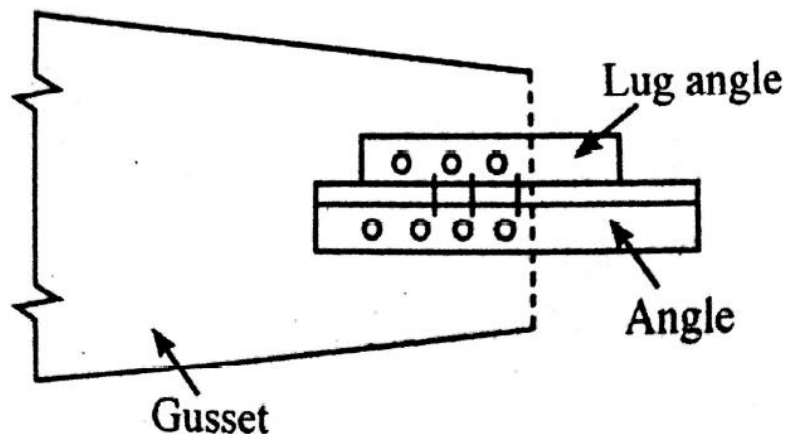
Disadvantages

- Lug angle connection requires extra piece of angle.
- The overall number of bolts is more than the connection without lug angle.
- The loading is eccentric.

A lug angle provided at the beginning of the joint, so that it can be effective in sharing the load.

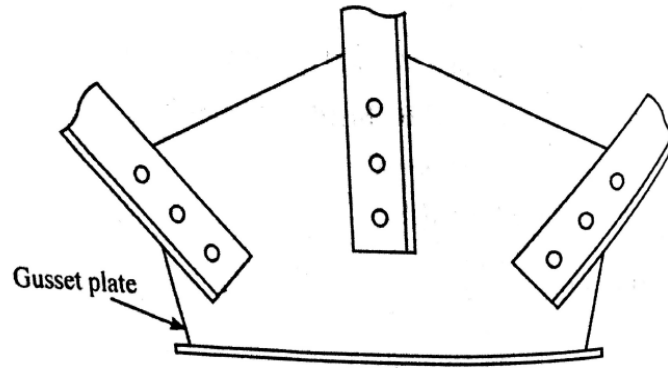
Design specification

- For the design of lug angle, the force in the outstanding leg is increased by 20% (for channel sections – 10%).
- For the connection between lug angle and gusset plate, the force in the outstanding leg is increased by 20% (for channel section – 10%).
- The connection between lug angle and main angle, the force in the outstanding leg is increased by 40% (for channel sections – 20%).



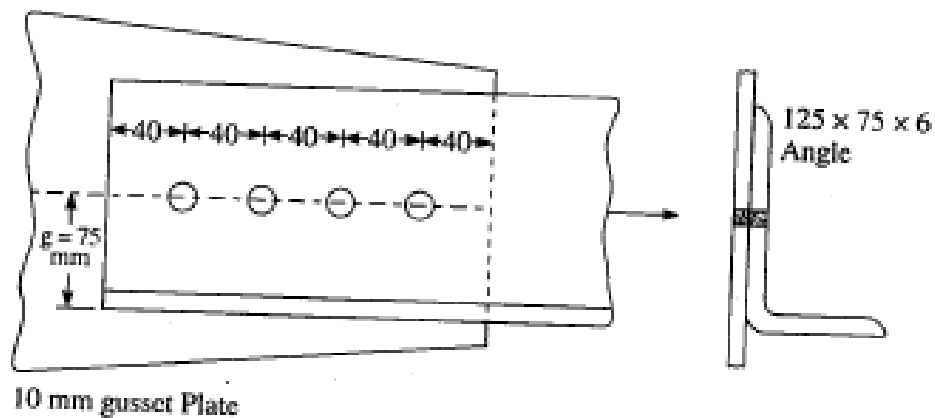
4.9 Gusset plate

A gusset plate is a plate provided to make connections at the plate where more than one member is to be joined. For eg; joints of truss, truss girder etc. the line of action of truss members meeting at a joint are assumed to coincide as shown in figure below. The size and shape of the gusset plate are usually decided from the direction of the member meeting at the joint.



4.10 Problems

1. The longer leg of a single angle $125 \times 75 \times 6 \text{ mm}$ is connected to a 10 mm thick gusset plate by 4 bolts of 16 mm dia arranged as shown in figure. Determine the design tensile strength of the angle. Take $f_y = 250 \text{ N/mm}^2$ and $f_u = 410 \text{ N/mm}^2$.



Solution. Bolt diameter = $d = 16 \text{ mm}$. Bolt hole diameter = $d_o = 16 + 2 = 18 \text{ mm}$

Sectional properties of the angle, $125 \times 75 \times 6$

$$\text{Gross area} = A_g = 1166 \text{ mm}^2$$

$$\text{Net area of the connected leg} = A_{nc} = \left[125 - \frac{6}{2} - 18 \right] 6 = 624 \text{ mm}^2$$

$$\text{Gross area of the outstanding leg} = A_{go} = \left[75 - \frac{6}{2} \right] 6 = 432 \text{ mm}^2$$

(a) *Design tensile strength governed by yielding of the gross-section*

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1166 \times 250}{1.10} = 265000 \text{ N}$$

(b) *Design tensile strength governed by rupture of critical section*

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

where,

$$\beta = 1.4 - 0.076 \frac{w}{t} \frac{f_y}{f_u} \frac{b_s}{L_c} \leq 0.9 \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \geq 0.7$$

where,

$$w = \text{Outstand leg width} = 75 - \frac{6}{2} = 72 \text{ mm}$$

$$b_s = \text{Outstand} + \text{gauge distance} - t = 75 + 75 - 6 = 144 \text{ mm}$$

$$L_c = \text{Length of the end connection}$$

$$= \text{Distance between the outer most bolt holes}$$

$$= 3 \times 40 = 120 \text{ mm}$$

$$\therefore \beta = 1.4 - 0.076 \times \frac{72}{6} \times \frac{250}{410} \times \frac{144}{120} = 1.4 - 0.667 = 0.733$$

$$\beta \text{ should be } \leq 0.9 \frac{f_u}{f_y} \cdot \frac{\gamma_{mo}}{\gamma_{mf}} \text{ i.e., } \leq 0.9 \times \frac{410}{250} \times \frac{1.10}{1.25} = 1.30$$

and $\beta \geq 0.7$

From all the above considerations, $\beta = 0.733$

$$T_{db} = \frac{0.9 \times 624 \times 410}{1.25} + \frac{0.733 \times 432 \times 250}{1.10} = 184205 + 71967 = 256172 \text{ N}$$

(c) **Design tensile strength by block shear T_{db}**

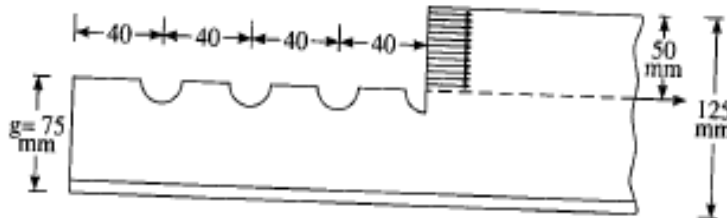
A_{vg} = Minimum gross area in shear along bolt line = $[3 \times 40 + 40]6 = 960 \text{ mm}^2$

A_{vm} = Minimum net area in shear along bolt line = $[3 \times 40 + 40 - 3.5 \times 18]6 = 582 \text{ mm}^2$

A_{tg} = Minimum gross area in tension from bolt hole to toe of angle perpendicular to the line of force = $6 \times 50 = 300 \text{ mm}^2$

A_{tm} = Minimum net area in tension from bolt hole to toe of angle perpendicular to the line of force

$$= 6(50 - 0.5 \times 18) = 246 \text{ mm}^2 \text{ (See Fig. 6.35.)}$$



Design tensile strength governed by block shear

$$\begin{aligned} T_{db} &= \frac{A_{vg} f_y}{\sqrt{3} \gamma_{mo}} + \frac{0.9 A_{tm} f_u}{\gamma_{mf}} \\ &= \frac{960 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 246 \times 410}{1.25} \\ &= 125971 + 72619 = 198590 \text{ N} \end{aligned}$$

OR

$$\begin{aligned} T_{db} &= \frac{0.9 A_{vm} f_u}{\sqrt{3} \gamma_{mf}} + \frac{A_{tg} f_y}{\gamma_{mo}} \\ &= \frac{0.9 \times 582 \times 410}{\sqrt{3} \times 1.25} + \frac{300 \times 250}{1.10} \\ &= 99195 + 68182 = 167377 \text{ N} \end{aligned}$$

$\therefore$ The design tensile strength of the angle

$$= \text{Least of the strengths determined above} = 167377 \text{ N}$$

2. Design an angle section to carry a factored load of 200kN. Bolts of 20mm diameter is to be provided for the connection of the member to the gusset plate. Take $f_y = 250\text{N/mm}^2$ and $f_u = 410\text{N/mm}^2$. The design strength of a 20mm diameter bolts = 45.3kN.

Solution.

Bolt diameter = $d = 20\text{ mm}$, Bolt hole diameter = $d_o = 20 + 2 = 22\text{ mm}$

$$\begin{aligned}\text{Approximate sectional area required} &= \frac{1.1 \times \text{Factored load}}{f_y} \\ &= \frac{1.1 \times 200 \times 10^3}{250} = 880\text{ mm}^2\end{aligned}$$

Provide a single angle $= 100 \times 65 \times 6$

Gross area of the angle $= A_g = 955\text{ mm}^2$

(a) Design tensile strength governed by yielding of the gross-section

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{955 \times 250}{1.1} = 217045\text{ N} = 2170.45\text{ kN}$$

Number of 20 mm diameter bolts required

$$\begin{aligned}&= \frac{\text{Factored load}}{\text{Design strength of a bolt}} \\ &= \frac{200}{45.3} = 4.4\end{aligned}$$

Provide 5 bolts of 20 mm diameter

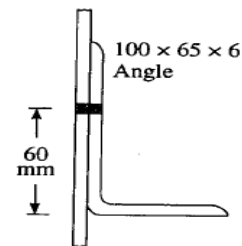
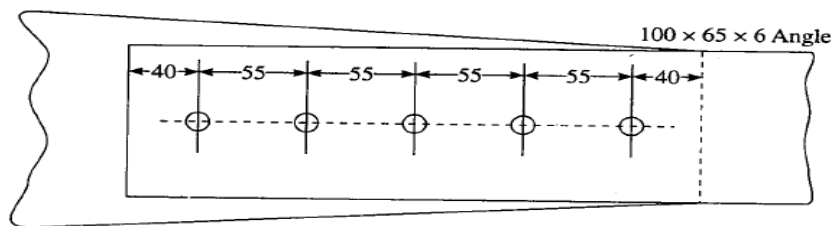


Fig. 6.36



(b) Design tensile strength governed by rupture of critical section

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076 \frac{w}{t} \frac{f_y}{f_u} \frac{b_s}{L_s} \leq \frac{f_u}{f_y} \frac{\gamma_{mo}}{\gamma_{ml}} \geq 0.7$$

A_{nc} = Net area of the connected leg

$$= \left(100 - \frac{6}{2} - 22 \right) 6 = 450 \text{ mm}^2$$

A_{go} = Gross area of the outstanding leg

$$= \left(65 - \frac{6}{2} \right) 6 = 372 \text{ mm}^2$$

w = Outstand leg width = $65 - \frac{6}{2} = 62 \text{ mm}$

b_s = Outstand + gauge distance - $t = 65 + 60 - 6 = 119 \text{ mm}$

L_c = Length of the end connection

= Distance between the outer most bolts = $4 \times 55 = 220 \text{ mm}$

$$\beta = 1.4 - 0.076 \times \frac{62}{6} \times \frac{250}{410} \times \frac{119}{220} = 1.4 - 0.259 = 1.141$$

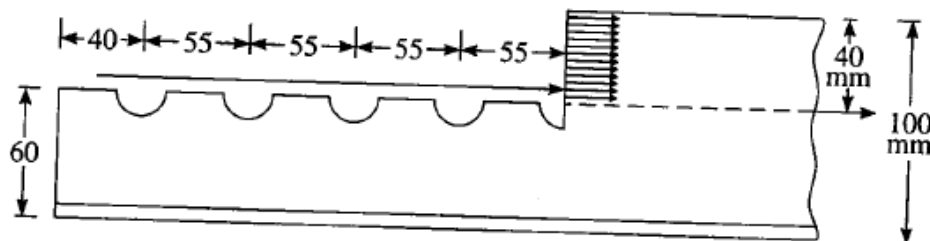
β should be less than or equal to $\frac{f_u}{f_y} \cdot \frac{\gamma_{mo}}{\gamma_{ml}} = \frac{410}{250} \times \frac{1.10}{1.25} = 1.443$

and β should be ≥ 0.7

From all the above consideration, $\beta = 1.141$

$$\begin{aligned} \therefore T_{dn} &= \frac{0.9 \times 450 \times 410}{1.25} + \frac{1.141 \times 372 \times 250}{1.10} \\ &= 132840 + 96466 = 229306 \text{ N} = 229.306 \text{ kN} \end{aligned}$$

(c) *Design tensile strength governed by block shear*



$$A_{vg} = \text{Minimum gross area in shear along the bolt line} \\ = 6[4 \times 55 + 40] = 1560 \text{ mm}^2$$

$$A_{vn} = \text{Minimum net area in shear along bolt line} \\ = 6[4 \times 55 + 40 - 4.5 \times 22] = 966 \text{ mm}^2$$

$$A_{tg} = \text{Minimum gross area in tension from bolt hole to toe of angle perpendicular to the line of force} \\ = 6 \times 40 = 240 \text{ mm}^2$$

$$A_m = \text{Minimum net area in tension from bolt hole to toe of angle perpendicular to the line of force.} \\ = 6[40 - 0.5 \times 22] = 174 \text{ mm}^2$$

Design tensile strength governed by block shear : T_{db}

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3} \gamma_{mo}} + \frac{0.9 A_m f_u}{\gamma_{ml}} \\ = \frac{1560 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 174 \times 410}{1.25} \\ = 204703 + 51365 = 256068 \text{ N} = 256.068 \text{ kN}$$

or

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}} \\ = \frac{0.9 \times 966 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.10} = 164644 + 54545 \text{ N} \\ = 219189 \text{ N} = 219.189 \text{ kN}$$

$\therefore$ The design tensile strength of the angle

= Least of strengths determined above

$$= 217.045 \text{ kN}$$

But factored tensile force

$$= 200 \text{ kN}$$

$\therefore$ The design is safe.

4.11 Assignment questions

1. What are tension members?
2. What are the types of tension members?
3. Explain the failure modes of tension members.
4. Write a note on lug angle.
5. Write a note on gusset plate.

4.12 Outcome

- Gives knowledge about behavior of tension members under loading.

4.13 Future Study

By understanding the behavior one can design the roof truss connection for an industrial or any other type of buildings.

Chapter – 6 Column Base

Contents:-

Introduction

Objective

Classification of column bases

Slab base

Problem on slab base

Gusseted base

Problem on gusseted base.

Assignment questions

Outcome'

Future study

Introduction

Steel columns are normally supported over a concrete block. However, the loads supported by these columns are large so the bearing may fail. Normal practice is to distribute the column loads to steel base plate which is placed over these concrete blocks. Column bases are subjected bearing pressure from below bending moment and shear force. The main problem in the design of column base is to determine the size of the base plate and its thickness. The size of the plates can be determined by the bearing area on foundation and the required thickness is determined so that bearing strength of the plate is not exceeded. The safety of the column and the structure mainly depends on the stability of the foundation and consequently on the bases.

Objective

To know the design criteria of column bases

Column bases classified into 2 types

- 1] **Transmitting direct axial load only:-** The pressure distribution between the base plate and concrete pedestal is assumed to be uniform.
- 2] **Carrying bending moment in addition to direct axial load:-** For column bases transmitting moment, angles anchored to the footing may be used and provided angles are designed for uplift forces.

Two prevalent column bases are

1. Slab Base
2. Column base

Slab Base:-

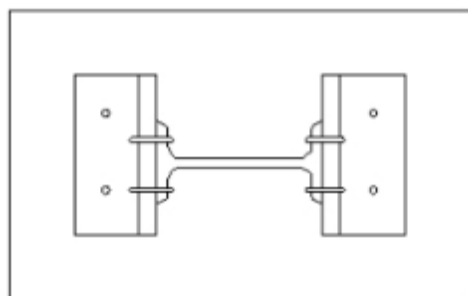
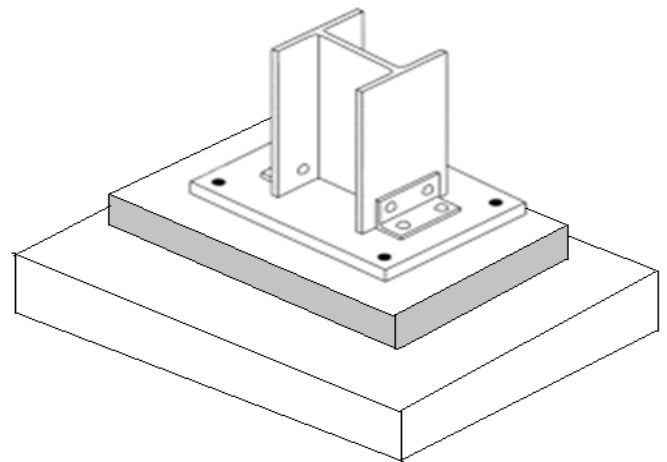
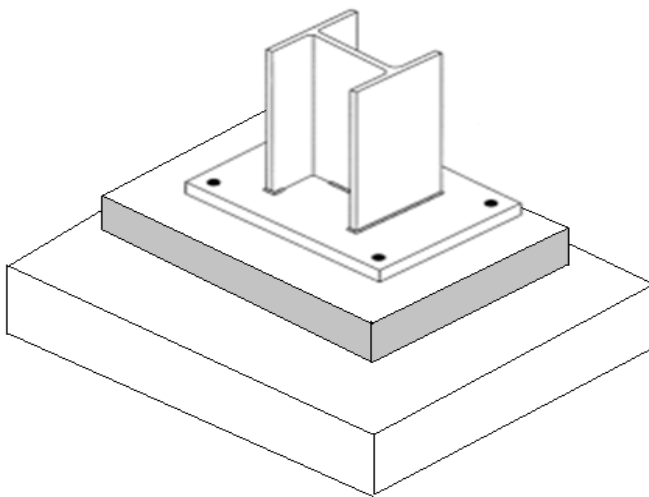
When the column is subjected to only direct loads, the base can be designed by assuming a uniform bearing pressure from below. For small loads, a steel plate alone shop welded to the column can be used to transmit the loads to the concrete pedestal. Such a base plate is called **Slab Base**.

Design of Steel Structural Elements 15CV62

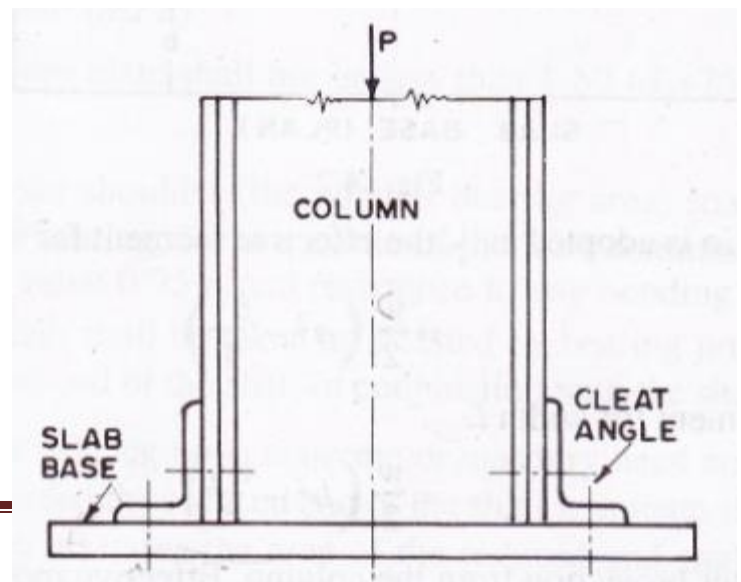
When the columns are subjected to bending moment along with direct forces, angle sections are attached to the flanges of the column. These angles are anchored with foundation bolts to the concrete pedestal. Depending upon the values of axial load and bending moment, there may be compression over the whole bases or compression over a part of the base and tension in the remaining part. Holding down, the bolts are provided to resist the tension forces and to prevent due to moments or applied tension. Horizontal loads are resisted by the welds provided to connect the column with the base plate

Advantages:-

- It is more suitable and economical for lightly loaded columns only.
- In case, where the projections are large or loads are heavy or moments are applied, the thickness can be reduced by the use of vertical plates or stiffeners.



Department of Civil Engineering, AUMCE



Problem

A steel column ISHB 400@759.3N/m is subjected to a factored axial load of 2000KN. Design a slab base for the column. Assume that the bearing surfaces of the column and base plate are machined. The concrete footing is of M20 grade.

Solution. Design bearing strength of concrete $= 0.45 f_{ck} = 0.45 \times 20 = 9 \text{ N/mm}^2$

$$\text{Area of base plate required} = \frac{2000 \times 10^3}{9} = 222222.2 \text{ mm}^2$$

$$\text{Overall depth of ISHB 400} = 400 \text{ mm, Flange width} = 250 \text{ mm}$$

The plan dimensions of the base plate will be selected so that the projections of the base plate beyond the overall dimensions of the column are nearly equal.

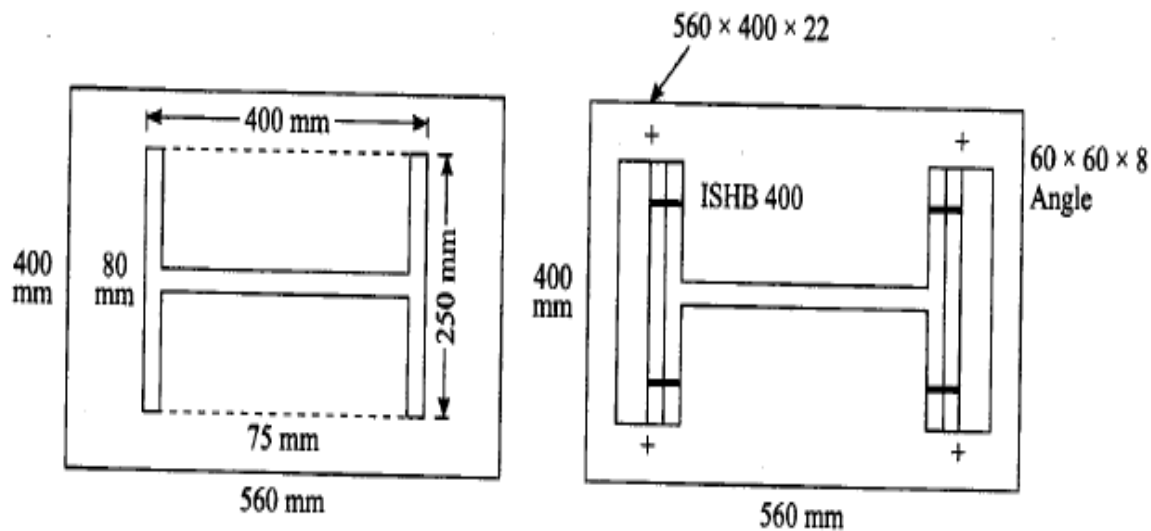


Fig. 7.62

$$\text{Suppose the base plate is } L \times B = 222222.2 \text{ mm}^2$$

$$\text{Try } L = 500 \text{ mm, } B = \frac{222222.2}{500} = 444.4 \text{ mm say } 445 \text{ mm}$$

$$\text{The projections are } \frac{500 - 400}{2} = 50 \text{ mm and } \frac{445 - 250}{2} = 97.5 \text{ mm}$$

Again try $L = 560 \text{ mm}$, $B = \frac{222222.2}{560} = 396.8 \text{ mm}$, say 400 mm

The projections area, $\frac{560 - 400}{2} = 80 \text{ mm}$ and $\frac{400 - 250}{2} = 75 \text{ mm}$

These projections are nearly equal.

We will therefore provide $560 \text{ mm} \times 400 \text{ mm}$ base plate.

Bearing stress on the plate $= w = \frac{2000 \times 10^3}{560 \times 400} = 8.928 \text{ N/mm}^2$

Thickness of the base plate

$$= t_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)\gamma_{mo}}{f_y}}$$

$$= \sqrt{\frac{2.5 \times 8.928(80^2 - 0.3 \times 75^2)1.1}{250}} \text{ mm}$$

$$= 21.5 \text{ mm say } 22 \text{ mm}$$

The base plate will be $560 \text{ mm} \times 400 \text{ mm} \times 22 \text{ mm}$, $60 \text{ mm} \times 60 \text{ mm} \times 8 \text{ mm}$ angles are used to connect the base plate and flanges of the column.

Gusseted base

When the load on the column section is too large or when the axial load is accompanied by bending moment. Usually a gusseted base is provided. It consists of a base plate, 2 gusset plates, 2 gusset angles; when bolted connection is made. The gusset plate and angles are placed on flanges as shown in figure. In case of welded connection, gusset angles are not required. The gusset materials need in the base increase the bearing area consequently reducing the thickness of base plate as compared to slab base. Gusset materials support the base plate against bending and consequently results in lesser thickness of base plate. This kind of base may be considered to be rigid.

The upward pressure from below the base causes bending of the gusset plate and puts their top edge in compression which may therefore buckle. This can be checked by limiting the width to thickness ratio for the gusset plate as given below.

$D \leq 29.3 \epsilon t_g$ = for the portion of gusset plate welded to column flange.

$S_o \leq 13.6 \epsilon t_g$ = for the outstanding of the gusset plate from the edge of the column flange.

$\epsilon = \sqrt{(250/f_y)}$ where $f_y = 250 \text{ N/mm}^2$.

The gusseted base plate may be designed as follows.

- (i) Divide the factored load on the column by the design bearing strength of concrete and find the area of the base plate required.
- (ii) Choose a thickness of gusset plate (16 mm) and gusset angle (say 150 × 115 × 15). The vertical leg of the angle must have a length to accommodate two bolts. To accommodate all these elements the width of the base plate is determined.

$$\text{Length of the base plate} = \frac{\text{Area of base plate required}}{\text{Width of the base plate}}$$

- (iii) Bearing stress on base plate

$$= w = \frac{\text{Column load}}{\text{Actual area of base plate}}$$

- (iv) Let b = distance between the gussets

c = cantilevering projection of the base plate

$$\text{Cantilever moment per unit width} = M_1 = \frac{wc^2}{2}$$

Thickness t required for this moment is calculated.

B.M. at the centre : The part of the base plate between the gusset plates bends in two principal rections. Considering two way slab action the B.M. at the centre may be taken as

$$M_2 = 0.2 \left[\frac{wb^2}{8} - \frac{wc^2}{2} \right]$$

Thickness required for this moment is calculated.

The greater of the thicknesses determined from the above two consideration is adopted.

- (v) Connections are designed for 50% of the column load.

If P = Factored load on the column

$$\text{Load per gusset plate} = \frac{1}{2} \left[\frac{P}{2} \right] = \frac{P}{4}$$

Number of bolts on a gusset plate and gusset angle

$$= \frac{\frac{P}{4}}{\text{Design strength of a bolt}}$$

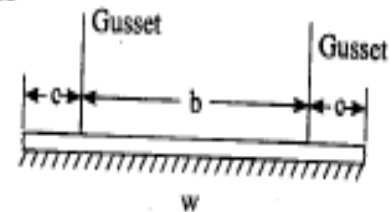


Fig. 7.65

Problem

An ISHB 300@710.2 N/m column carries a factored load of 2000KN. Design a gusset base with bolted connection. The design bearing strength of the concrete pedestal is 9N/mm^2 .

Solution. Factored load on the column = 2000 kN

$$\text{Area of base plate required} = \frac{2000 \times 10^3}{9} = 222222.2 \text{ mm}^2$$

Let us provide 16 mm gusset plates and $150 \times 115 \times 15$ gusset angles.

$$\begin{aligned} \text{Width of base plate required to accommodate these components} \\ = 350 + 2(16) + 2(115) = 612 \text{ mm say } 620 \text{ mm} \end{aligned}$$

$$\text{Length of base plate required} = \frac{222222.2}{620} = 358.4 \text{ mm say } 360 \text{ mm}$$

$\therefore$ The base plate will be 620 mm $\times$ 360 mm

$$\text{Bearing pressure on the base plate} = w = \frac{2000 \times 10^3}{620 \times 360} = 8.96 \text{ N/mm}^2$$

$$\text{Cantilevering length} = (115 - 15) + 4 = 100 + 4 = 104 \text{ mm}$$

Cantilever moment for a 1 mm wide strip

$$= M_1 = 8.96 \times \frac{104^2}{2} = 48455.7 \text{ Nmm}$$

$$1.2 \frac{f_y Z_e}{\gamma_{mo}} = \frac{1.2 \times 250}{1.1} \times 1 \times \frac{t^2}{6} = 48455.7$$

$\therefore$

$$t = 32.7 \text{ mm (including the thickness of the angle)}$$

$$\text{Thickness of base plate} = 32.7 - 15 = 17.7 \text{ mm say } 18 \text{ mm}$$

Maximum span moment for 1 mm wide strip

$$= M_2$$

Allowing for two way bending, the B.M. at the centre

$$\begin{aligned} = M_2 &= 0.2 \left[8.96 \times \frac{382^2}{8} - 8.96 \times \frac{119^2}{2} \right] \text{ Nmm} \\ &= 19999 \text{ N/m} \end{aligned}$$

$$\frac{1.2 \times 250}{1.1} \times \frac{1 \times t^2}{6} = 19999 \quad \therefore t = 21 \text{ mm say } 25 \text{ mm}$$

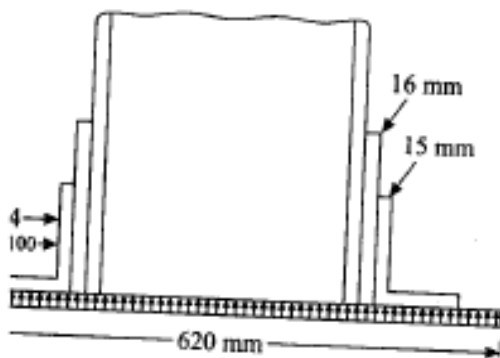


Fig. 7.66

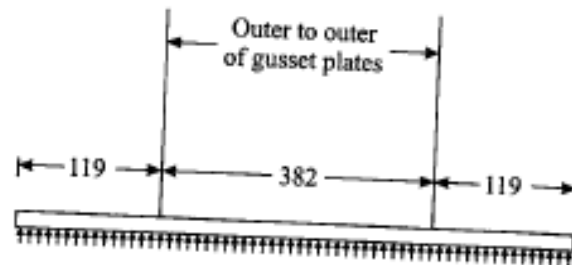


Fig. 7.67

From the above two considerations, thickness required = 25 mm
 Hence the base plate will be 620 mm × 360 mm × 25 mm.

Connections

The bolt connections will be designed on the assumption that 50 percent of the column load will be transmitted to the base plate by direct bearing and the balance through the gusset plates.

∴ Load transmitted to each gusset plate

$$= \frac{1}{2} \left[\frac{P}{2} \right] = \frac{P}{4} = \frac{2000}{4} = 500 \text{ kN}$$

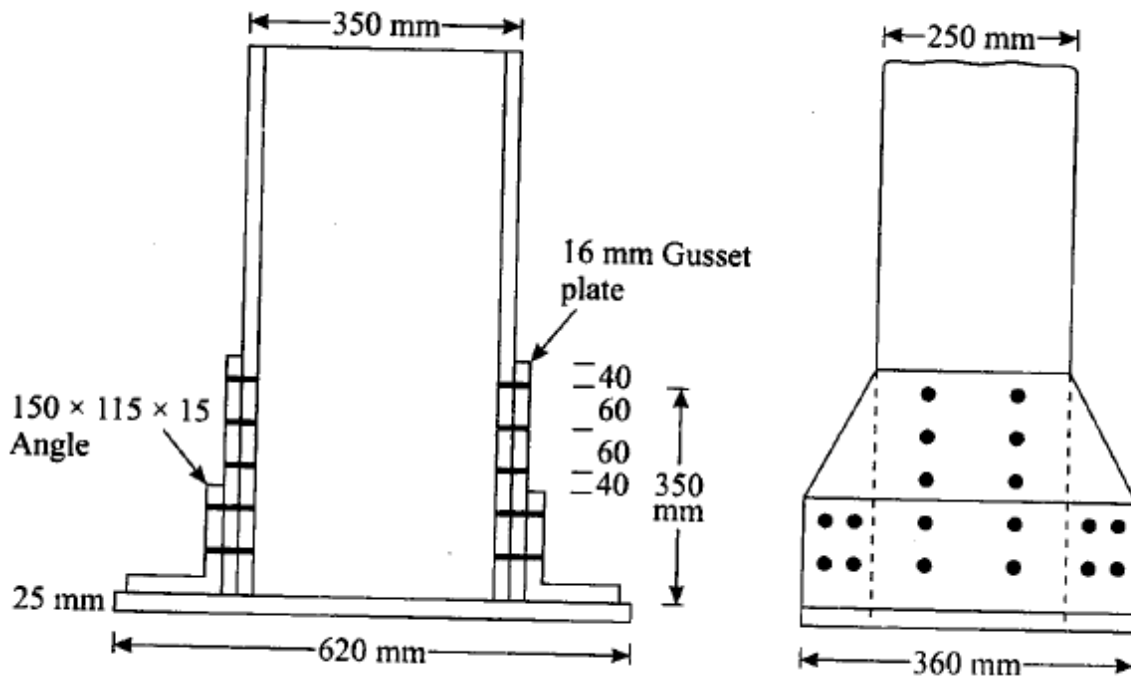
Provide 22 mm diameter bolts.

Shearing strength of bolt (in single shear)

$$\frac{f_u}{\sqrt{3}\gamma_{mb}} \cdot A_n = \frac{400}{\sqrt{3} \times 1.25} (0.78) \pi \times \frac{22^2}{4} \times 10^{-3} \text{ kN} = 54.78 \text{ kN}$$

Number of bolts required on one gusset plate

$$= \frac{500}{54.78} = 9.1 \text{ say } 10 \text{ bolts.}$$



Assignment questions

What are column bases?

What are the types of column bases?

Write a note on slab base.

Write a note on gusseted base.

Outcome

Gives knowledge about design criteria of slab and gusseted base

Chapter – 8 Designs of Beams

Contents:-

Introduction

Objective

Classification of beam

Failure modes of beams

Factors affecting lateral stability of beam

Classification of cross section

Web buckling and web crippling

Problems

Assignment questions

Outcome

Future study

Introduction

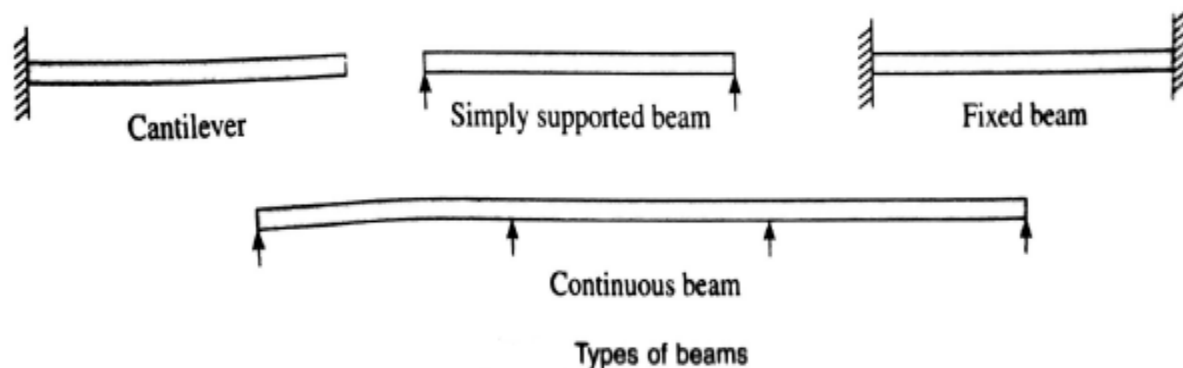
Beam is a structural member that supports load which are applied transverse to their longitudinal axis which are resisted by their resistance due to bending and shear.

Objective

- To study the design criteria of laterally supported and laterally unsupported beams

Depending upon conditions at the supports, beams are classified as

- Simply Supported Beam
- Cantilever Beam
- Fixed Beam
- Propped Beam
- Continuous Beam



Failure of beam can occur in several ways such as

- Bending Failure
- Shear Failure
- Deflection Failure

The size of the structural steel beam is designated based on Bending, shear and deflection.

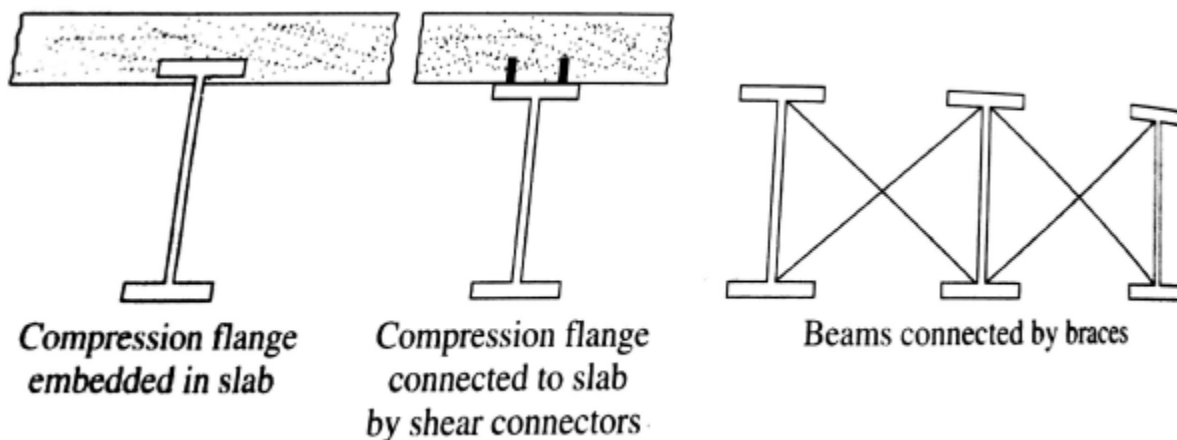
Beams are classified into two types

- Laterally Supported (Restrained) Beams
- Laterally Unsupported (Unrestrained) Beams

Laterally Supported (Restrained) Beams

Loads on a beam cause bending, due to which tension is induced in one flange and compression is induced in other flange. The compression flange behaves somewhat like a column and will tend to buckle to the side or laterally as the stress increases, if it is not restrained in some way. Most beams have some lateral support – especially those which support the floor slab and it may also be restrained by its connection to cross beam or bracings.

Figure shows the methods of providing lateral support to compression flange.



Laterally unsupported (Unrestrained) Beams

If a beam is subjected to a loading, the compressive flange due to its considerable length acts as a long column and is liable to lateral buckling, if it is not adequately stiff or if it is not restrained by continuous lateral support to prevent lateral buckling. Laterally unsupported beams are also called laterally unrestrained beams when loaded undergoes lateral buckling of compression flange.

Laterally Stability of beams

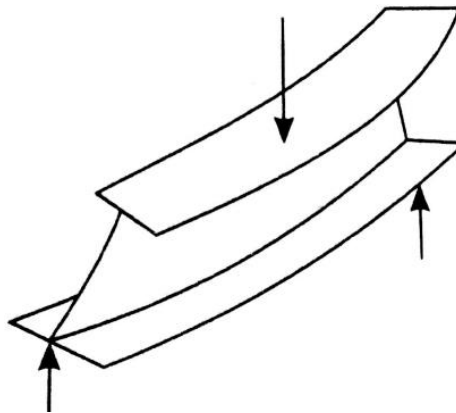
The beam will attain its full moment capacity when it is loaded in flexure and if the lateral instabilities of the beam are avoided. Lateral stability of beams can be increased by providing adequate lateral restraints. For example in steel buildings; beams are laterally restrained by floor decks. The lateral buckling of unrestrained beams is a function of its slenderness. Higher the slenderness ratio, higher will be the lateral buckling.

Failure modes of beams

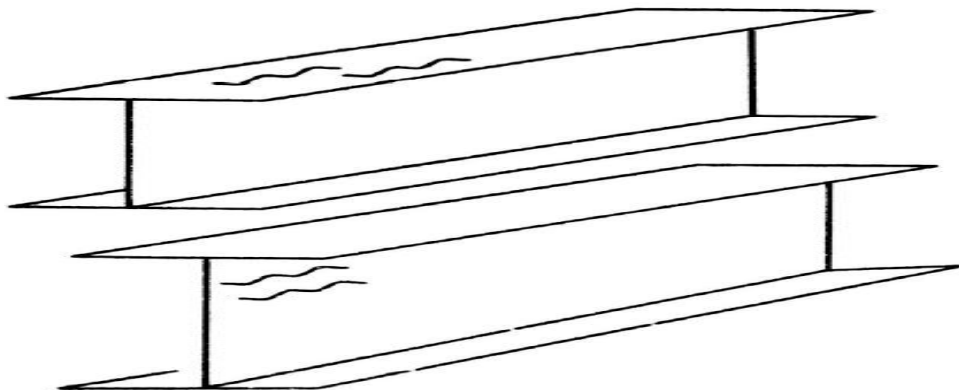
1. **Excessive bending:-** This failure occurs when a beam undergoes excessive deflection in the plane of loading. It is deflection caused by bending.



2. **Failure due to lateral torsional buckling:-** In this case, failure occurs due to lateral deflection as well as twist. The load at which this failure occurs depends upon the proportions of beam, manner of loading and support conditions.



3. **Local buckling:-** Flange may buckle due to compression. Web may buckle due to shear or due to combined effect of shear and bending or due to vertical compression under a concentrated load.



4. Local failure:- This may occur due to the following reasons,

- Web may yield due to shear
- Web may suffer local crushing
- Local failure around any openings if present in the web.

Factors affecting lateral stability

- **Cross-sectional shape:-** Closed cross sections have torsional stiffness. I-section and box sections are commonly used because of their higher lateral stiffness.
- **Support conditions:-** Fixed conditions provide higher lateral stability when compared to simply supported conditions.
- **Effective length:-** Effective length will be less when lateral restraint is provided. IS800:2007 table 10.4 provides effective lengths KL of the compression flanges for different end restraints.
- **Transverse loads:-** The arrangement and applications of transverse loads greatly affects the lateral stability of beams. When the load applied above the centroid of the cross section increases the overturning moment and becomes more critical.
- **Type of loading:-** Eccentric loading causes more lateral buckling than the load acts along CG.

Classification of beam cross-sections

- 5) **Class I or Plastic section:-** This type of sections are subjected to bending moments equal to plastic moments and can undergo local rotation means plastic hinge is formed.

Plastic hinge refers to the deformation of part of beam wherever plastic bending takes place.

Plastic moment means moment at which entire cross section reaches to yield stress.

- 6) **Class II or Compact section:-** This type of sections are subjected to bending moments equal to plastic moment but cannot undergo local rotation.
- 7) **Class III or Semi-compact sections:-** This type of sections are subjected to bending moments equal to yield moments where cross section starts to buckle after outermost fiber yields.

Yield moment is the moment at which only exterior fibers reaches yield stress.

- 8) **Class IV or Slender sections:-** This type of section fails before yield moment is obtained.

Web buckling and Web Crippling

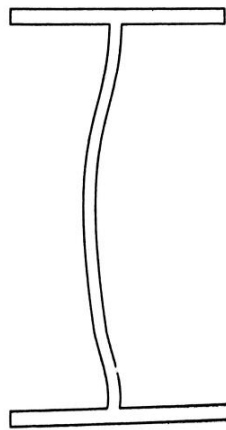
Due to the action of heavy concentrated loads on the beam, a region of high compressive stresses is directly formed under the loads. The upward reaction at the support also produces region of high compressive stress in the web. Such high compressive stresses in the web may lead to web failure like web buckling (also called as vertical buckling) and web crippling (also called as web crushing).

Web buckling:-

Web buckling occurs when the vertical compressive stress in the web at the mid height reaches the critical buckling stress, as the thin web acts as a long slender column. It is generally found that rolled steel is likely to develop web crippling before developing web buckling.

The web buckling strength at support will be

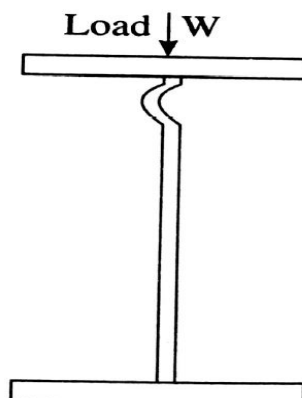
$F_{wb} = (b_1 + h_1) * t_w * f_{cd}$ where, f_{cd} is the allowable compressive stress and b_1 is the bearing length



Web Crippling

In this case due to a concentrated load on the beam or reaction at a support, high compressive stresses are produced in the thin web close to the upper flange or lower flange as a consequence of this the thin web may develop or fold at a place close to the flange.

$F_w = (b_1 + n_2) * t_w * f_{yw} / \gamma_{m0}$ where, $f_{yw} = f_y = 250 \text{ N/mm}^2$.



Problem

A simply supported beam of span 5m has its compression flange laterally supported. It has to support the following loads.

Dead load excluding the self weight of the beam = 80 kN
Point load at mid span = 65 kN
Live load = 120 kN

Solution.

$$\text{Factored } DL = 1.5 \times 80 = 120 \text{ kN}$$

$$\text{Factored } LL = 1.5 \times 120 = 180 \text{ kN}$$

$$\text{Total factored distributed load} = 300 \text{ kN}$$

$$\text{Factored point load} = 1.5 \times 65 = 97.5 \text{ kN}$$

$$\text{Factored moment} = M = \frac{300 \times 5}{8} + \frac{97.5 \times 5}{4} = 309.375 \text{ kNm}$$

$$\text{Plastic modulus required} = Z_p = \frac{309.375 \times 10^6}{250} \times 1.1 = 1361.25 \times 10^3 \text{ mm}^3$$

Let us select ISLB 450 @ 640.6 N/m

Properties of the section

Plastic modulus = $Z_p = 1401.35 \times 10^3 \text{ mm}^3$, Section modulus = $Z_e = 1223.8 \times 10^3 \text{ mm}^3$,
 $h = 450 \text{ mm}$, $b_f = 170 \text{ mm}$, $t_f = 13.4 \text{ mm}$, $t_w = 8.6 \text{ mm}$, Radius at root = $r_1 = 16 \text{ mm}$, $Z_p = 1401.35 \times 10^3 \text{ mm}^3$

$$d = h - 2(t_f + r_1) = 450 - 2(13.4 + 16) = 391.2 \text{ mm}, \quad b = \frac{b_f}{2} = \frac{170}{2} = 85 \text{ mm}$$

Section classification

$$\left. \begin{aligned} \frac{b}{t_f} &= \frac{85}{13.4} = 6.34 < 9.4 \\ \frac{d}{t_w} &= \frac{391.2}{8.6} = 45.5 < 84 \end{aligned} \right\} \therefore \text{This is a plastic section.}$$

Actual Total factored distributed load

$$= 300 + 1.5 \times 0.64 = 300.96 \text{ kN}$$

Actual factored moment

$$= M = \frac{300.96 \times 5}{8} + \frac{97.5 \times 5}{4} = 309.98 \text{ kNm}$$

Plastic modulus required

$$= Z_e = \frac{309.98 \times 10^6}{250} \times 1.1 = 1363.91 \times 10^3 \text{ mm}^3$$

Plastic modulus of ISLB 450 $= 1401.35 \times 10^3 \text{ mm}^3$

Factored shear $= V = \frac{120 + 180 + 0.96 \times 5 + 97.5}{2} = 201.15 \text{ kN}$

Design shear strength of the section

$$= V_d = \frac{f_y}{\gamma_{mo} \sqrt{3}} h t_w = \frac{250}{1.1 \sqrt{3}} (450 \times 8.6) \times 10^{-3} \text{ kN} = 507.82 \text{ kN}$$

$$0.6 V_d = 0.6 \times 507.82 = 304.69 \text{ kN} \quad \text{But } V = 201.15 \text{ kN}$$

$\therefore V < 0.6 V_d$ We also note $\frac{d}{t_w} < 67$

Design bending strength $M_d = \text{Lesser of the following}$

(i) $\beta_b Z_p \frac{f_y}{\gamma_{mo}}$ (Note $\beta_b = 1$ Since this is a plastic section)

$$= \frac{1 \times 1401.35 \times 10^3 \times 250}{1.1} \times 10^{-6} \text{ kNm}$$

$$= 318.49 \text{ kNm}$$

(ii) $\frac{1.2 Z_e f_y}{\gamma_{mo}} = \frac{1.2 \times 1223.8 \times 10^3 \times 250}{1.1} \times 10^{-6} \text{ kNm}$

$$= 333.76 \text{ kNm}$$

$\therefore M_d = 318.49 \text{ kNm}$ But $M = 309.98 \text{ kNm} \therefore M < M_d$

Deflection

Deflection is determined due to unfactored live loads

$$\text{Deflection} = \frac{5}{384} \frac{wl^4}{EI} + \frac{Wl^3}{48EI} \quad [\text{For ISLB 450, } I = 27536.1 \times 10^4 \text{ mm}^4]$$

$$\frac{5}{384} \frac{120 \times 5^3}{200 \times 27536.1 \times 10^4} \times 10^9 + \frac{65 \times 5^3}{48 \times 200 \times 27536.1 \times 10^4} \times 10^9 \text{ mm} = 3.546 + 3.074 = 6.62 \text{ mm}$$

$$\text{Permissible deflection} = \frac{\text{Span}}{300} = \frac{5000}{300} = 16.67 \text{ mm}$$

Check for web buckling at support

Provide a stiff bearing length of 80 mm i.e., $b_1 = 80 \text{ mm}$

$$\text{Depth of the web} = d = h - 2(t_f + r) = 450 - 2(13.4 + 16) = 391.2 \text{ mm}$$

$$\text{Least radius of gyration of web section} = r = \frac{t_w}{2\sqrt{3}} = \frac{8.6}{2\sqrt{3}} = 2.483 \text{ mm}$$

$$\text{Slenderness ratio} = 0.7 \times \frac{391.2}{2.483} = 110.3$$

As a slender column of buckling class c , design compressive stress corresponding to the above slenderness ratio

$$= f_{cd} = 94.27 \text{ N/mm}^2$$

$$n_1 = \frac{h}{2} = \frac{450}{2} = 225 \text{ mm} \quad b_1 + n_1 = 80 + 225 = 305 \text{ mm}$$

$$\begin{aligned} \text{Design buckling strength} &= (b_1 + n_1) t_w f_{cd} \\ &= 305 \times 8.6 \times 94.27 \times 10^{-3} \text{ kN} = 247.3 \text{ kN} \\ &\quad (\text{Greater than } V = 201.15 \text{ kN}) \end{aligned}$$

Check for web crippling at support

$$b_1 = 80 \text{ mm}, n_2 = 2.5(t_f + r_1) = 2.5(13.4 + 16) = 73.5 \text{ mm},$$

$$\begin{aligned} \text{Design crippling strength} &= \frac{(b_1 + n_2) t_w f_y}{\gamma_{mo}} \\ &= \frac{(80 + 73.5) 8.6 \times 250}{1.1} \times 10^{-3} \text{ kN} \\ &= 300.02 \text{ kN} > 201.15 \text{ kN} \end{aligned}$$

Assignment questions

1. What is a beam?
2. What are the classifications of beams?
3. Write a note on laterally supported beams.
4. Write a note on laterally unsupported beams.
5. What are the failure modes of beam?
6. Write a note on web buckling and web crippling.

Outcome

Gives knowledge about laterally supported and laterally unsupported beams.

Future Study

By studying this one design, one can apply this design criteria in design a multi storey buildings.